Impact Factors for Fatigue Design of Steel I-Girder Bridges Considering the Deterioration of Road Surface Condition

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Abstract: The purpose of this paper is to evaluate the impact factor (IM) in LRFD bridge design specifications for fatigue design and to propose a method for determining reasonable IMs for the fatigue design of steel I-girder bridges that can more rationally consider the effect of the deterioration of the road surface condition (RSC) during its whole lifecycle. The deterioration process of the RSC was investigated under the given traffic and environmental conditions, and the number of truck passages taken for the RSC to deteriorate from one class to the next was investigated. A three-dimensional coupled vehicle–bridge model was developed to simulate the interaction between the bridge and vehicle, with both the bridge and fatigue load models adopted from an existing LRFD code. The IM of the stress range (IM_SR), which is calculated using the stress range instead of the maximum stress used traditionally, was used for the fatigue analysis of steel girders. Numerical simulations were performed to study the IM_SR of steel I-girder bridges under different RSCs while taking into consideration the effect of two other important parameters: bridge span length and vehicle speed. Results show that the RSC has a greater impact on the IM_SR than on the traditional IM, and the IM_SR is greater than the traditional IM calculated using the maximum stress. By considering the cumulative fatigue damage caused by the passage of each truck under different RSCs and the deterioration process of the RSC during its whole lifecycle, simple and reasonable expressions were proposed for the IMs for fatigue design of steel I-girder bridges under the given traffic and environmental conditions.

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Author keywords: Impact factor; Fatigue design; Deterioration process; Road surface condition; Steel I-girder bridge.

Introduction

The fatigue life of steel bridge girders is dominated by the stresses caused by the trucks crossing the bridge. The stress range (an algebraic difference between the maximum and minimum stresses) due to each truck passage and the number of stress cycles are two key parameters in the fatigue analysis. In the current AASHTO LRFD bridge design specifications (AASHTO 2012), the stress range for fatigue design is calculated by applying an impact factor (IM) of 0.15 to the calculated static stress range caused by the passage of the specified fatigue truck. This IM of 0.15 adopted in the current AASHTO LRFD code for fatigue design was based on the numerical simulation results of Hwang and Nowak (1991), in which four steel I-girder composite bridges with span lengths between 12 and 30 m were studied while an average road surface condition (RSC) was adopted. However, previous studies have shown that the real stress ranges of bridge components due to the dynamic vehicle loading can be significantly underestimated when the RSC is poor (Rao and Talukdar 2003; Zhang and Cai 2012; Zhang and Cai 2013). Therefore, the IM of 0.15 currently adopted in the AASHTO LRFD code (AASHTO 2012) may not truly reflect the dynamic effect of vehicle loading on the stress ranges of bridge components during the whole lifecycle of a RSC.

During the service life of a steel bridge, the dynamic impacts due to the traffic loads and deteriorated RSCs can induce serious fatigue issues for the bridge components (Zhang and Cai 2012). Although there have been a number of studies on the effect of RSC on the behavior of vehicle–bridge systems (Deng and Cai 2010a; O’Brien et al. 2006; Schenk et al. 2003; da Silva 2004) and other related applications, such as vehicle load or parameter identification (Deng and Cai 2009; Deng and Cai 2010b; Wu and Law 2011) and bridge damage detection (Jaksic et al. 2014), few studies have investigated the effect of the road-surface deterioration process on the fatigue behavior of steel bridges. Zhang and Cai (2012, 2013) studied the effect of vehicle speed and RSC on the fatigue life of a steel bridge and proposed an approach for the fatigue design of steel bridges in which the equivalent stress ranges induced by each truck passage and the number of stress cycles were combined into one variable based on the equivalent fatigue damage. They found that the road-surface deterioration rate had a significant influence on the fatigue reliability of bridge components. However, the deterioration process of the RSC was only considered qualitatively in their study, and the cause for the deterioration of RSC, i.e., the cumulative fatigue damage by each truck passage during its whole lifecycle, was not considered in their study, which may lead to inaccurate predicted fatigue life.

The purpose of this paper is to evaluate the IM specified in the AASHTO LRFD code for bridge-fatigue design and propose a method for determining reasonable IMs for the fatigue design (IMs_fD) of steel I-girder bridges that can more rationally consider the effect of the deterioration of the RSC during the whole lifecycle. The paper is organized in three main parts. First, under the given traffic and environmental conditions, the deterioration process of the RSC is investigated in terms of the number of truck passages and the time taken for the RSC to deteriorate from one class to the next, for instance, from the good class to the average class. Second, a three-dimensional coupled vehicle–bridge model is developed to

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analyze the IM of the stress range (IM_{SR}) of the steel I-girder bridges under consideration. The relationships among three parameters, including the bridge span length, RSC, and vehicle speed, and the IM_{SR} were investigated by numerical simulations. Finally, simple and reasonable expressions of the IMs_{FD} for steel I-girder bridges are proposed.

Analytical Bridges

In this study, five typical steel I-girder bridges with span lengths ranging from 10.67 m (35 ft) to 36.58 m (120 ft) were designed according to the AASHTO LRFD bridge design specifications (AASHTO 2012). All five bridges, each consisting of five identical girders with a girder spacing of 2.13 m (7 ft), are simply supported and have a roadway width of 9.75 m (32 ft) and a bridge-deck thickness of 0.20 m (8 in). A typical cross section of the bridges is shown in Fig. 1. Besides the end diaphragms, which are used for all five bridges, intermediate diaphragms are also used to connect the five girders depending on their span lengths, as provided in Table 1. In this study, the steel I-girder bridges were modeled with ANSYS 14.5. Fig. 2 shows the finite-element model of Bridge 2. A summary of the detailed properties and the fundamental frequencies of the five bridges obtained from the modal analysis is provided in Table 1.

Analytical Vehicle Model

The HS20-44 truck specified in the AASHTO LRFD bridge design specifications (AASHTO 2012) was used as the fatigue truck for the five bridges. The analytical model for this truck is illustrated in Fig. 3. The properties of the truck, including the geometry, mass distribution, damping, and stiffness of the tires and suspension systems, are given in Table 2 (Shi 2006).

Coupled Vehicle–Bridge System

Equation of Motion of the Vehicle

The equation of motion for a vehicle can be expressed as follows:

\[
[M_v] \{\ddot{d}_v\} + [C_v] \{\dot{d}_v\} + [K_v] \{d_v\} = \{F_G\} + \{F_v\}
\]  

(1)

where \([M_v]\), \([C_v]\), and \([K_v]\) = mass, damping, and stiffness matrices of the vehicle, respectively; \(\{d_v\}\) = displacement vector of the vehicle; \(\{F_G\}\) = gravity force vector of the vehicle; and \(\{F_v\}\) = vector of the wheel–road contact forces acting on the vehicle.

Equation of Motion of the Bridge

The equation of motion for a bridge can be written as follows:

\[
[M_b] \{\ddot{d}_b\} + [C_b] \{\dot{d}_b\} + [K_b] \{d_b\} = \{F_b\}
\]  

(2)

where \([M_b]\), \([C_b]\), and \([K_b]\) = mass, damping, and stiffness matrices of the bridge, respectively; \(\{d_b\}\) = displacement vector of the bridge; and \(\{F_b\}\) = vector of the wheel–road contact forces acting on the bridge.

Assembling the Vehicle–Bridge Coupled System

Based on the displacement relationship and the interaction force relationship at the contact points, the vehicle–bridge Coupled system can be established by combining the equations of motion of both the bridge and vehicle (Deng and Cai 2009)

![Fig. 1. Typical cross section of bridges considered and vehicle loading position adopted](image)

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Span length (m)</th>
<th>Fundamental natural frequency (Hz)</th>
<th>Cross-sectional area (m²)</th>
<th>Inertia moment of cross section (10⁻² m⁴)</th>
<th>No. of intermediate diaphragms</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.67</td>
<td>12.40</td>
<td>0.018</td>
<td>0.040</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>16.76</td>
<td>8.62</td>
<td>0.020</td>
<td>0.109</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>22.86</td>
<td>6.10</td>
<td>0.023</td>
<td>0.219</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>30.48</td>
<td>4.39</td>
<td>0.026</td>
<td>0.421</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>36.58</td>
<td>3.49</td>
<td>0.028</td>
<td>0.641</td>
<td>4</td>
</tr>
</tbody>
</table>

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where $C_{b-v}, C_{b-v}, C_{v-b}, C_{v-b}, K_{b-v}, K_{b-v}, K_{v-b}, K_{v-b}, F_{b-r},$ and $F_{v-r}$ are due to the wheel–road contact forces and are time-dependent terms.

To simplify the bridge model and therefore save computational effort, the modal superposition technique can be used (Deng and Cai 2010b), and Eq. (3) can be simplified into the following:

$$\begin{bmatrix}
M_b \\
M_v
\end{bmatrix}
\begin{Bmatrix}
\ddot{d}_b \\
\ddot{d}_v
\end{Bmatrix}
+ \begin{bmatrix}
C_{b-b} & C_{b-v} \\
C_{v-b} & C_v
\end{bmatrix}
\begin{Bmatrix}
d_b \\
v
\end{Bmatrix}
+ \begin{bmatrix}
K_{b-b} & K_{b-v} \\
K_{v-b} & K_v
\end{bmatrix}
\begin{Bmatrix}
\dot{d}_b \\
\dot{d}_v
\end{Bmatrix}
= \begin{Bmatrix}
F_{b-r} \\
F_{v-r} + F_G
\end{Bmatrix}
$$

where $C_{b-b}, C_{b-v}, C_{v-b}, K_{b-b}, K_{b-v}, K_{v-b}, F_{b-r},$ and $F_{v-r}$ are due to the wheel–road contact forces and are time-dependent terms.

Modeling the Deterioration Process of RSC

**Generation of Road Surface Profile**

RSC is a very important factor that affects the dynamic interaction between the bridge and vehicle. A road surface profile is usually assumed to be a zero-mean stationary Gaussian random process and can be generated through an inverse Fourier transformation based
Table 2. Major Parameters of the Fatigue Truck Model Used in This Study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of Truck Body 1 (kg)</td>
<td>2,612</td>
</tr>
<tr>
<td>Pitching moment of inertia of Truck Body 1 (kg·m²)</td>
<td>2,022</td>
</tr>
<tr>
<td>Mass of Truck Body 2 (kg)</td>
<td>26,113</td>
</tr>
<tr>
<td>Pitching moment of inertia of Truck Body 2 (kg·m²)</td>
<td>33,153</td>
</tr>
<tr>
<td>Rolling moment of inertia of Truck Body 2 (kg·m²)</td>
<td>181,216</td>
</tr>
<tr>
<td>Mass of first axle suspension (kg)</td>
<td>490</td>
</tr>
<tr>
<td>Upper spring stiffness of first axle (N/m)</td>
<td>242,604</td>
</tr>
<tr>
<td>Upper damper coefficient of first axle (N/s/m)</td>
<td>2,190</td>
</tr>
<tr>
<td>Lower spring stiffness of first axle (N/m)</td>
<td>87,085</td>
</tr>
<tr>
<td>Lower damper coefficient of first axle (N/s/m)</td>
<td>2,000</td>
</tr>
<tr>
<td>Mass of second axle suspension (kg)</td>
<td>808</td>
</tr>
<tr>
<td>Upper spring stiffness of second axle (N/m)</td>
<td>1,903,172</td>
</tr>
<tr>
<td>Upper damper coefficient of second axle (N/s/m)</td>
<td>7,882</td>
</tr>
<tr>
<td>Lower spring stiffness of second axle (N/m)</td>
<td>3,503,307</td>
</tr>
<tr>
<td>Lower damper coefficient of second axle (N/s/m)</td>
<td>2,000</td>
</tr>
<tr>
<td>Mass of third axle suspension (kg)</td>
<td>653</td>
</tr>
<tr>
<td>Upper spring stiffness of third axle (N/m)</td>
<td>1,969,034</td>
</tr>
<tr>
<td>Upper damper coefficient of third axle (N/s/m)</td>
<td>7,182</td>
</tr>
<tr>
<td>Lower spring stiffness of third axle (N/m)</td>
<td>3,507,429</td>
</tr>
<tr>
<td>Lower damper coefficient of third axle (N/s/m)</td>
<td>2,000</td>
</tr>
<tr>
<td>Distance from front axle to mass center of tractor L1 (m)</td>
<td>1.698</td>
</tr>
<tr>
<td>Distance from middle axle to mass center of tractor L2 (m)</td>
<td>2.569</td>
</tr>
<tr>
<td>Distance from middle axle to mass center of trailer L3 (m)</td>
<td>4.452</td>
</tr>
<tr>
<td>Distance from rear axle to mass center of trailer L4 (m)</td>
<td>4.692</td>
</tr>
<tr>
<td>Distance from rivet joint to mass center of tractor L5 (m)</td>
<td>2.215</td>
</tr>
<tr>
<td>Distance from rivet joint to mass center of trailer L6 (m)</td>
<td>4.806</td>
</tr>
<tr>
<td>Half distance between wheels in the same axle b (m)</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Table 3. RRCs for Different Road-Roughness Classifications

<table>
<thead>
<tr>
<th>Road-roughness classification</th>
<th>Range for RRC (m³/cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very good</td>
<td>2 × 10⁻⁶ to 8 × 10⁻⁶</td>
</tr>
<tr>
<td>Good</td>
<td>8 × 10⁻⁶ to 32 × 10⁻⁶</td>
</tr>
<tr>
<td>Average</td>
<td>32 × 10⁻⁶ to 128 × 10⁻⁶</td>
</tr>
<tr>
<td>Poor</td>
<td>128 × 10⁻⁶ to 512 × 10⁻⁶</td>
</tr>
<tr>
<td>Very poor</td>
<td>512 × 10⁻⁶ to 2,048 × 10⁻⁶</td>
</tr>
</tbody>
</table>

Table 4. Number of Truck Passages and Time Taken for the RSC to Deteriorate to the Next Class

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Very good</th>
<th>Good</th>
<th>Average</th>
<th>Poor</th>
<th>Very poor</th>
</tr>
</thead>
<tbody>
<tr>
<td>N_t</td>
<td>4,113,464</td>
<td>1,156,504</td>
<td>938,321</td>
<td>839,119</td>
<td>768,396</td>
</tr>
<tr>
<td>t (years)</td>
<td>6.63</td>
<td>1.86</td>
<td>1.52</td>
<td>1.35</td>
<td>1.24</td>
</tr>
<tr>
<td>T (years)</td>
<td>6.63</td>
<td>8.49</td>
<td>10.01</td>
<td>11.36</td>
<td>12.6</td>
</tr>
<tr>
<td>r (%)</td>
<td>52.63</td>
<td>14.80</td>
<td>12.01</td>
<td>10.74</td>
<td>9.82</td>
</tr>
</tbody>
</table>

Progressive Deterioration of RSC

To reflect the progressive damage of the RSC due to traffic loading and/or environment corrosions, a proper progressive deterioration model for the RSC is necessary. Based on Paterson’s (1986) study, the IRI values at any time since opened to traffic can be calculated as

\[
\text{IRI}_t = 1.04 \epsilon^{\eta t} \cdot \text{IRI}_0 + 263(1 + \text{SNC})^{-5} \cdot \text{(CESAL)}_t \tag{9}
\]

where IRI_t is the IRI value at time t; IRI_0 is the initial roughness value before opened to traffic; t is the time in years; \( \eta \) is the environmental coefficient varying from 0.01 to 0.7 depending on dry or wet and freezing or nonfreezing conditions; the structural number (SNC) is a parameter calculated from the strength and thickness of each layer in the pavement; and (CESAL)_t is the estimated number of traffic in millions in terms of the AASHTO 80-kN (18-kip) equivalent single-axle load at time t. It should be noted that Eq. (9) was initially developed for pavement management systems when initiating the maintenance and rehabilitation of asphalt-surfaced pavements. However, the deterioration of RSCs is affected mainly by three factors: initial roughness level, traffic loading, and age. Other factors, such as pavement thickness and stiffness, have a smaller influence on the roughness deterioration (Shiyab 2007). Therefore, the stiffness difference between bridge pavements and pavements on subgrade was not considered when analyzing the roughness deterioration in this study. Therefore, the RRC at any
time since opened to traffic can be predicted with the following equation:

$$w(n_0) = \frac{6.1972 \times 10^{-6} \times \exp \left\{ \left[ 1.04 e^{0.1 \cdot IRI} \right] + 263(1 + SNC)^{-2} \cdot (CESAL) \right\} / 0.42808 \right\} + 2 \times 10^{-6}$$ (10)

The purpose of modeling the deterioration process of the RSC is to obtain the required number of truck passages, $N_i$ ($i = 1, 2, 3, 4, 5$), taken for the RSC to deteriorate from one class to another. For instance, $N_1$ is the number of truck passages required for the RSC to deteriorate from the very good class to the good class. Based on Shiyab (2007), the SNC is calculated to be 6.19, and $H$ is usually adopted as 0.1 for bridges exposed in general environment. In this study, the average daily truck traffic (ADTT) and fraction of traffic in a single lane were assumed to be 2,000 and 0.85, respectively, as suggested by the AASHTO LRFD code (AASHTO 2012). The traffic increase rate was not considered, and thus the CESAL was assumed to be 12.42 for each lane each year (Shiyab 2007). Based on Eq. (10), the deterioration process of the RSC can be analyzed, and the results are shown in Table 4. In Table 4, $N_i$ and $t_i$ are the number of truck passages required and years taken for each class of RSC to deteriorate to the next class, respectively; $T$ is the total years taken for the RSC to deteriorate to the end of each class of the RSC since opened to traffic; and $r_i = N_i/\sum N_i$ ($i = 1, 2, 3, 4, 5$), the proportion of the number of truck passages required for each class of RSC to deteriorate to the next class.

**Numerical Studies**

In the literature, the influence of a number of parameters on the dynamic IM has been studied, including the vehicle loading position, vehicle weight, vehicle traveling speed, number of loading lanes, RSC, and road-surface roughness correlation (Chang and Lee 1994; Liu et al. 2002; Yang et al. 1995). In this study, three important parameters commonly considered to have a significant effect on the IM were investigated: bridge span length, vehicle speed, and RSC.

It should be noted that the accuracy and reliability of the vehicle–bridge model used in this study were verified in other works (Deng 2009; Deng and Cai 2010c), in which a series of field tests were conducted on an existing slab-on-girder concrete bridge in Louisiana. It was found that the field-measured bridge responses agreed very well with the numerical simulation results.

The span lengths and other parameters of the five bridges used in this study are listed in Table 1. Seven vehicle speeds, ranging from 30 to 120 km/h with an interval of 15 km/h, were considered, and
five different RSCs according to ISO (1995) were studied: very good, good, average, poor, and very poor. Fig. 1 shows the vehicle position for the loading case specified in the AASHTO LRFD code (AASHTO 2012), where the vehicle travels along the centerline of the lane. It should be noted that the road surface profiles along left-wheel and right-wheel tracks of the truck were assumed to be exactly the same. In other words, the variation of road surface in the lateral direction was not considered in this study. Actually, it was shown by Liu et al. (2002) that, in practice, the use of two identical profiles does not cause much deviation in the resulting dynamic bridge responses. It should also be noted that the IM due to the fatigue truck may not necessarily equal the IM due to the real traffic (the former is usually slightly larger than the latter). Actually, the fatigue truck adopted in the current AASHTO LRFD code (AASHTO 2012) was developed based on the actual truck-traffic spectrum obtained from weigh-in-motion studies, in which data of more than 27,000 trucks and 30 sites around the United States (Snyder et al. 1985) were collected. A fatigue truck is typically used to represent the equivalent fatigue damage accumulation resulting from the truck traffic at a specific site with a variety of gross vehicle weights and axle configurations (Chotickai and Bowman 2006). Because of this reason, the IM due to the fatigue truck is usually used to represent the IM for the real traffic by researchers for the sake of simplicity and conservativeness.

To investigate the relationship among the three parameters and the IM, for each specific case with a given bridge span length, vehicle speed, and RSC, the vehicle–bridge interaction analysis was set to run 20 times with 20 sets of randomly generated road surface profiles under the given RSC. Then, the average value of the 20 IMs was obtained and used in the following result analysis. A total of 20 simulations were also considered to be sufficient by other researchers (Liu et al. 2002).

Stress range is a key parameter that affects the fatigue behavior of steel bridges. As illustrated in Fig. 4, the IM_{SR} is defined as follows (Patrick et al. 1992):

\[
\text{IM}_{SR} = \frac{R_{\text{dyn,s}} - R_{\text{sta}}}{R_{\text{sta}}}
\]

whereas the traditional IM is defined using stress as follows:
where $R_{\text{dyn,s}}, R_{\text{dyn,a}}$ and $R_{\text{sta}}$ are the maximum dynamic stress range and maximum dynamic stress and static stress caused by the vehicle loading, respectively. The stress range at the midspan of the girder carrying the largest amount of load was selected as the bridge response for calculating the IMs in this study. It should be noted that the numerical simulation results show that the maximum dynamic stress either occurred at the midspan or was very close to the dynamic stress at the midspan, depending on factors including the bridge span length, diaphragm position, vehicle speed, and RSC. Therefore, for the purpose of convenience, the stress at the midspan was chosen in the analysis.

Fig. 5 shows the maximum static stress ranges at the midspan of all five girders of each bridge under the loading case considered. It can be easily seen that the largest stress range occurred at the midspan of Girder 4. Therefore, the stress ranges of Girder 4 were used for calculating the IMs. It should be noted that the reason why the stress ranges of Bridge 3 were larger than those of Bridge 4 on Girders 3 and 4 is that Bridge 4 has a diaphragm right at the midspan; thus, the vehicle loading is distributed more evenly laterally.

The average IMs obtained from the numerical simulations for each RSC based on Eq. (11) were plotted against the vehicle speed in Fig. 6, where the results for bridges with different span lengths were plotted separately. For comparison, the average IMs calculated based on Eq. (12) were also plotted in Fig. 6.

With the average IMs varying from greater than 3.0 when the RSC is very poor to less than 0.10 when the RSC is very good, it is evident from Fig. 6 that, all five bridges, the RSC has a significant impact on both the IMs and the traditional IMs. However, an increase in vehicle speed does not necessarily lead to an increase in the IM, as also reported by other researchers (Brady et al. 2006; Liu et al. 2002). Interestingly, Fig. 6 also shows that the RSC has a greater impact on the IMs than on the traditional IMs, and the average IMs are larger than the average traditional IMs under each RSC.

To examine the effect of each parameter on the IM more clearly, the average IMs were plotted against each of the three parameters separately in Fig. 7. It can be easily seen that the average IMs for all the RSCs are all above the AASHTO-specified value of 0.15 with the exception of the very good RSC. This is easily understandable because the value of 0.15 adopted in the AASHTO LRFD code for fatigue design is based on the study of Hwang and Nowak (1991), in which only an average RSC was considered in the numerical simulations.

To propose rational IMs for steel I-girder bridges, the following steps were taken. First, based on the regression analysis on the simulation results, the expressions for predicting the IMs for each RSC were obtained. Then, by considering the cumulative fatigue damage caused by the passage of each truck under different RSCs and the number of truck passages taken to cause the RSC to deteriorate from one class to the next, the IMs for steel I-girder bridges were proposed, taking into consideration the whole lifecycle of the RSC.

Because it has been demonstrated that the IM is highly dependent on the RSC, it would be natural to express the IM as a function of RSC. In this study, the following expressions for predicting the IM under each RSC were suggested based on a regression analysis:

\[
IM_{\text{SR}} = \frac{R_{\text{dyn}} - R_{\text{sta}}}{R_{\text{sta}}} \quad (12)
\]

\[
IM_{\text{SR}} = \text{RSL} \times \left\{ \begin{array}{ll}
0.46 + 0.057 \times (22.86 - L) & L < 22.86 \\
0.46 & L \geq 22.86
\end{array} \right.
\]

where \(R_{\text{SL}}\) = road surface index, which is taken as 0.12, 0.27, 0.62, 1.21, or 2.71, corresponding to very good, good, average, poor, or very poor RSC, respectively, based on the regression analysis results; and \(L\) = bridge span length.

The reason why vehicle speed was not considered in this expression is that vehicles can usually run at speeds within a wide range,
whereas an increase in vehicle speed does not necessarily cause the IM_SR to increase or decrease monotonically, as shown in Fig. 6. Moreover, the influence of vehicle speed on the IM_SR is considerably smaller than that of bridge span length and RSC. Therefore, as usually done in the codes, vehicle velocity is not included as a variable in the proposed expressions for predicting the IM_SR. In addition, it should also be noted that, in real life, drivers tend to drive at lower speeds under worse RSCs, and therefore it may not be appropriate to calculate the average IMs for each RSC by taking the average of the results for all seven vehicle speeds considered. However, due to the same reasons discussed above, this influence was not considered in this study.

Based on the proposed expressions, the IMs_SR of the five bridges studied were predicted for each RSC, as summarized in Table 5.

![Fig. 7. Variation of the average IM_SR with change of each parameter: (a) bridge span length; (b) vehicle speed; (c) RSC](image)

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Very poor</th>
<th>Poor</th>
<th>Average</th>
<th>Good</th>
<th>Very good</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.13</td>
<td>1.40</td>
<td>0.72</td>
<td>0.31</td>
<td>0.14</td>
</tr>
<tr>
<td>2</td>
<td>2.18</td>
<td>0.98</td>
<td>0.50</td>
<td>0.22</td>
<td>0.10</td>
</tr>
<tr>
<td>3–5</td>
<td>1.25</td>
<td>0.56</td>
<td>0.29</td>
<td>0.13</td>
<td>0.06</td>
</tr>
<tr>
<td>6</td>
<td>2.17</td>
<td>0.96</td>
<td>0.52</td>
<td>0.21</td>
<td>0.10</td>
</tr>
</tbody>
</table>

![Table 5. IMs_SR for the Six Bridges](image)

Proposed IMs_FD

As fatigue failure is related to the cumulative damage caused by the passage of each truck, a reasonable IM_FD should take into consideration the effect of each truck passage on the cumulative fatigue damage during the lifecycle of the bridge. Based on this consideration, the following IMs were proposed for the fatigue design of steel I-girder bridges under the traffic and environmental conditions suggested by the AASHTO LRFD code:

\[
IM_{FD} = \sum r_i \times IM_{SR} = \sum r_i \times RSI \times \begin{cases} 
0.46 + 0.057 \times (22.86 - L) & L < 22.86 \text{ m} \\
0.46 & L \geq 22.86 \text{ m}
\end{cases} \quad (14)
\]

where \( r_i \) (\( i = 1, 2, 3, 4, 5 \)) is the ratio of the numbers of truck passages needed for each class of the RSC to deteriorate to the next class, as summarized in Table 4.

Based on Eq. (14), the proposed IMs_FD for the five bridges studied were calculated and are summarized in Table 6. It should be noted that, in Table 6, that two conditions were considered: Condition A and Condition B, where the only difference between the two conditions is that Condition A included all five RSCs when calculating the IM_FD using Eq. (14), whereas Condition B does not include the very poor RSC in the calculation. The purpose of not
considering the very poor RSC in Condition B is that, in real life, maintenance has usually been performed before the road-wearing surface turns into a very poor condition. Therefore, Condition B may be more realistic when considering the whole lifecycle of the road-wearing surface.

Table 6 shows that the proposed IMs_FD for relatively short bridges (Bridges 1 and 2) are significantly larger than those for relatively long bridges (Bridges 3–5). In addition, the IMs_FD under Condition A are considerably larger than those under Condition B, indicating the importance of maintaining the RSC, especially very poor RSC, in reducing the fatigue damage of bridge components due to the traffic load. It is also very interesting to note that, for bridges with span lengths of no shorter than 22.86 m (Bridges 3–5), the proposed IMs_FD under Condition B has a constant value of 0.14, which is slightly smaller than the IM of 0.15 specified in the AASHTO LRFD code (AASHTO 2012) for fatigue design. This indicates that the IM of 0.15 specified in the current AASHTO LRFD code is suitable for the fatigue design of medium-span to long-span simply supported steel I-girder bridges. However, for the fatigue design of short-span bridges, larger IMs should be adopted.

Conclusions

In this study, the IMs_SR for steel I-girder bridges were studied. Simple and reasonable IMs_FD for steel I-girder bridges were proposed by considering the cumulative fatigue damage caused by the passage of each truck under different RSCs and the deterioration process of the road surface during its whole lifecycle. Whereas previous researchers who have based their studies on either numerical simulation or field test results have raised the concern that the IM currently adopted for fatigue design may be unconservative under certain circumstances, this paper attempted to provide more reliable quantitative results that can more accurately describe the relationships between the IM and its influencing parameters and simple IMs that can be used as reference by researchers and engineers. Based on the results from this study, the following conclusions can be drawn:

1. Under the traffic and environment conditions suggested by the AASHTO LRFD code, it takes 12.6 years for the RSC to deteriorate from the very good class to the very poor class. The time taken for the RSC to deteriorate from one class to the next class becomes shorter as the RSC becomes worse.
2. The RSC has a greater impact on the IMs_SR than on the traditional IMs calculated using the maximum stress, and the average IMs_SR are larger than the average traditional IMs under each RSC.
3. The proposed IMs_FD for steel I-girder bridges can more accurately consider the dynamic effect of vehicle loading on the fatigue life of steel I-girder bridges. The proposed IMs also indicate that the IM of 0.15 adopted in the current AASHTO LRFD code is still suitable for the fatigue design of medium-span to long-span steel I-girder bridges under the assumed traffic and environmental condition. However, for the fatigue design of short-span steel bridges, larger IMs should be adopted.

It should be noted that the expression for IMs_FD in Eq. (14) was obtained with an ADTT of 2,000 under the environmental condition suggested by the AASHTO LRFD code. Therefore, IMs calculated using Eq. (14) can be used as supplementary specifications to the AASHTO LRFD bridge design specifications when dealing with fatigue design of short steel bridges under the assumed traffic and environmental conditions. However, the method proposed in this study can be used for determining reasonable IMs_FD for different types of steel bridges and under different traffic and environmental conditions by adopting different environmental coefficients (η) and traffic numbers (CESAL) in Eq. (10).

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