Vehicle Weight Limits and Overload Permit Checking Considering the Cumulative Fatigue Damage of Bridges

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Abstract: The ever-increasing demand in freight transportation results in a fast-growing number of overload permit requests every year. Overweight trucks, if not properly managed, can induce excessive fatigue damage that could significantly reduce bridges' load-carrying capacity and affect their durability. Therefore, developing rational vehicle weight limits and procedures for overload permit checking is very important to ensure the safety of bridges. In the current practice of overweight vehicle management, a permit decision is usually made by checking the ratio of the load effect imposed by the overweight truck to the design vehicle load effect against an allowable limit. The fatigue damage on the bridge due to the repeated vehicular loads and its influence on bridges' load-carrying capacity are usually ignored. The widely adopted federal bridge formula in the United States has also been criticized as being too restrictive for vehicles with certain axle configurations. In this article, a method for determining vehicle weight limit and overload permit checking is proposed based on the consideration of the cumulative fatigue damage of bridges. A typical steel–concrete composite girder bridge is used as an example for illustrating the proposed method. Based on the results from this study, the rationality of the federal bridge formula is discussed. The results from this study can not only be used to develop vehicle weight limits and assist in overload permit checking but also to assess the fatigue damage and predict the remaining fatigue life of existing bridges. **DOI: 10.1061/(ASCE)BE.1943-5592.0001267.** *© 2018 American Society of Civil Engineers.*

Author keywords: Vehicle weight limit; Overload permit checking; Cumulative fatigue damage; Federal bridge formula; Composite girder bridge.

Introduction

Despite the existence of various regulations on truck weight and size, the phenomenon of truck overloading is still very common around the world. Repetitive vehicular overloading not only poses a great threat to the safety of bridges but also brings a fast-growing number of overload permit requests for road authorities to deal with. Rational vehicle weight regulations and effective systems for overload permit checking are therefore highly desired.

When evaluating the permit request of an overweight vehicle, the basic idea is to check the ratio of the stress imposed by the overweight vehicle to the design load effect and make sure that the overstress ratio is controlled under an acceptable level (TRB 1990; Correia and Branco 2006). Various methods are available for calculating the bridge stresses, and the key is to accurately predict the longitudinal stresses on the bridge girders. Detailed structural analysis may be performed to obtain the bridge stresses, but it is very time-consuming and requires detailed information on the bridge structure that may be difficult to collect (Vigh and Kollár 2006, 2007). Simplified structural analysis methods, such as the beamline analysis method, the influence line method, and two-dimensional (2D) finite-element (FE) analysis, may also be adopted to calculate the stresses (Vigh and Kollár 2006, 2007; Correia and Branco 2006; Correia et al. 2014). However, these simplified analysis methods have often been argued to be too conservative (Wood et al. 2007).

Another simple way for overload permit checking is to compare the axle loads of the vehicle with the allowable limits. In the United States, the federal bridge gross weight formula, also known as Bridge Formula B, as defined in Eq. (1), is one of the widely used methods.

$$W = 2{,}224\left(\frac{0.305N}{N-1}B + 12N + 36\right) \tag{1}$$

where W = allowable gross weight in newtons on any group of two or more consecutive axles; N = number of axles in the group under consideration; and B = distance in meters between the outer axles of the group under consideration. The formula also sets a cap of 363 kN on the gross vehicle weight (GVW).

Bridge Formula B was first implemented in 1974 to restrict the weight-to-length ratio of a vehicle crossing a bridge. Although several improvements have been made since then, it has been argued that the formula is overly restrictive for shorter trucks (TRB 1990). In the meantime, there have been a lot of criticisms of the cap of 363 kN set on the GVW, which is believed to be arbitrary and too restrictive for long combination vehicles (TRB 1990; Moshiri and Montufar 2016). In fact, it is believed that interstate highway bridges can carry more loads than this cap as long as a vehicle does not overstress the bridges, which are designed according to different codes, by their permissible limits (James et al. 1986; Jaykishan 2005).

In addition, the overstress criterion adopted in developing existing bridge formulas has been criticized by researchers for not considering the damage of bridges due to repeated loads or overloads (Ghosn 2000). In addition, the allowable overstress ratios adopted by different organizations are not consistent (FHWA 1994; Vigh and Kollár 2006; Correia and Branco 2006). In this article, a method for determining vehicle weight limit and overload permit checking is proposed that considers the cumulative fatigue damage of bridges

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due to repeated vehicular loads. A typical steel–concrete composite girder bridge is used as an example for illustrating the proposed method. The rationality of the federal bridge formula is also discussed. The results from this study can not only provide a useful reference for the fatigue design of new bridges but also assist in developing vehicle weight regulations and assessing the fatigue damage and the remaining fatigue life of existing bridges.

Vehicle Loads

The axle configuration of vehicles has a considerable influence on the resulting stresses of bridges and is therefore important in setting the vehicle weight limit and conducting the overload permit checking. In this study, two typical truck models were used. One was the HS-20 design truck specified in the AASHTO *Load and Resistance Factor Design* (LRFD) code (AASHTO 2012), which was used in this study for calculating the design load effect. The other was the design fatigue truck specified in the AASHTO fatigue guide specifications (AASHTO 1990). The fatigue truck was utilized to calculate the cumulative fatigue damage caused by the traffic loads on the bridge and was also adopted as the reference model for overweight trucks.

Fig. 1 shows a sketch of these two trucks. It is noted that these two trucks have very similar configurations except for the axle spacing between the middle and rear axles (denoted as L in Fig. 1), which is 4.27 and 9.14 m, respectively, for the HS-20 truck and the design fatigue truck. Moreover, the configuration of the two trucks was developed based on the axle-weight ratios and axle spacings of the four- and five-axle trucks, which account for the major types of trucks that induce the fatigue damage on typical bridges, and the sets of two closely spaced tandem axles in the actual four- and five-axle trucks were replaced by the single axle (Schilling and Klippstein 1978; Schilling 1984). In addition, the two truck models have the same GVW of 320 kN as specified in the AASHTO fatigue guide specifications (AASHTO 2012). This weight was determined based on the actual truck traffic spectrum obtained from the weigh-in-motion (WIM) data covering over 27,000 trucks and 30 sites across the United States (Snyder et al. 1985).

To study the effect of the GVW of the overweight trucks on the fatigue damage of bridges, five more GVWs for the overweight trucks, namely, 363, 400, 445, 489, and 534 kN, were investigated. To simplify the analysis, the overweight trucks are assumed to have the same axle spacing as the design fatigue truck except the GVW. The extra weight of the overweight truck is proportionally distributed to each truck axle. In addition, the effect of traffic volume was also investigated in this study. Studies have shown that the majority of design lane average daily truck traffic (ADTT) lies between 1,500 and 2,500 (Ghosn et al. 2015). Therefore, five different ADTT values, namely, 1,000, 1,500, 2,000, 2,500, and 3,000, were considered in this study.

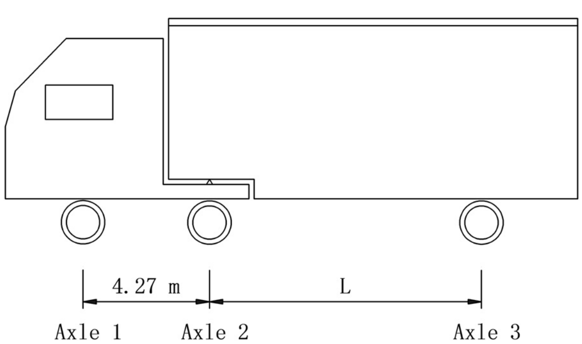

Fig. 1. Adopted truck models.

Bridge Model

In this study, a steel–concrete composite girder bridge designed according to the AASHTO LRFD code (AASHTO 2012) was adopted for the purpose of illustration. The bridge consists of five identical steel I-girders with details as listed in Table 1. It has a span length of 30.48 m and a roadway width of 9.75 m and is a good representative of the simply-supported multigirder bridges in the United States. The cross section of the bridge is illustrated in Fig. 2. In addition to the end diaphragms, three intermediate diaphragms were arranged for the bridge. Intermediate diaphragms are generally utilized to stabilize the girders during construction and placement of the deck and also to distribute traffic load transversely among the girders to some extent (Stallings et al. 1997).

The complexity of the FE model may have a considerable influence on the accuracy of the calculated bridge stresses. Therefore, in the present study, two FE bridge models with different levels of complexities were investigated, namely, a three-dimensional (3D) bridge model, as illustrated in Fig. 3, and a 2D FE bridge model. In the 3D FE bridge model, the concrete bridge deck, the steel girder, and the guardrail were modeled by solid elements, whereas the diaphragms were modeled by shell elements. For the 2D FE bridge model, the concrete bridge deck, the top flange, and the bottom flange of the steel I-girders were modeled by beam elements, whereas the web of the steel I-girder was modeled with plane elements.

Finite-element analysis was adopted in calculating the girder stresses. For the 3D FE bridge model, the girder stresses can be obtained directly from a detailed FE analysis. For the 2D FE bridge model, the girder stress was calculated using the beamline analysis method combined with the corresponding girder distribution factor (GDF), which can be calculated using Eq. (2) in the AASHTO LRFD code (AASHTO 2012), as done by other researchers (Wood et al. 2007; Harris and Gheitasi 2013):

$$\mathrm{GDF} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \tag{2}$$

where S = girder spacing of 2.13 m; L = span length of 30.4 m; K_g = longitudinal stiffness parameter of the girder; and t_s = depth of concrete slab, namely, 0.2 m.

Vehicle Weight Limit Analysis Method

Accumulated Fatigue Damage on Bridges

In the present study, the Miner's cumulative damage model (Miner 1945), which is usually referred to as Miner's rule, was

Table 1. Properties of the steel I-girders of the bridge considered

Property	Value
Number of girders	5
Girder spacing	2.13 m
Girder height	1.61 m
Cross-sectional area	0.02 m^2
Moment of inertia	0.0011 m^4
Initial Young's modulus	210 GPa
Poisson's ratio	0.25

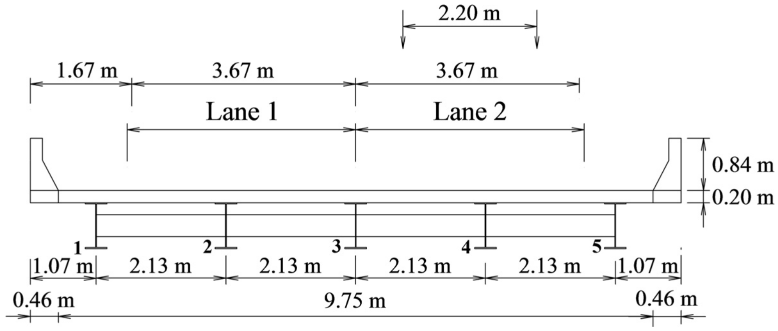

Fig. 2. Cross section of the bridge considered.

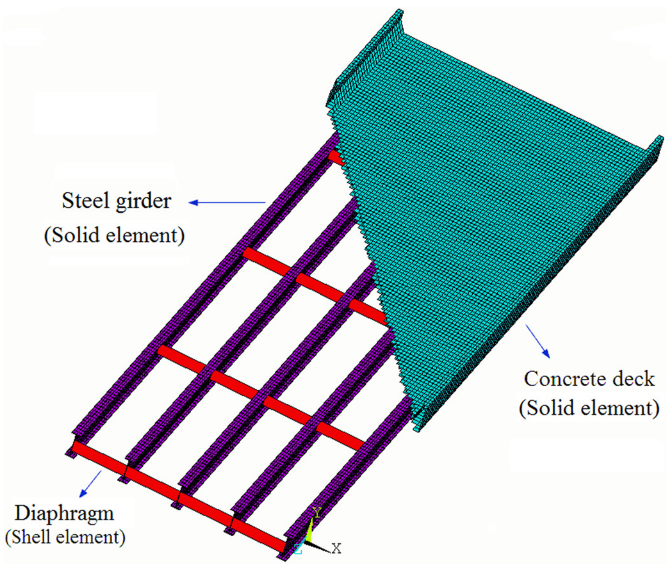

Fig. 3. 3D FE model of the bridge considered.

adopted when calculating the cumulative fatigue damage of bridges. Miner's rule is suggested by the AASHTO LRFD code (AASHTO 2012) for calculating the effective stress range and has been widely used in bridge design (Fatemi and Yang 1998). Based on the stress time history obtained, Eq. (3) was adopted to calculate the vehicle-induced cumulative fatigue damage on the bridge girders:

$$\mathrm{CFD}(t) = \sum_i \frac{n_i}{N_i} \tag{3}$$

where n_i and N_i = actual number of stress cycles experienced and the fatigue life corresponding to the ith stress-range bin S_i, respectively. In addition, the relationship between the two parameters N_i and S_i can be expressed as follows (AASHTO 2012):

$$N_i = \frac{A}{S_i^m} \tag{4}$$

where A = fatigue-strength coefficient; and m = slope constant of the S-N curve. For the details investigated in this study (i.e., the welds connecting the bottom flange and the web of the steel girders), the slope constant m can be approximately taken as 3 based on their fatigue category defined in the AASHTO LRFD code (AASHTO 2012). In addition, the fatigue constant A corresponding to Category B (Item 3.1) for welded joints was adopted herein, and the value of 3.93×10^{12} MPa was taken from Table 6.6.1.2.3–1 in the AASHTO LRFD code (AASHTO 2012).

It was demonstrated by Schilling (1984) that the cumulative fatigue damage of complex stress cycles due to an individual truck passage can be regarded as the fatigue damage resulting from the maximum stress range (MSR) with an equivalent number of stress cycles (ENSC) that can be determined from the following relationship:

$$\mathrm{ENSC} = num + \left(\frac{S_{r1}}{S_{rp}}\right)^m + \left(\frac{S_{r2}}{S_{rp}}\right)^m + \cdots + \left(\frac{S_{ri}}{S_{rp}}\right)^m \tag{5}$$

where num = the number of maximum stress range caused by individual truck passage; S_{ri} = higher-order stress range; and S_{rp} = maximum stress range, which can be calculated as the algebraic difference between the maximum stress and the minimum stress.

The number of stress cycles caused by the passage of a truck can be determined by the rainflow counting algorithm (Downing and Socie 1982). Stress ranges below 3.45 MPa are believed to have a negligible influence on the fatigue behavior (Kwon et al. 2012). Therefore, 3.45 MPa was selected herein to be the threshold when counting the number of stress cycles. The upper limit of a cutoff threshold is typically approximately 25–33% of the constant-amplitude fatigue limit (CAFL) for welded steel details (Connor et al. 2005). Therefore, the applicable stress-range cutoff level was considered within a range from 3.45 MPa to 33% of the CAFL when calculating the ENSC.

Finally, based on the linear fatigue damage model, the cumulative fatigue damage caused by the truck loading during a given time step ΔT can be calculated as follows:

$$\mathrm{CFD}(t) = Num \cdot \frac{\mathrm{ENSC} \times \mathrm{MSR}^3}{A} \tag{6}$$

where Num = number of truck passages during the given time period ΔT.

It should be noted that the dynamic load allowance as specified in the AASHTO LRFD code (AASHTO 2012) was added to the static bending stress range to consider the dynamic vehicle load effect when calculating the CFD.

To illustrate the calculation of the CFD and related parameters, a loading case in which the design fatigue truck passed through the bridge was conducted. The maximum static stress at the most unfavorable position (i.e., the bottom of Girder 4 at the midspan position) when the fatigue truck passes through the bridge is shown in Fig. 4. Based on Fig. 4, the maximum stress range was calculated to be 28.13 MPa. Moreover, because the CAFL for Category B details is 110 MPa according to Table 6.6.1.2.5–3 in the AASHTO LRFD code (AASHTO 2012), the applicable stress-range cutoff level was calculated to be from 3.45 to 36.3 MPa. Based on Eq. (5), the ENSC

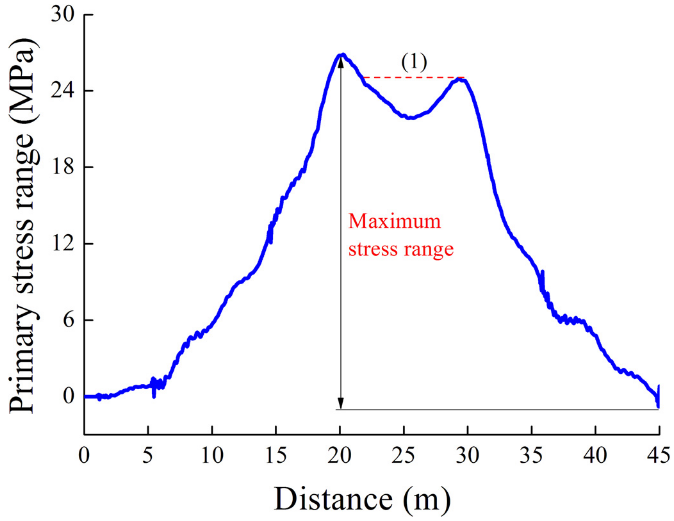

Fig. 4. Static stress history of the design fatigue truck.

Table 2. Calculation of equivalent number of cycles

Cycle number	Cycle order	Stress range S_{ri} (MPa)	S_{ri}/S_{rp}	$(S_{ri}/S_{rp})^3$
Primary	1	28.13	1.000	1.000
(1)	2	3.31	0.118	0.002
ENSC	—	—	—	1.002

was then calculated to be 1.002, as shown in Table 2. Finally, the fatigue caused by each truck passage and the CFD in the first 5 years can be calculated as 1.247×10^{-8} and 0.046, respectively.

The fatigue properties may vary significantly between different fatigue details. Despite some other fatigue-prone details, the condition of the girders generally controls the safety of girder bridges. Therefore, the fatigue details of the girders are usually the main targets in the fatigue design of girder bridges (Wang et al. 2005). Previous studies have shown that for small- to medium-span bridges, the bending moment is generally more critical than the shear force to satisfy the bridge section requirement (Brühwiler and Herwig 2008; González et al. 2011). Therefore, the bending stress of the fatigue details at the girder midspan generally governs the fatigue design of the type of girder bridges under consideration. Thus, in the present study, only the bending stress on the bridge girders was considered.

Because the chance of two overweight trucks being present on the bridge at the same time is very small (Fu and You 2009; Zhang and Cai 2012), as suggested by the AASHTO LRFD code (AASHTO 2012) for fatigue consideration, only one loading case with the truck loading in the slow lane was considered, as shown in Fig. 2. Assuming that the bridge is intact (i.e., free from any fatigue damage), the maximum static bending stresses on the bridge girders under the action of different trucks are shown in Table 3. It should be noted that in each case, the truck was loaded at the most unfavorable longitudinal position.

Vehicle Weight Limit Method

The goal of overweight vehicle management is to maintain the overstress due to the overweight vehicles at an acceptable level to ensure the safety of the bridge structure. The axle-load-based permit-checking procedures, for instance, Bridge Formula B, are simple to implement. However, they are argued to be too conservative for long combination vehicles, as discussed previously. In fact, the vehicle-induced stresses of bridges do not only depend on the axle weights of the vehicle but also on other factors, such as the axle spacing and the width of the vehicle. If an overweight vehicle fails to meet the criteria by the axle-load-based permit-checking procedure, structural analysis may be performed to evaluate the level of overstress caused by the overweight vehicle. Based on the experience from previous studies (Vigh and Kollár 2007; Correia and Branco 2006), in the present study, the stress ratio (SR), which is defined as the ratio of the stress due to the overload to the stress caused by the design load vehicle [as shown in Eq. (7)], was used as the basis for permit checking. If the calculated SR was less than a predetermined threshold value, the vehicle would be granted a permit. Otherwise, the permit request would be denied. The detailed permitting procedure is illustrated in Fig. 5.

Table 3. Maximum static bending stress on the bridge girders caused by different trucks

Truck	GVW (kN)	Maximum static bending stress (MPa) 2D FE bridge model	3D FE bridge model
HS-20 design truck	320	38.27	31.88
Design fatigue truck	320	29.59	26.86
Overweight truck	363	34.01	30.28
	400	37.86	33.38
	445	42.59	37.13
	489	47.24	40.80
	534	51.99	44.56

$$\mathrm{SR} = \frac{S^{\mathrm{OV}}}{S^{\mathrm{DLV}}} \tag{7}$$

where S^{OV} and S^{DLV} = bending stresses caused by the overweight truck and the design load vehicle, respectively.

Vehicle-Induced CFD on the Bridge

In this section, the effect of traffic growth, including the increase in both traffic volume and vehicle weight, on the CFD of bridge structures was investigated. For the purpose of comparison, the design fatigue truck with a weight of 320 kN and five other overweight trucks of 363, 400, 445, 489, and 534 kN, respectively, were studied. Five different ADTTs were considered, namely, 1,000, 1,500, 2,000, 2,500, and 3,000.

Effect of GVW on the CFD

Fig. 6 shows the increase of CFD with time under repeated loading of trucks with different GVWs under an assumed ADTT of 2,000. It can be seen from Fig. 6 that the CFD obtained from both the 2D and the 3D FE bridge models increased sharply with an increase of the GVW, which will lead to significantly reduced bridge fatigue life. For example, it can be seen from Fig. 6 and Table 3 that at the end of 20 years, when the GVW increased from 320 to 363 kN (by 13%), the corresponding stress increased from 26.86 to 30.28 MPa (by 13%). However, the CFD caused by the design fatigue truck increased from 0.192 to 0.275 (by 44%). This clearly shows that the fatigue damage is proportional to the third power of the stress (and therefore that of the GVW), as demonstrated by Eq. (6). This

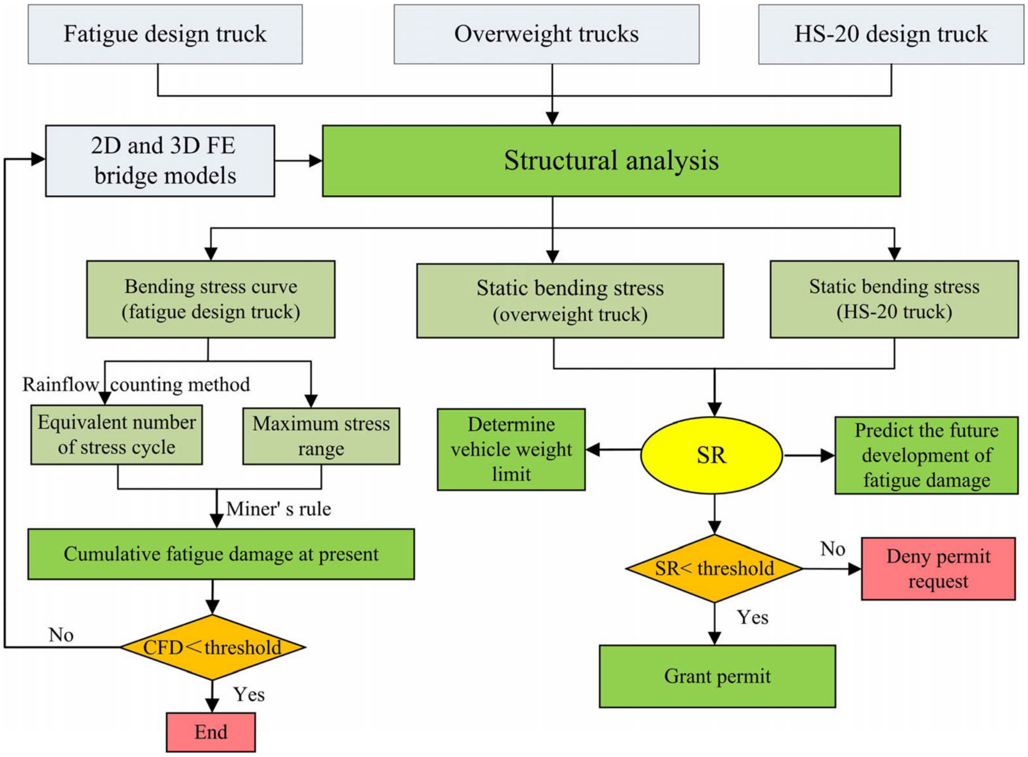

Fig. 5. Flowchart for determining vehicle weight limit and making permit decision.

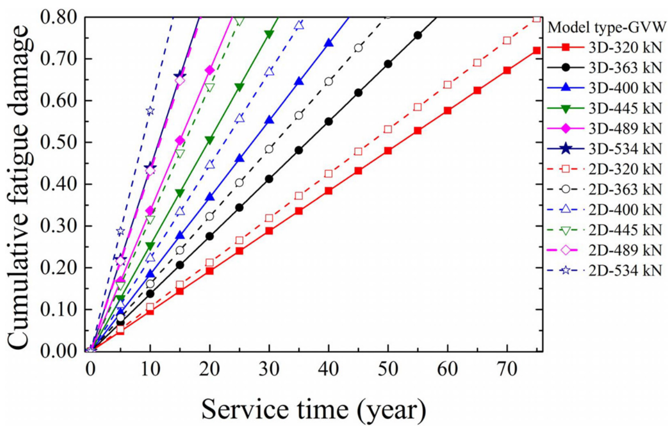

Fig. 6. CFD due to trucks with different GVWs with an assumed ADTT of 2,000.

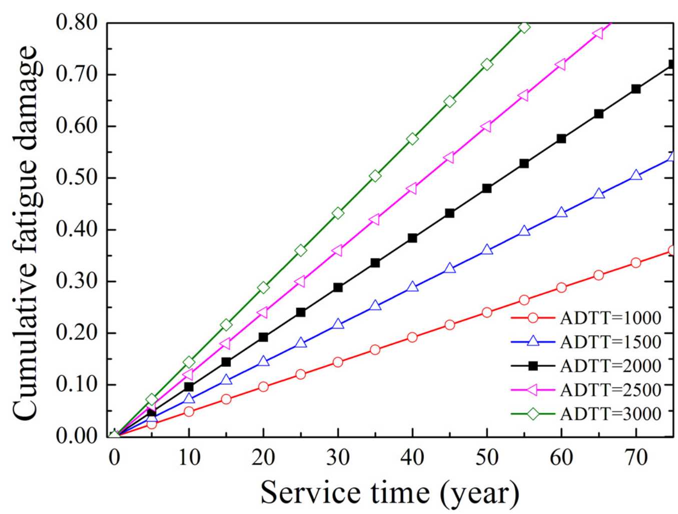

Fig. 7. CFD due to various ADTTs.

highlights the importance of enforcing vehicle weight regulations in ensuring the service life of bridges.

In addition, it was also found that the calculated CFD based on the 2D FE bridge model was always larger than the corresponding CFD obtained based on the 3D FE bridge model, and this difference increased as the GVW increased. For example, with a GVW of 400 kN, at the end of 30 years, the difference in the CFD calculated based on the two bridge models exceeded 0.1, which is significant and could largely influence the decision making. Many studies (Eom and Nowak 2001; Wood et al. 2007; Cha et al. 2016) have proven that combining beamline analysis and the transverse live-load distribution factor in the AASHTO LRFD code (AASHTO 2012) may result in an overestimation of the girder stress. In fact, adopting beamline analysis and the transverse distribution factor is a simplified empirical method in that it does not consider all the influence factors, such as the diaphragms, and may overestimate the stresses of bridge girders (Wood et al. 2007). Therefore, the 3D FE bridge model was adopted in the following analysis.

Effect of ADTT on the CFD

Fig. 7 shows the variation in CFD with time under different ADTT values when the bridge was subjected to the repeated loading of the design fatigue truck of 320 kN. It can be seen from Fig. 7 that the CFD of the bridge increases linearly with an increase of ADTT. This is because a linear fatigue damage model was adopted in the present study.

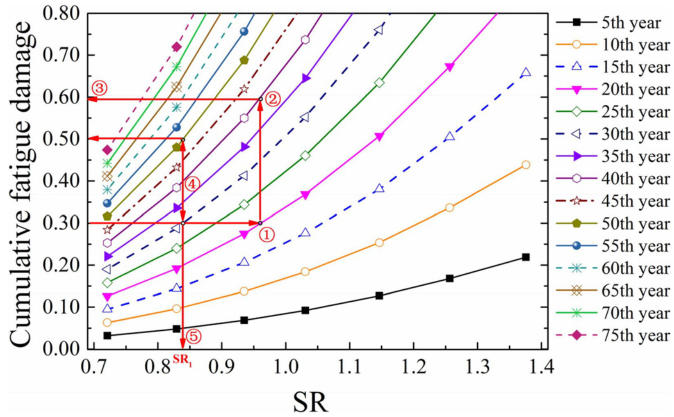

Fig. 8. Development of fatigue damage under different threshold values of SR.

Based on the information in Fig. 7, the bridge CFD under the design ADTT can be predicted during a bridge's whole life cycle, which provides useful information for bridge fatigue design. In addition, the CFD of an in-service bridge can be reasonably estimated based on the available/assumed traffic data in the past, which also provides valuable information for bridge condition assessment and maintenance.

Based on the results in Figs. 6 and 7, it can be found that either the growth of traffic volume or the growth of truck gross weight would result in increased CFD. If the data on traffic volume is available or can be reasonably assumed, the fatigue condition of the bridge at present or in the future can be reasonably predicted according to the results in Figs. 6 and 7.

Vehicle Weight Limit Analysis considering the CFD of Bridges

Selecting a proper threshold value for the SR is critical in determining whether an overweight vehicle should be granted a permit. Once determined, the threshold SR value generally sets the limits on the GVW for different trucks. In general, if the SR value due to an overweight vehicle is less than the threshold value adopted, the vehicle can be granted an automatic permit. For a new bridge, adopting different threshold values of SR means allowing vehicles with different GVWs to cross the bridge, which will cause different levels of fatigue damage after a certain period of time.

For the purpose of illustration, two assumptions were made in the following analysis. First, an ADTT of 2,000 was assumed. Second, it was assumed that bridge girders are replaced when the CFD index of the bridge girders reaches 0.5. A CFD index of 0.5 usually corresponds to a poor condition rating (Cha et al. 2016). It should be noted that in real applications, a proper ADTT value would be determined based on the recorded traffic data in the past or the best estimation. In addition, the decision of bridge girder replacement is never simply determined by the CFD index. Rather, it is influenced by many factors, including the condition rating of the bridge components, the priorities of the DOT, and the availability of funds (IDOT 2010).

Fig. 8 shows the development of bridge fatigue damage with time when different threshold values for the SR are adopted, assuming that the trucks strictly obey the weight limits. It should be noted that the CFD under different SR thresholds was calculated following the procedure illustrated in Fig. 5. From Fig. 8, it can be seen that the selection of the threshold value of SR has a significant effect on the CFD of the bridge structure. For example, for the bridge under consideration, it takes more than 75 years for the CFD on the bridge girders to reach 0.5 if a SR of 0.7 is adopted, whereas it only takes approximately 10 years if a SR of nearly 1.4 is adopted. Therefore, selecting different SR values can make a big difference in the resulting fatigue life of bridges.

It should be pointed out that Bridge Formula B was developed with the aim to control the stresses imposed by passing vehicles to be within a certain range as compared with the design stresses. As such, the fatigue damage due to repeated vehicle loads would not be excessively large (TRB 1990). For example, the formula allows a 5% overstress on the HS-20 bridges, which corresponds to a SR value of 1.05. In addition to the 5% overstress criterion, the formula also sets a cap of 363 kN on the GVW. For the bridge under consideration in the present study, based on the results in Table 3 or Eq. (7), a 363-kN vehicle will cause a SR value of 0.95, which is less than 1.05. This indicates that the cap of 363 kN on the GVW does not necessarily produce a consistent SR value of 1.05 for all bridges and that the cap of 363 kN on the GVW is stricter than the 5% overstress criterion for the bridge under consideration. The implementation of these two different criteria can cause a significant difference in fatigue damage development. Taking the bridge considered in this study as an example, from Fig. 8, it can be seen that the time taken for the CFD of the bridge girders to reach 0.5 is 25 years and 35 years when the SR value is taken as 0.95 (which corresponds to the cap of 363 kN as discussed previously) and 1.05, respectively. This indicates that the threshold value of SR adopted in practice has a significant effect on the fatigue life of bridge girders and may have a considerable influence on the life-cycle cost of bridges.

Based on the relationships between the CFD and the influencing factors, such as the GVW, ADTT, and SR, a method for determining vehicle weight limit was developed considering the CFD of bridges. This method works for both new bridges and existing bridges.

For a new bridge, if the expected service life of the key bridge components (e.g., girders) is predetermined, the threshold value of the SR can then be obtained based on the relationships between the CFD and the two parameters, namely, the SR and the service time, as shown in Fig. 8. The truck weight limit can then be obtained based on the threshold value of SR by back-calculation using Eq. (7). Taking the relationships shown in Fig. 8, for example, assuming the expected service life of the bridge girders is set at 25 years, from Fig. 8, it can be found that when a SR value of 1.05 is selected, it takes 25 years for the CFD of the bridge girders to reach 0.5. Therefore, a SR value of 1.05 should be taken as the threshold value in this case. The design stress of the bridge girders is calculated to be 31.88 MPa (as listed in Table 3). Therefore, the allowable stress due to an overweight truck is 1.05 times 31.88 MPa, which gives 33.47 MPa, which corresponds to a GVW of approximately 400 kN, as shown in Table 3. It is noted that in this case, the overweight truck was assumed to have the same geometric size as the design fatigue truck. When implementing the weight regulations in practice, vehicles with weights under 400 kN would be granted an automatic pass. If a truck has a weight over 400 kN, structural analysis would be performed to assist the decision making. If the truck causes a stress larger than 33.47 MPa on the bridge girders, then the request of the truck would be rejected. Otherwise, the truck can be issued a permit.

In addition, Fig. 8 can also be used to guide the determination of vehicle weight limits for existing bridges. The procedure is illustrated in the following steps. For the purpose of illustration, a steel–concrete composite girder bridge is again used as an example. It is assumed that this bridge has been in service for T_0 (= 20) years, and the CFD on the bridge girder is estimated at CFD_0 (= 0.3). The expected service life is T_{exp} (= 40) years for the bridge girder when the CFD reaches the threshold value of 0.5. For the purpose of convenience, each step is also marked with a number in Fig. 8.

Step 1: Find the intersection (numbered as Point ①) of the curve for the T_0th year and the horizontal line with CFD = 0.3. Because the CFD curves are drawn for every 5 years, the interpolation method can be used to find the CFD curve for the 20th year.

Step 2: Draw a vertical line from Point ① obtained in Step 1. Find the intersection point (numbered as Point ②) of this vertical line and the CFD curve for the T_{exp}th (= 40th) year.

Step 3: Find out the CFD value (denoted as CFD_{exp}) at Point ② and compare it with the threshold value of 0.5 (Point ③). If the CFD_{exp} is greater than the threshold value of 0.5, a more restrictive vehicle weight limit should be enforced to achieve the expected service life. Otherwise, a less restrictive vehicle weight limit may be adopted. In this example, CFD_{exp} is equal to 0.59 (>0.5). Therefore, a more restrictive vehicle weight limit should be enforced.

Step 4: Draw a vertical line that intersects with the CFD curves for the T_{exp}th (= 40) year and the T_0th (= 20) year, calculate the vertical distance between the two lines, and compare it with the margin (0.20 = 0.50–0.30) between the threshold CFD (0.50) and the current CFD (0.30). Move the vertical line left or right until this vertical distance is equal to the margin (0.20), as shown by the number ④ in Fig. 8.

Step 5: Find the intersection of this vertical line (at its final position) with the horizontal axis at Point ⑤. The SR value at Point ⑤ is the threshold SR value to be adopted for determining the vehicle weight limit. In this case, a value of 0.84 should be adopted as the threshold for weight limit purposes in the future.

Summary and Conclusions

A method for determining vehicle weight limit was proposed considering the cumulative fatigue damage of bridges due to repeated vehicle loading. The proposed method can be applied to determine the vehicle weight limit and assist overload permit checking for both new bridges and existing bridges so that the bridges (components) achieve the desired service life.

A typical steel–concrete composite girder bridge was used as an example for illustrating the proposed method. Parametric studies were also performed to investigate the effect of a few important parameters, including GVW and ADTT, on the CFD. Based on the results of this study, the following conclusions can be drawn:

1. Both the ADTT and the GVW have a significant influence on the CFD of bridges. If the ADTT data in the past are available or can be reasonably estimated, the CFD for existing bridges can be reasonably estimated; the future development of fatigue damage and the remaining fatigue life of bridges can also be predicted.
2. Based on the bridge considered in this study, it was found that the overstress criterion of 1.05 adopted in Bridge Formula B does not necessarily correspond to a GVW limit of 363 kN for passing vehicles. It was found that the implementation of these two different criteria can lead to a significant difference in the resulting fatigue life of the bridge girders and may cause a considerable difference in the life-cycle cost of bridges.
3. The development of fatigue damage with time under different threshold values of SR was obtained. This information can be used to determine the weight limit for both new and existing bridges. It can also be used to predict the future development of fatigue damage with assumed traffic information in the future.

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