WIM based Live Load Model for Advanced Analysis of Simply Supported Short and Medium-Span Highway Bridges

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Abstract

The accuracy of bridge system safety evaluations and reliability assessments obtained through refined structural analysis procedures depends on the proper modeling of traffic load effects. While the live load models specified in the AASHTO procedures were calibrated for use in combination with the approximate analysis methods and load distribution factors commonly used in the U.S., these existing models may not produce accurate results when used in association with advanced finite element analyses of bridge structures.

This paper proposes a procedure for calibrating appropriate live load models that can be used for advanced analyses of multi-girder bridges. The calibration procedure is demonstrated using actual truck data collected at a representative set of weigh-in-motion (WIM) stations in New York State. Extreme value theory is used to project traffic load effects to different service periods. The results are presented as live load models developed for a 5-year typical rating interval and for a 75-year design life. The outcome of the calibration indicates that maximum traffic load effects can be calculated using finite element models with the help of a single truck for short to medium one-lane multi-girder bridges and two side-by-side truck configurations for multi-lane bridges. The proposed analysis trucks have the axle configurations of the standard AASHTO 3-S2 and Type 3 Legal Rating trucks with appropriate factors to amplify their nominal weights. The amplification factors reflect the presence of overweight trucks in the traffic stream and the probability of multiple-

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presence. The proposed live load models are readily implementable for deterministic refined analyses of highway bridges and for evaluating the reliability of bridges at ultimate limit states considering the system's behavior.

Key words: Weigh-In-Motion (WIM), Load Modeling, Live Load, Reliability Calibration, Loads for FEA, Multi-Girder Bridges, Refined Analysis.

1. Introduction

Typical bridge design and evaluation processes as well as refined reliability analyses are highly sensitive to the live load models used to simulate the effects of traffic load on highway bridges. This sensitivity is related to the high levels of uncertainty that are associated with estimating traffic load characteristics at bridge sites. Design codes and specifications attempt to compensate for these sensitivities and uncertainties by using accordingly calibrated live load safety factors in combination with simple generic live load models that can be used during the design of new bridges or the safety evaluation of existing ones. When analyzing multi-girder superstructures, these models require positioning a set of concentrated loads to describe the axle weights (sometimes in combination with distributed loads) having specific intensities at critical locations along the length of bridge decks (AASHTO 2014; CEN 2003) to determine critical system load effects. In the AASHTO specifications, the global load effects are subsequently multiplied by the load distribution factor to give the maximum effect on an individual beam. Alternatively, live load models may be presented as load effects on specific bridge components (e.g. maximum moment at mid-span of a beam in a multi-beam bridge) (Nowak and Rakoczy 2013; Reid and Yaiaroon 2012). Either way, codes and specifications present live load models that are often considered as "general purpose" nominal models applicable to all types of bridges.

The AASHTO load analysis procedure for two-lane short to medium simple span bridges, which are used as the base line, assumes that the maximum load effect is caused by side-by-side trucks of equal weight. This assumption has been found to work well for implementation with the specified AASHTO load distribution factors. However, to improve the bridge safety assessment process, efforts are being currently directed at developing refined analysis procedures that require the placement of "design or analysis" truck models both longitudinally and transversely on the bridge deck to perform finite element analyses for calculating load effects on particular components. The relevance of such refined analyses for accurately evaluating the safety of bridges can be undermined if the applied live load model is itself too rough to represent actual loading conditions as pointed out by previous research (Cheung and Li 2002; Fu and Hag-Elsafi 2000; Žnidarič et al. 2012). Furthermore, truck traffic intensity, volume and truck weights may vary considerably between regions and between sites. Optimized live load models may be necessary to perform a refined safety evaluation of bridges using site-specific or state-specific live load models that reflect actual traffic conditions (Cohen et al. 2003; Sivakumar et al. 2011). Other researchers have also proposed approaches to refine the load models. For example, Leahy et al. (2015) proposed a model that consists of a distributed load of variable intensity that depends mainly on span length based on WIM data collected on a state level. A procedure to adapt AASHTO LRFR (2003) load factors to measured data was also proposed by Pelphrey et al. (2008) for bridge evaluation. But, previous efforts have mainly concentrated on live load models for application with existing AASHTO load distribution factors. This paper presents an approach for the reliability calibration of a live load model applicable for the Finite Element Analysis of Grillage and 3-D Models of bridge systems rather than the traditional single line analysis in combination with load distribution factors.

Engineers have used the maximum single girder analysis to find the moments and shear forces on typical multi-girder bridges because a moving load analysis for single girders is relatively easy to perform and computationally inexpensive. However, because the live load models in combination with the standard load distribution factors specified in current bridge standards are calibrated for typical bridges subjected to regular traffic loads assuming that the trucks follow pre-specified lane paths, they may not accurately reflect actual load effects on specific bridges exposed to particular loading conditions. The consideration of specific conditions is especially important when evaluating existing bridges. The traditional girder analysis with the AASHTO live loads and lateral load distribution factors may not provide the level of rating accuracy that may be needed in special cases such as when a bridge does not pass the safety evaluation process by a reasonable margin (mainly during the assessment of existing bridges) or when the bridge may be exposed to exceptionally large overweight trucks. An accurate bridge rating process should take into account local truck traffic conditions, as observed through WIM records, using a refined structural analysis model that reflects the actual bridge behavior when it is loaded by multiple trucks of different weights and configurations placed in the most critical longitudinal and lateral positions. A main objective of this paper is to provide live load models and a methodology for analyzing bridges that are in the “borderline” and where a more detailed analysis can save bridges from expensive strengthening and/or posting or where the AASHTO design and rating trucks do not reflect the intensity of the truck traffic observed at the site. In the AASHTO calibration of the live load model and the load distribution factors, typically a system of forces was used as moving load and only afterwards the effect due to the most critical position were distributed to the most critical girders of the bridge. In this study a different approach, based on influence surfaces is recommended to improve the calculation of the load effect distribution among members.

The calibration of the proposed approach is illustrated with the specific goal of developing state-specific live load models for evaluating the ultimate strength capacity of highway bridge superstructures. The live load models should be applicable for analyzing the effect of vehicular traffic on individual components or alternatively could be used to study the reserve strength or the reliability of the entire structural system using refined structural analysis procedures that take into account the non-linear behavior of bridges under extreme live loading conditions. For strength limit state analyses, the live load models should reflect the effect on the structure of the small percentage of trucks defining the very upper tail of the load effect histogram, as the goal is to estimate the maximum load expected to cross the bridge within a specific design or service period. The tail end of the truck weight distribution spectrum can best be captured through field truck data collected using Weigh-In-Motion (WIM) installations at representative highway sites (Sivakumar et al. 2011).

The processing and statistical analysis of WIM data for live load modeling purposes has been extensively studied as early as the 1980's (Ghosn and Moses 1986; Moses et al. 1984; Moses and Ghosn 1983, 1985) and continued throughout the years, (see for example Caprani et al. 2002; Cheung and Li 2002; Nowak 1999; O'Connor et al. 2002; Qu et al. 1997; Soriano et al. 2016 among others). WIM systems generally measure each passing truck's axle weights, axle spacings, Gross Vehicle Weight (GVW), traffic lane, and time of arrival. Previous studies have developed specific step-by-step procedures to translate this large amount of traffic data into user-friendly bridge live load models for implementation with traditional AASHTO beam analysis methods (Sivakumar and Ghosn 2011). This paper extends the approach to develop live load models appropriate for use when performing refined 3-D advanced deterministic or reliability analyses of bridge structural systems. The new models are meant to be applicable for the design of new bridges assuming a 75-year design life or for the rating of existing bridges using a 5-year rating period (Moses 2001; Nowak 1999). The focus of this paper is on short to medium length simple-span multi-beam steel composite bridges because they represent a very common configuration in many countries and US states (Ghosn et al. 2015a). However, the same procedure can be applied to other structural types such as simple span or continuous I-beam, box girder steel and prestressed concrete bridges. The calibration procedure is illustrated in this paper using truck data collected at several WIM stations in the state of New York. When the live load models are used to perform direct reliability analyses, they must include the statistical parameters necessary for defining the random nature of the applied live loads. A proposal for a probabilistic live load model is also presented in the paper.

An example is presented to illustrate how the resulting live load model can be implemented in engineering practice and to highlight the differences between the results compared to those obtained when using the current AASHTO model.

2. Statistical Analysis of Live Load Effects

WIM Database

The truck traffic information and truck weight data used in this study are extracted from a set of one year worth of records from twenty WIM stations spread around the New York State network of highways (Ghosn et al. 2015a). Each

WIM station dataset was filtered using the approach recommended by Sivakumar et al. (2011) in order to remove unreliable data. The WIM sites are classified based on a number of site characteristics which include the total number of vehicles recorded at each site, the ADTT (Average Daily Truck Traffic) defined as the number of daily trucks recorded at each site averaged over one year of measurements and the number of OW (Over-Weight) trucks defined as the number of trucks that exceed the legal weight limits applicable to the state (Fiorillo and Ghosn 2014; Ghosn et al. 2015a). Table 1 summarizes these site characteristics for each WIM measurement site. The data show that the percentage of overweight trucks varies between 11.7% and 26.6% of the total number of recorded trucks. These percentages in combination with the ADTT may have a significant effect on the number of heavy trucks that may simultaneously cross a bridge which is an important determinant of the maximum load that would be expected on a multi-lane bridge during a given service period.

Table 1. NYS WIM stations along with their main characteristics

Grillage Analysis of Representative Bridge Models

As observed from studying common truck configurations and the low probability of fitting two consecutive trucks in the same lane, the maximum load effect on short to medium span bridges in the range of 15 o 60 m is governed by the presence of a single truck per lane. The analysis of the load effect on a single lane is performed by sending truck data collected from a WIM site through the appropriate influence surfaces. The calculations carried out in this work are specifically adapted to composite steel girder bridges, as it is a very common structural type in many countries and US states, especially the state of New York, whose WIM data is being used to illustrate the proposed methodology (Ghosn et al. 2015a). A set of 100 bridge configurations having representative characteristics of the population of steel-composite bridges are considered. The bridge population consists of structures having the combination of the geometric parameters presented in Table 2.

Table 2. Geometric parameters of the bridge population (Steel girder-concrete slab bridges).

Designs for steel I-girder bridges having the geometric configurations summarized in Table 2 are performed according to the standard AASHTO (1996) specifications to reflect the basic design of the majority of existing New York steel girder bridges. The selection of the optimal cross section for each configuration is set to minimize the weight among all possible cross sections that satisfy the moment demand. The steel I-girder section design process is described by Ghosn et al. (2015b). Besides the main longitudinal elements, the design includes the definition of diaphragms in the form of K-shaped bracing and of the deck based on the FHWA (2003) recommendations.

The analysis of the effect of heavy trucks on each bridge is performed through a grillage model based on the modeling approach recommended by Hambly (1991). 2-D grillage models are known to provide a simple yet accurate approach for calculating the response of bridge superstructures in a computationally efficient procedure.

Figure 1. Example of moment influence surfaces for the section at midspan of a 30 m four-girder bridge with 1.8 m beam spacing, for the (a) first and (b) second member.

The results of the grillage analysis are summarized in influence surfaces for different bridge configurations that are used to calculate the maximum effect of each truck in the WIM database for all possible positions of the truck within its particular lane. The advantage of using influence surfaces is that the bridge response, which is needed for the millions of trucks in the WIM database, is not calculated by a numerical model but is computed by interpolation making the process more efficient computationally. Examples of moment influence surfaces for two longitudinal girders of a 30 m bridge with four girders at 1.8 m center to center are shown in Figure 1. The contour plots in the figure help give the response of a main member when one point load is placed on a specific location on the surface of the bridge deck. The influence surfaces in Figure 1 are plotted using normalized coordinates; both longitudinally (normalized with respect to span length) and transversally (normalized with respect to bridge width). For example, when a 1 kN load is placed at 0.5 of the span length and 0.2 of the width of the bridge, the moment effect is 12 kN.m (see the arrow in Figure 1**a**). With the same procedure, the response of a system of forces representing the load of each tire of a truck can be found by multiplying the load effect for each 1 kN force extracted from the influence surface for the position of the force by the actual value of the concentrated force and adding the contributions of all the forces that represent the truck.

For example, when a single or two side-by-side AASHTO HL-93 design trucks are moved along the 30 m four-girder bridge with 1.8 m beam spacing whose influence surfaces are depicted in Figure 1 along a path where the exterior wheel is half way between the two external girders, the fraction of the load effect of one truck carried by the most loaded internal girder is 0.34 and it is 0.50 for the external member for the 1-lane case and 0.54 for the internal and 0.57 for the external member for the 2-lane case. These are compared to the AASHTO distribution factors that would be equal to 0.32 for the internal and 0.50 for the external member when the bridge is loaded by a single lane after removing the multiple lane factor, and 0.45 for the internal and 0.55 for the external member for the 2-lane case. The differences between the relatively small results may be due to the specific characteristics of the bridge analyzed in this example and the positioning of the loads. The AASHTO LRFD load distribution factors are calibrated by fitting the equations through the results of hundreds of bridge grillage analyses similar to the analyses performed in this paper after verifying the accuracy of the grillage models with more advanced 3-D finite element analyses and field test results (Zokaie et al. 1995).Naturally, because the AASHTO load distributions are fitted equations, individual analysis results may differ from those in the equations.

Single Lane Loading

Each truck in the WIM data files is analyzed using the influence surfaces described earlier to find the maximum moment and maximum shear in each beam of each of the set of bridges listed in Table 2. The maximum moment effects are evaluated at the midpoint of each beam and the maximum shear near the end of each span. The maximum load effects for all the trucks in each WIM file are then assembled into histograms for each main member of each bridge configuration. For example, the maximum moment effects calculated for the WIM database for site number 9121, which consists of about half a million trucks collected over a 1-year period, are assembled into the histograms presented in Figure 2 for the 30 m bridge with 8 beams at 1.8 m spacing. The plots, which show a very large distribution of load effects, indicate that the load is primarily carried by the external two girders when the trucks are travelling in the right main lane of the bridge with some load in the third girder from the right and a negligible proportion of load being distributed to the remaining 5 girders (see Figure 2a). Also, the graphs show how the load shifts to girders 3 and 4 when the trucks are travelling in the passing lane (see Figure 2b).

Figure 2. Histogram of maximum moment in the first four members due to traffic load in (a) lane 1 and (b) lane 2, for the 30 m bridge with 8 beams at 1.8 m.

While the histograms give the distribution of the load effects from all trucks, ensuring the safety of a bridge requires verifying that the bridge will be able to carry the maximum load that it will be exposed to within a pre-determined service or design life. The study carried out by Nowak (1999) employed a simple method to extrapolate the maximum load effect from truck data collected during a survey undertaken in 1975 in Ontario Canada. The simple method was found to be reasonable because the Ontario data available at the time was biased toward the heaviest 20% of the trucks crossing the survey site. The approach may not necessarily be accurate for other data sets but it was widely used in other research projects (Jo et al. 2005; Khorasani 2010). According to Nowak’s method, the load effect data are assumed to fit a normal distribution. Assuming a certain number, N, of trucks crossing the bridge during the reference period, the expected maximum load effect is obtained using the 1/N fractile of the standard normal probability function. This truck weight fractile provides the number of standard deviations by which the maximum load exceeds the mean value.

While Nowak's approach is valid if the load effect from the truck database follows a normal distribution, Ghosn et al. (2011) and Soriano et al. (Soriano et al. 2014) generalized the approach of Nowak (1999) by studying a large set of truck Weigh-In-Motion (WIM) databases and observing that only the upper tail of the load effect histogram approaches that of a normal distribution and proposed an approach for fitting the upper tail end of each truck load effect histogram into a normal distribution whose own tail end matches the upper 5% of the actual histogram. In this study, the model proposed by Sivakumar et al. (2011) and Ghosn et al. (2011, 2013) is adopted as will be described further below because it focuses on the tail of the traffic load effects which defines the extreme loads that a bridge must carry.

Two-Lane Loading

As is the case with many WIM databases, the WIM data files available for this work do not provide arrival times with sufficient precision to analyze the probability of multiple-presence on two-lane bridges (OBrien and Caprani 2005). For this reason, general headway data collected by (Sivakumar et al. 2011) are used in this paper for the analysis of two-lane loadings. Analyzing large numbers of WIM sites in New York, Sivakumar et al. (2011) observed that the percentage of trucks involved in multi-presence events varies with the Average Daily Truck Traffic (ADTT). Based on those observations, in this paper 0.5%, 1.25% and 2% of truck loading events are assumed to take place with two side-by-side trucks for ADTT<100, 100<ADTT<5000 and ADTT>5000 respectively as described by Ghosn et al. (2011, 2013) and Soriano et al. (2016). The probability of having three trucks side-by-side contributing to the maximum load effect in a main girder is very small due to the low probability of simultaneous presence. Furthermore, the nature of the influence surface for multi-beam bridges shows that the effects on the girders away from the loaded lane are somewhat limited. Therefore, the focus of this study is on single lane and two-lane loading events.

The probability density function of the effect of two trucks simultaneously on the bridge in two lanes $f_s(S)$ can be calculated using a convolution equation presented as:

$$f_s(S) = \int_{-\infty}^{+\infty} f_{x_2}(S - x_1) f_{x_1}(x_1) dx_1 \tag{1}$$

where x_1 is the effect of the trucks in lane 1 and x_2 is the effect of the trucks in lane 2, $f_s(S)$ is the probability distribution of the combined multi-lane effects $S = x_1 + x_2$, $f_{x_1}(\ldots)$ is the probability distribution of the effects of trucks in lane 1 and $f_{x_2}(\ldots)$ is the probability distribution of the effects of trucks in lane 2.

Eq. (1) assumes no correlation between the effects of the trucks in each of the lanes. In fact, the WIM data for the sites analyzed in this study show no correlation between the weights of trucks close to each other in the same lane or in adjacent lanes as shown in Figures 3 for station 9121. These data justify the use of Eq. (1) for modeling the effects of multi-presence events and later on the extreme value distribution model adopted in this study.

Figure 3. Relationship between weights showing lack of correlation between consecutive vehicles in different lanes.

Statistical Projection of Extreme Load Effects

The lack of correlation between the effects of the trucks within the same lane and those following each other in different lanes and the assumption that the tail ends of the load effect histograms approach those of a normal distribution allow for the application of extreme value theory to obtain the maximum combined load effect for trucks in a single lane or two adjacent lanes. In fact, the tail end of the histogram of the combined load effect extracted from two normal distributions can also be modeled by the tail end of a normal distribution with means and standard deviations that can be extracted from those obtained by fitting the tail ends of each lane's histogram.

The procedure followed in this paper mirrors the one proposed by Ghosn et al. (2011, 2013) and Sivakumar et al. (2011) except that the load effect is directly obtained from the analysis of an individual beam using the grillage model and associated influence surfaces rather than calculating it as the total load effect in the entire superstructure.

The mean and standard deviation of the combined load effect are used to calculate the statistical parameters of the Type I (Gumbel) distribution that describes the maximum load in a specific time interval by means of the following equations (Ang and Tang 2007):

$$\alpha_N = \frac{\sqrt{2\ln(N)}}{\sigma_S} \tag{2}$$

$$u_N = \mu_S + \sigma_S \left(\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right) \tag{3}$$

where μ_S and σ_S are the mean and standard deviation of the load effect (one or two lanes), α_N and u_N are respectively the inverse measure of dispersion and the most probable value of the Type I distribution; μ_S and σ_S are the mean value and the standard deviation of the normal distribution whose tail end matches that of the actual histogram of the single lane or the combined load effect; N is the number of load repetitions related to the bridge service period.

α_N and u_N are then used to find the mean of the maximum load effect, $L_{\max}$, its standard deviation, $\sigma_{L\max}$ and its coefficient of variation $V_{L\max}$ for any reference time during which N loading events take place:

$$L_{\max} = \mu_{\max} = u_N \frac{0.577216}{\alpha_N} \tag{4}$$

$$V_{L\max} = \frac{\sigma_{L\max}}{L_{\max}} = \frac{\pi}{L_{\max}\sqrt{6}\alpha_N} \tag{5}$$

3. Live Load Model

The results obtained by Eq. (4) and (5) present a probabilistic model of the maximum load effects on bridge members. This model can be either used directly during the reliability analysis of bridge components or implemented during a reliability-based calibration of LRFD procedures to propose nominal live load models that can be applied in traditional engineering practice during the deterministic linear analysis of bridges. The basic concept for proposing a live load model for advanced analyses consists of defining a set of axle forces, representing a design truck configuration, where the effects of the design truck's axle weights model the expected maximum load effect on the most critical members as obtained by Eq. (4) and (5). It is noted that the nominal truck's configuration and its axle weights may not necessarily be unique in the sense that different combinations of truck configurations and axle weights may reproduce the same desired load effects. In this work, the nominal truck configurations are adopted from standard AASHTO trucks to provide a live load model with a familiar configuration for practicing engineers and researchers. Specifically, Ghosn et al. (2011, 2013, 2015b) showed that 3-S2 semi-trailers form the vast majority of trucks traveling on US highways and that these trucks produce the largest load effects on the main members of medium span bridges. Similarly, the effects of heavy trucks on short span bridges (less than 30 m) are mostly governed by single unit trucks which can best be modeled by AASHTO Type 3 legal trucks. For these reasons, it is proposed that the maximum load effects on short to medium span single lane and two-lane bridges be simulated by analyzing the combined effects of either one or two side-by-side trucks having the configurations of the AASHTO 3-S2 or Type 3 Legal Trucks presented in Figure 4. While the configurations of the trucks are representative of the vast majority of trucks observed on U.S. highways, the axle weights of the AASHTO Legal Trucks are considerably lower than the weights of illegally overweight or permit trucks as observed from the WIM data. Therefore, the analysis needs to account for the expected maximum load effect which can be simulated by using design trucks having the configurations of the AASHTO Legal Trucks but with amplified intensities as explained further below.

Figure 4. AASHTO Type 3 and 3-S2 Legal Truck configurations.

To perform a refined analysis of a bridge, the engineer needs to develop a structural model and apply the nominal loads on the structure to study the effects that the nominal loads will produce at a particular section of a beam. When checking the ultimate limit states (ULS), these nominal load effects should simulate the maximum load effects expected in the bridge service life to ensure that the factored calculated load effects at the beam section or bridge component of interest are lower than the factored capacity of that component. In this work, two lane loading conditions are defined: a) in the first one nominal legal truck is placed in the external lane, and b) in the second side-by-side trucks are placed on the most critical position on the bridge deck. To simulate the effects of overweight trucks that travel on US highways, it is proposed that the axle weights of one of the design trucks be amplified by a factor α. This factor α is needed because the WIM data shows that the AASHTO HL-93 load model, that was designed to envelope the effects of exclusion vehicles, does not cover the large variety and the high numbers of overweight trucks observed on many US highway systems. Furthermore, statistical analyses of the data shows that it is highly unlikely that the maximum load on a bridge would be governed by two side-by-side trucks of the same weight. The proposed α factor would serve as both a multiple presence factor and an overweight factor.

Figure 5 shows the placement of one lane and side-by-side nominal legal trucks introducing the parameter α as a multiplier of the weight of the truck in the main drive lane. The calibration of the parameter α is executed such that the maximum load effect produced by the single AASHTO legal truck of weight αP or two side-by-side trucks of weights P and αP on the most critical main member is equal to the maximum load effect obtained from Eq. (4) for the member. Different values of the parameter α may be used for studying the shear forces and moments due to one-lane and two-lane loadings.

Figure 5. Section view of side-by-side truck loading pattern.

During the structural analyses performed in this study, the following assumptions are made regarding the transverse placement of the trucks to simulate the worst loading conditions:

- The distance between the most external wheel and the edge of the deck is 1.2 m (barrier + curb + clearance).
- The truck wheels are spaced at 1.8 m.

- For the two-truck cases, the transversal distance between trucks is 1.2 m, unless the deck width is smaller than 7.2 m in which case the distance between trucks is reduced to satisfy the distance from the edge criteria set in the first bullet.

The longitudinal position of the trucks to be employed is the one producing the highest value for the effect (moment or shear) under consideration. The worst position of the one or two trucks has to be calculated for each truck in its own lane separately.

The calibration of α is carried out by equating the mean of the maximum load effect produced by Eq. (4) to the legal truck load effect using the following equation:

$$\alpha = \frac{L_{\max,1}}{E_1^{legal}} \tag{6}$$

For two-lane loadings, the parameter α of the legal truck in the main drive lane is obtained according to the following equation:

$$\alpha = \frac{L_{\max,2} - E_2^{legal}}{E_1^{legal}} \tag{7}$$

where $L_{\max,k}$ is the mean value of the maximum load effect obtained from Eq. (4) over *k* loaded traffic lanes, and E_i^{legal} is the effect of the legal truck (3-S2 or type 3) located in such a way as to produce the maximum effect position in lane *i*.

The load effects considered for calculating α are bending moment and shear in the most critical longitudinal beam. The uncertainties related to the system of forces proposed in here are directly related to the corresponding load effect. The analysis process consists of calculating L_{max} using Eq. (4) for all the combinations of the bridge configurations listed in Table 2 repeated for each of the truck records collected from the twenty WIM stations. L_{max} and the corresponding parameter α are calculated for shear and bending moments assuming a design life equal to 75 years and also for typical rating interval of 5 years.

As an example, the calculation of the parameter α is presented for the WIM data collected in station ID 9121, for the analysis of the moment effect for the one-lane loading case. The example refers to the results of a 30 m steel composite bridge, 11 m wide with 8 beams spaced at 1.8 m center to center. The maximum moment at mid-span of the external member is calculated through influence surfaces by moving the centroid of system of forces (the vehicle wheels) within all the possible positions on the external traffic lane. For example, for a truck having the axle configuration of the 3S-2 truck, the most critical position is when the centroid of the system of axle forces is calculated to be 16.8 m. The maximum moment at the mid-span section is found to be 741.0 kNm. Such calculations are performed for each of the 1149657 vehicles in the WIM station ID 9121 dataset. A set of 1,149,657 maximum moments are subsequently assembled into a histogram similar to those shown in Figure 2. The subset consisting of the largest 5% of the values is then fitted to match the upper 5% of a fictitious normal distribution defined by a mean value μ_s =130.0 kN.m and a standard deviation σ_s =320.4 kN.m. μ_s and σ_s thus obtained are implemented into Eq. (2) and (3) which for the 75-year design life are associated with a number of truck loading events *N*=*ADTT**365*75=105,705,261 to obtain $u_N = 1934$ and $\alpha_N = 1.89 \cdot 10^{-2}$. Using Eq. (4) and (5) we find L_{max} =1964.7 kNm and V_{max}=3.4%. Finally, the parameter α that should be used to amplify the weight of the AASHTO 3-S2 Legal truck is obtained from Eq. (6) as α =2.65 (=1964.0/741.0). The process is repeated to analyze the truck data in each WIM site for all the bridge configurations defined in Table 1 for the one-lane and two-lane loading cases.

Some of the results of the calculation of the parameter α according to Eq. (7) for the 75-year design life are plotted in Figure 6 for -different bridge configurations where the nominal trucks used are those of the 3-S2 legal truck configuration. Similar results are obtained for the Type 3 truck for span lengths smaller than 30 m and for the one-lane case obtained using Eq. (6).

The variability in the calculated value of the parameter α is found to be relatively small leading to a COV for the maximum load effect on the most critical beams for the entire population of steel composite bridges ranging between 4.5 to 6% when analyzing the data from one WIM site.

Figure 6 helps study the sensitivity of the parameter α to beam spacing (BS), span length (SL) and number of beams (NB). The plots are generated from the analysis of the trucks of WIM station 9121 for both maximum girder moment (Figure 6a) and shear (Figure 6b) for the 75-year case.

Figure 6. Variation of the parameter α (a) moment and (b) shear, for different combinations of beam spacing and number of beams for a 40 m span bridge.

The plots in Figure 6 show that increasing the number of beams results in higher values of α for moment effects and lower values for shear effects but an asymptotic value is reached at about 8 beams. The different trends in the shear and moment values are due to the higher increase of the denominator than the numerator of Eq. (7) for the shear and the opposite behavior for moments as the number of beams increases. This is caused by the differences between the axle spacings of the actual trucks as compared to the AASHTO Legal Trucks and also to the differences in the lateral load distribution for loads near the supports where shear dominates and loads near the middle of the span where the moment dominates.

The sensitivity of α to different numbers of beams, beam spacing and span length, as can be observed in the shapes of the curves plotted in Figure 6 and results obtained for other span-lengths, suggest that the parameter α can be estimated from a quadratic equation of the form:

$$\begin{aligned}\alpha_{eq.} &= const + a_1\frac{SL}{30.5} + b_1\frac{BS}{1.8} + c_1\frac{NB}{6} + \\ &+ a_2\left(\frac{SL}{30.5}\right)^2 + b_2\left(\frac{BS}{1.8}\right)^2 + c_2\left(\frac{NB}{6}\right)^2\end{aligned} \tag{8}$$

where SL is the span length in meters, BS is the beam spacing in meters, NB is the number of beams, $const$ is a constant coefficient and a_1, b_1, c_1, a_2, b_2, c_2 are coefficients calibrated to minimize the error in estimating a value of the parameter $\alpha_{eq.}$, compared to the actual α values obtained directly by Eq. (6) or (7). The seven coefficients that appear in Eq. (8) are calibrated using the data from of the twenty New York WIM stations and for all bridges in the database by minimizing the following mean error index:

$$\mu_{err} = \frac{\sum\left|err_{\mu}\right|}{n_{tot}\ n_{st}} = \frac{\sum\left|\left(\alpha-\alpha_{eq.}\right)/\alpha\right|}{n_{tot}\ n_{st}} \tag{9}$$

where n_{tot} is the total number of bridges analyzed (in this case 100) and n_{st} is the number of WIM stations used (in this case 20).

The coefficients a_1, b_1, …, c_2 are curve shape coefficients that depend on the load effect under study (moment of bridges less than 30 m with the Type 3 truck and moment of bridges more than 30 meters and shear for all bridges with the 3-S2 truck) and on the load case (one or two side-by-side trucks). On the other hand, the parameter $const$ of Eq. (8) is originally calculated independently for each WIM station. Subsequently, the parameters $const$ from the different sites are assembled into groups as will be discussed further below .

The minimization of the error between the results obtained from the simulation and those obtained from Eq. (8) is performed by automatically testing different sets of coefficients $const$, a_1, b_1, …, c_2 and minimizing the error defined by Eq. (9) through a trial and error process. In order to reduce the computational effort, the Evolutionary minimization algorithm built into “Microsoft Excel 2013” was used. The coefficients are summarized in Table 3 for the one-lane and two-lane load models obtained after analyzing the full twenty WIM station datasets ($n_{st} = 20$).

Table 3. Coefficients of Eq. (8) for analysis of shear and moments under one-lane and two-lane loadings.

Because of the different influences they produce, it is not always straight forward to relate WIM station characteristics to actual load effects. Nevertheless, the results obtained in this study show that the overweight (OW) percentages have an important effect on the parameter α as shown in Figure 7.

Figure 7. Relationship between OW percentage and parameter α for the one-lane loading of an 8-beam bridge with beam spacing of 1.8 m and a span length equal to 30 m.

This observed trend is used to define three levels of traffic intensity based on the percentage of overweight (OW) trucks in each WIM station dataset. Specifically, light, medium and heavy traffic enforcement levels are respectively defined based on observed OW percentages of about 12, 19 and 25%. As already mentioned, while the coefficients a_i, b_i and c_i, are observed to remain essentially constant for all traffic sites, the value of the parameter $const$ of Eq. (8) is different for each of the three groups of OW levels. The minimization algorithm already mentioned for the full set of 20 WIM stations is therefore repeated to find the appropriate parameter $const$ for each of the three subsets of stations, grouped based on the three levels of overweight percentages. The different values for $const$ are listed in Table 4 for shear and moment effects on one-lane and multi-lane bridges. The latter effect is divided into effect on short spans governed by the AASHTO Legal single unit truck and longer spans governed by the AASHTO Legal 3-S2 truck.

The values presented in Table 4 can be used for the evaluation and rating of existing bridges where a bridge site's overweight truck intensity can be estimated based on legal weight enforcement levels or WIM data analysis. For short span bridges, when the live load is applied, a preliminary comparison between the effect of the AASHTO 3-S2 and Type 3 Legal Trucks should be checked, and the most critical truck model considered when performing a bridge system analysis.

While the values provided in Table 4 can be used in combination with Eq. (8) when evaluating bridges at sites where the truck traffic characteristics are reasonably well known such as when rating an existing bridge, it is often difficult to have such information particularly when designing new bridges. In such cases, a similar set of coefficients is calibrated from all WIM stations, leading to the results of Table 4 which are obtained by executing the evaluation of the constant of Eq. (8) over the data collected from all the WIM stations. It is well understood that by covering a wider range of stations, there will be a higher variability in the value of the parameter. This higher variability should be compensated by using a higher live load (or safety) factor when designing new bridges as compared to the evaluation of existing bridges.

Table 4. Value of the constant of Eq.(8) for different OW percentages and for design of new structures (all).

The constants listed in Table 4 along with the coefficients in Table 3 applied to Eq. (8) and the results of a finite element analysis of bridges under the effect of truck loads arranged as shown in Figure 5 can be used for deterministic evaluation of the safety of multi-girder bridges.

4. Implementation: Example Analysis of a Multi-Girder Bridge

An example is presented in this section to illustrate how the live load model developed in this work can be used to estimate the applied load effect for the design of a single-span bridge. The example is for a 30 m steel composite bridge, 11 m wide with 8 beams spaced at 1.8 m center to center. The moment on the most loaded beam calculated according to the AASHTO LRFD Bridge Design Specifications (2014) including the use of the load distribution factor gives a maximum moment equal to 1879 kN m. During the design process, this nominal live load moment is associated with a live load factor $\gamma_L = 1.75$. This indicates that the factored live load effect excluding the dynamic amplification factor should be equal to 3288 kN m.

Because the AASHTO tabulated load distribution factors are meant to represent a wide range of bridge structures, they may not provide a very accurate representation of the live load effects on a particular bridge. Therefore, when a more refined 3-D or grillage bridge analysis is required, the engineer may choose to use the live load model proposed in this work which can be found according to the following steps:

1. Considering a span length equal to 30 m and 8 beams at 1.8 m spacing, the parameter α is obtained by applying the coefficients in Tables 3 and 4 to Eq. (8) to find the maximum moment effect in 75 years for the one lane case:

$$\alpha_{1-lane} = 2.56 + 0.263\frac{30}{30.5} + 0.001\frac{1.8}{1.8} + 0.245\frac{8}{6} - 0.086\left(\frac{30}{30.5}\right)^2 + 0.001\left(\frac{1.8}{1.8}\right)^2 - 0.103\left(\frac{8}{6}\right)^2 = 2.88$$

and for the two lane case:

$$\alpha_{2-lane} = 1.91 + 0.380\frac{30}{30.5} + 0.066\frac{1.8}{1.8} + 0.178\frac{8}{6} - 0.112\left(\frac{30}{30.5}\right)^2 + 0.006\left(\frac{1.8}{1.8}\right)^2 + -0.059\left(\frac{8}{6}\right)^2 = 2.37$$

2. A system of forces representing two side-by-side trucks having the configuration of the AASHTO 3-S2 Legal Trucks as depicted in Figure 4 are applied on a grillage model of the bridge set up using the approach proposed by Hambly (1991) to calculate the maximum moment in the bridge beams. In this case, a grillage model is used, although 3-D finite element models including 3-D solid elements or a combination of shell elements may also be employed.
3. The worst position of the two side-by-side trucks is found by varying the positions of the applied trucks in the model until the maximum effect on the most critical section of the longitudinal member is found. In this case, assuming that the most critical section is at the midspan of the bridge, the maximum moment is found when the truck is placed in such a way that the front axle is 10.39 m from the end of the span. The lateral spacing of the wheels is set as depicted in Figure 5.
4. The grillage analysis indicates that the moment on the most external member due to the presence of a 3-S2 Truck is $M_{lane1} = 741 kNm$ when the truck is placed in lane 1 and $M_{lane2} = 385 kNm$ when it is in lane 2.
5. The moment on the most external beam for the one-lane case is:

$$M_{1-lane} = \alpha M_{lane1} = 2.88 \times 741 = 2132\ kNm$$

while the moment on the most external beam for the two-lane case is:

$$M_{2-lane} = \alpha M_{lane1} + M_{lane1} = 2.37 \times 741 + 385 = 2138\ kNm$$

6. Because the parameter α calculated in this study is based on matching the expected 75-year maximum live load effect and because the AASHTO HL-93 nominal live load as derived by Nowak (1999) has an inherent bias which on the average is approximately equal to 1/1.25 (i.e. the actual expected maximum live load effect is 1.25 times the HL-93 load effect), then the factored live load that should be used in designing the bridge members should be calculated as:

$$LL_{fac} = \frac{1.75}{1.25} 2138\ kN\ m = 2993\ kN\ m$$

This example shows that the value of the maximum moment found using a refined analysis where one of the applied AASHTO Legal truck loads is multiplied the factor α of Eq. (8) gives a factored live load moment for the most critical member equal to 2993 kNm which is lower than the 3288 kNm obtained when using the AASHTO (2014) HL-93 live load in combination with the load distribution equations. The lower value from the grillage analysis reflects the improved accuracy of the live load model and the analysis process performed using the proposed approach as compared to the approximate analysis performed when using the AASHTO (2014) method. It is understood that such refined analysis may not be necessary during the design of new bridges in regions where no large numbers of overweight trucks are observed. However, such a refined analysis may be useful when rating existing bridges which had shown borderline safety levels when analyzed using traditional AASHTO methods or when the WIM data shows large deviations in truck weights compared to normal traffic on typical bridge sites.

5. Probabilistic Live Load Model

While the coefficients and constants in Tables 3 and 4 are sufficient for performing deterministic bridge analyses, a probabilistic format for the live load model is needed if the engineer decides to carry out a reliability analysis of a bridge structure. Specifically, the probabilistic format must account for the variability in the applied load and the associated modeling uncertainties (Ghosn et al. 2011, 2013). Therefore, the load effect in a reliability analysis using the results for the α parameter generated in this paper or similar simulations can be represented as the product of the following random variables for the two-lane loading case:

$$LL_{2-lane} = W_{truck}(1+\alpha \cdot StS) \cdot Mod \cdot Dyn \tag{10}$$

or the following for the one-lane case

$$LL_{1-lane} = W_{truck}\alpha \cdot StS \cdot Mod \cdot Dyn \tag{11}$$

where W_{truck} is the deterministic load effect of the nominal weight of the AASHTO 3-S2 Legal Truck having a total weight equal to 320 kN or the Type 3 Legal Truck with a gross weight equal to 222 kN; Mod is the load effect model uncertainties, StS is the site to site variability accounting for the uncertainty in defining a load value representing different WIM stations, LL is the total live load effect intensity. Detailed statistics of the random variables in Eq. (15) and (16) are provided in Table 5.

Table 5. Random variables associated with the parameter α

The statistical values for the dynamic amplification factors listed in Table 5 as suggested by Nowak (1999) are found to be in line with the ones proposed by numerical studies and experimental investigations (Deng et al. 2011; González 2010; OBrien et al. 2012).

6. Conclusions

A procedure is described to calibrate a live load model that can be used to perform advanced deterministic analyses for the design or evaluation of simply-supported multi-girder bridges. The procedure is illustrated by calibrating a model that produces similar maximum load moment and shear effects as those of trucks collected from a set of WIM stations in New York State.

It has been found that the configuration of the AASHTO 3-S2 Legal Truck with amplified axle weight intensities can provide acceptable live load configurations for simulating the maximum traffic load effects on medium span bridges between 30 m and 60 m in length. For short spans (less than 30 m) the overall bending behavior of bridges under maximum truck loads can best be modeled using the AASHTO Type 3 Legal Truck configuration.

The gross vehicle weights of the Type 3 and 3-S2 truck configurations must be amplified to reflect the maximum load effects expected during the service life of the bridge which may be caused by a combination of overweight trucks. For two-lane cases, the weights of the axles of one truck are exactly those of the AASHTO Legal trucks while the axle weights of the other truck are scaled by a factor α that varies as a function of span length, number of beams and beam spacing. For the one-lane case, the nominal legal truck weight is also multiplied by an appropriate value of the parameter α.

The proposed parameter α that depends on the percentage of overweight trucks in the traffic stream, would serve as both a multiple presence factor and an overweight factor to amplify the weights of the nominal analysis AASHTO 3-S2 and Type 3 trucks when performing a refined structural analysis of a bridge.

This paper proposes a quadratic equation for calculating the parameter α based on the maximum effect on typical bridge configurations that would be caused by a combination of heavy trucks the characteristics of which are collected by WIM stations in the state of New York.

The calibration process described in this paper is meant to provide similar bending moments and shear forces as the maximum values expected during the design lives or rating cycle of multi-beam bridges. The process has been presented for the case of composite steel girder bridges. The same approach can be used to develop live-load models suitable for other bridge types and load effects. Also, the same approach can be followed to calibrate live load models representing truck traffic in different regions and states.

The proposed model can be used to carry out deterministic analyses of bridge systems if accompanied with adjusted live load factors when rating existing bridges which had shown borderline safety levels when analyzed using traditional AASHTO methods or when the WIM data for the bridge site shows large deviations in truck weights compared to normal traffic on typical bridge sites. Also, the proposed live load model complemented with the statistical data obtained during the calibration process described in this paper can be used for the reliability analysis of complete bridge structural systems.

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Table 1

Station ID	No. of vehicles	% OW	ADTT
1281	27221	19.2	76
1400	652304	18.6	1832
1800	305100	22.6	851
2680	154740	24.4	429
3311	1225061	18.4	3481
4342	477552	14.2	1329
4483	113220	16.3	322
5183	467485	21.2	1298
5281	114761	26.4	317
6282	67350	21.3	186
6340	107884	20.1	335
6482	822958	17.3	2299
7100	454588	23.3	1293
7181	149752	26.6	414
7381	273144	12.4	779
8280	1733022	11.7	4804
8382	1277280	14.3	3605
9121	1149657	15.2	3861
9580	561431	21	1555
9631	226993	25.5	651

Table 2

Span length (m)	15, 20, 30, 40, 60
Number of beams	4, 6, 8, 10
Beam spacing (m)	1.2, 1.8, 2.4, 3.0, 3.6

Table 3

	M				V	
	3-S2		Type 3		3-S2	
	1 Lane	2 Lanes	1 Lane	2 Lanes	1 Lane	2 Lanes
a_1	0.263	0.380	1.828	1.378	0.228	0.115
b_1	0.001	0.066	0.047	0.152	-0.020	-0.013
c_1	0.245	0.178	0.908	0.273	0.160	0.262
a_2	-0.086	-0.112	-0.074	-0.017	-0.081	-0.047
b_2	0.001	-0.006	-0.016	-0.051	0.000	0.000
c_2	-0.103	-0.059	-0.389	-0.113	-0.080	-0.122

Table 4

		const (V)		*const* (M)			
		3-S2		3-S2		Type 3	
	Traffic (OW %)	1-lane	2-lane	1-lane	2-lane	1-lane	2-lane
5 years							
	12	1.77	1.39	1.77	1.05	0.50	0.48
	19	2.38	1.88	2.32	1.68	1.28	1.22
	26	3.84	2.87	3.76	2.77	2.53	2.19
	all	2.39	1.89	2.34	1.57	2.56	1.91
75 years							
	12	1.92	1.58	1.93	1.32	0.66	0.65
	19	2.58	2.15	2.55	2.00	1.56	1.55
	26	4.19	3.33	4.06	3.25	3.00	2.65
	all	2.59	2.15	2.56	1.91	1.30	1.11

Table 5

Symbol	Random variable	Mean	COV	Distribution	Reference
α	Weight multiplier of the truck on main drive lane	From Eq. (8)	1-lane loading 4.0%	Gumbel	Present study
			2-lane loading 8.0%		
StS	Site to site variability from different WIM stations	1.00	Mean 15.0%	Normal	Present study
			Low, Medium and High OW 6.0%		
Dyn	Dynamic amplification due to moving vehicles	1-lane loading 1.13	1-lane loading 9.0%	Normal	(Nowak 1999)
		2-lane loading 1.10	2-lane loading 5.5%		
Mod	Load effect model (grillage)	1.00	8.0%	Normal	(Sivakumar and Ghosn 2011)