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Retraction

Retracted: Calibration of the Live Load Factor for Highway Bridges with Different Requirements of Loading

Advances in Civil Engineering

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Advances in Civil Engineering has retracted the article titled "Calibration of the Live Load Factor for Highway Bridges with Different Requirements of Loading" [1].

The data are from the California Department of Transportation (Caltrans) research project "P285 LRFD and LRFR Specifications for Permits and Fatigue Truck Loads" managed by Dr. Gongkang Fu, who did not provide permission to use these data or to use unpublished reports by Dr. Fu.

The Chief Editor has approved retraction, and the authors apologise.

References

[1] L. Liu, Q. Ren, and X. Wang, "Calibration of the Live Load Factor for Highway Bridges with Different Requirements of Loading," *Advances in Civil Engineering*, vol. 2020, Article ID 7347593, 10 pages, 2020.

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Research Article

Calibration of the Live Load Factor for Highway Bridges with Different Requirements of Loading

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Highway bridge load rating has been moving toward structural reliability since the issuance of AASHTO LRFR specifications; however, the recommended load factors were carried out by a few reliable truck data. The objective of this study is to calibrate the live load factor in AASHTO LRFR Rating Specification by using huge amount of WIM data collected in California for more than ten years between 2001 and 2013. Since traffic volumes, vehicular overloads, and traffic components are highly related to the load effect induced, a set of calibration equations is proposed here, in which the nominal standard load effect models are used and different requirements of loading are taken into account. By the analytical model of platoons of trucks and the extrapolation of the gathered WIM data over a short period of time to remote future over a longer time period, the expected maximum live load effects over the rating period of 5 years are also obtained. Then, the live load factor is calibrated as the product of the codified value multiplied by the ratio between the nominal standard load effect and the expected mean value. The results show that the products of the two ratios present rather constant, implying the proposed method and load configurations selected are effective. In the end, the live load factors of 1.0 and 0.7 along with load configurations are recommended for a simple span length less than 300 ft. The recommended calibration method and live load factors will eliminate the unnecessary overconservatism in rating specifications.

1. Introduction

Highway bridge design and load rating in the US have been moving toward structural reliability-based practice milestoned by the LRFD (load and resistance factor design) Bridge Design Specification of the American Association of State Highway and Transportation Officials (AASHTO) in 1994. Over the past two decades or so, the effort of this movement has been steadily growing, including continuously developing new LRFD provisions and improving existing provisions, developing load and resistance factor rating (LRFR) specifications. With the issuance of the AASHTO Manual for Condition and Load Resistance Factor Rating of Highway Bridges, this country has entered an ear of LRFR for bridge rating.

In the effort of implementing the new LRFR specifications, the load factors have been calibrated through several NCHRP research projects, producing NCHRP Report 368 [1], NCHRP Report 454 [2], and NCHRP Report 20-7/186 [3]. However, when the research studies were carried out, there were a few reliable truck data available. Hence, the static weight data collected by the truck survey were used in these studies complying with some assumptions such as the Average Daily Truck Traffic (ADTT), the headway distances between trucks, the multiple presence factor (MPF) of trucks on bridge span and the correlation between weights of trucks in adjacent lanes, and so on.

Nowadays, a large amount of reliable truck data has been systematically collected through Weigh-in-Motion (WIM) systems. Taking advantage of this novel technique, some State Department of Transportation such as the Michigan State Department of Transportation (MDOT) refined their live load factors and proposed an adjustment factor to increase the live load to account for heavy truck traffic in metropolitan areas [4]. It was also concluded that the truck loads may vary significantly from state to state and bridge to bridge, due to different locations, traffic volumes, and operating status, resulting in different risk levels for highway bridge structures. Similarly, a comparative study supported by the New York State Department of Transportation

(NYDOT) was conducted by Ghosn et al., in which the live load factors were calibrated based on live load models developed using WIM data collected from several representative New York sites [5]. In Oregon, truck WIM data were also used to develop state-specific LRFR live load factors, and the factors were calibrated using the same statistical methods that were used in the original development of LRFR [6]. Recently, Eamon et al. conducted a reliability-based calibration of live load factors for bridge design specific to the state of Michigan, and it was found that Michigan load effects were greater than those previously assumed, requiring higher load factors than those in current use [7]; Rasheed et al. implemented a structural reliability analysis of superstructure of highway bridges on the China-Pakistan Economic Corridor [8]; Oudah et al. calibrated live load factors for bridge systems conveying extremely heavy mining trucks [9]; Anitori et al. proposed a WIM-based live load model for advanced analysis of simply supported short- and medium-span highway bridges [10], and Siavashi and Eamon developed traffic live load models for bridge superstructure rating [11]. Besides, some efforts have been made to enhance the accuracy of load effect projections to longer period of time needed for design and rating [12–17]. These previous works could be regarded as precursors to this study. Although valuable, because of limitations of data availability, some concerns are still needed to be clarified. For example, no side-by-side truck configurations were explicitly investigated; a small number of WIM sites; a small selection of bridges; and other approximations in converting WIM data to load effects, especially in the projection of short-term WIM data to longer period of time (i.e., the design working life or evaluation period).

This work team has been making efforts to collect WIM data in the US and in China, and the data collected from California State for more than ten years between 2001 and 2013 are found to have a time stamp to 0.01 s, the highest time-stamp resolution available for WIM data. The objective of this study is to calibrate the live load factor in AASHTO LRFR Rating Specification by using this huge amount of data. Note that NCHRP Report 454 presents a generic method for calculating specific live load factors, which is the same as that found in the LRFR manual [18, 19]. It was therefore decided to use the same approximate calibration concept in this study, specified by the equation 29 of NCHRP Report 454:

$$\gamma_L = \gamma_{L,\text{ref}} \times \frac{\text{expected maximum live load effect for case } L}{\text{expected maximum live load effect for reference case}}, \tag{1}$$

where γ_L is the live load factor for the case of interest and $\gamma_{L,\text{ref}}$ is the reference live load factor already in practice such as the design live load factor or rating live load factor. Live load factor γ_L to be found in this research is expressed here as the reference live load factor $\gamma_{L,\text{ref}}$ multiplied by a correction factor, a ratio between the expected maximum load effect for the case of interest and the expected live load effect for the reference case. Note that for simplicity, the gross weights of the loads were used instead of the load effects as stated in equation (1) in NCHRP 454. It is also seen in equation (1) that the reliability indices β for evaluation are not explicitly involved in the calibration but implicitly referred to.

Since the target reliability index β_0 has been embedded in structural design/assessment codes, this research tried to fix the problems mentioned above by taking into account the load model configuration reflected in the nominal load effect of load models specified in AASHTO LRFD, to fully fill the same concept of reliability and implement the reliability-based relative calibration; consequently, equation (1) could be modified as follows:

$$\gamma_L \times \text{nominal standard load effect for case of interest} = \gamma_{L,\text{ref}} \times \text{nominal standard load effect for reference case} \times \frac{\text{expected maximum live load effect for case } L}{\text{expected maximum live load effect for reference case}}. \tag{2}$$

Equation (2) represents the complete concept actually used in the LRFR calibration report NCHRP Report 454, and explicitly indicates that it is the product of the live load factor and the nominal standard load effect (such as the HL93, Legal Vehicles) that is being calibrated, not only one of the two alone. In other words, one may use many different combinations of the two to reach the same structural reliability level as long as the product of the two remains unchanged. Therefore, whether a prescribed load configuration may reach a target safety requirement needs to be examined along with the associated live load factor. As a matter of fact, adjusting the load factor and not the loading configuration is more convenient and practical. For implementation, the user of the proposed checking equation would more likely prefer a new live load factor over a new truck configuration because a new live load factor is easier to implement. Therefore, this research effort has taken the option of fixing the load configuration based on practical implementation consideration and then calibrating the live load factor.

2. WIM Data Processing

After roughly comparing and analyzing, the records of 70 sites collected in California between 2001 and 2013 were firstly selected for this research to cover as many bridge types and bridges having different number of lanes in one direction as possible. For the case of interest, traffic volume and truck weights are highly concerned, since the Average Daily Truck Traffic (ADTT), Gross Vehicle Weight (GVW), and Axle Load (AL) are the main variables that determine the effects of live load on bridge girders, and it is necessary to understand the distributions of those random parameters to increase the confidence in the reliability analysis.

Specifically, the histograms of ADTT, GVW, and AL are plotted for each site of each year to understand their characteristics and distributions. According to the ranks of the three parameters, namely, if the magnitude of ADTT, GVW, and AL of a site had been remarkable during the past twelve years, indicating a stable and reliable traffic situation,

then it could be a representative site and used in the next analysis, taking into account the reality that ADTT usually corresponds to a variety of truck types, and GVW and AL are directly related to the live load effects induced to bridges. The overview of processing is illustrated in Figure 1.

As a consequence, 15 sites are selected. Among them, 9 sites have three traffic lanes and others have two lanes in one direction. The ADTTs range from 5,218 to 12,816 for heavy traffic volume and 777 to 2,560 for light sites. The data were scrubbed before the analysis, and typically less than 4% of the recorded data was eliminated when apparent inconsistency was seen. The calculated results of top 5 sites selected are listed in Table 1 as a typical example, where the yearly WIM data set of 2011 was used and the moment and shear are both for the span of 30 ft. Since there are too many years, sites, and span lengths used in this study, more details are not convenient to be listed here.

3. Load Effect Analysis Based on the Markov Chain Random Process

The previous research [20] has pointed out that the Markov Chain random theory is an ideal mathematical model to simulate the process of trucks travelling on bridges. To cover all trucks locating on the span simultaneously, a platoon of trucks is treated as a basic analysis cell herein, as illustrated in Figure 2, where the structural response of a bridge system subjected to a moving platoon of trucks is described by an influence line; the one in the middle of this platoon referred to as the main truck (T_m) hereafter is placed on the influence line to induce the maximum load effect, and then other trucks in the platoon are placed on the span according to their distances from the main truck. If truck arrivals follow a Markov Chain Process, the bridge acts as a filter represented by its moment influence line at a given location, and the moment response $E(t)$ follows a filtered Markov process. If L is the influence line representation of a bridge structure, the response at time t, at the point on the bridge due to truck n is

$$E_n(t) = L(t\text{-}T_n, X_n = i), \tag{3}$$

in which t-T_n describes the location of the truck n with respect to the time t, at which the response of the main truck is being measured. This truck location is calculated by its speed and the time difference from the arrival of the main truck. X_n describes the occupied lane, which will be denoted by i. The term i can take any of the values 1, 2, 3, ... depending on the number of lanes available. The total response on the bridge due to one truck event (a platoon) between a reference time $T_m = 0$ (arrival of the main truck and t, the time at which the response is measured) is given by

$$E(t) = \sum_{n=1}^{N} L(t - T_n, X_n). \tag{4}$$

It should be noted that, if by any time a truck has left the bridge span, then the influence line function automatically assigns a value of zero to $E(t)$. Since the arrival of a new platoon of trucks is conditioned on the previous one, every platoon of trucks can be studied by examining the arrival of the previous one accordingly. In addition, the number of trucks in the platoon (denoted as N) can vary, and $N = 21$ in this study, which is adequate to cover all trucks simultaneously moving on a span less than 300 ft, and the longitudinal spacings between N trucks can be found using time stamps and speeds recorded in WIM data. The transverse distribution of these trucks on the span is also recorded in WIM data as the lane occupied by each truck.

As expressed in equation (4), the individual contribution from each truck is then added to that of the main truck to find the total load effect of that platoon of trucks. Note that the superposition of these trucks' load effects needs to consider effects in two perpendicular directions. One is in the traffic direction and the other the transverse direction, which will involve lateral distribution of load to the interested bridge component. Therefore, the total load effect considering lateral superposition of trucks in different lanes is modeled as follows:

$$\begin{aligned} E_{i\text{-total}} &= \mathrm{DF}_1 E_{i1} + \mathrm{DF}_2 E_{i2} + \ldots \mathrm{DF}_L E_{iL} \\ &= \mathrm{DF}_1 \left(E_{i1} + \frac{\mathrm{DF}_2}{\mathrm{DF}_1} E_{i2} + \ldots \frac{\mathrm{DF}_L}{\mathrm{DF}_1} E_{iL} \right), \end{aligned} \tag{5}$$

where $E_{i\text{-total}}$ = total load effect of the ith load event (i.e., the ith platoon of trucks) in the WIM data; E_{i1} to E_{iL} = load effects of trucks in lanes 1 to L, respectively, up to all the available lanes; DF_1-DF_L are lateral distribution factors to distribute loads in lanes 1-L to the focused bridge component. Based on a review of available research results for a variety of highway bridge types and span lengths [21–23] as well as the previous work carried out by the first author [24], the lateral distribution factors of 0.45, 0.15, and 0 are used for two-lane, three-lane, and four or more lane cases, respectively. As an example, the calculated moment and shears of the top 5 sites can be seen in Table 1.

On the other hand, WIM data usually are gathered over a short period of time and compared with the time intervals covered by bridge evaluation and design specifications (typically from several years to several decades or a century), and it is impossible to gather WIM data for such a long time span, so that projection of the measured data to remote future over a longer time period is needed to assess the structural reliability assured by the specifications and is critical in developing code requirements as interested here.

The procedure developed by Fu et al. [24] and Fu and You [25] is used here to obtain the statistics of load effects over the evaluation time period. Based on the extreme I distribution, it uses the short-term maximum value distribution to obtain the long-term maximum value distribution as follows:

$$\mu_M = \mu_1 + \frac{\ln M}{\pi} \sqrt{6}\sigma_1, \tag{6}$$

$$\sigma_M = \sigma_1, \tag{7}$$

where μ and σ = mean and standard deviation (SD) of the maximum load effect over the number of time periods indicated by the subscripts. Subscripts M and 1 represent M

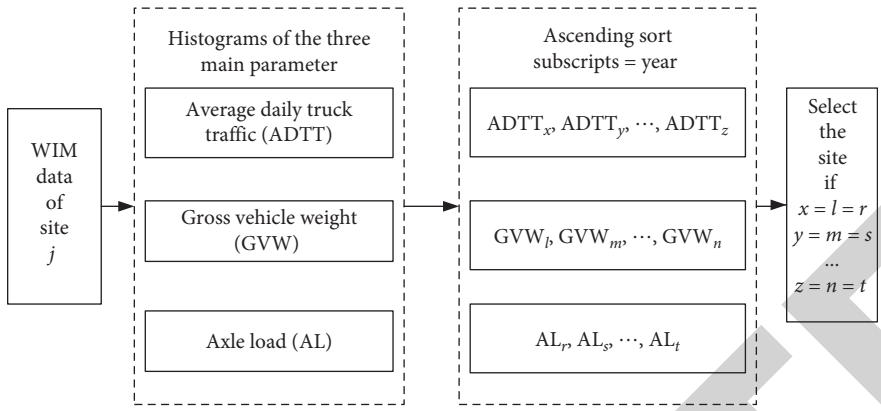

Figure 1: The overview of the process of the recorded data.

Table 1: The calculation results for the top 5 sites selected as an example.

Site ID	ADTT	Maximum GVW (kN)	Maximum AL (kN)	Maximum moment (kN∗m)	Maximum left shear (kN)	Maximum right shear (kN)
60	13,566	574.24	182.81	545.26	59.82	59.82
10	12,995	602.26	188.60	784.23	85.74	85.74
96	12,658	585.36	192.15	583.51	67.07	67.07
95	11,843	577.35	187.71	583.64	71.06	71.06
74	11,596	579.13	182.81	587.80	63.29	63.29

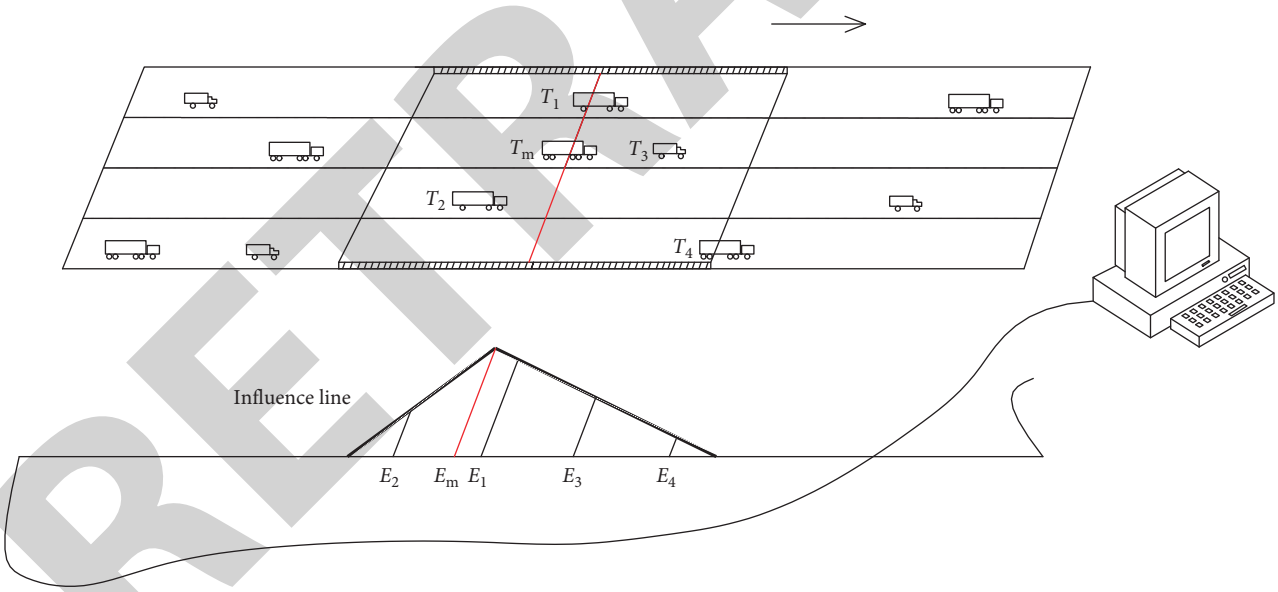

Figure 2: Illustration for computing load effect of midspan moment.

periods of long term and the counterpart one period, respectively. There needs to be a number of WIM data sets with short-term time period to perform statistic estimation of the mean μ_1 and SD σ_1. In other words, one year of WIM data may be divided into 12 months of data for 12 monthly maximum load effects, which then can be used to estimate μ_1 and σ_1. Consequently, they can be used in equations (6) and (7) to find the long-term maximum load effect mean μ_M and SD σ_M. The accuracy and efficiency of this extrapolation method have been verified by using "long-term" WIM data gathered (e.g., using yearly data to check the projection of daily/monthly data sets), and more details are available in Liu [26, 27].

4. Calibration of the Live Load Factor in LRFR

This section has a focus on deriving bridge-structural-reliability-based vehicle configurations and associated load factors for highway bridges with different loading levels. Specifically, four requirements of loading are considered here, consisting of case (a): light traffic situation without overloaded vehicles travelling on bridges; case (b): heavy

traffic with many overloaded trucks frequently moving on bridge spans; case (c): medium one with a few of overloaded trucks occasionally passing through, and case (d): the last one whose traffic components are unknown. In all cases, overloaded vehicles as a particular class are taken into account as well as their volume (or frequency), being consistent with the permit checking required by many states [28, 29]. To calibrate the live load factor for each load condition, equation (2) is then explicitly written with the nominal standard load models as equations (8)–(11) for cases (a)–(d), respectively.

$$\gamma_L = \gamma_{L,\mathrm{ref}} \times \frac{M_L}{M_{5\mathrm{CA}}} \times \frac{E_{R-5\mathrm{CA}}}{E_{R-L}}, \tag{8}$$

$$\gamma_L = \gamma_{L,\mathrm{ref}} \times \frac{M_L}{M_{\mathrm{CA}}} \times \frac{E_{R-\mathrm{CA}}}{E_{R-L}}, \tag{9}$$

$$\gamma_L = \gamma_{L,\mathrm{ref}} \times \frac{M_L}{1.3 \times M_{\mathrm{CA}}} \times \frac{E_{R-\mathrm{CA}}}{1.3 \times E_{R-L}}, \tag{10}$$

$$\gamma_L = \gamma_{L,\mathrm{ref}} \times \frac{M_L}{M_{\mathrm{CA}}} \times \frac{E}{E_{R-L}}, \tag{11}$$

in which M represents the nominal standard load effect and the subscripts represent the standard truck models fixed; E represents the expected maximum live load effect, and the subscripts represent the reference cases involved. Specifically, M_L = nominal standard load effect of AASHTO legal loads; $M_{5\mathrm{CA}}$ = nominal standard load effect of the 5-axle CA permit loads; M_{CA} = nominal standard load effect of the CA permit loads; $E_{R-5\mathrm{CA}}$ = expected maximum live load effect up to the 5-axle CA permit loads; E_{R-L} = expected maximum live load effect up to federal legal loads; $E_{R-\mathrm{CA}}$ = expected maximum live load effect up to CA permit loads; E = expected maximum all live load effect.

Compared with equation (2), equations (8)–(11) include the California permit load as the load case of interest and the truck load traffic complying with the federal legal requirement as the reference case. Note that the live load factor γ_L to be found in equations (8)–(11) is expressed as the reference live load factor $\gamma_{L,\mathrm{ref}}$ multiplied by two ratios that can be viewed as correction factors. Besides the nominal value ratio, the other is the ratio of expected values of the case of interest (truck traffic including California permit loads) and the case of reference (truck traffic complying with the federal legal load requirement). For example, the former load in equation (8) is modeled here using WIM data excluding those vehicles whose load effect exceeds that of the 5-axle CA permit truck, and the latter is modeled also using WIM data but excluding those above the federal bridge formula. The similar ruler is applied to other three equations. All expected maximum live load effects are obtained using the approach presented in the previous section. In which, the basic period of time is one year. A total of 13 years of WIM data are used to extract 13 yearly maximum values, based on which the projection is performed, using $N = 13 \times 5 = 65$ for the 5-year period. In addition, for realistic modeling for estimating the expected values, 10% and 20% allowance are separately given when excluding those trucks above the referred upper limits. The entire calibration process is demonstrated by a flowchart below (Figure 3).

Based on this concept, Figures 4–6 show the ratios of the two expected values of the two truck traffic loads in equation (8), given 0%, 10%, and 20% allowance separately. They are plotted as functions of span length for moment, left shear, and right shear, respectively. Due to the space limit, only the results from one site are displayed in this paper.

When the expected maximum load effect ratios in Figure 4 multiplied by the nominal load effect ratios between the AASHTO legal loads and the 5-axle CA permit truck are required in equation (8), the resulting products are shown in Figure 7 also as functions of simple span length for the midspan moment and support shears. It is seen in Figure 7 that the product of the two ratios appears to be fairly constant over span length. Statistically, the standard deviations of the expected maximum load effect ratios are 0.23–0.12, and those of the products of the two ratios have reduced to 0.11–0.06. It indicates that the AASHTO legal load and CA permit truck as the load configurations proposed for rating can reach reasonably uniform reliability over the span length range used here. This is seen to be true for both midspan moment and support shears.

Figures 8 and 9 display the counterpart results with 10% and 20% allowance respectively, from which, it also can be seen that the products of the two ratios present rather constant, implying the proposed method and load configurations selected are effective. Note that bridges with span length less than 300 ft can be defined as short-to-medium span category, and for implementation and practical convenience, the calculated ratio for each span length selected here can be averaged as one, since these ratios appear to be relative constant and applicable to short-to-medium bridges. Similarly, the ratios for midspan moment and support shears as well as different allowances could also be averaged to propose one ratio as the final product for equation (8) of case (a). As a consequence, a value of 0.35 is obtained for site 002. Due to the limited space, the calibration process for other 14 sites is not repeated here, but the results are displayed in Table 1. The same calibration process is then carried out for equations (9) to (11), respectively, with a different nominal load effect described above; their results are shown in Tables 2–4. It should be remarked that equations (8)–(11) are approximate ignoring the influence of dead load effect, which can become significant when the span is long. Next, substituting the final values in Tables 2–5 into equations (8)–(11) results in the required live load factor γ_L of 0.59, 0.91, 0.60, and 0.66, respectively, for $\gamma_{L,\mathrm{ref}} = 1.3$ being the live load factor for operating rating in LRFR. For being reasonably conservative, the values of 0.7 and 1.0 are recommended for practice.

To summarize the recommendation derived in this section, the following live load factors along with load configurations are recommended for spans less than 300 ft.

(1) For rating light traffic sites without overloaded vehicles travelling on bridges, use the 5-axle CA permit

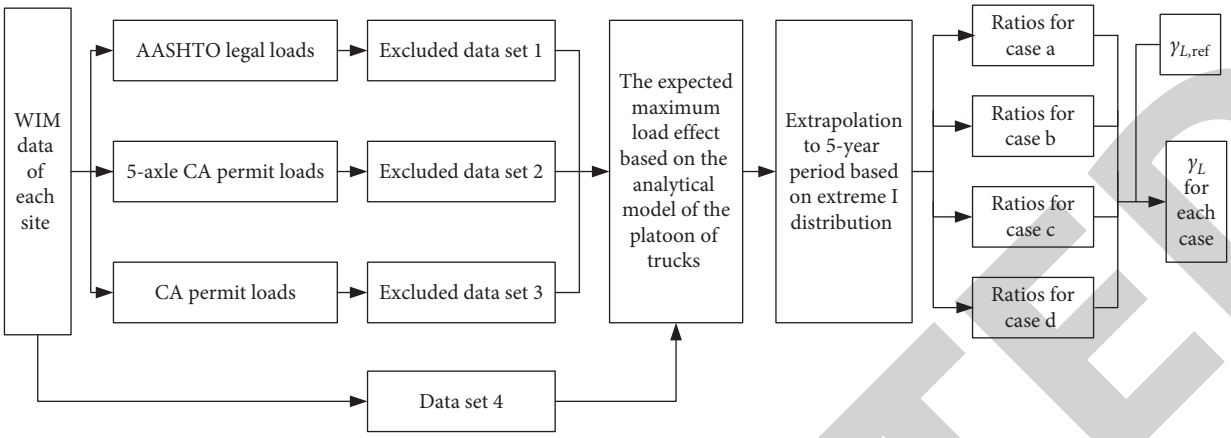

Figure 3: The flowchart of the calibration process.

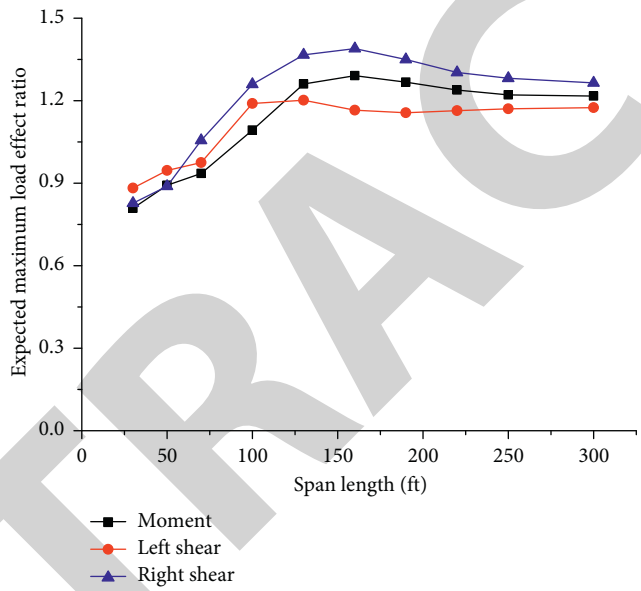

Figure 4: Mean value ratio in equation (8) for site 002 (with 0% allowance).

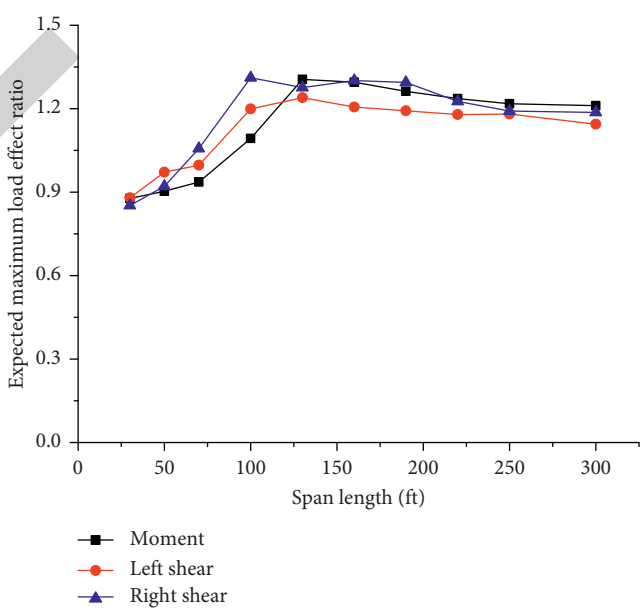

Figure 5: Mean value ratio in equation (8) for site 002 (with 10% allowance).

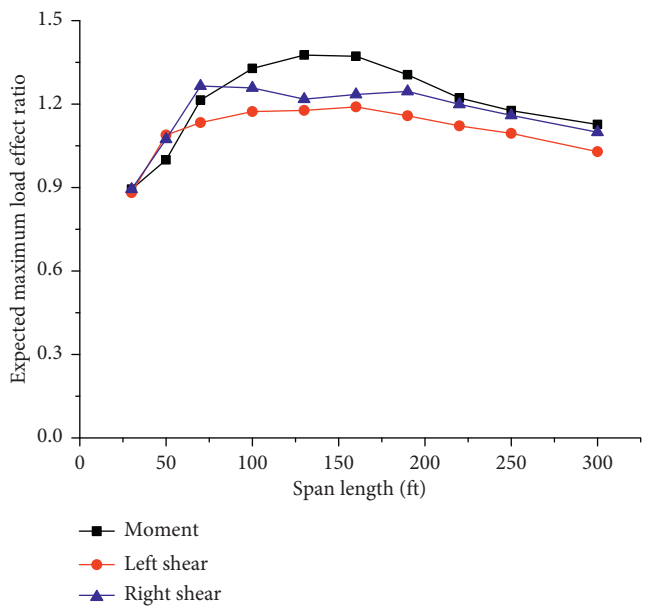

Figure 6: Mean value ratio in equation (8) for site 002 (with 20% allowance).

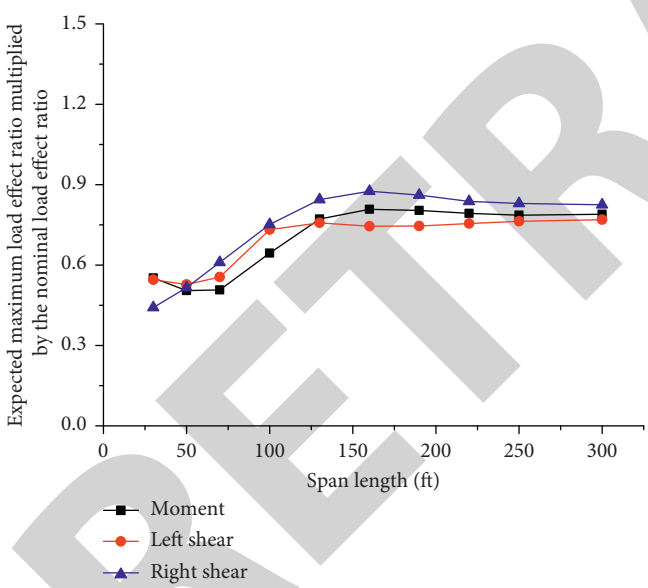

Figure 7: Expected value ratio multiplied by nominal load effect ratio in equation (8) for site 002 (with 0% allowance).

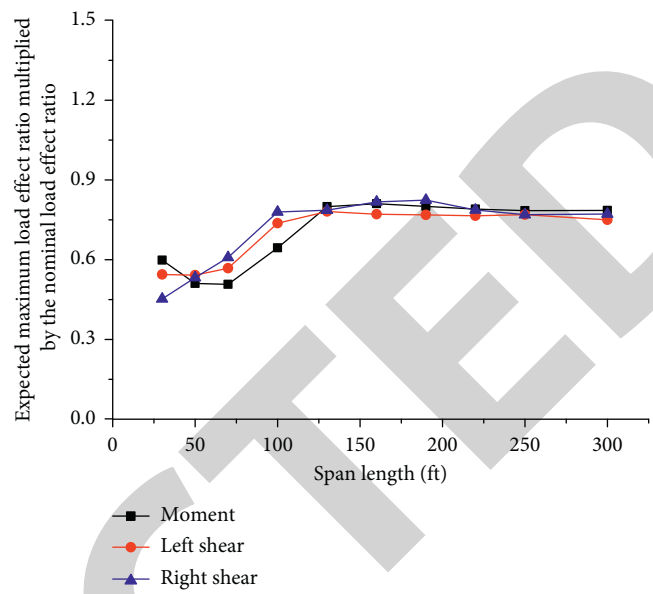

Figure 8: Expected value ratio multiplied by nominal load effect ratio in equation (8) for site 002 (with 10% allowance).

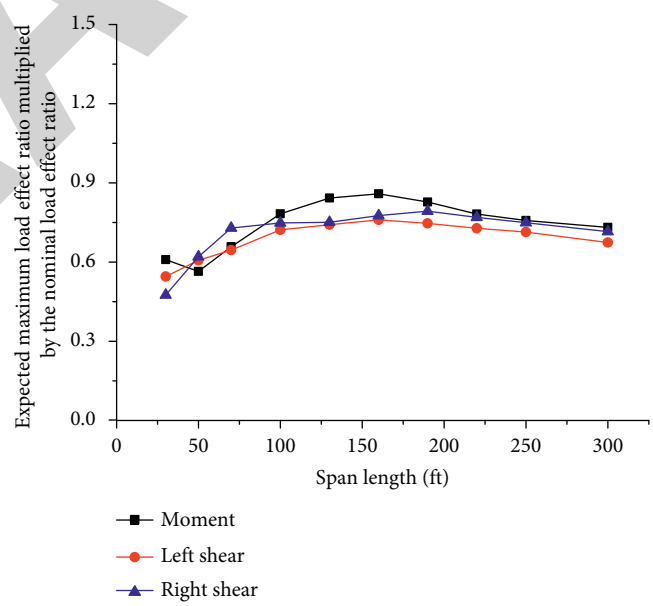

Figure 9: Expected value ratio multiplied by nominal load effect ratio in equation (8) for site 002 (with 20% allowance).

truck as the load configuration and a live load factor of 0.70

(2) For rating heavy traffic with overloaded trucks moving on bridge spans (both frequently and occasionally), use the set of CA permit trucks and a live load factor of 1.0

(3) For rating bridges with an unknown traffic situation, use the same set of CA permit trucks as (2) to be reasonably conservative, and a live load factor of 0.7 is recommended

Obviously, the recommended associated live load factors are less than 1.3, which is the reference live load factor $\gamma_{L,\mathrm{ref}}$ in equations (8) to (11). The fact indicates that the AASHTO train load is overconservative or unnecessary. Furthermore, traffic condition would significantly affect bridge rating,

Table 2: Ratios for equations (8) and the 15 sites.

Site	Moment			Left shear			Right shear		
	0%	10%	20%	0%	10%	20%	0%	10%	20%
2	0.36	0.36	0.36	0.34	0.34	0.33	0.35	0.34	0.33
20	0.38	0.37	0.34	0.37	0.35	0.33	0.37	0.35	0.31
22	0.45	0.43	0.41	0.45	0.42	0.41	0.40	0.38	0.35
25	0.32	0.32	0.32	0.31	0.31	0.31	0.32	0.32	0.32
60	0.35	0.33	0.33	0.34	0.33	0.32	0.36	0.35	0.33
70	0.51	0.47	0.47	0.54	0.50	0.49	0.51	0.48	0.47
88	0.49	0.44	0.34	0.44	0.48	0.36	0.50	0.47	0.32
89	0.46	0.49	0.49	0.46	0.48	0.48	0.49	0.49	0.49
90	0.51	0.50	0.50	0.56	0.53	0.52	0.53	0.51	0.49
95	0.52	0.49	0.47	0.53	0.51	0.49	0.51	0.47	0.46
96	0.47	0.47	0.45	0.49	0.48	0.47	0.48	0.47	0.44
111	0.55	0.60	0.59	0.61	0.67	0.66	0.59	0.62	0.61
112	0.54	0.53	0.52	0.53	0.52	0.52	0.52	0.53	0.50
115	0.43	0.39	0.39	0.47	0.40	0.39	0.45	0.43	0.42
816	0.50	0.50	0.50	0.56	0.53	0.52	0.50	0.48	0.50
Average	0.46	0.45	0.43	0.47	0.46	0.44	0.46	0.45	0.42
Final value									**0.45**

Table 3: Ratios for equation (9) and the 15 sites.

Site	Moment			Left shear			Right shear		
	0%	10%	20%	0%	10%	20%	0%	10%	20%
2	0.61	0.62	0.62	0.65	0.65	0.62	0.70	0.68	0.65
20	0.68	0.71	0.68	0.70	0.66	0.64	0.72	0.71	0.67
22	0.67	0.65	0.68	0.70	0.65	0.68	0.67	0.64	0.65
25	0.61	0.62	0.63	0.63	0.63	0.63	0.63	0.65	0.68
60	0.66	0.63	0.63	0.68	0.65	0.65	0.67	0.67	0.69
70	0.66	0.67	0.72	0.71	0.67	0.78	0.69	0.67	0.74
88	0.74	0.65	0.63	0.61	0.67	0.63	0.74	0.71	0.61
89	0.64	0.66	0.74	0.63	0.68	0.74	0.69	0.68	0.77
90	0.67	0.68	0.73	0.69	0.70	0.73	0.71	0.74	0.77
95	0.72	0.77	0.77	0.74	0.78	0.79	0.76	0.77	0.76
96	0.71	0.75	0.72	0.75	0.75	0.75	0.77	0.77	0.74
111	0.67	0.73	0.75	0.71	0.75	0.81	0.72	0.77	0.80
112	0.67	0.77	0.80	0.68	0.75	0.76	0.7	0.8	0.80
115	0.72	0.63	0.67	0.77	0.64	0.65	0.67	0.65	0.70
816	0.67	0.69	0.73	0.73	0.70	0.71	0.67	0.69	0.75
Average	0.67	0.68	0.70	0.69	0.69	0.70	0.70	0.71	0.72
Final value									**0.70**

Table 4: Ratios for equation (10) and the 15 sites.

Site	Moment			Left shear			Right shear		
	0%	10%	20%	0%	10%	20%	0%	10%	20%
2	0.36	0.36	0.36	0.34	0.34	0.33	0.35	0.34	0.33
20	0.38	0.37	0.34	0.37	0.35	0.33	0.37	0.35	0.31
22	0.45	0.43	0.41	0.45	0.42	0.41	0.40	0.38	0.35
25	0.32	0.32	0.32	0.31	0.31	0.31	0.32	0.32	0.32
60	0.35	0.33	0.33	0.34	0.33	0.32	0.36	0.35	0.33
70	0.50	0.47	0.46	0.54	0.50	0.49	0.50	0.46	0.46
88	0.54	0.49	0.38	0.46	0.50	0.38	0.54	0.51	0.35
89	0.49	0.49	0.49	0.48	0.48	0.48	0.52	0.49	0.49
90	0.53	0.50	0.50	0.56	0.53	0.52	0.54	0.51	0.49
95	0.53	0.50	0.47	0.53	0.51	0.49	0.52	0.48	0.46
96	0.48	0.47	0.45	0.49	0.48	0.47	0.48	0.47	0.44
111	0.61	0.60	0.59	0.68	0.67	0.66	0.65	0.62	0.61
112	0.55	0.53	0.52	0.54	0.52	0.52	0.54	0.53	0.51
115	0.44	0.40	0.40	0.47	0.40	0.40	0.45	0.43	0.42
816	0.50	0.50	0.50	0.57	0.53	0.52	0.50	0.48	0.50
Average	0.47	0.45	0.43	0.48	0.46	0.44	0.47	0.45	0.42
Final value									**0.46**

Table 5: Ratios for equation (11) and the 15 sites.

Site	Moment			Left shear			Right shear		
	0%	10%	20%	0%	10%	20%	0%	10%	20%
2	0.43	0.44	0.47	0.42	0.43	0.39	0.43	0.42	0.37
20	0.49	0.48	0.44	0.49	0.45	0.43	0.48	0.45	0.41
22	0.57	0.56	0.54	0.58	0.54	0.53	0.53	0.49	0.46
25	0.41	0.41	0.41	0.40	0.40	0.40	0.42	0.42	0.42
60	0.42	0.43	0.43	0.44	0.42	0.41	0.45	0.45	0.43
70	0.55	0.53	0.59	0.57	0.54	0.61	0.55	0.53	0.58
88	0.59	0.55	0.43	0.50	0.58	0.45	0.60	0.59	0.40
89	0.47	0.51	0.58	0.45	0.50	0.58	0.50	0.50	0.59
90	0.54	0.52	0.61	0.55	0.55	0.64	0.56	0.52	0.60
95	0.53	0.55	0.60	0.53	0.55	0.64	0.53	0.54	0.59
96	0.51	0.53	0.52	0.50	0.52	0.52	0.5	0.51	0.49
111	0.53	0.57	0.63	0.55	0.60	0.69	0.55	0.59	0.64
112	0.53	0.60	0.60	0.51	0.60	0.62	0.54	0.61	0.59
115	0.48	0.48	0.49	0.49	0.46	0.48	0.49	0.51	0.52
816	0.55	0.56	0.57	0.58	0.56	0.56	0.55	0.54	0.56
Average	0.51	0.51	0.53	0.50	0.51	0.53	0.51	0.51	0.51
Final value									**0.51**

especially when bridges have short-to-medium spans and different requirements of loading. Therefore, it would be better to use different live load factors to reflect the reality and reduce unnecessary costs.

5. Conclusions

In the present study, structural reliability-based live load factors have been developed by the truck load effect statistics using the analytical model of platoons of trucks and the extrapolation approach, taking a number of temporal maximum values. Then, the live load factors are recommended along with their approximate load configurations for rating short-to-medium bridge spans less than 300 ft, with or without overloaded vehicles travelling. The recommended calibration method and live load factors will eliminate the unnecessary overconservatism in rating specifications. Also, the nominal load models are recommended to be used as load configurations, with the live load factor of 0.7 and 1.0, respectively. It should be noted that all WIM data were collected from California, and the research and the conclusions in this paper focus on simply supported bridges with a span less than 300 ft, and further investigations are needed for more bridge types and other sites outside California.

Data Availability

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

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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