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Weigh-in-Motion load effects and statistical approaches for development of live load factors

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Abstract. The aim of this paper is to simply present live load factor calculation methodology formulation with the addition of a simple new future load projection procedure to previously proposed two methods. For this purpose, Oregon Weigh-in-Motion (WIM) data were used to calculate live load factors by using WIM data. These factors were calculated with two different approaches and by presenting new simple modifications in these methods. A very simple future load projection method is presented in this paper. Using four different WIM sites with different average daily truck traffic (ADTT) volume, and all year data, live load factors were obtained. The live load factors, were proposed as a function of ADTT. ADTT values of these sites correspond to three different levels which are approximately ADTT= 5,000, ADTT = 1,500 and ADTT ≤ 500 cases. WIM data for a full year were used from each site in the calibration procedure. Load effects were projected into the future for the different span lengths considering five-year evaluation period and seventy-five-years design life. The live load factor for ADTT=5,000, AASHTO HS20 loading case and five-year evaluation period was obtained as 1.8. In the second approach, the methodology established in the Manual for Bridge Evaluation (MBE) was used to calibrate the live load factors. It was obtained that the calculated live load factors were smaller than those in the MBE specifications, and smaller than those used in the initial calibration which did not convert to the gross vehicle weight (GVW) into truck type 3S2 defined by AASHTO equivalents.

Keywords: bridges; load rating; weigh-in-motion; truck loadings; live load factors

1. Introduction

Statistical and probabilistic approaches in bridge engineering subjects has been widely studied in recent years (Zhou *et al*. 2019, Kaloop *et al*. 2019, Jeon *et al*. 2018, Yurdakul and Ates 2018, Ye *et al*. 2017, Tabsh and Mitchell 2016). In addition load factors and multi-lane load factors have been an interesting area of research recently (Zhou *et al*. 2018, Stewart 2018). Moreover considering Weigh-in-Motion (WIM) measurements in statistical bridge engineering studies is another topic which is gaining interest (Chan *et al*. 2005, Yan *et al*. 2017, Miao and Gosh, 2016). Very short summary of some of these studies and some other interesting studies on these subjects are presented below.

A computational framework for probabilistic modeling of the fatigue damage accumulation of short to medium span bridges under actual traffic loading was described by (Yan *et al*. 2017). Stochastic truck-load models were simulated based on site-specific WIM measurements in the same study. A Markov-chain based advanced simulation technique to perform a probabilistic dynamic analysis of the mechanisms that may lead to the progressive collapse of bridge structures was used in Miao and Gosh (2016), the flexibility of the approach allows for the use of site-specific truck weight and traffic data collected using WIM systems.

D'Angelo and Nussbaumer (2015) provided an original framework for the fatigue reliability analysis of a road bridge, the framework was applied to the Venoge bridge, a composite steel concrete bridge within the A1 Swiss highway. Ghasemi and Nowak (2016) studied the reliability analysis for serviceability limit state of bridges considering deflection criteria. They updated the statistical parameters (mean and standard deviation of the bridge deflections) based on the WIM data (from several states across the United States) at different lifetimes. Soto *et al*. (2015) assessed live load factors by using WIM data for Mexican Highway bridge design. Nowak (1999) used Canada Ontario truck WIM for live load factor analysis. In Nowak (1999) it was assumed that the economic lifetime for newly designed bridges was 75 years and the maximum values of live loads and environmental loads were extrapolated to 75 years from an available Ontario truck database collected in 1972.

Similar to the study that is presented in this paper, as defined above some researchers used specific bridges in Switzerland, Mexico, Canada and in the USA for analyzing WIM data applications (D'Angelo and Nussbaumer 2015, Soto *et al*. 2015, Nowak 1999, Pelphrey *et al*. 2008). A method was proposed for calculating site-specific load factors, using truck weight data from WIM sites in (AASHTO 2003). The format used in the derivation of live load factors contained in the specifications of (AASHTO 2012) was followed. The jurisdictional and enforcement characteristics of Oregon, the modifications used to describe the alongside truck population based on the unique

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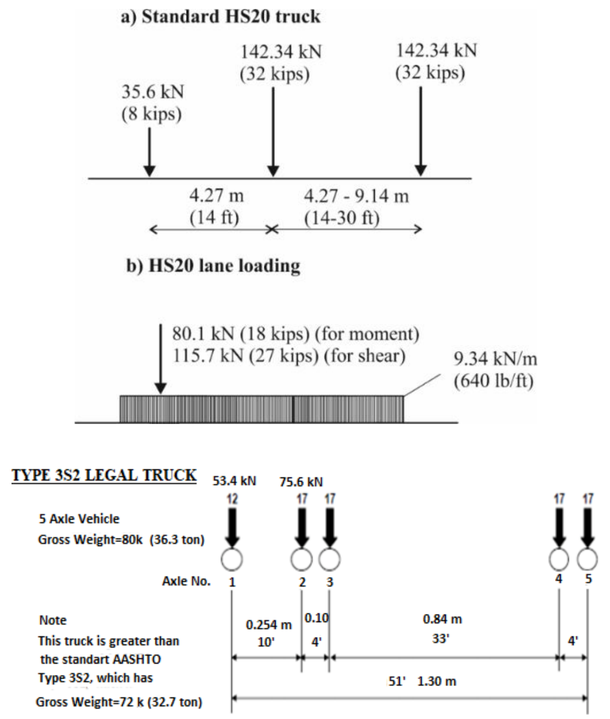

Fig. 1 HS20 live load in AASHTO and Oregon 3S2 Truck

truck permitting conditions in the state, the WIM data filtering, sorting, and quality control, as well as the calibration process, and the computed live load factors, were presented in by (Pelphrey *et al*. 2008). Another example application of obtaining lane load factors by using WIM data was demonstrated by (Zhou *et al*. 2018). An important recent study on bridge vehicle load based on WIM usage was carried out by (Chen *et al*. 2018). Anitori *et al*. (2017) mentioned that existing AASHTO live load models may not produce accurate results when used in association with advanced finite-element analyses of bridge structures and proposed a procedure for calibrating appropriate live-load models for multi-girder bridges. Another interesting study investigating dynamic load allowance factors for highway bridges was conducted by (Zhou *et al*. 2015). A research similar to this study on account of WIM consideration, uses one entire year WIM of State of Wisconsin for evaluating extreme load effects on bridges due to different truck classes, was presented by (Tabatabai *et al*. 2017).

In this study, live load factors and load effects were calculated using the WIM data by implementing statistical and probabilistic approaches. New simple modifications on these approaches were defined. WIM trucks were converted to load effects on an equivalent HS20 frame for moments and shears. WIM data were analyzed from four different sites. The key criterion for the selection of sites was average daily traffic data ADTT. Analyses for each (ADTT) volume category were performed. One full year of WIM data from the 4 sites located throughout Oregon were used in this study. Load effects were calculated for bridge spans ranging from 9.14 to 60.96 m (30 to 200 ft). To minimize the uncertainty in projecting the upper tail of the load effects, one year of WIM data were used for the live load factor computations. The live load factors were computed for ODOT rating vehicles ODOT (2019a). Load effects results were put into AASHTO 3S2 equivalents defined in AASHTO (2012). The top 20 % of the load effects were taken into account in the calibration. Live load factor calibration considered a full year of data for each WIM site. Extrapolations to 5 years of recurrence were taken into account. A simple future load prediction method is defined. The approach defined in Section 2.4 of this paper is very simple to use and projects the future loads efficiently. The analysis methods used to determine the statistical load effects, site-specific live load factors, and the resulting live load factors based on Oregon WIM data are presented below.

Table 1 WIM sites, locations, and ADTT

Corridor	Site Location	Site Designation	ADTT	ADTT % of ADT
I-5	Woodburn NB	WBNB	5550	13%
US97	Bend NB	BNB	607	8%
OR58	Lowell WB	LWB	581	7%
I-84	Emigrant Hill WB	EHWB	1786	36%

2. Live load factors analysis and methodology

WIM trucks were converted to load effects on an equivalent HS20 frame for moments and shears in this section. HS20 loading scheme is given in Fig.1. Oregon 3S2 truck configuration is also shown in Fig. 1 ODOT (2019b). Details of the live load factor methodology is given below.

2.1 Selection of oregon sites and wim data

WIM data collected at 4 sites in Oregon with different ADTT values were used in this paper. Seasonal and data collection windows were included. Four highways/interstates from Oregon were used that collected WIM data. These highways/interstates are US97, I-84, I-5 and OR58. The ADTT values for shorter WIM data windows ranged from 581 to 5550 for the sites, as shown in Table 1. In addition, the fraction of truck traffic relative to average daily traffic is given in Table 1. ADT (Average Daily Traffic) values are shown in Table 1 as well.

Four different time periods were considered in the calibration process:1 month, 2 months, 6 months and all year data. The results were compared and one year of data was used for each WIM site to eliminate possible seasonal influences and minimize errors in projection of the maximum evaluation period load effects. The raw WIM records were provided in text format and were subsequently processed. The raw data were carefully sorted, cleaned and filtered. For developing live load factors data classification and sorting is very important. The total number records for each site for the full year of data, along with their corresponding Oregon Department of Transportation (ODOT) Motor Carrier Transportation Division (MCTD) weight table classification (Table 1 to Table 5 and Table X in MCTD) are shown in Table 2. Table 2 of this paper has a second row which has the labels of Table 1 to Table 5 and

Table 2 Details of WIM records and numbers of records in ODOT weight tables

Site	All Year Data	Table 1	Table 2	Table 3	Table 4	Table 5	Table X
I-5 Woodburn NB	1,787,624	1,589,458	138,821	57,498	1090	117	639
US97 Bend NB	193,121	166,029	7247	19,363	175	60	244
OR58 Lowell WB	245,796	228,876	15,340	1425	98	3	54
I-84 Emigrant Hill WB	601,677	554,825	38,238	8447	82	3	82

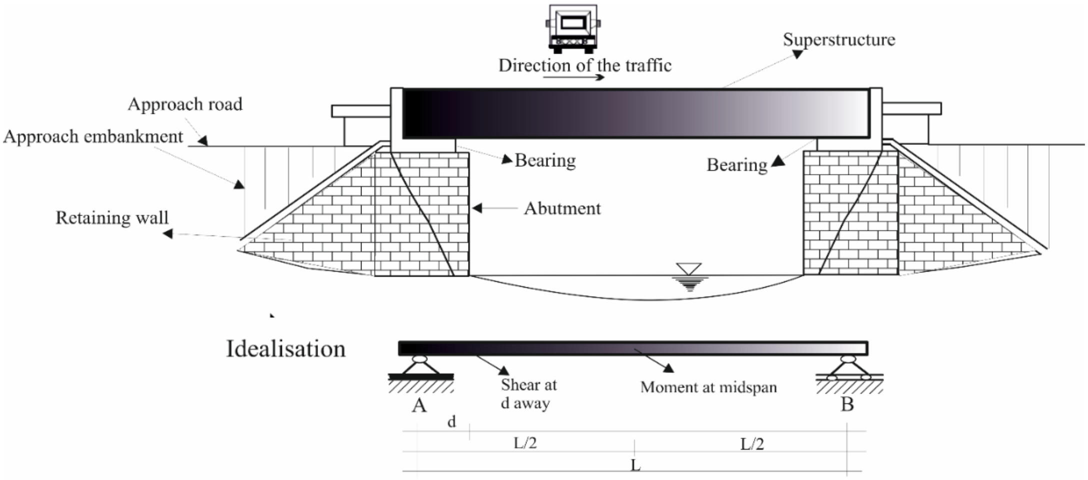

Fig. 2 Simple span bridge idealization and the locations of shear and moment

Table X. Table 1 to Table 5 and Table X represent the trucks with different weight characteristics that are defined in ODOT manuals. More information on these weight tables can be obtained in ODOT (2019a). Actual ADTT values of the yearly data from each site used in this study are also presented in Table 2. The calculated number of permitted vehicles per day is also shown in Table 2. Statistics were generated as defined considering the GVW for the ODOT rating truck and the alongside truck population from the WIM data. Only the top 20% of the truck weight data were considered to be consistent with the projection of the upper tail of the weight histogram. ODOT rating vehicles are shown in Table 3 ODOT (2019a). These rating vehicles were used for the determination of live load factors.

2.2 Calibration approaches for live load factors

In the first approach, Nowak (1999) used data from a truck survey performed in 1975 by the Ontario Ministry of Transportation. However, the data used in this study are WIM data collected over a year period from the four different sites as previously described. HS20 loading was used in the LRFD live load calibration. All WIM truck loads were converted into HS20 equivalents. Girder distribution factors were calculated to obtain the load effects in interior girders. Bias factors were computed for HS20 design loading considering a two-lane loaded condition. HS20 equivalent load effects were computed for an evaluation period of five years and used in the live load factor calibration procedure. In the second approach, live load factors were calibrated using the MBE approach. The live factors were calculated for ODOT rating vehicles. The load effects from the WIM truck population were converted into equivalent 3S2 GVW. Two lane loaded cases were chosen as the controlling case during calibration process. Reliability indices which were computed by Yanik and Higgins (2019) were taken into account as the reliability levels of the computed live load factors in this study. Yanik and Higgins (2019) used the same WIM data that were used in this study, and also considered HS20 loading with five years of an evaluation period. During the live load factor calculations, the load effect for truck in the first lane was considered as the ODOT rating truck load effect (converted to 3S2 equivalent). The load effect of the truck in the second lane (alongside truck) was computed from the entire truck population including STPs (converted into 3S2 equivalents).

2.3 Maximum truck load effect

Moving load analyses were performed for each truck WIM record, using the individual axle weights and relative spacing. Static elastic analyses were performed to determine the truck induced load effects. The truck axles were incrementally moved across a line element bridge model in 1,000 uniform increment (from the steer axle located on the near abutment to the last trailer axle leaving the far abutment). At each position, the moment and shear responses at selected locations on the bridge models were computed. For this paper, moment values were calculated at mid-span (positive moments) or over continuous support locations (negative moments) and shear values were calculated at a distance L/10 from the support, where L is the span length. Spans ranged from 6.1 to 76.2 m (20 to 250 ft) with 3.05 m (10 ft) increments. Both simple and two-span continuous models were used in the analyses. The locations where load effects were calculated, are shown schematically for simple span bridges in Fig 2. The same locations are simply shown for two-span continuous bridge in

Table 3 ODOT rating vehicle classifications

Rating Vehicle	Live Load Factor Designation	GVW (tons and kips)	Rating Vehicle	Live Load Factor Designation	GVW (tons and kips)
Legal Type 3	Oregon Legal Loads	22.7 - 50	OR-STP-4A	STP-4A	44.9 - 99
Legal Type 3S2		36.3 - 80	OR-STP-4B	STP-4B	83.9 - 185
Legal Type 3-3		36.3 - 80	OR-STP-5A	STP-5A	68.3 - 150.5
OR-CTP-2A	CTP-2A,2B	47.9 - 105.5	OR-STP-5B	STP-5B	73.7 - 162.5
OR-CTP-2B		47.9 - 105.5	OR-STP-5C	STP-5C	117 - 258
OR-CTP-3	CTP-3	44.5 - 98	OR-STP-5BW	STP-5BW	92.5 - 204
OR-STP-3	STP-3	54.7 - 120.5			

Table 4 Simple span load effects for HS20 loading

Span (m and ft)	MSHT (kN)	MSHL (kN)	CS (kN)	RS (kN)	MMHT (kNm)	MMHL (kNm)	CM (kNm)	RM (kNm)
9.1- 30.0	190	158.4	190	220.6	67.2	352.5	280.7	427.1
18.3-60.0	238.4	201.1	238.4	270.5	82.4	1084.7	756.5	1094.1
27.4-90.0	254.9	243.8	254.9	286.9	87.5	1816.8	1427.7	1822.2
36.6-120.0	263.3	286.5	286.5	295.4	90.0	2548.9	2294.0	2553.0
61 - 200.0	273.1	400.3	400.3	400.3	122.0	4501. 3	5558.9	5558.9

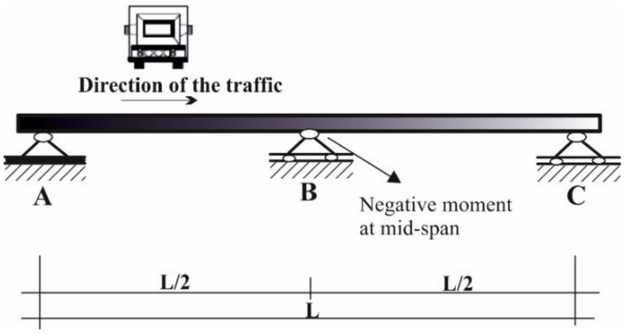

Fig. 3 Negative moment location for 2-span continuous bridge analysis

Fig. 3. The moments and shears calculated for each WIM truck were converted to an equivalent HS20 (using the 32.7 tons or 72 kips GVW reference) truck or lane load effect, whichever governed for the span length considered by dividing it with the HS20 reference value as:

$$\text{Moment: } \varphi_{iM} = M_{T,i}/M_{HS20}\ ;\ \text{Shear: } \varphi_{iV} = V_{T,i}/V_{HS20}$$
$$\text{Negative moment; } \varphi_{iM^-} = M^-_{T,i}/M^-_{HS20} \tag{1}$$

where $M_{T,i}$, $V_{T,i}$ and $M^-_{T,i}$ are the simple span moment, simple span shear, and continuous span negative moment for the i-th truck. M_{HS20}, V_{HS20} and M^-_{HS20} are the corresponding moment, shear and negative moment of the controlling HS20 reference load effect. The reference load effect values for the HS20 along with the values reported in Nowak (1999) are shown in Table 4 for simple spans (Abbreviations in Table 4 are; MSHT: maximum shear of HS20 truck loading, MSHL: maximum shear of HS20 lane loading, CS: controlling shear, RS: reported Shear in Nowak (1999), MMHT: Maximum moment of HS20 truck loading, MMHL: max. moment of HS20 lane, CM: controlling moment, RM: reported moment by Nowak (1999). The calculated load effects from the WIM data, divided by the reference HS20 load effects produced WIM load effect ratios and these were sorted into ascending order. The mean values and COVs (coefficient of variation) were computed and the data were used to project the expected maximum loading event for each WIM site.

2.4 Projection of maximum loading event with a simple approach

Using the sorted WIM load effect ratios computed above the inverse standard normal distributions (the ordinate of the extrapolated load effect curves) were calculated. The load effect was taken as a random variable with a normal distribution defined by the mean (μ_x) and standard deviation(σ_x) of the population. The maximum loading event depends on the number of possible events over the evaluation period. The probability for the i-th truck load can be written as

$$P_i = i/(N+1) \tag{2}$$

where N is the total number of trucks in the population. The probability can be transformed to inverse standard normal distribution as follows:

$$z = \Phi^{-1}(P) \tag{3}$$

where Φ^{-1} is the inverse standard normal distribution function. In this study, the built-in statistical functions in Microsoft Excel and Matlab software were used to obtain inverse standard normal distributions. Many researchers have studied and used different models for fitting WIM survey data to project the future maximum load effect value. Nowak (1999) used an approximate method to project the upper tail of the inverse standard normal distribution curves. Ditlevsen (1998) and Nowak (1993) considered global linear regression models. Caers and Maes (1998) used tail portion linear regression models in future maximum value projection. Tail projection procedures and comparison of different estimation methods were presented by (Fu *et al*. 2006). They expressed that, when there is a significant curvature in the tail, linear regression models will not work satisfactorily. The 75-year maximum load effect using extreme value theory (Gumbel distribution model) was projected by (Kwon *et al*. 2011). In their study, they discussed that the extreme value theory was used for projection as the method was more systematic than the normal probability plot and provides consistent and conservative projected values.

As described above, although there are several available models for projection there is not any rule or systematic method to determine future load effects. In this study, a new simple projection approach was used. The method considered tail linear, tail-nonlinear, and polynomial models as projection models. The reason to use three different estimation models is that the inverse standard normal distribution function curves of moment and shear data have slightly different characteristics and one method does not fit all. Nonlinear power first degree functions were used by concentrating on the tail portion as nonlinear models. The maximum order for the polynomial model was three in the present work. The critical points (when the tail of the curve has a sharp bend with high curvature which define the tail portion), were also taken into account during the projection procedure. The top 50% of the monthly data and the top 5% of the two months to all year data of the truck moment or shear load effect ratios within each category were considered to be consistent with the projection of the upper tail of the curves used by Nowak (1999). The available WIM data were extrapolated using the methods defined above to determine the maximum expected load effects for various periods of time up to 75 years. The number of trucks, N, in 75 years will be 900, 450, 150 and 75 times larger than the one month, two months, six months and one year WIM data, respectively.

In this new simple approach, three alternative projected curves were applied to the upper tail: nonlinear, linear, and a second degree polynomial. For each projected curve throughout this paper, the best projected curve was chosen from the three alternatives in order to represent the future load effects in 5 and 75 years. The projection procedure was repeated for each WIM site, each span, and each load effect (moment and shear). All year data results for each site are given in this section. The results of I-5 Woodburn NB and I-84 Emigrant Hill WB are presented in Figs. 4 and 5 for moment and shears (figures on the left side of the page represent moments while figures on the right side represent shears). The mean maximum moments and shears corresponding to various periods of time can be read from the graph. For OR58 Lowell WB, and US97 Bend NB WIM sites, all year WIM data were used to compute the maximum truck moment and shear calculations. However, the results are not shown here. Additionally, in this paper, because of space limitations, only simple span and continuous span results for Woodburn NB WIM data are given in the following part. However, the conclusions derived from the projection curves by using 1 & 6 months of WIM data for Emigrant Hill, Bend and Lowell NB traffic sites are presented in the following paragraphs. The same comparison for all year WIM data from US97 Bend, I-84 Emigrant Hill, and OR58 Lowell were carried out, but not given in this paper because of space limitations. It is obtained that, the inverse normal standard distribution values for one year, five years and 75 years for Woodburn NB site were higher than those in Nowak (1999). These values correspond to the actual ADTT of 4898, while the data used in NCHRP 368 was 9,250 trucks (2 weeks Ontario data) which correspond to an ADTT of approximately 1000. On the other hand, for Bend NB WIM data, it was observed that the *z* scores for 75 years, 5 years and 1 year are close to the ones described by Nowak (1999) because US97 Bend NB has an ADTT of 529 closer to the Ontario data.

To analyze the influence of considering two different data evaluation methods on the results, two different truck data population were used in the calculation procedure. These truck data populations were; top 20% of the truck weight data and top 20% of the load effect data (moment or shear) for each month. The controlling moments and shears for HS20 truck are given in Table 5. Table 5 also presents the HS20 moments and shears that were presented by Nowak (1999). All the moments and shears were divided by the corresponding HS20 moment (or shear) for spans from 6.1 to 76.2 m to obtain HS20 equivalent load effects. The mean HS20 equivalent shear ratios were also investigated in a comparative way for the top 20% of the weight data and top 20% of the load effects.

2.5 Bias factors

The maximum projected live load effect bias factors relative to the HS20 design load were calculated as:

$$\lambda_{HS20} = \mathrm{M}(5) / \mathrm{M}(\mathrm{HS20}) \tag{4}$$

where M(5) is the five year projected truck live load effect (simple span moment, simple span shear, or continuous span negative moment) while M(HS20) is the live load effect from the HS20 loading.

The results for ADTT=5,000 are shown in Fig. 6 for simple span moment (upper figure) and continuous span negative moment (lower figure). Emigrant Hill data and the bias factors for US97 Bend data for simple span moments and shears are not shown in this section, although they were used for calculating live load factors presented in the next sections of this paper. For the Woodburn NB WIM data, bias factors for an evaluation period of 75 years were

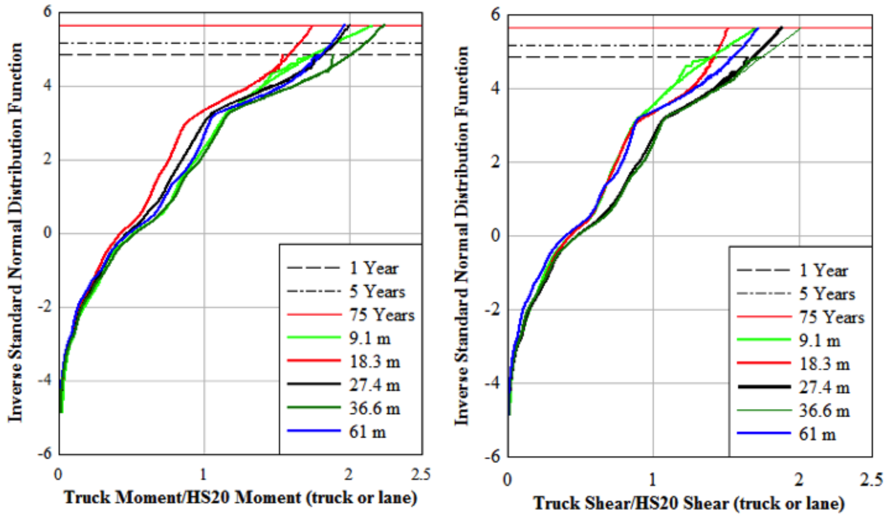

Fig. 4 Moments and shears for simple spans, all year WIM data for I-5 Woodburn NB.

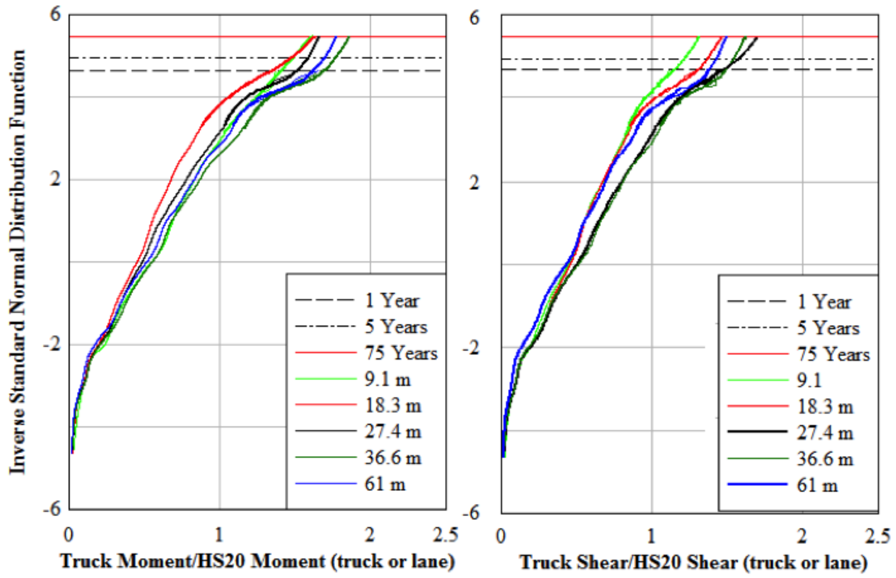

Fig. 5 Moments and shears for simple spans, all year WIM data for I-84 Emigrant Hill.

computed for simple span moment, shear, and negative moment and were compared with those obtained by Nowak (1999). The biases for 75 years of moment and shear for HS20 loading are similar to each other in both studies. However bias factors for negative moments show some differences for longer spans.

2.6 Analysis for two lane loaded case

To analyze the two lane loading case, the distribution of truck load to interior girders must be calculated. To obtain the maximum load events in multi-girder bridges, girder distribution factors are needed. In this paper, girder distribution factors were computed using the formulas obtained by Zokaie (2000). The formula which was proposed by Zokaie (2000) and (Zokaie *et al*. 1992) calculates girder distribution factor as a function of girder spacing. The formula for moment in interior girders (non-composite steel girder, reinforced concrete T-beam, and prestressed concrete girder) for bridges designed for two or more traffic lanes is expressed as:

$$GDF_{2M} = 0.15 + (s/3)^{0.6}(s/L)^{0.2} \quad (5)$$

while the distribution factor for the same bridge types for one traffic lane loaded is taken as:

$$GDF_{1M} = 0.1 + (s/4)^{0.4}(s/L)^{0.3} \quad (6)$$

where s is the girder spacing and L is the span length. For shear, the distribution factor for two lanes loaded case can be expressed as $GDF_{2S} = 0.4 + (s/6) - (s/25)^2$, and for the bridges designed for one traffic lane loaded case, the distribution factor can be expressed as $GDF_{1S} = 0.6 + (s/15)$. Girder distribution factors were calculated for spans of 9.1, 18.3, 27.4, 36.6 and 61 m and five different girder spacing (1.2, 1.8, 2.4, 3.1 and 3.7 m). The formulas for moment/shear distribution to interior girders in one lane loaded cases in AASHTO (2012) include multiple presence factor which is 1.2. The moment/shear distribution factor equations for interior girder developed by

Table 5 Simple span moment and shear for HS20 loading

Span (m)	M (HS20) (kNm) Controlling	(Nowak 1999)(kNm) M (HS20)	V (HS20) (kN) Controlling	(Nowak 1999)(kN) V (HS20)
6.1	216.9	245.4	160.1	185.0
9.1	352.5	427.1	189.9	220.6
12.2	596.6	610.1	213.5	245.5
15.2	806.7	851.5	234.9	260.2
18.3	1084.7	1094.1	238.4	270.5
21.3	1328.7	1336.8	248.2	277.6
24.4	1572.8	1579.5	252.7	282.9
27.4	1816.8	1822.2	254.9	286.9
30.5	2060.8	2066.3	258.0	290.5
33.5	2304.9	2310.3	272.2	293.1
36.6	2548.9	2553.0	286.5	295.4
39.6	2793.0	2797.1	300.7	300.7
42.7	3037.0	3041.1	314.9	314.9
45.7	3355.7	3355.7	329.2	329.2
48.8	3752.9	3752.9	343.4	343.4
51.8	4171.9	4171.9	357.6	357.6
54.9	4612.5	4612.5	371.9	371.9
57.9	5074.8	5074.8	386.1	386.1
61.0	5558.9	5558.9	400.3	400.3
64.0	6064.6	-	414.6	-
67.1	6592.0	-	428.8	-
70.1	7141.1	-	443.0	-
73.2	7711.9	-	457.3	-
76.2	8304.4	-	471.5	-

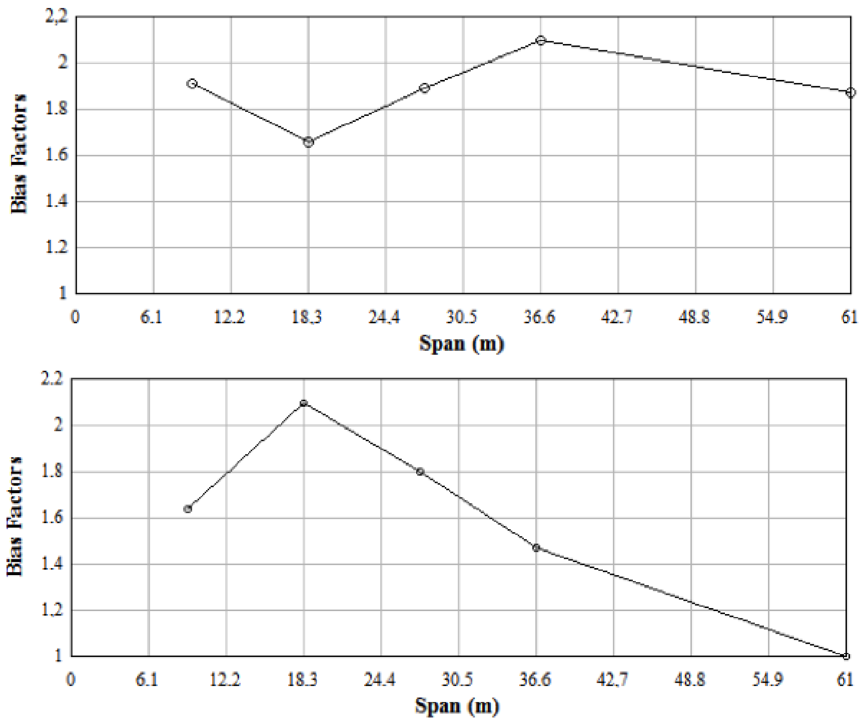

Fig. 6 Bias Factors for ADTT:5000 (Woodburn NB).

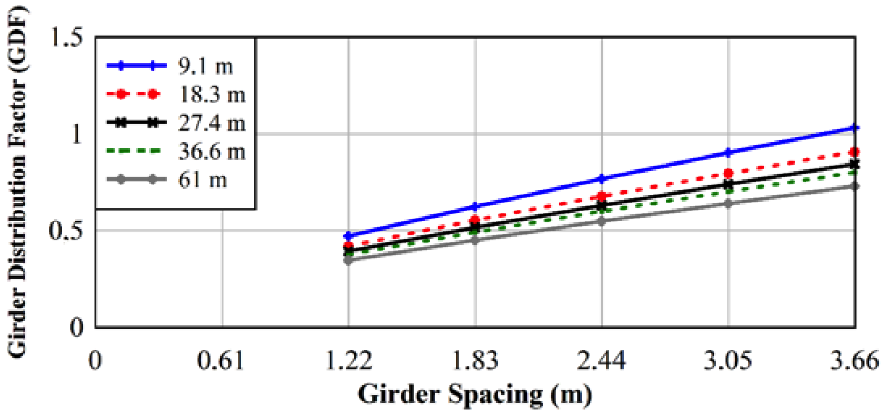

Fig. 7 Girder distribution factors of moment for two or more traffic lanes.

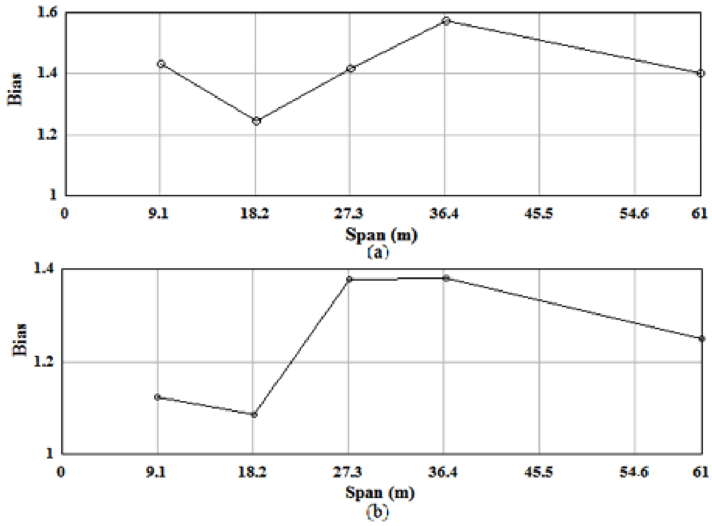

Fig. 8 Live load bias factors for two lane shear per girder (ADTT=5,000).

Zokaie (2000), have multiple presence factors of 1.0. Another point which must be clarified is the formulas which were developed by Zokaie (2000) that presented above give the wheel distribution factors. In this study, the lane distribution factors were calculated and used. Hence, the girder distribution factors that have been computed by the equations given above were divided by two to produce the lane fractions. The girder distribution factors that were calculated for moments are presented in Fig. 7 for a bridge designed for two or more traffic lanes. Girder distribution factors for shear are span independent. Girder distribution factors for a bridge designed for one traffic lane and for shear were calculated and used in the calculation of live load factors.

3. Live load factors

Live load factors were estimated using the following equation

$$g_{LL} = \lambda_{LL}(1 + k\, V_{LL}) \tag{7}$$

where λ_{LL} is the two lane loaded bias factor that can be calculated using Eq. (7), k is a constant value which was used as (=2), V_{LL} is the COV of the live load (with impact).

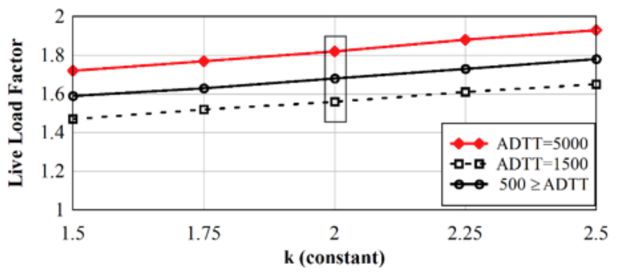

Fig. 9 Largest live load factors found for HS20 loading considering a 5 years of evaluation period

For this study, the COV of live load (with impact) was considered as 0.15. The bias factors for two lane loaded cases are given in Figs. 8 a&b for moment and shear, respectively for the ADTT=5,000 case.

In addition to the bias calculations for ADTT=5,000, live load bias factors for two lane moments and shears for ADTT=1500 and ADTT $\leq$ 500 cases were also calculated. These factors were obtained by using Emigrant Hill and OR58 Lowell WIM data, which produced higher factors than US97 Bend data. The graphical representations are not shown here, while the biases were used to calculate the live load factors. Using the bias factors calculated in this section

Table 6 Live load factors (k=2)

ADTT	Load Factor	COV	Average Bias Factor
5,000	1.60-1.80	0.15	1.24-1.40
1,500	1.40-1.56	0.15	1.07-1.27
≤500	1.56-1.68	0.15	1.10-1.30

for the three different ADTT levels and using Eq. (7), the live load factors were calculated. The live load factors are given in Fig. 9. Fig. 9 was composed for different k constant values.

For k=2 values the results are presented within a rectangular dashed box. The most conservative live load factors are shown in Fig. 9. The variation of the live load factors is presented in Table 6. As seen here, the live load factors for the lowest volume sites are larger than the intermediate site due to differences in the projection of the maximum expected load event based on the data in the upper tails. For the ADTT case of 5000, based on the live load factors computed, the 1.8 factor for HS20 loading (controlled by bending) would produce reliability index (β) values on average of 3.45 and a minimum β of 2.75 (Yanik and Higgins, 2019). More information on the selection of k constant parameter and reliability indices can be obtained from Yanik and Higgins (2019).

Because HS20 produces higher load effects compared to 3S2, this would correspond roughly to a load factor of 2.22 for 3S2 loading (based on the average ratio of HS20 moment/3S2 moment for the span lengths considered). To produce the level of reliability closer to that in AASHTO (2008) calibration of β of 2.5, this would need to be multiplied by approximately 0.76. Thus, to produce a target β of 2.5 for the present WIM data from the ADTT=5000 site, the live load factor for 3S2 loading should be 1.7. This is lower than the 1.8 used in the calibration in AASHTO (2008).

4. Live load factor calibration using another methodology

In this section, the live load factor calibration methodology that was developed by Moses (2001) was followed. The following sections describe the methods used to determine the site-specific live load factors based on WIM data and the resulting live load factors.

4.1 Load effects

The WIM truck load effects (moment and shear) were converted to those on an equivalent 3S2 frame. The WIM GVW truck weight was converted to an equivalent 3S2 GVW by dividing the WIM load effect on the span considered by the reference load effect from the 72 kip GVW 3S2. For these results, the mean and standard deviation of top 20% GVW were computed. The side-by-side probabilities considered in Moses (2001) were used and load effects were extrapolated to recurrence of five years. The same WIM sites described in the previous sections of this paper were analyzed. Similarly, all year WIM data of each site were used in the calibration procedure. The 3S2 equivalent moments and shears for simple spans were calculated for two different cases. The first case includes all trucks, including Legals, CTPs and STPs. The second loading case includes the truck population from legal trucks (Weight Table 1 of ODOT Rating Vehicle Classifications), Extended Weight Table 2 (47.85 tons maximum), and 44.45 tons continuous trip permit trucks (CTPs) from Weight Table 3 as given in ODOT (2019a). The legal and continuous trip permit truck statistics for each WIM site are given in Table 7. Permit trucks per day are also included in Table 7. Number of permit trucks per day were calculated as:

$$N_{P,\mathrm{day}} = (N_{all} - N_P)/365 \tag{8}$$

where N_{all} is the number of all trucks in the truck population for all year data and N_P is the number of legal and continuous trip permit trucks. Table 7 also presents the number of top 20% of the total and legal and continuous trip permit truck population. The previous sections of this paper investigated HS20 loading for live load calibration. This section of the paper considers the 3S2 truck, with 32.66 tons (322.27 kN or 72 kip) GVW as the reference vehicle, in the calibration procedure. The load effect ratios between HS20 loading and 3S2 truck loading are shown in Table 8 for the various spans that were analyzed in this section.

4.2 Live load factor methodology

Site specific load factors calculation using truck weigh data from WIM sites that follow the format used in the LRFD specifications were presented in AASHTO (2008). This is to determine the statistics associated with the 3S2 truck population, to characterize the uncertainty associated with the alongside truck. In the study of (Pelphrey *et al*. 2008), the maximum loading event for calibration of live load factors for two lane loaded case considered the alongside truck population as Weight Table 1, Weight Table 2 and CTPs from Weight Table 3 trucks. One critique of this approach that was raised was that alongside truck population should not just be restricted to Legal and CTPs. Therefore, in this paper, two different alongside truck loading cases were taken into account as the maximum loading event. These cases are; Alongside truck population as Legals (Table 1 and 2), CTPs (from Table 3), and random truck population as alongside trucks (included STP). The maximum loading event for case 1, for the live load calibration is illustrated in Fig. 10, and the maximum loading event for case 2, is presented in Fig. 11. The main distinctions between the live load calibration methodology in this paper and Moses (2001) are; In Moses (2001), for legal loads, there is a variability in both lanes, in this paper both lanes are the same. In Moses (2001) for permit trucks, variability is only in the alongside truck population while this paper was probabilistic in both lanes. Moses (2001) uses higher side-by-side probability varies with ADTT (1/15, 1/100 and 1/1,000), in this paper side-by-side probability of 1/30 was used for all different ADTT levels. Moses (2001) considers Np*Ps/s (side-by-side probability)

Table 7 Legal and permit truck statistics for WIM sites

	I5 Woodburn NB	US 97 Bend	OR58 Lowell	I-84 Emigrant Hill
Total Truck Number	1,787,624	193,118	245,796	601,677
Legal + Continuous Trip Permits	1,748,445	185,637	243,372	597,118
Permit Trucks	39,139	7,481	2,424	4,559
Permit Trucks per Day	107	20	7	13
Total Top 20 %	357,525	38,624	49,159	120,335
Legal +CTP Top 20 %	349,689	37,127	48,674	119,424

Table 8 Load effect ratios for HS20 loading and 3S2 truck

Span (m)	M_{HS20} (kNm)	M_{3S2} (kNm)	M_{HS20}/M_{3S2}	V_{HS20} (kN)	V_{3S2} (kN)	V_{HS20}/V_{3S2}
9.1	352.5	299.6	1.17	189.9	132.6	1.43
18.3	1084.7	816.2	1.33	238.4	189.0	1.26
27.4	1816.8	1548.3	1.17	254.9	221.5	1.15
36.6	2548.9	2279.1	1.12	286.5	238.4	1.20
61	5558.9	4232.9	1.31	400.3	258.4	1.55

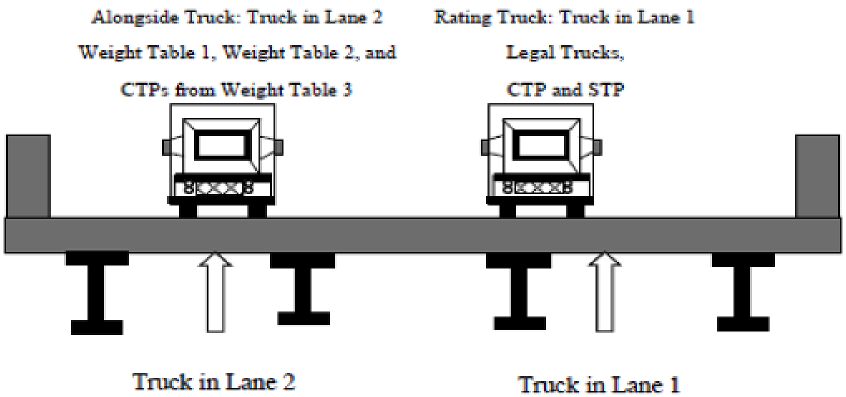

Fig. 10 Maximum loading event calibration for live load factors (Case 1).

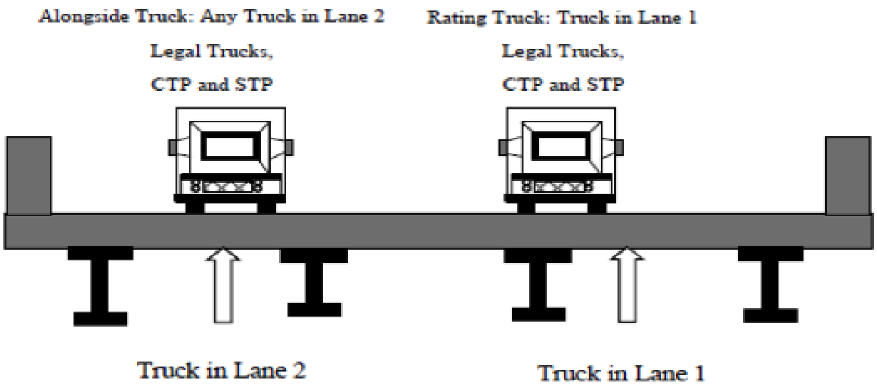

Fig. 11 Maximum loading event calibration for live load factors (Case 2).

as the number of events, this research used Np as the number of events. ODOT uses a suite of 13 trucks to represent the permit categories for rating purposes.

These trucks were presented in Table 3 of this paper. In Figs. 10 and 11, the truck in Lane 1 is the rating truck from ODOT classifications and the live load factor methodology explained below. For more information of side by side probability calculations and loading cases, please refer to Yanik and Higgins (2020). Live load factor for rating can be presented with the following formula

$$g_L = 1.8(W_T / 240)\,(72 / W) \tag{9}$$

where W is the GVW of the vehicle (legal truck or permit truck with units of kips), and W_T is the expected maximum total weight of rating and alongside vehicles. In this paper, W is the 3S2 equivalent weight of the legal truck or permit truck. Total weight W_T can be expressed as

$$W_T = R_T + A_T \tag{10}$$

where R_T =rating truck and can be calculated for legal loads as

$$R_T = W^* + t_{ADTT}\ s^*_{3S2} \tag{11}$$

or for permit loads as

$$R_T = P + t_{ADTT}\ s^*_{along} \tag{12}$$

where W^* is the mean value of top 20 % of legal trucks taken from the 3S2 population, σ^*_{3S2} is the standard deviation of the top 20 % of legal trucks, P= weight of permit truck, and σ^*_{along} is the standard deviation of the top 20 % of the alongside trucks. The alongside truck, A_T, is computed for legal loads as

$$A_T = W^*_{along} + t_{ADTT}\ \sqrt{2}\ s^*_{along} \tag{13}$$

where W^*_{along}= mean of the top 20% of alongside trucks and t_{ADTT} is the fractile value corresponding to the number of side-by-side events. In this study, there are two legal trucks in both lanes for the legal loads case. Therefore $\sqrt{2}$ comes from Equation 14 of Moses (2001), which can be written as, $\sigma_{W_T} = \sqrt{2}\sigma_{W_1}$, where σ_{W_1} is the standard deviation of the trucks in both lanes (standard deviation of 3S2 equivalent alongside truck in this study). For routine and permit loads, A_T can be calculated as

$$A_T = W^*_{along} + t_{ADTT}\ s^*_{along} \tag{14}$$

The number of side-by-side crossings is computed for legal trucks as; N(legals)= (ADTT) × (365 days/year) × (evaluation period) × ($P_{s/s}$) ×(% of record). And for permit truck, N is computed as; N(permits)=(N×(365 days/year) × (evaluation period) × ($P_{s/s}$). Here; N_P= number of observed single trip permits (STPs) in the WIM data extrapolated over the evaluation period, and $P_{s/s}$= probability of side-by-side concurrence. AASHTO (2008) and AASHTO (2012) calibrations assumed a 1/15 (6.7%), probability of side-by-side events for truck passages. This assumption was based on visual observations, and is conservative for most sites. The side-by-side probabilities that were used in the live load calibration are shown in Table 9.

Table 9 Side-by-side probabilities

ADTT	WIM Site	$P_{s/s}$
5,000	I-5 Woodburn	1/15 (6.7%)
1,500	I-84 Emigrant Hill	1/100 (1%)
≥ 500	US97 Bend	1/100 (1%)
≥ 500	OR58 Lowell	1/100 (1%)

As it can be indicated from the equations given in this section, GVW is considered in live load factor calibration defined by Moses (2001). The distinction of this study from Moses (2001) and AASHTO (2008) is, load effects were taken into account in the live load calibration procedure. Therefore, load effect results were put into 3S2 equivalent GVWs. 3S2 equivalent moments and shears were computed for two cases as described above. For obtaining the alongside truck, 3S2 equivalent GVW, the load effects were calculated and converted to 3S2 equivalents for all trucks in the first case. In the second case, the load effects were converted to 3S2 equivalents, for the population that consists of Weight Table 1, Weight Table 2 and CTPs from Weight Table 3 trucks. 3S2 equivalent load effects, for the two cases are analyzed in this paper. 3S2 equivalent load effects for all WIM sites have been analyzed throughout this paper. And all year WIM data for each site were used for this procedure. The graphics for these cases are not given here, despite the 3S2 equivalent load effects analysis was used in the conclusion section of this paper. The mean and standard deviation for 3S2 equivalent GVW (alongside truck GVW) were computed for moment as:

$$W^*_{along,M} = \mu_{M/M_{3S2}} \times 72\ ;\ \sigma^*_{along,M} = \sigma_{M/M_{3S2}} \times 72 \tag{15}$$

where $\mu_{M/M_{3S2}}$ is the mean of the top 20% of the 3S2 equivalent moment ratios, while $\sigma_{M/M_{3S2}}$ is the standard deviation of the top 20% of the 3S2 equivalent moment ratios. The mean and standard deviation for 3S2 equivalent GVW (alongside truck GVW) were computed for shear as:

$$W^*_{along,V} = \mu_{V/V_{3S2}} \times 72\ ;\ \sigma^*_{along,V} = \sigma_{V/V_{3S2}} \times 72 \tag{16}$$

where $\mu_{V/V_{3S2}}$ is the mean of the top 20 % of the 3S2 equivalent shear ratios, and $\sigma_{V/V_{3S2}}$ is the standard deviation of the top 20% of the 3S2 equivalent shear ratios. In Eqns. 15 and 16, (72) is the GVW of 3S2 truck in units of kips. This procedure was applied to all spans. The largest 3S2 equivalent GVW was obtained with the largest mean and standard deviation values. The resulting mean and standard deviation of the top 20% of the equivalent 3S2 load effect ratios are presented in Table 10 for all trucks. The statistical parameters are given for all truck population in Table 10 for all sites. μ and σ represent the resulting mean and standard deviation of the top 20% of the load effects. The results from I5 Woodburn NB site is presented in Table 11. Rating truck load effects were also converted to 3S2 equivalents for live load factor computations. 3S2 equivalent GVW with respect to moment of a rating truck was calculated as $W^*_M = (M_R / M_{3S2}) \times 72$. where M_R and M_{3S2} are the moment of the rating truck and 3S2 truck, respectively. 3S2 equivalent GVW with respect to shear of a rating truck can be written as (The United States

Table 10 3S2 Equivalent GVW statistics considering top 20% of load effects for all trucks

Site	μ (Controlling)	W^*_{along} (tons)	σ (Controlling)	σ^*_{along} (tons)
I-5 Woodburn NB	0.96	31.25	0.09	2.95
I-84 Emigrant Hill WB	0.85	27.85	0.11	2.99
US97 Bend NB	0.96	31.30	0.08	2.54
OR58 Lowell WB	0.89	29.17	0.08	2.68

Table 11 3S2 Equivalent GVW statistics considering Top 20% of the load effects for legal + continuous trip permits

Site	μ (Controlling)	W^*_{along} (tons)	σ (Controlling)	σ^*_{along} (tons)
I-5 Woodburn NB	0.95	30.89	0.08	2.45

Table 12 3S2 Equivalent GVW statistics for ODOT rating vehicles

Rating Truck Parameters				AVG 3S2 Equiv. GVW		Max		Permit Type
Rating Vehicle	Actual GVW	Length	Nominal	Shear	Moment		Equiv.	
ID	(tons)	(m)	GVW/3S2	(tons)	(tons)	Min/Max	GVW/3S2	
Type3	22.5	5.8	0.69	28.2	28.4	28.4	0.88	LEGAL
3S2	36.0	15.5	1.11	33.2	33.3	33.3	1.03	LEGAL
3-3	36.0	16.5	1.11	32.3	31.5	32.3	1.00	LEGAL
CTP-2A	47.5	25.0	1.47	36.0	37.4	36.0	1.11	ROUTINE
CTP-2B	47.5	23.0	1.47	37.3	36.0	36.0	1.11	ROUTINE
CTP-3	44.1	13.1	1.36	43.7	43.2	43.2	1.33	ROUTINE
STP-3	54.2	21.3	1.67	44.3	43.6	43.6	1.35	SINGLE
STP-4A	44.6	11.9	1.38	45.0	45.4	45.0	1.39	SINGLE
STP-4B	83.3	30.5	2.57	54.7	51.9	51.9	1.60	SINGLE
STP-5A	67.7	22.4	2.09	53.9	54.6	53.9	1.66	SINGLE
STP-5B	73.1	19.8	2.26	60.1	62.1	60.1	1.85	SINGLE
STP-5C	116.1	38.4	3.58	63.0	63.1	63.0	1.95	SINGLE
STP-5BW	91.8	30.2	2.83	61.2	58.3	58.3	1.80	SINGLE
3S2	32.4	12.5	1.00	32.4	32.4	32.4	1.00	LEGAL

Table 13 Live Load Factors for WIM sites (All traffic case for alongside truck)

ODOT Rating Vehicle	Load Factor						
ADTT	ADTT≥5,000		ADTT=1,500		ADTT≤500		
	I5 Woodburn NB	Oregon-specific	I-84 Emigrant Hill	Oregon-specific	US97 Bend NB	OR58 Lowell WB	Oregon-specific
Type3	1.47	1.4	1.35	1.35	1.35	1.32	1.4
3S2	1.33	1.4	1.23	1.35	1.24	1.21	1.4
3-3	1.35	1.4	1.25	1.35	1.26	1.23	1.4
CTP-2A	1.20	1.35	1.12	1.35	1.14	1.11	1.25
CTP-2B	1.20	1.35	1.12	1.35	1.14	1.11	1.25
CTP-3	1.09	1.45	1.02	1.4	1.04	1.01	1.3
STP-3	1.06	1.25	1.00	1.2	1.02	1.00	1.1
STP-4A	1.05	1.4	0.99	1.35	1.01	0.99	1.25
STP-4B	1.00	1	0.94	1	0.96	0.94	1
STP-5A	0.98	1.1	0.93	1.05	0.95	0.93	1
STP-5B	0.95	1.05	0.90	1.05	0.92	0.90	1
STP-5C	0.93	1	0.89	1	0.90	0.89	1
STP-5BW	0.96	1	0.91	1	0.93	0.91	1

customary units are used during these calculations and results are converted to SI units).

$$W_V^* = V_R / V_{3S2} \times 72 \tag{17}$$

here V_R and V_{3S2} are the shear of the rating truck and 3S2 truck in a respective way. The lengths, GVW, 3S2 equivalent GVW with respect to moment and shear and controlling equivalent 3S2 GVW are shown in Table 12 for ODOT rating vehicles with 3S2 truck (72 kips). The column heading for “Max” in Table 12 represents the equivalent 3S2 GVW ratios that were used in the live load factor calibration.

4.3 Live load factor results

The computed live load factor for all sites using all year data and for all ODOT rating vehicles are presented in Table 13 with the Oregon-specific values that are effective in ODOT policies. The live load factors in Table 13 were computed for case 2 that was presented in Fig. 11. Case 2 correspond to all traffic case for alongside truck calculations. The live load factors for Case 1 that correspond to continuous + legal trucks case for alongside truck calculations is given in Table 14. Case 1 is illustrated in Fig. 10. Table 14 shows the results for ADTT≥5,000 case. Table 14 shows the load factors for Type 3, Type 3S2,

Table 14 Live load factors for ADTT ≥5,000 (Continuous + legals for alongside truck)

ODOT Rating Vehicle	Load Factor		ODOT Rating Vehicle	Load Factor	
ADTT	ADTT≥5,000		ADTT	ADTT≥5,000	
	I5 Woodburn NB	Oregon-specific		I5 Woodburn NB	Oregon-specific
Type3	1.40	1.4	STP-3	1.04	1.25
3S2	1.28	1.4	STP-4A	1.03	1.4
3-3	1.30	1.4	STP-4B	0.98	1
CTP-2A	1.16	1.35	STP-5A	0.96	1.1
CTP-2B	1.16	1.35	STP-5B	0.93	1.05
CTP-3	1.06	1.45	STP-5C	0.92	1
			STP-5BW	0.94	1

Table 15 Computed Oregon-specific live load factors for permit loads and upper portion of AASHTO (2008) Table 6.A.4.5.2.a.1 values

Permit Type	Frequency	Loading Condition	DF	Permit Vehicle	Live load Factor γ_L by ADTT (one direction)					
					> 5,000		= 1,500		< 500	
					AASHTO (2008)	Or.-Spec.	AASHTO (2008)	Or.-Spec.	AASHTO (2008)	Or.-Spec.
Routine or Annual	Unlimited Crossings	Mix w/traffic (other vehicles may be on the bridge)	2 or more lanes	CTP-2A	1.75	1.20	1.58	1.12	1.45	1.14
				CTP-2B	1.75	1.20	1.58	1.12	1.45	1.14
				CTP-3	1.80	1.09	1.63	1.02	1.49	1.04
Single Trip	Route-Specific Limited Crossings	Mix w/traffic (other vehicles may be on the bridge)	2 or more lanes	STP-3	1.60	1.06	1.46	1.00	1.35	1.02
				STP-4A	1.80	1.05	1.63	0.99	1.49	1.01
				STP-4B	1.30	1.00	1.21	0.94	1.14	0.96
				STP-5A	1.30	0.98	1.21	0.93	1.14	0.95
				STP-5B	1.30	0.95	1.21	0.90	1.14	0.92
				STP-5C	1.30	0.93	1.21	0.89	1.14	0.90
				STP-5BW	1.30	0.96	1.21	0.91	1.14	0.93

Type 3-3 and lane loads. The generalized live load factors for the STRENGTH I limit state that were defined in AASHTO (2008) (Table 6A.4.4.2.3a-1 of MBE), are given in Table 15 for comparison purposes. Table 6A.4.4.2.3a-1 of AASHTO (2008) gives the load factors for ADTT=1,000 and ADDT≤100 traffic volumes. However, it recommends linear interpolation for other ADTT levels. Therefore, for ADTT=1,500 and ADDT≤500 traffic volumes, live load factors were obtained by linear interpolation in Table 15. The permit load factors from AASHTO (2008) are shown in Table 15 (Or.-Spec. abbreviation correspond to Oregon-specific in Table 15) with Oregon-specific live load factors that were computed in this paper (Case 2: All traffic for alongside truck population). It must be noted that AASHTO (2008) recommendation for routine permits between 100 kips and 150 kips, the load factor must be interpolated by weight and ADTT value. Hence the live load factors for ODOT CTP-2A, CTP-2B, STP-3 rating trucks, and ADTT levels of =1,500 and < 500, were interpolated in Table 15. Generalized live load factors are given in Table 16. As indicated from Tables 13 to 16, the computed live load factors in this study for ODOT rating vehicles, are lower than Oregon-Specific live load factors that were implemented by ODOT. These obtained live load factors are also lower than the values found in the AASHTO (2008) Permit Load Factors table ‘Table 6A.4.5.2a-1’, and for legal loads ‘Table 6A.4.4.2.3a-1’.

5. Conclusion

A very simple future load projection method is proposed along with two live load factor calibration case study in this paper. The load projection method is easy to fit and gives good results with different types of tails. The load factors were calibrated for HS20 design loads in the first part of this paper. All the findings in this study are applicable for an evaluation period of 5 years. If a researcher or designer uses the same methodology that has been explained in this paper and considers a different design load, different live load factors can be estimated. Additionally, if a researcher or designer considers a different evaluation period than 5 years, different live load factors can be obtained.

Main findings are listed below for HS20 loading calibration. The live load factor for ADTT=5,000 and AASHTO HS20 loading case and five-year evaluation period was 1.8. This would correspond to the live load factor of 1.7 in 3S2 equivalents relative to (Moses 2001), as the reference load factor, to convert LRFD factors for design to AASHTO (2008) factors, for five-year evaluation period with a target reliability index of 2.5. The findings obtained from live load calibration of ODOT rating vehicles, with respect to AASHTO 3S2 truck loading are given below. WIM trucks load effects were converted on equivalent 3S2 frame (either moment or shear) in the live

Table 16 Generalized live load factors, γ_L for routine commercial traffic

Traffic Volume (One Direction)	Load Factor for Type 3, Type 3S2, Type 3-3 and Lane Loads
Unknown	1.80
$ADTT \geq 5{,}000$	1.80
$ADTT = 1{,}500$	1.67
$ADTT \leq 500$	1.51

load factor calibration. In this study, alongside truck population also considered continuous trip permit vehicles and legal vehicles. The conservative factors were obtained with alongside truck population including all traffic. Therefore, the live load factors presented in this paper are the largest computed live load factors. The resulting Oregon-specific live load factors of ODOT rating vehicles were smaller than the national standard. Moreover, the computed Oregon specific live load factors were also smaller than the ones defined in ODOT policy. Therefore, no immediate changes are necessary for the suite of ODOT rating vehicles. Secondary findings obtained throughout this paper are; Considering top 20% of the load effects, and top 20% of the GVW, does not have a significant influence on the extrapolated moments. As a result, this fact does not have a big influence on load factors as well. The correlation between load effects and GVW is over 90 %. The quantity of the WIM data has distinctive effects on the results. The higher quantity of WIM is analyzed the more accurate results can be obtained. Therefore, one year of WIM data for each site was analyzed in this paper.

Acknowledgments

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