Truck Weight Limit for Simply Supported Steel Girder Bridges Based on Bridge Fatigue Reliability

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Abstract: The objective of this study was to present a new approach for determining the truck weight limit of simply supported steel girder bridges. This approach is based on the fatigue reliability of bridge girders under the action of random traffic flows considering the presence of multiple trucks. A simply supported steel girder bridge designed according to the current codes was used as the bridge model to illustrate the presented approach. Based on the collected traffic data from Wisconsin, random traffic flows were generated using the Monte Carlo method and were used as the traffic loading. Numerical simulations were carried out to investigate the effects of three important parameters, including the fraction of traffic in the fast lane ($FTFL$), violation rate (VR), and truck weight limit (TWL), on the average fatigue damage accumulation ($AFDA$) induced by each truck in the random traffic flow. Based on the Miner's cumulative damage model and the S-N curve, the effects of these parameters on the fatigue reliability index of bridge girders were also analyzed. The truck weight limit that can ensure the target fatigue reliability of the bridge girders after 75 years of service was determined based on the fatigue reliability analysis of the bridge girder.
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Author keywords: Truck weight limit; Violation rate; Fatigue damage accumulation; Monte Carlo method; Fatigue reliability.

Introduction

Setting regulations on truck weights and sizes is a challenge because of the conflicting interests of different groups (Tabsh and Tabatabai 2001). Shippers and carriers desire fewer strict regulations on truck weights and sizes to reduce their operation costs, while owners and government agencies may give the safety and sustainability of infrastructures a higher priority. Therefore, a proper truck weight limit regulation should be able to achieve a good balance among many factors, such as the truck productivity, bridge safety, transportation safety and efficiency, economy, environment, etc. (Luskin and Walton 2001; Moshiri and Montufar 2016).

Plenty of research effort has been devoted to achieve this goal. Some researchers intended to find the optimal routings for overweight and oversized vehicles to cross bridges (Chou et al. 1999; Correia and Branco 2006; Fu and Hag-Elsafi 2000; Vigh and Kollár 2006; Vigh and Kollár 2007). Other researchers attempted to propose a proper truck weight limit and to explore its effect on the maximum live load and the fatigue life of bridges (Asantey and Bartlett 2005; Cohen et al. 2003; Ghosn 2000; Ghosn and Moses 2000; Han et al. 2018; James et al. 1986; Moshiri and Montufar 2016). Fu et al. (2008) reviewed the previous studies on the cost impacts from the increase of the truck weight limit and proposed a new approach to estimating the costs of truck weight limit changes for bridges in a highway infrastructure system. However, none of these studies on truck weight limits were based on the fatigue safety of bridge girders from the reliability point of view.

The reliability method has been widely used to investigate the fatigue behavior of steel bridges. Some researchers investigated the effect of increasing traffic loads on the fatigue reliability of welded bridge details (Liu et al. 2017; Lu et al. 2017; Righiniotis 2006) while other researchers conducted fatigue reliability assessment of steel bridges based on health-monitoring data (D'Angelo and Nussbaumer 2015; Guo and Chen 2013; Guo et al. 2012; Kwon and Frangopol 2010; Liu et al. 2010; Ni et al. 2010; Soliman et al. 2013). Fatigue reliability analysis of long-span bridges under combined dynamic loads from vehicles and wind was also carried out by some researchers (Chen et al. 2012; Wu 2012; Zhang et al. 2013). In addition, some researchers proposed different models to evaluate the fatigue reliability of existing steel bridges by using updated monitoring data (Leander et al. 2015; Wang et al. 2012; Zhao et al. 1994). However, few of these studies were focused on the effect of the truck weight limit on the fatigue reliability of steel bridges.

The objective of this study was to present a new approach for determining a proper truck weight limit for simply supported steel girder bridges. This approach is based on the fatigue reliability of bridge girders under the action of the random traffic flows with consideration of multiple truck presence. A simply supported steel girder bridge designed according to the AASHTO LRFD (AASHTO 2012) was used as the bridge model to illustrate the presented approach. Based on the collected traffic data from Wisconsin, random traffic flows considering the presence of multiple trucks were generated with the Monte Carlo method and were used as the vehicle loading. Numerical simulations were carried out to investigate the effects of three parameters, including the fraction of traffic in the fast lane ($FTFL$), violation rate (VR), and truck

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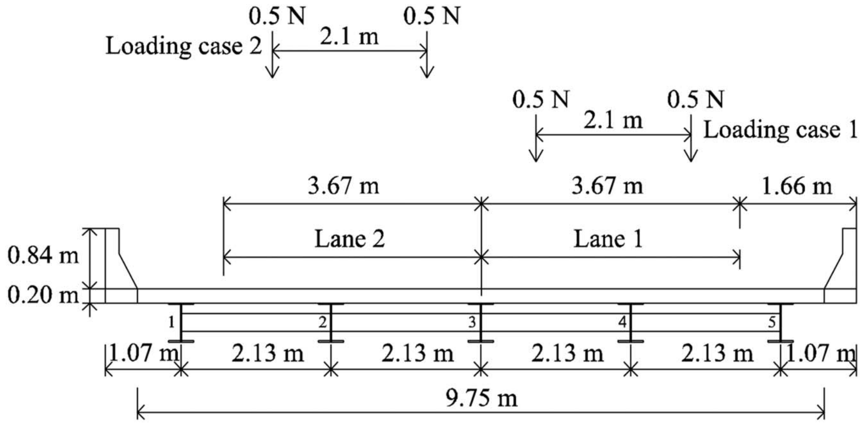

Fig. 1. Cross section and loading cases adopted in the present study.

weight limit (TWL), on the average fatigue damage accumulation induced by each truck from the randomly generated traffic flows. The fatigue reliability index of the bridge girders was obtained based on the limit state function deduced with the Miner's cumulative damage model and the S-N curve. The relationship between the fatigue reliability index of the steel bridge girder and the number of the average daily truck traffic ($ADTT$) was obtained under different $FTFL$, VR, and TWL values. A procedure for determining a proper truck weight limit that can ensure the target fatigue reliability of the bridge girders after 75 years of service was proposed.

Analytical Bridge Model

In the present study, a typical simply supported steel girder bridge designed according to the AASHTO LRFD (AASHTO 2012) code was used as the bridge model for illustration. This bridge has five identical I-girders spaced at 2.13 m (7 ft). The span length, roadway width, and deck thickness of the bridge are 36.60 m (120 ft), 9.75 m (32 ft), and 0.20 m (8 in.), respectively. The cross section of the bridge is illustrated in Fig. 1. Besides the end diaphragms, the bridge also has four intermediate diaphragms that connect the adjacent bridge girders. Each steel I-girder has a height of 1.21 m, a cross-sectional area of 0.028 m^2, and a moment of inertia of 0.0064 m^4. In the present study, the steel I-girder bridge was modeled using the ANSYS 14.5 program as shown in Fig. 2.

Stress Influence Line of the Point of Interest

In real traffic situations, the proportion of traffic present in the slow lane (Lane 1 in Fig. 1) is usually much larger than that in the fast lane (Lane 2 in Fig. 1). Therefore, the traffic in the slow lane is the main contributor to the fatigue damage accumulation in steel bridges. It was also found that under a moving unit load applied in the slow lane (achieved by applying a pair of 0.5-N point loads side by side, as shown in Fig. 1), the maximum static stress (130.96 Pa) occurred at the midspan of Girder 4. Therefore, the stress of Girder 4 is used for fatigue analysis hereafter. The effect of the transverse location of loading on the maximum static stress was investigated and it was found that the maximum static stress occurs where the center of the applied point loads is around the center line of Line 1. Therefore, it is safe to use the stress influence line when the point loads are applied at the center of the lane. The fatigue detail considered in this research is the connection between the bottom flange and the web at the midspan of Girder 4 of the bridge under consideration. This detail belongs to Category B for welded joints as specified in the AASHTO LRFD (AASHTO 2012) code.

The stress influence line of the point of interest was obtained by moving the unit load across the bridge step-by-step, at 0.1 m each step. The influences lines for both loading Case 1 and loading Case 2 in Fig. 1 were obtained. Three different wheel gauges shown in Fig. 3, including 1.8, 2.1, and 2.4 m, were considered. From Fig. 3, it can be observed that the effect of the wheel gauge on the stress influence line of the point of interest is insignificant under the two loading cases considered. Therefore, the average value of the three stress influence lines under the three different wheel gauges was adopted in this analysis.

Traffic Data

The traffic data collected from the weigh-in-motion sites in Wisconsin was used as reference to generate the random traffic flow (Haider and Harichandran 2007a). Haider and Harichandran (2007b) analyzed the traffic data and found that the bimodal shape of the gross vehicle weight of each truck class, classified by the Federal Highway Administration, could be effectively described through a mixture of two normal distributions, as also found by

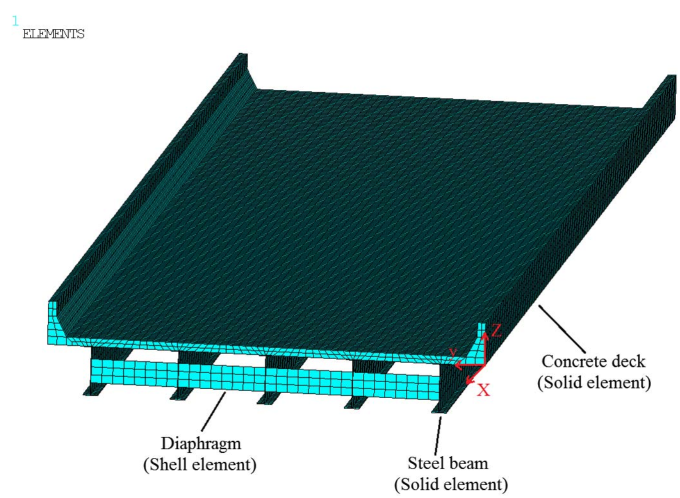

Fig. 2. Finite element model of the adopted bridge.

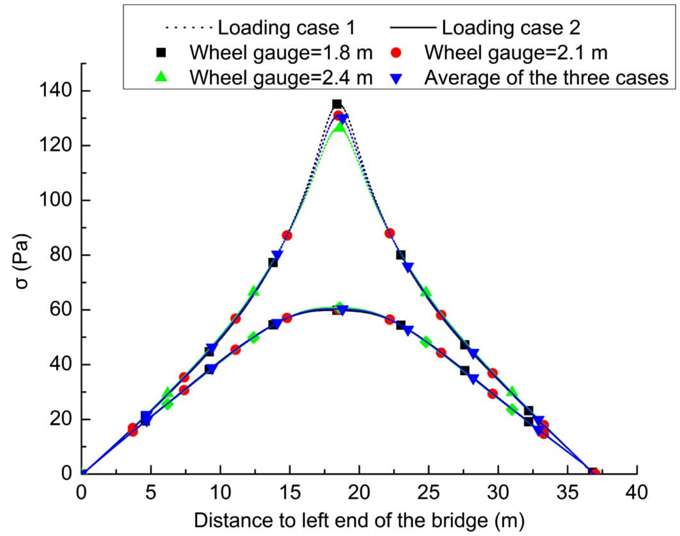

Fig. 3. Stress influence lines of the point of interest under the loading cases considered.

other scholars (Al-Yagout et al. 2002; Prozzi and Hong 2007). They obtained the distribution properties of the gross vehicle weight and the proportion of each truck class, as summarized in Table 1 (Haider and Harichandran 2007a). In Table 1, μ_i, σ_i, and p_i ($i = 1, 2$) represent the mean, standard deviation, and proportion of each normal distribution, respectively, while P represents the proportion of trucks in each truck class. It should be noted that since the gross vehicle weight distribution parameters for Class 4 trucks and Class 12 trucks were not provided in the study of Haider and Harichandran (2007a), in Table 1 the Class 4 trucks (7.31%) have been merged into the Class 5 trucks, and the Class 12 trucks (0.17%) have been merged into the Class 13 trucks. This mergence may result in a slightly conservative estimation of the fatigue damage accumulation as a result of the following two facts: (1) the gross vehicle weights of the Class 5 and Class 13 trucks are commonly larger than those of the Class 4 and Class 12 trucks, respectively; and (2) the proportions of the Class 4 trucks and Class 12 trucks are relatively small. Fig. 4 shows the axle load split ratio and the mean axle spacing of each truck considered in the present study (FHWA 2001; Kwigizile et al. 2004).

Violation Rate

The efficiency of vehicle weight regulation enforcement can be reflected by the violation rate, which is defined as the proportion of the number of trucks with weights surpassing a truck weight limit that cross the bridge to the total number of trucks with weight surpassing the same weight limit (Asantey and Bartlett 2005). For example, $VR = 0\%$ means a "perfect" compliance condition. Similarly, $VR = 30\%$ means that among all the trucks with weight over the posted weight limit, 30% of them violate the rule and pass the bridge. In reality, compliance to a posted weight limit is unlikely to be perfect. Violation rates larger than 5% are very common (Wyatt and Hassan 1985). Large violation rates may lead to excessive loading and a reduction of bridge reliability. Therefore, for a given load limit it is necessary to investigate the effect of the violation rate on the bridge reliability.

Random Traffic Flow and Stress History of the Point of Interest

Based on the distribution properties of the gross vehicle weight (GVW) in Table 1 and the truck configuration in Fig. 4, the Monte Carlo method (MCM) was used to generate the random traffic flow. The headway distance of trucks was assumed to follow a uniform distribution with the minimum headway distance larger than 5 m (15 ft), as did by Nowak and Hong (1991). The *rand* function in Matlab was used to determine whether a truck with weight exceeding the TWL will cross the bridge or not. For example, the condition with rand $(1,1) \leq VR$ denotes that the truck will cross the bridge and vice versa. After applying the loads of the random traffic flow to the stress influence lines of the point of interest in Fig. 3, the stress history under the action of the random traffic flow can be obtained. The flowchart for generating the random traffic flow and obtaining the stress time history of the point of interest is illustrated in Fig. 5. The procedure for considering multiple truck presence in Fig. 5 is introduced in more details as follows. First, two random traffic flows that have the same length and the same starting point were obtained for the fast lane and slow lane, respectively. Then, the two traffic flows were applied to the corresponding influence lines step-by-step with a step length of 0.1 m, and two stress time histories of the point of interest were obtained. Finally, the two stress time histories were added up at each load step correspondingly to obtain the total stress history of the point of interest. An impact factor of 0.15, as specified in the AASHTO LRFD (AASHTO 2012) code, was adopted to consider the vehicle dynamic effect when obtaining the stress time history of the point of interest. In Fig. 5, N_{T1} and N_{T2} are the total number of trucks that travel along the slow lane and fast lane, respectively, and were determined based on the total number of trucks crossing the bridge and the fraction of traffic (FT) in each lane. It should be noted that the distribution pattern of the headway distance has a significant influence on the random traffic flow as well as the presence of multiple trucks. Therefore, it is suggested that a real distribution pattern of the headway distance be adopted when using the

Table 1. Parameters of the mixture distribution of the gross vehicle weight (GVW) in Wisconsin

Vehicle classification	μ_1 (kN)	μ_2 (kN)	σ_1 (kN)	σ_2 (kN)	p_1	p_2	P (%)
5	72.81	106.91	18.91	31.27	0.37	0.63	28.88
6	126.39	217.75	30.39	26.66	0.83	0.17	8.48
7	95.01	259.42	22.55	60.96	0.02	0.98	2.15
8	134.28	185.34	22.49	40.5	0.49	0.51	9.63
9	163.61	304.28	30.89	65.69	0.21	0.79	45.08
10	170.28	343.43	18.91	82.29	0.04	0.96	0.80
11	227.62	287.53	77.76	4.54	0.92	0.08	0.63
13	538.42	804.55	141.21	148.68	0.53	0.47	4.34

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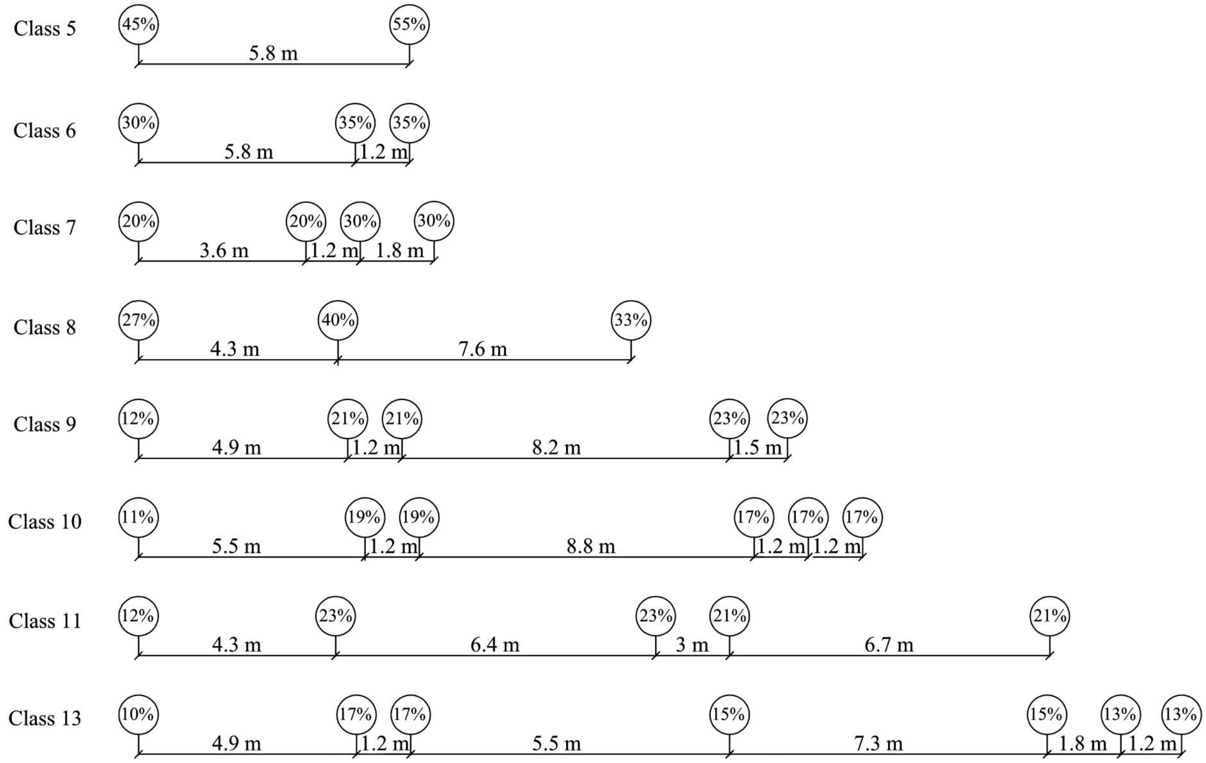

Fig. 4. Axle load split ratio and mean axle spacing of each truck class considered in the present study.

proposed method for determining vehicle weight limits if such information is available.

Fatigue Evaluation of the Bridge Considered

The fatigue damage (FD) of bridge components will accumulate under the action of continuous traffic flows. In theory, a fatigue detail would have finite fatigue life if the maximum stress range surpasses the constant-amplitude fatigue limit (CAFL). In fact, previous studies have shown that as long as the frequency of the stress range surpassing the CAFL is higher than the limit of 0.01%, the fatigue life of the detail is no longer infinite (Hodgson et al. 2006). Fig. 6 shows one case of the stress range experienced by the point of interest under the action of the random traffic flow with an ADTT of 1,000. For this case, the frequency of the stress range surpassing the CAFL was calculated to be 0.28%, which is significantly higher than the limit of 0.01%, indicating that the fatigue live of the bridge component is not infinite. In the present study, every stress range larger than the cutoff value was taken into account when calculating the fatigue damage accumulation under the action of the generated random traffic flow.

Miner's rule (1945) has been used widely to estimate the fatigue damage accumulation (FDA) as follows:

$$FDA = \sum_i \frac{n_i}{N_i} \tag{1}$$

where N_i = number of stress cycles to fatigue failure under a stress range level of S_i; and n_i = actual number of stress ranges experienced. Miner's rule (1945) assumes that the FDA is linear and ignores the influence of sequence of stress application, which has been demonstrated to have an insignificant effect on the accuracy of fatigue evaluation (Albrecht and Friedland 1979; Schilling et al. 1978). According to the AASHTO LRFD (AASHTO 2012) code, N_i and S_i hold the following relationship:

$$N_i = \frac{A}{S_i^m} \tag{2}$$

where A = fatigue-strength coefficient and was found to follow a lognormal distribution with a mean value of 2.57×10^{13} MPa^3 (7.85×10^{10} ksi^3) and a coefficient of variation of 0.35 for Category B; and m = slope constant that is usually taken as 3 for all AASHTO fatigue category details (Keating and Fisher 1986). After substituting Eq. (2) into Eq. (1), the following equation can be obtained:

$$FDA = \sum_i \frac{n_i S_i^3}{A} \tag{3}$$

According to Schilling (1984), the cumulative fatigue damage resulting from a series of complex stress cycles can be calculated with the maximum stress range (MSR) and the equivalent number of stress cycles ($ENSC$) determined using the following equation:

$$ENSC = num + \left(\frac{S_{r1}}{MSR}\right)^m + \left(\frac{S_{r2}}{MSR}\right)^m + \cdots + \left(\frac{S_{ri}}{MSR}\right)^m + \cdots + \left(\frac{S_{rcut}}{MSR}\right)^m \tag{4}$$

where num = number of the maximum stress range; m = slope constant of the S-N curve; S_{ri} ($i = 1 \cdots cut$) = higher-order stress ranges; and S_{rcut} = cutoff stress range, which was selected as 3.45 MPa (0.5 ksi) in this analysis (Connor et al. 2004). The slope constant m was taken as 3. In the present study, the rain flow counting algorithm was adopted to calculate the number of stress ranges from the stress time history (Schilling 1982).

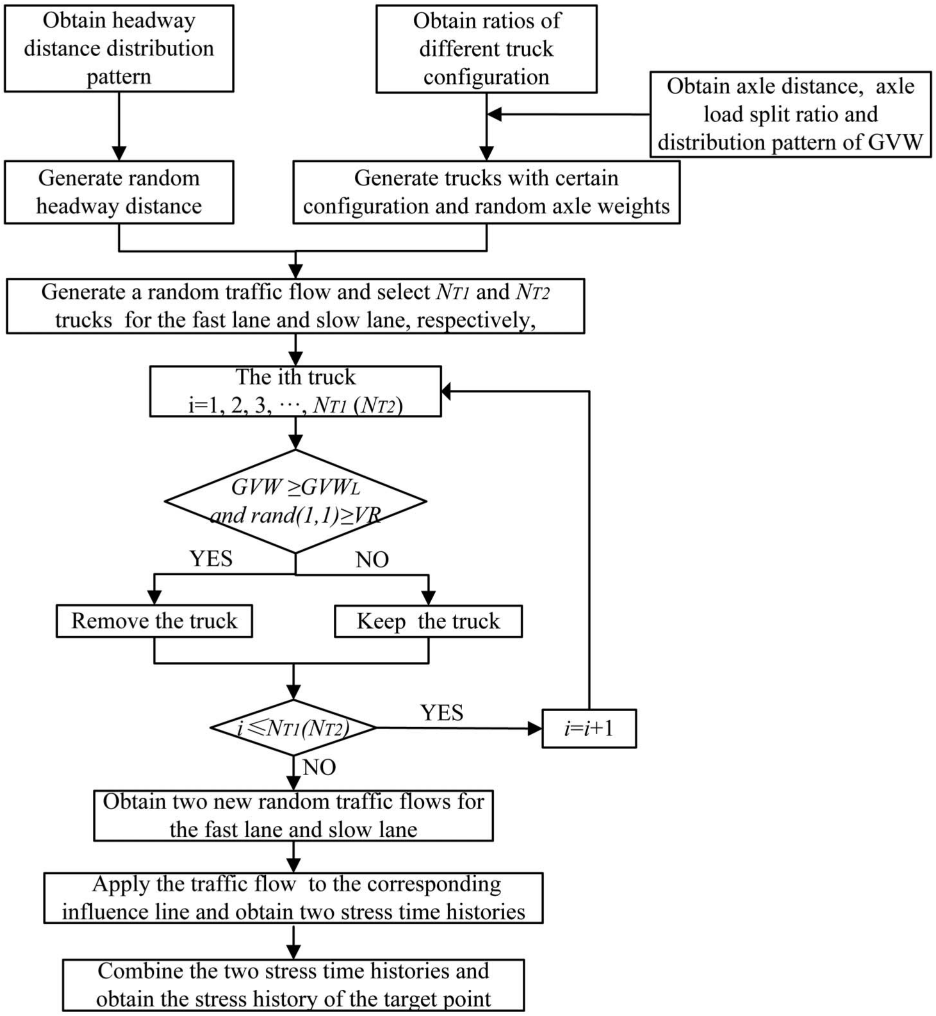

Fig. 5. Flowchart for generating the random traffic flow and obtaining the stress time history of the point of interest.

Assume N_T ($N_T = N_{T1} + N_{T2}$) is the number of trucks selected from the random traffic flow, and MSR_T and $ENSC_T$ are the maximum stress range and equivalent number of stress range experienced by the point of interest under the action of the N_T trucks selected, respectively. Based on Eq. (3), the fatigue damage (FD) caused by the N_T trucks is defined as follows:

$$FD_{N_T} = ENSC_T \cdot MSR_T^3 \tag{5}$$

The average fatigue damage accumulation ($AFDA$) caused by each of the N_T trucks can be calculated as:

$$AFDA = \frac{FD_{N_T}}{N_T} = \frac{\mathrm{ENSC}_T \cdot MSR_T^3}{N_T} \tag{6}$$

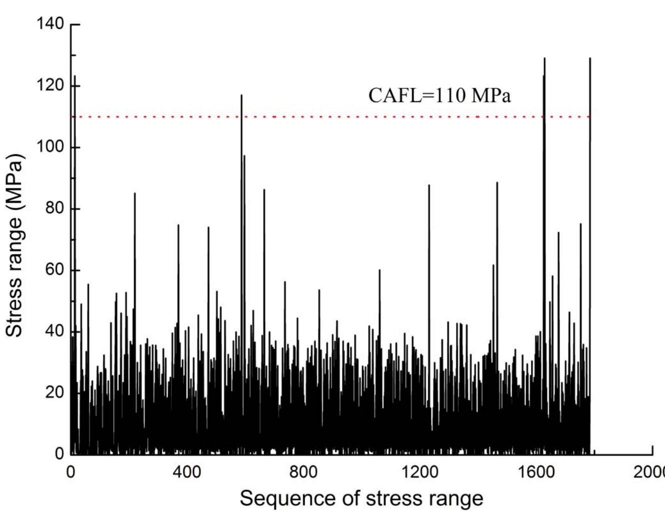

Fig. 6. One case of the stress range experienced by the point of interest under the action of the random traffic flow with an $ADTT$ of 1,000.

Sensitivity Analysis

To investigate the effect of N_T and the number of Monte Carlo simulations (N_{sim}) on the $AFDA$ and to find a balance between the accuracy of the $AFDA$ and the computational cost, sensitivity analysis was conducted to obtain the $AFDA$s under different N_T and N_{sim}, and the results are shown in Fig. 7. It should be noted that the $AFDA$s in Fig. 7 were obtained under the action of the random traffic flow with a $FTFL$ of 0.15. From Fig. 7, it can be observed that the variation of the $AFDA$s becomes insignificant when $N_{sim} = 10{,}000$ and $N_T \geq 100$. Therefore, $N_{sim} = 10{,}000$ and $N_T = 100$ were adopted to calculate the $AFDA$ based on Eq. (6) hereafter. In fact, $N_{sim} = 10{,}000$ has also been adopted by other researchers (Miranda and Deodatis 2012; Naess and Gaidai 2008).

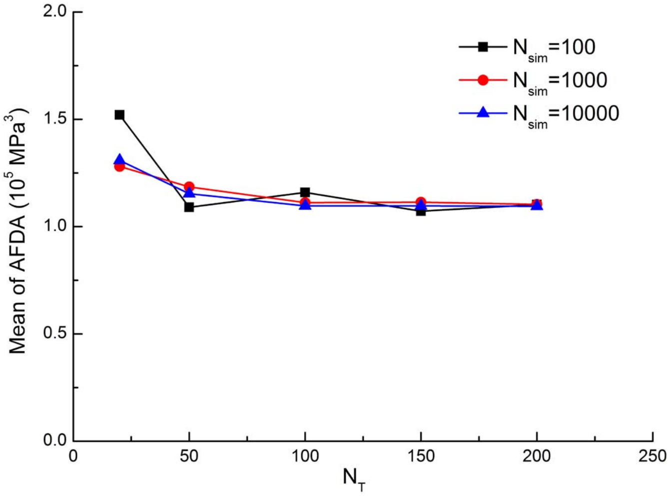

Fig. 7. $AFDA$s under different N_T and N_{sim}.

Numerical Simulations

In the present study, the fatigue reliability of the bridge girder considered was first investigated under the action of the generated random traffic flow, which intends to represent the real traffic. Four different $FTFL$s were considered, namely, 0.10, 0.15, 0.20, and 0.25. Next, the effect of VR on the efficiency of enforcement of truck weight regulations was analyzed with a $FTFL$ of 0.15 and a TWL of 55 ton. Then, the effect of TWL on the fatigue reliability of the bridge girder was studied with a $FTFL$ of 0.15 and a VR of 0.30. Five $TWLs$ ranging from 40 to 60 tons with an interval of 5 tons were investigated. Finally, under the assumed $FTFL$, VR, and $ADTT$, the TWL that can ensure the target fatigue reliability for the bridge girder after 75 years of service was determined.

Distribution Pattern of the AFDA

To determine the distribution type that best fits the $AFDA$ data, the chi-square test was used in the present study. The chi-square test determines the type of distribution of a set of data by comparing the test value against a threshold value. The threshold value can be determined based on the significance level assumed for the test and the number of intervals used for the histogram. The chi-square test value can be estimated as follows (Deng and Cai 2010):

$$\chi^2 = \sum_{j=1}^{k} \frac{(O_j - E_j)^2}{E_j} \tag{7}$$

where k = total number of intervals of histogram; and O_j and E_j = actual number of data and the theoretical number of data in the jth interval, respectively. In the present study, the number of intervals of the histogram was set to be 10, and the significance level was adopted as 0.99, leading to a threshold value of 23.21.

To obtain the statistical properties of the $AFDA$, under each condition considered, one hundred $AFDA$s were obtained and tested against the normal and lognormal distributions, both of which are widely used in the engineering field. The test results were summarized in Table 2. From Table 2 it can be observed that the chi-square test values for the two distributions tested are close to each other and are all far below the threshold value of 23.21 under all the conditions considered, indicating that both the lognormal and normal distributions are suitable for describing the $AFDA$s. In the present study, the lognormal distribution was adopted when calculating the reliability index. The statistical properties of the $AFDA$s under the conditions considered are summarized in Table 3.

Table 2. Chi-square test results on the distribution of the $AFDA$ under different $FTFL$, TWL, and VR values

Parameter	Lognormal	Normal
	FTFL	
0.10	3.76	3.50
0.15	3.89	5.24
0.20	4.31	3.59
0.25	1.91	1.63
	TWL	
40	3.05	2.88
45	2.99	2.93
45.5	11.33	10.93
46	2.09	1.92
47	2.24	2.25
50	3.91	3.89
55	2.57	2.51
60	4.35	4.89
	VR	
0	1.79	1.93
0.10	4.82	4.74
0.20	10.52	10.53
0.30	2.57	2.51
0.40	6.32	5.81
1	3.89	5.24

Table 3. Statistical properties of the $AFDA$ under different $FTFL$, TWL, and VR values

Parameter	Mean (MPa^3)	COV
	FTFL	
0.10	1.12×10^5	0.0047
0.15	1.10×10^5	0.0043
0.20	1.08×10^5	0.0041
0.25	1.06×10^5	0.0046
	TWL	
40	6.47×10^4	0.0042
45	6.82×10^4	0.0040
45.5	6.84×10^4	0.0043
46	6.86×10^4	0.0041
47	6.89×10^4	0.0043
50	6.96×10^4	0.0045
55	7.02×10^4	0.0040
60	7.10×10^4	0.0033
	VR	
0	5.26×10^4	0.0014
0.10	5.85×10^4	0.0037
0.20	6.44×10^4	0.0040
0.30	7.02×10^4	0.0040
0.40	7.60×10^4	0.0037
1	1.10×10^5	0.0043

Limit State Function

In the present study, the limit state function (LSF) is defined as follows:

$$g = \Delta - FDA \tag{8}$$

where Δ = critical damage accumulation index and was found to follow a lognormal distribution with a mean value of 1.0 and a coefficient of variation of 0.3 (Chung 2004). Based on Eqs. (3), (5), and (6), during the expected 75-year service life of the bridge, the FDA can be estimated as:

$$FDA = \sum_i \frac{n_i S_i^3}{A} = \frac{75 \cdot 365 \cdot ADTT}{N_T} \cdot \frac{ENSC_T \cdot MSR_T^3}{A} = \frac{75 \cdot 365 \cdot ADTT \cdot AFDA}{A} \tag{9}$$

Substituting Eq. (9) into Eq. (8), the following equation can be obtained:

$$g = \Delta - \frac{75 \cdot 365 \cdot ADTT \cdot AFDA}{A} \tag{10}$$

In Eq. (10), the random variables (i.e., Δ, A, $AFDA$) were assumed to follow lognormal distributions based on the results from previous studies (Chung 2004; Zhang and Cai 2012) and the $ADTT$ was treated as a constant. However, the effect of $ADTT$ on the fatigue reliability was investigated through a parametric study in which different $ADTT$ values were adopted. The fatigue reliability index of the bridge girder, β, was deduced as follows:

$$\beta = \frac{\kappa_\Delta + \kappa_A - (\kappa_{AFDA} + \ln(75 \cdot 365 \cdot ADTT))}{\sqrt{\xi_\Delta^2 + \xi_A^2 + \xi_{AFDA}^2}} \tag{11}$$

where the parameters κ_y and ξ_y ($y = \Delta$, A, $AFDA$) are given as follows:

$$\xi_y = \sqrt{\ln(1+\delta_y^2)}$$
$$\kappa_y = \ln(\mu_y) - \xi_y^2 \tag{12}$$

where μ_y and δ_y = mean value and the coefficient of variation of y, respectively. In the present study, the target reliability index was adopted to be 2.5, as specified in the AASHTO LRFD (AASHTO 2012) code. Based on the statistical properties of y discussed previously and in Eqs. (11) and (12), the effects of $FTFL$, TWL, and VR on the fatigue reliability of the bridge girder considered were studied and discussed in the following part, respectively. Besides, the TWL that can ensure the target reliability index after 75 years of service for the bridge girder considered is determined under the assumed $FTFL$, VR, and $ADTT$ values.

Effect of the FTFL

The effect of the $FTFL$ on the fatigue reliability of the bridge girder considered is shown in Fig. 8. It can be observed from Fig. 8 that the fatigue reliability index of the bridge girder increases slightly with the increase of the $FTFL$, which may be from the combined effect of the following two aspects. On one hand, with the increase of the $FTFL$, the fraction of traffic in the slow lane will decrease. A truck traveling in the slow lane will cause much larger stress than the same truck traveling in the fast lane (as shown by the influence lines in Fig. 3); therefore, with the total number of trucks being the same, a smaller fraction of traffic in the slow lane will lead to a smaller number of large stress ranges. On the other hand, the increase of $FTFL$ also means a larger probability of multiple presence of trucks on the bridge, leading to a higher probability of experiencing larger stress ranges caused by the multiple trucks on the bridge. It can also be observed from Fig. 8 that the $ADTT$ required for the reliability index of the bridge girder to decrease to the target reliability index after at the end of 75 years' service life is around 2,500 under the $FTFL$s considered.

Effect of the VR

Fig. 9 shows the effect of the VR on the fatigue reliability of the bridge girder under consideration in which $FTFL = 0.15$ and $TWL = 55$ ton were adopted. In Fig. 9, the only difference in the traffic composition between the two cases with $VR = 0$ and $VR > 1$ is that trucks with weight exceeding 55 tons were still in the traffic flow depending on the value of VR while those trucks were all removed from the traffic flow when $VR = 0$. From Table 1, it can be observed that the proportion of heavy vehicles is small while the TWL was adopted to be 55 tons. However, even under this condition, the results in Fig. 9 still shows that VR has a considerable effect on the bridge fatigue reliability. Specifically, the $ADTT$ needed for the reliability index of the bridge girder to decrease to the target reliability index after 75 years of service decreases from 5,320 to 2,540 when the VR increases from 0.0

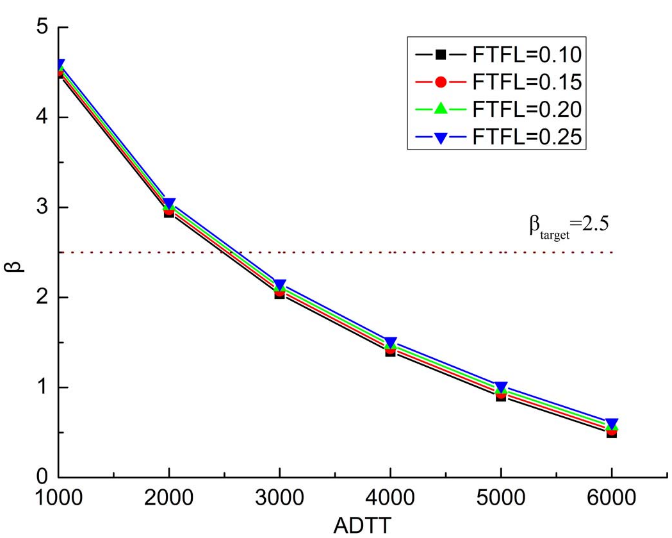

Fig. 8. Variation of fatigue reliability index with change in $ADTT$ and $FTFL$ for the bridge under the loading case considered.

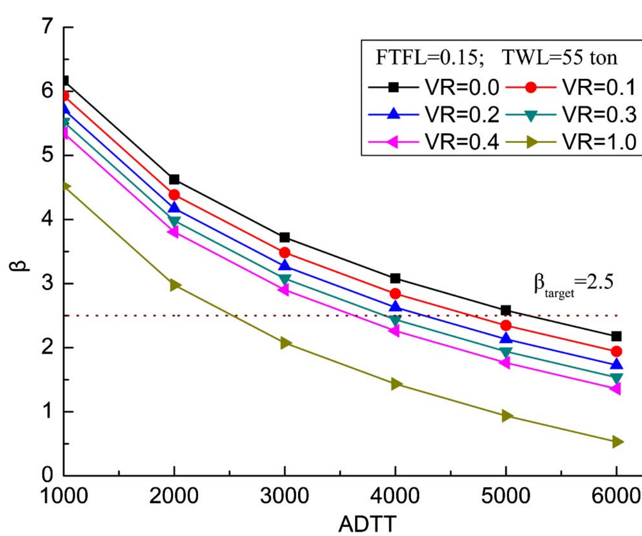

Fig. 9. Variation of fatigue reliability index with change in $ADTT$ and VR for the bridge under the loading case considered.

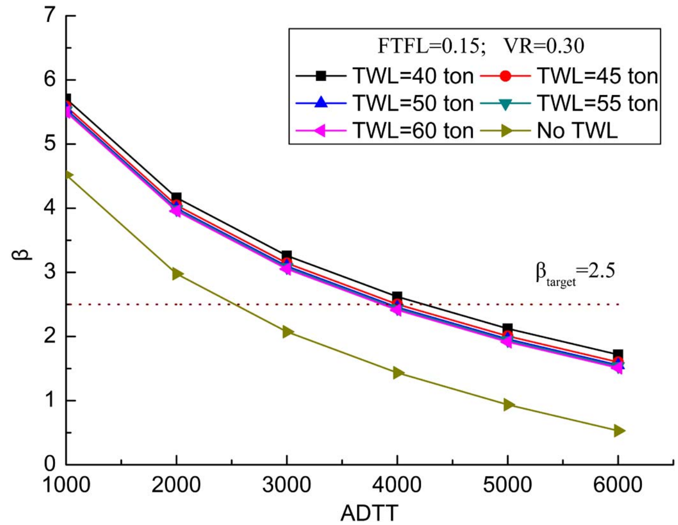

Fig. 10. Variation of fatigue reliability index with change in $ADTT$ and TWL for the bridge under the loading case considered.

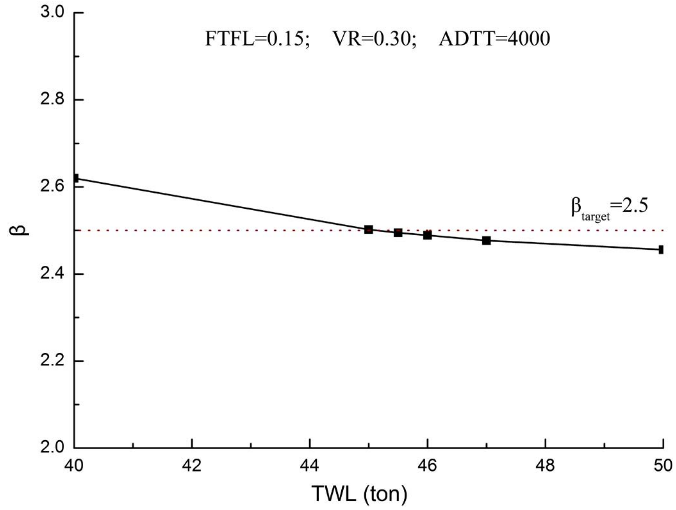

Fig. 11. Variation of fatigue reliability index with change in TWL under the loading case considered and the $FTFL$, VR, and $ADTT$ assumed.

to 1.0, indicating that the efficiency of enforcement of truck weight regulation is significantly affected by the VR.

Effect of the TWL

Fig. 10 shows the effect of the TWL on the fatigue reliability of the bridge girder considered, where $FTFL = 0.15$ and $VR = 0.3$ were adopted. From Fig. 10, $ADTT$ required for the reliability index of the bridge girder to decrease to the target reliability index is about 4,330, in which $FTFL = 0.15$, $TWL = 40$ ton, and $VR = 0.30$. In contrast, if no TWL is posted, the $ADTT$ required for the reliability index of the bridge girder to decrease to the target reliability index is around 2,500. This indicates the importance of setting the TWL on the fatigue reliability of the bridge girders. Interestingly, the difference between the $ADTTs$ required for the reliability index of the bridge girder to decrease to the target reliability index with $TWL = 40$ tons and $TWL = 60$ tons is small. This may be because the proportion of trucks in the traffic flow with weight ranging from 40 to 60 tons is low. Actually, from the statistic information of GVW in Table 1, it can be estimated that the proportion of trucks with weight exceeding 37 ton is roughly 7%. The estimation can be performed based on the means and coefficient of variations (COVs) of the GVW of each class and will not be given in detail here.

Determining the Truck Weight Limit Based on Bridge Fatigue Reliability

To determine the TWL for the bridge girder to achieve the target fatigue reliability index at the end of 75 years of service, the variation of fatigue reliability index β with change in TWL was obtained based on Eq. (11), where $FTFL$, VR, and $ADTT$ were taken as 0.15, 0.30, and 4,000, respectively. The results are shown in Fig. 11. As expected, the fatigue reliability index decreases with the increase of the TWL. Specifically, a TWL of 45 tons is needed for the bridge girder to achieve the target fatigue reliability index of 2.5 after 75 years of service.

Although the TWL determined in the present study was based on a particular bridge component and the assumed traffic data, the proposed approach can be used for different bridges to determine the TWL under different traffic load conditions. A summary of the procedures to carry out the proposed approach is given as follows:

1. Obtain the stress influence line of the point of interest based on the static analysis of the three-dimensional finite element model of the bridge under consideration.
2. Based on the real traffic data or the assumed traffic data for the bridge considered, obtain the stress time history of the point of interest based on the flowchart illustrated in Fig. 5; calculate the $AFDA$ based on Eq. (6), and obtain its statistical properties under the given $FTFL$, VR, and $ADTT$ values.
3. Based on Miner's rule and the obtained statistical properties of the $AFDA$ under the given $FTFL$, VR, and $ADTT$, determine the TWL for the bridge to achieve the target reliability index by using Eq. (11).

To prevent the concentrated weight on a truck's axle from producing excessive stress on bridge members, the United States Congress enacted the federal bridge formula to limit the weight-to-length ratio of a vehicle crossing a bridge and set regulations on the axle weight and the gross vehicle weight (DOT 2006). However, in the present study, the truck weight limit was determined with the purpose of making sure that the fatigue reliability of the steel bridge girders can achieve their target reliability index at the end of the 75 years' service life under the code-specified traffic conditions. Since the increase of the average daily truck traffic ($ADTT$) will lead to a decrease of the TWL to achieve the same target reliability index, it is possible that the truck weight limit obtained from the proposed method may be smaller than that obtained from the strength limit state under certain $ADTT$. Therefore, the truck weight limit obtained from the proposed method can be used as a supplement to the truck weight limit obtained from the strength limit state.

Summary and Conclusions

Most previous studies have focused on the effect of truck weight limit on the maximum live load that bridges may experience. However, very few studies have investigated the effect of the truck weight limit on the fatigue reliability of steel bridge girders. In the present study, a new method of studying the effect of the truck weight limit on the fatigue reliability of steel bridge girders was presented and illustrated with a simply supported steel girder bridge. The effect of three parameters, including the fraction of traffic in the fast lane, the violation rate, and the truck weight limit, on the fatigue damage accumulation and fatigue reliability of steel bridge girders was investigated. A detailed procedure for determining the truck weight limit for steel bridges was also proposed. Based on the results from this study, the following conclusions can be drawn:

1. The effect of the fraction of traffic in the fast lane on the fatigue reliability of the bridge girder considered is insignificant.
2. Enforcing truck weight limit has a significant effect on the fatigue reliability of the bridge girders. A large violation rate can lead to significant reduction in the fatigue reliability of bridge girders.

The influence of the environmental erosion on the steel fatigue strength was not considered in the fatigue analysis in the present study, which will be the focus of future studies.

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