www.dergipark.gov.tr
ISSN:2148-3736
El-Cezerî Fen ve Mühendislik Dergisi
Cilt: 6, No: 3, 2019 (736-747)

El-Cezerî Journal of Science and Engineering
Vol: 6, No: 3, 2019 (736-747)
DOI :10.31202/ecjse.577400

ECJSE

Research Paper / Makale

Reliability Analysis of Oregon Bridges Using Weigh-in-Motion (WIM) Data

Arcan YANİK[a,b*], Christopher HIGGINS[a]

[a]Oregon State University, School of Civil and Construction Engineering, Corvallis, 97331, Oregon, USA.
[b]Istanbul Technical University, Department of Civil Engineering, Maslak, 34469, Istanbul-Turkey,
yanikar@itu.edu.tr

Received/Geliş: 13.06.2019 **Accepted/Kabul:** 29.07.2019

Abstract: In this study a reliability analysis was carried out for Oregon bridges. Different bridge types that are non-composite steel girder, reinforced concrete T-beam, and prestressed concrete girder bridges were considered. Reliability indices were calculated for various span lengths and girder spacing. Reliability indices were estimated for AASHTO HS20 design loading considering a five-year evaluation period. The actual average daily truck traffic (ADTT) values were used during the calculation. Four WIM sites from Oregon state were taken into account in the analysis. These sites correspond to three different levels of ADTT which are approximately ADTT= 5,000, ADTT = 1,500 and ADTT ≤ 500 cases. The distinction of this paper from the existing literature is the large amount of WIM data used in the analysis. The resulting reliability indices ranged from 2.4 to 6.4 for different span lengths, girder spacing, load effects (moment or shear), type of girder and ADTT level. These indices were compared with the existing ones presented in the literature.

Keywords: Bridges, Reliability analysis, Weigh-in-Motion, Traffic

Oregon Köprülerinde Hareket Halinde Tartma (WIM) Verileri Kullanılan Bir Güvenilirlik Analizi

Öz: Bu çalışmada Oregon köprüleri için bir güvenilirlik analizi gerçekleştirilmiştir. Çalışma kapsamında kompozit olmayan çelik kirişli, betonarme T kesit kirişli ve öngerilmeli beton kirişli köprüler göz önüne alınmıştır. Güvenilirlik indisleri, çeşitli kiriş açıklıkları ve aralıkları için hesaplanmıştır. Yüklemelerde Amerikan otoyol ve taşıma standartlarını belirleyen kurum olan AASHTO'nun HS20 yükü ve 5 yıllık değerlendirme zamanı kullanılmıştır. Çalışma kapsamında gerçek ortalama günlük trafik (ADTT) değerleri kullanılmıştır. Hesaplamalarda Oregon eyaletine ait dört tane trafik bölgesine ait hareket halinde tartma (WIM) verileri kullanılmıştır. Bu trafik bölgeleri dört adet ADTT değerine karşı gelmektedir. Bu durumlar ADTT= 5,000, ADTT = 1,500 ve ADTT ≤ 500 değerlerine tekabül etmektedir. Bu çalışmanın mevcut çalışmalardan farkı kullanılan WIM verilerinin oldukça fazla sayıda olmasıdır. Bir yıl boyunca toplanan WIM verileri kullanılmıştır. Analizler sonucunda güvenilirlik indislerinin farklı kiriş aralıkları ve açıkları için moment ve kesme kuvveti etkisine göre 2.4 ten 6.4 e kadar değiştiği görülmüştür. Bulunan değerler literatürde yer alan değerler ile karşılaştırılmıştır.

Anahtar kelimeler: Köprüler; Güvenilirlik Analizi; Hareket Halinde Tartma; Trafik

1. Introduction

Weigh-in-motion (WIM) devices are designed to capture and record the axle weights, and gross vehicle weights as vehicles drive over a traffic measurement site. WIM data are obtained from the measurement of these devices, during the traffic. On the other hand implementing statistical and probabilistic approaches in bridge engineering [1] subjects has been an interesting area of research in recent years [2-3]. In addition, recently bridge and transportation engineering researchers have

Bu makaleye atıf yapmak için
Yanik, A., Higgins, C., *"Oregon Köprülerinde Hareket Halinde Tartma (WIM) Verileri Kullanılan Bir Güvenilirlik Analizi"* El-Cezerî Fen ve Mühendislik Dergisi 2019, 6(3); 736-747.

How to cite this article
Yanik, A., Higgins, C., "*Reliability Analysis of Oregon Bridges Using Weigh-in-Motion (WIM) Data*" El-Cezerî Journal of Science and Engineering, 2019, 6(3); 736-747.

been using WIM data in their studies. Examples of these recent studies include; steel girder bridge analysis [4], accuracy determination of bridge WIM systems [5], axle detection of prestressed concrete bridges [6], investigation of health monitoring of railway bridges with WIM data [7]. State of the art review of bridge WIM research was presented in [8] and recent developments in bridge WIM was given in [9]. Portable bridge WIM research was carried out in [10]. The usage of bridge WIM for overweight truck enforcement was written in [11]. Determination of the effect of live load on simply supported bridges using WIM was presented in [12]. Investigation of the feasibility of using a single-span bridge as a WIM tool to quantify the gross vehicle weights (GVWs) of trucks with a small number of sensors and without using axle detectors was studied in [13].

Another important research related to the content of this paper is about probabilistic assessment of live load factors, by using WIM data for Mexican Highway bridge design [14]. As this study is related with statistic, probabilistic and reliability analysis, some recent research on these subjects, considering WIM data and bridges are given below. A probabilistic model for fatigue life prediction was suggested based on vehicle data from bridge WIM in [15]. The author also reviewed the reliability achieved with the Eurocode model, for fatigue assessment of road bridges. The truck-load characteristics were investigated based on long-term WIM data, and the typical truck types were extracted by [16]. Reliability analysis for serviceability limit state of bridges considering deflection criteria was given in [17]. 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 [17].

In this paper reliability indices for common Oregon Department of Transportation highway bridge types were obtained. These bridge types were non-composite steel girder (ncst), reinforced concrete T-beam (rctb), and prestressed concrete girder bridges (pcg). During the reliability analysis live load factor and statistical data obtained for four different WIM sites from Oregon [18], were used. One year of WIM data were used in the reliability index calculations. Next sections contain the load and WIM sites information, reliability analysis formulation and the results and conclusions obtained in this study.

2. Load Definition and WIM Sites

In this study WIM trucks were converted to load effects on an equivalent HS20 frame loading. HS20 loading scheme is given in Figure 1 [19].

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 of interest in Oregon were used which collected WIM data: I-5, I-84, OR58, and US97. 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.

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%

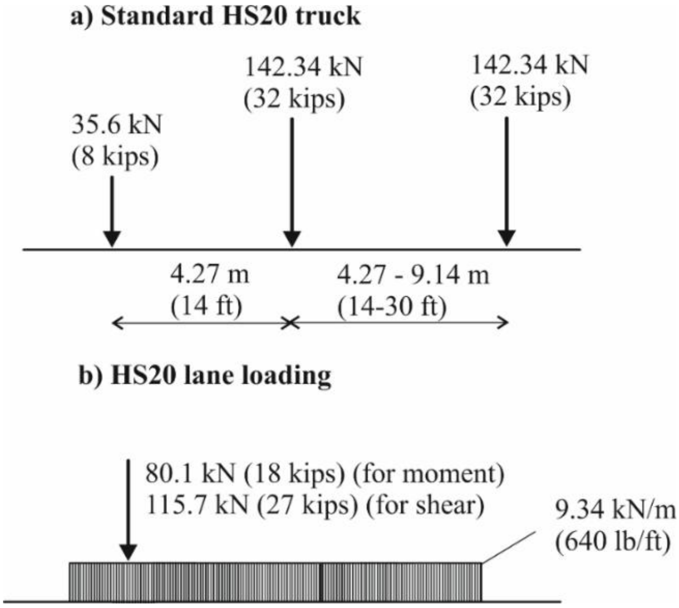

Figure 1. HS20 live load in AASHTO

As this study focuses on reliability analysis, live load factors that were presented in [18] were used in the corresponding equations defined in the following section. Live load factors that have been calculated for moments and shears in [18] were taken into account in this study. 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 in [18]. 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 bridge in Figure 2. The same locations are shown simply for two-span continuous bridge in Figure 3. Moments, shears and negative moments calculated at the points defined in Figure 2 and 3 were used in the reliability analysis part of this paper. Detailed information and the necessary formulations on the live load factor analysis can be found in [18].

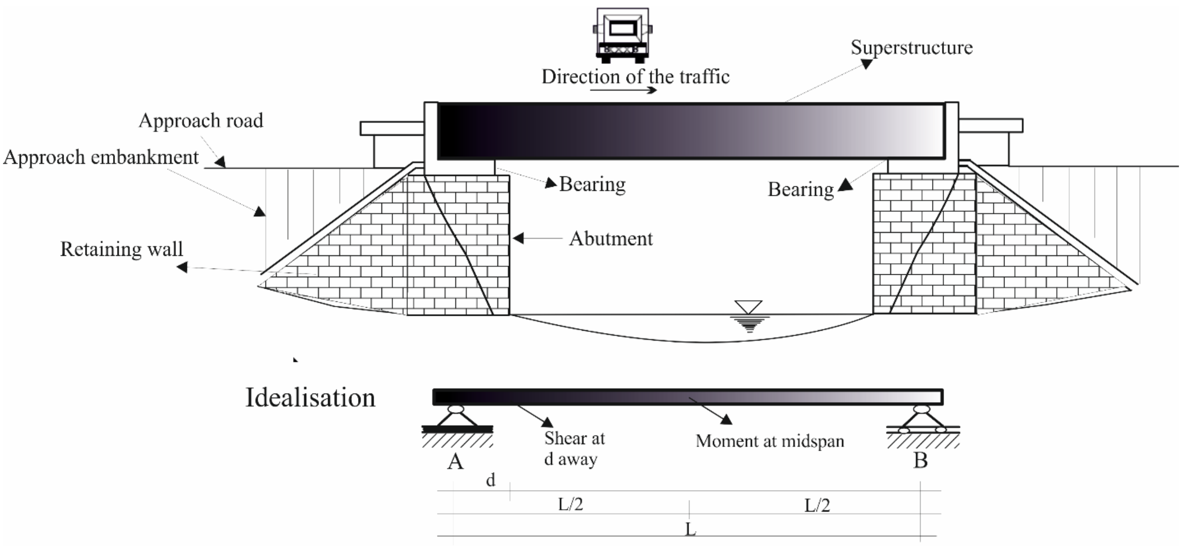

Figure 2. Simple span bridge idealization and shear and moment locations

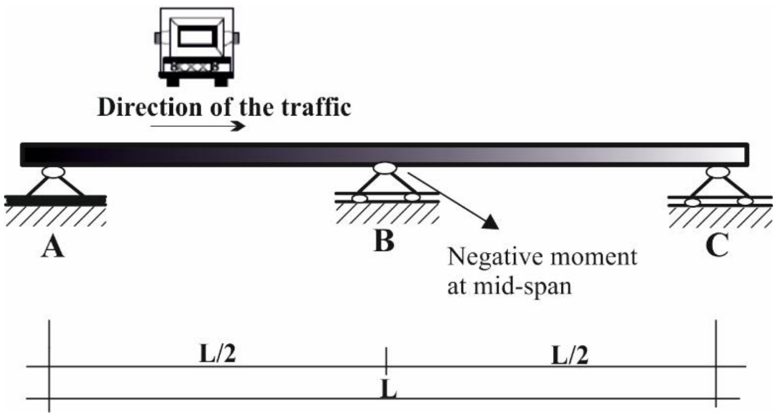

Figure 3. Negative moment location for 2-span continuous bridge analysis.

3. Reliability Analysis

In engineering structures, a limit state function, g, can be written as g=R-Q, here R represents the resistance (load carrying capacity) and Q represents the load effect (total moment or shear applied to the structure in the present case). If g>0, the structure is safe, otherwise it will fail. The probability of failure is the probability that load effects will exceed the strength. The probability of the failure PF can be written as [1]:

$$P_F = \text{Prob}\ (R-Q<0) = \text{Prob}\ (g<0) \tag{1}$$

A case considering the demand of the system (load on the system, Q) and the capacity of the system (resistance of the structure, R) are shown in Figure 4 with both Q and R taken as random variables. The mean and standard deviation of the load effects on the system can be shown by μ_Q and σ_Q while the characteristics for resistance are μ_R and σ_R . Probability density functions of these random variables are shown with $f_Q(q)$ and $f_R(r)$ for load and resistance respectively [20].

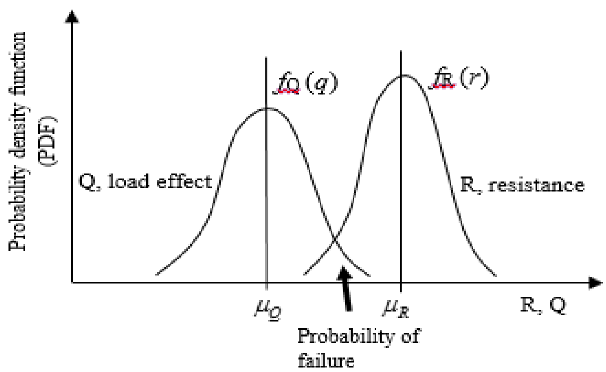

Figure 4. Probability density functions of load and resistance [20].

3.1 Reliability Index

The reliability index is the function of probability of failure, P_F, and expressed as:

$$\beta = -\Phi^{-1}(P_F) \tag{2}$$

where Φ^{-1} is the inverse standard normal distribution function. If load effects Q and resistance R are both normal random variables, the reliability index can be written as:

$$\beta = (\mu_R - \mu_Q) / \sqrt{\sigma_R^2 - \sigma_Q^2} \tag{3}$$

if both Q and R, are lognormal variables then the reliability index is expressed as:

$$\beta = \ln(\mu_R / \mu_Q) / \sqrt{Cv_R^2 - Cv_Q^2} \tag{4}$$

where Cv_R and Cv_Q are the coefficient of variation (COV) of load effects and resistance, respectively. In the design of structural components, members need to be designed to meet the maximum demand considering all possible loads that could act on them during their lifetime. The reliability index methodology in this study considered the live loads, impact loads and dead loads that may act on the bridge structures. Reliability indices were computed for ncsg, rctb, and pcg bridges. The input data for calculating reliability analysis were the statistical parameters for each load component and resistance. These parameters were the nominal value, mean value, bias factor (λ= ratio of the mean-to-nominal), COV (ratio of standard deviation and the mean value) and the type of cumulative distribution function. The reliability index can be computed as [1]:

$$\beta = \left(R_n\lambda_R(1-kV_R)[1-\ln(1-kV_R)] - \mu_Q\right) / \left(\sqrt{R_n V_R \lambda_R (1-kV_R)^2 + \sigma_Q^2}\right) \tag{5}$$

where V_R is the COV of resistance μ_Q is mean total applied load, λ_R is the bias factor of the resistance, R_n is the nominal resistance, (R_n λ_R) is the mean unfactored resistance (actual) or mean factored applied load, σ_Q is the standard deviation of total applied load. Taking the parameter, k, as 2, the reliability index can be computed as [1,21]:

$$\beta = \left((1-2\ V_R)(1+\ln(1/1-2V_R)) - \mu_Q / (\lambda_R R_n)\right) / \left(\sqrt{(V_R - 2V_R^2)^2 + \sigma_Q^2 / (\lambda_R R_n)^2}\right) \tag{6}$$

The mean total load and standard deviation of the total load can be calculated with the following expressions [22]:

$$\mu_Q = \mu_{LL} + \mu_{IM} + \mu_{DL} \ , \ \sigma_Q^2 = \sigma_{DL}^2 + \sigma_{LL}^2 + \sigma_{IM}^2 \tag{7}$$

where μ_{LL} = mean live load, μ_{DL}= mean dead load and μ_{IM}=mean dynamic load. The mean values of are calculated using bias factors, λ, and nominal design values of considered load component. COV of the total load can be calculated with $V_Q = \sigma_Q / \mu_Q$.

3.1.1 Statistical Parameters of Dead Loads

Statistical parameters for dead loads are taken from [1]. Dead loads due to factory-made members (steel and precast concrete girders) are shown with D_1, dead load due to cast in place concrete are shown with D_2, dead loads due to the weight wearing surface (asphalt) are shown with D_3 in this paper. The mean (μ_D) and standard deviations (σ_D) for the dead load effects were estimated using the bias factors factor (λ_D) and COV which are presented in Table 2. The mean and standard deviations of dead load were calculated as:

$$\mu_{DL} = D_n \ \lambda_D; \quad \sigma_{DL} = \mu_n \ COV \tag{8}$$

Table 2. Statistical Parameters of Dead Load

Component	Bias factor λ	COV
Factory-made members (D_1)	1.03	0.08
Cast-in-place members (D_2)	1.05	0.10
Wearing surface (D_3)	1.00	0.25

3.1.2 Statistical Parameters of Live Loads

Statistical parameters of live loads namely bias factors and COV were obtained considering Woodburn NB WIM data converted to HS20 equivalent load effects and projected to a maximum expected loading event in [18]. The statistical parameters were derived for ADTT=5,000 and 5-year evaluation period (projection). The COV of the live load effects was calculated as :

$$COV_L = \sigma_{L,\%20} / \mu_{L,\%20} \tag{9}$$

where $\mu_{L,\%20}$ is the mean of the top 20% of the load effects (either shear or moment), $\sigma_{L,\%20}$ is the standard deviation of the top 20% of the load effects (either shear or moment). Bias factors were determined by reading the values from the plots of each WIM site for moments and shears in [18]. They were calculated for each WIM site given in this paper. The bias factors were obtained for a time period of 5 years in their study [18]. These bias factors are for the one loaded lane case. In this study, the ratio of the mean maximum 4 days of truck loading, and 5 years of maximum live load effect is found to be about 0.75 for most of the spans. Therefore, bias factors for the two lanes loaded case can be calculated as:

$$\lambda_g = 0.75\ \lambda_L \tag{10}$$

where λ_L is the bias factor for the one lane loaded case. The mean live load per girder can be written as $\mu_{LL} = \lambda_g LL$ where LL is the live load acting on the girder caused by HS20 loading. All the bias factors and load factors calculated in [3,18] were used in this study.

3.1.3 Impact Load

The impact load was calculated as $\mu_{IM} = f_{IM}\ \mu_{LL}$ where f_{IM} is the impact factor. And according to [23] it can be expressed as:

$$f_{IM} = 50/(L+125) \leq 0.3 \tag{11}$$

where L is the span length. The COV of the impact loading is obtained as 0.80 from the study of [24] and [25].

3.1.4 Live Load and Dynamic Load Effect

The mean value of the combination of the live load and impact load was calculated as:

$$\mu_{LL+IM} = \mu_{LL} + \mu_{IM} \tag{12}$$

It must be noted that for two lane loaded cases multiple presence factor is 1.0 in the above equation as recommended by [1]. The COV of the combination of live load and dynamic load effect was determined as:

$$COV_{LL+IM} = \sigma_{LL,S+IM} / \mu_{LL,S+IM} \tag{13}$$

where live load part $\mu_{LL,S}$ in this equation does not include the bias factor. The standard deviation $\sigma_{LL,S+IM}$ was determined as:

$$\sigma_{LL,S+IM} = \sqrt{\sigma_{LL,S}^{2} + \sigma_{IM}^{2}} \tag{14}$$

where $\sigma_{LL,S}$ is the standard deviation of the live load and $\sigma_{LL,IM}$ is the standard deviation of the dynamic load. The standard deviation of the live load and impact load was calculated as:

$$\sigma_{LL,S} = COV_{LL}\ \mu_{LL,S}\ ;\ \sigma_{IM} = COV_{IM}\ \mu_{IM} \tag{15}$$

where COV_{LL} and COV_{IM} represent the COV of the live load which was 0.095 for this study and the COV of the impact load from the literature was 0.80. For two lane bridges, the COV for most spans ranged from 0.13-0.15, while for shorter spans the COV were slightly higher. In [1] COV for two lane bridges were presented as 0.18 for most spans and 0.19 for very short spans considering the HL93 loading. In this study, the COV was computed for HS20 loading. In the reliability index and live load factor calculations [18], the COV for the combination of live and impact loads was taken as 0.15.

3.1.5 Statistical Parameters of Resistance

The resistance statistics for ncsg, rctb, and pcg bridges were taken from [1]. For given loads, D, L, and I the minimum required resistance R_{min} can be calculated for load factor design as:

$$R_{min} = [1.3\ D + 2.17\ (L+I)]/\phi \tag{16}$$

The statistical parameters for the different bridge types are shown in Table 3. (abbreviations in Table 3 are; M is moment, S is shear and LE is load effect)

Table 3. Statistical parameters of resistance.

Type	LE	λ	COV	Type	LE	λ	COV	Type	LE	λ	COV
ncsg	M	1.12	0.10	rctb	M	1.14	0.13	pcg	M	1.05	0.075
	S	1.14	0.105		S	1.20	0.155		S	1.15	0.14

4. Reliability Index Calculations

Reliability indices were calculated using Eqs. 5 and 6 for HS20 design load and considering 5-year evaluation period. The reliability indices were computed for ncsg, rctb and pcg bridges. Girder spacing of 1.2, 1.8, 2.4, 3.1 and 3.7 m were taken into account. Span lengths for ncsg and pcg bridges are 9.1, 18.3, 27.4, 36.6 and 61 m while these lengths for rctb are 9.1, 18.3, 27.4 and 36.6 m. The same impact load ratio used by [1] (which varies based on span length) was implemented to the reliability index equations in this section. The reliability indices using I-5 Woodburn NB WIM data and archival statistical data for the ncsg are given in Figure 5. Figure 5 includes results from [1] for comparison. Reliability indices for ncsg, presented in [1] for a 75-years period are also shown in this figure along with β=3.5 and β=2.5 as reference lines. The reliability indices for different ADTT levels that have been investigated, are given in the conclusion section. In a second approach, the impact load ratio was taken as 30% for all span lengths. The reliability indices for this case were also calculated in this paper for ADTT=5,000 level, and the findings for this case were used in conclusions. The reliability indices for moments in rctb, are shown in Figure 6 along with the results from [1].

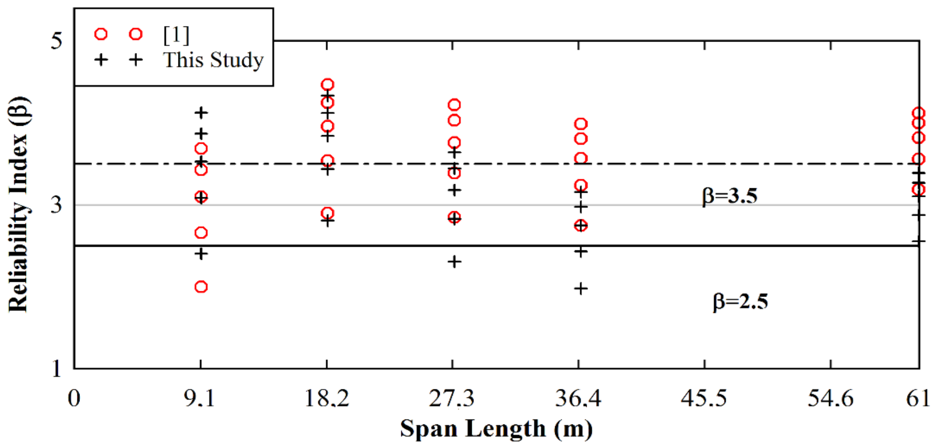

Figure 5. Reliability indices for moment in ncsg.

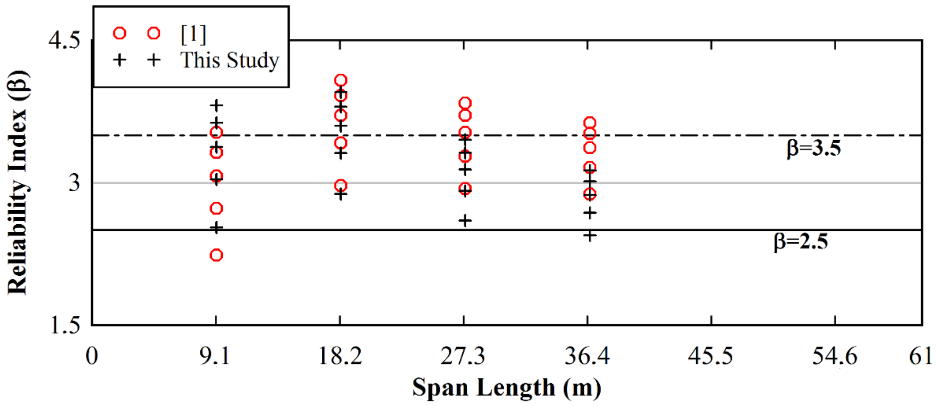

Figure 6. Reliability indices for moments in rctb.

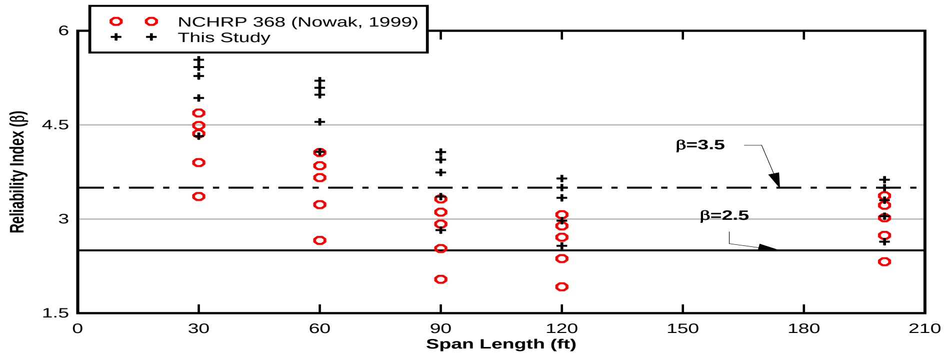

Figure 7. Reliability indices for shear in ncsg. (bridge lengths in ft)

The reliability indices for moments in pcg bridges are also compared along with results from [1]. The reliability indices for shears in ncsg and rctb are shown in Figure 7 and Figure 8 respectively. Figures 7 and 8 are given in a comparative way with [1] as well. It should be noted here that bridge lengths are shown with ft units in those figures. The shear results of pcg bridges are not shown in this paper because of space limitations. Results obtained from the comparisons, that are not presented in this paper, are used to obtain the comments written below. As it can be indicated from the reliability index results, there are differences between the reliability indices with respect to the girder spacing and span length. The calculated reliability indices for shears in ncsg and rctb are higher than those presented in [1].

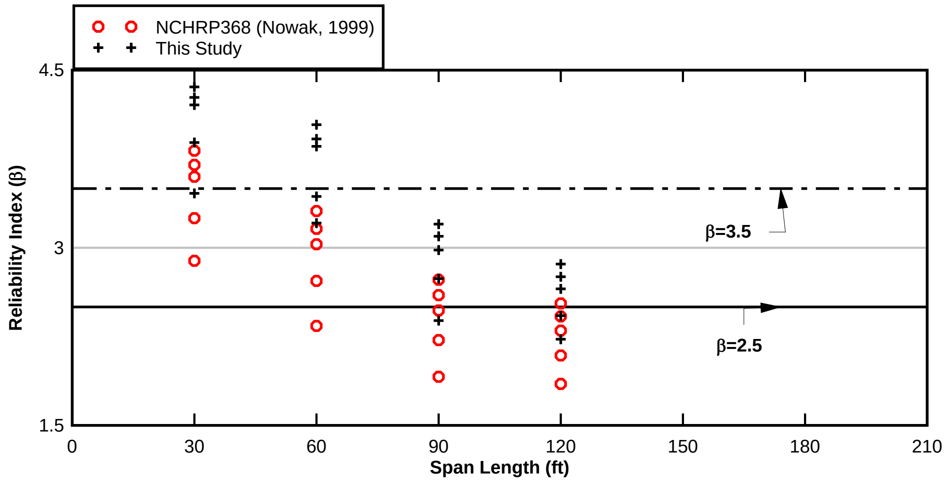

Figure 8. Reliability indices for shear in rctb. (bridge lengths in ft)

While the reliability indices of short span moments in ncsg and rctb are higher or at the same extent with the ones in [1], for longer spans, the reliability indices for moments are at the same extent, or lower than the reliability indices calculated in [1]. One of the main reasons of this difference is, the COV the live and impact load is different with the COV in [1]. Moreover, the bias factors for moments of longer spans in this study, are different than the ones obtained in [1]. The reliability indices for ncsg, rctb bridges and pcg bridges are tabulated in Table 4. The mean, the COV (with respect to girder spacing) of β and the minimum of the reliability indices for each span length are presented.

Table 4. Statistical characteristics of reliability indices.

ADTT	Type	L.E.	Span Length (m) 9.1			18.3			27.4			36.6			61		
			β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}
=5,000	ncsg	M	2.4	3.4	0.18	2.8	3.7	0.15	2.3	3.1	0.15	2.0	2.6	0.16	2.5	3.0	0.10
		V	4.3	5.1	0.09	4.1	4.8	0.09	2.8	3.6	0.13	2.6	3.2	0.12	2.6	3.2	0.11
	rctb	M	2.5	3.3	0.14	2.9	3.5	0.11	2.6	3.1	0.10	2.4	2.8	0.09	-	-	-
		V	3.5	4.0	0.08	3.2	3.7	0.09	2.4	2.9	0.10	2.2	2.6	0.09	-	-	-
	pcg	M	2.3	3.4	0.20	2.8	3.8	0.16	2.3	3.1	0.15	2.1	2.7	0.14	2.5	3.0	0.10
		V	3.6	4.2	0.09	3.3	3.9	0.09	2.4	3.0	0.11	2.3	2.7	0.10	2.2	2.7	0.10
=1,500	ncsg	M	3.8	4.7	0.12	3.3	4.2	0.12	3.0	3.8	0.12	2.6	3.3	0.12	2.8	3.3	0.09
		V	5.4	6.1	0.06	4.3	5.0	0.08	3.4	4.2	0.10	3.2	3.8	0.09	3.0	3.6	0.09
	rctb	M	3.5	4.2	0.10	3.2	3.8	0.09	3.0	3.5	0.08	2.8	3.2	0.08	-	-	-
		V	4.2	4.7	0.06	3.4	3.8	0.08	2.7	3.2	0.09	2.6	2.9	0.06	-	-	-
	pcg	M	2.5	3.5	0.17	3.5	4.4	0.12	3.0	3.8	0.12	2.4	3.0	0.13	2.6	3.1	0.09
		V	3.8	4.9	0.14	3.3	4.3	0.14	3.0	3.8	0.13	2.7	3.3	0.12	2.7	3.3	0.10
≤500	ncsg	M	2.8	3.8	0.16	3.2	4.1	0.13	2.8	3.6	0.13	2.6	3.2	0.12	2.6	3.1	0.10
		V	4.5	5.3	0.08	4.4	5.1	0.08	3.0	3.8	0.12	2.9	3.5	0.11	2.8	3.4	0.10
	rctb	M	2.8	3.6	0.13	3.2	3.8	0.10	2.9	3.4	0.09	2.8	3.2	0.08	-	-	-
		V	3.6	4.1	0.08	3.4	3.9	0.08	2.5	3.0	0.10	2.4	2.7	0.07	-	-	-
	pcg	M	2.7	3.8	0.18	3.2	4.2	0.14	2.8	3.6	0.14	2.6	3.2	0.12	2.5	3.0	0.10
		V	3.7	4.3	0.08	3.6	4.1	0.08	2.6	3.1	0.11	2.5	2.9	0.09	2.4	2.8	0.10
[1]	ncsg	M	2.0	3.0	0.20	2.9	3.8	0.15	2.9	3.7	0.13	2.8	3.5	0.13	3.2	3.7	0.09
		V	3.4	4.2	0.11	2.7	3.5	0.14	2.0	2.8	0.16	1.9	2.6	0.16	2.3	2.9	0.13
	rctb	M	2.2	3.0	0.15	2.9	3.6	0.11	2.9	3.4	0.10	2.3	2.9	0.20	-	-	-
		V	2.9	3.5	0.10	2.3	2.9	0.12	1.9	2.4	0.12	1.9	2.2	0.11	-	-	-
	pcg	M	1.9	2.9	0.22	3.0	4.0	0.15	3.0	3.8	0.14	2.9	3.6	0.12	3.2	3.9	0.10
		V	2.9	3.6	0.11	2.4	3.0	0.12	1.9	2.4	0.13	1.9	2.3	0.11	2.1	2.5	0.10

A summary of the reliability indices statistics is shown in Table 5. The mean values are given for shear and moment separately. The last column in Table 5 shows the mean of all spans. As seen in Table 5, there is only slightly higher variation in the reliability indices with respect to girder spacing for moment (0.12) compared to shear (0.10).

Table 5. Summary of statistical characteristics of reliability indices.

L. E.	Span Length (m)															Mean μ_{all}		
	Mean Values																	
	9.1			18.3			27.4			36.6			61					
	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}	β_{min}	$\beta\mu$	β_{COV}
M	2.8	3.7	0.2	3.1	3.9	0.1	2.7	3.4	0.1	2.5	3.0	0.1	2.6	3.1	0.1	2.75	3.45	0.12
V	4.1	4.7	0.1	3.7	4.3	0.1	2.8	3.4	0.1	2.6	3.1	0.1	2.6	3.2	0.1	3.14	3.73	0.10

5. Conclusions

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 reliability indices can be estimated. Additionally, if a researcher or designer considers a different evaluation period than 5 years, different reliability indices can be obtained.

For HS20 loading, ADTT=5,000 and five years of evaluation period the reliability indices ranged from 2.4 to 5.5 for ncsg, while they ranged from 2.4 to 4.4 for rctb. Girder spacing, span length as well as bias factors had effect on the variation of the reliability indices. Live load factors for ADTT=5,000 case, and HS20 loading with five years of evaluation period ranged from 1.60 to 1.80. These values correspond to the reliability indices presented above. For HS20 loading, ADTT=1,500, and five years of evaluation period the reliability indices ranged from 2.6 to 6.4 for ncsg, equally they ranged from 2.8 to 4.9 for rctb. For HS20 loading, ADTT≤500 case and five years of evaluation period, the reliability indices ranged from 2.8 to 5.7 for ncsg, similarly for rctb they ranged from 2.4 to 4.5. For ADTT of 5,000, the average reliability indices of bridges, based on minimum resistance are obtained as 3.2 for moment and 3.5 for shear. For ADTT of 1,500, the average reliability indices of bridges, based on minimum resistance are obtained as 3.7 for moment and 4.0 for shear. For ADTT of 500, the average reliability indices of bridges, based on minimum resistance are obtained as 3.6 for moment and 3.7 for shear. The reliability index for ADTT of 5,000, and moment, is found as 3.2 which is lower than 3.5 which was used as a target reliability index in current [26] specifications.

Acknowledgements

This work was partially supported by the Oregon Department of Transportation (ODOT). Any opinions and conclusions are those of the authors and do not necessarily reflect the views of the supporter.

References

[1]. Nowak AS. NCHRP Report 368: Calibration of LRFD Bridge Design Code. Washington, DC: Transportation Research Board, National Research Council; 1999.

[2]. Koç, M., Esen, İ., Eroğlu, M., Çay, Y., Çerlek, Ö., "Esnek Yapıların Çoklu Taşıt Hareketi Etkisi Altında Dinamik Analizi". El-Cezeri Journal of Science and Engineering, 2018, 5: 176-181.

[3]. Yanık A. ve Higgins C., "A Monte-Carlo Simulation for the Estimation of Side-by-Side Loading Events on Oregon Bridges", Politeknik Dergisi, *(*): *, (*), DOI:10.2339/politeknik.469495.
[4]. Ma R., Xu S., Wang D., Chen A., "Vehicle models for fatigue loading on steel box-girder bridges based on weigh-in-motion data.", Structure and Infrastructure Engineering, 2018, 14(6): 701-713.
[5]. Znidaric A., Kalin J., Kreslin M., Favai P., Kolakowski P., "Railway bridge weigh-in-motion system." Transportation Research Procedia, 2016, 14: 4010–4019.
[6]. Kalyankar R., Uddin N., "Axle detection on prestressed concrete bridge using bridge weigh-in-motion system.", Journal of Civil Structural Health Monitoring, 2017, 7(2): 191-205.
[7]. Znidaric A., Kalin J., Kreslin M., "Improved accuracy and robustness of bridge weigh-in-motion systems.", Structure and Infrastructure Engineering, 2018, 14(4): 412-424.
[8]. Yang Y., Cai CS., Deng L., "State-of-the-art review on bridge weigh-in-motion technology.", Advances in Structural Engineering, 2016, 19(9): 1514-1530.
[9]. Lydon M., Taylor S.E., Robinson D., Mufti A., Brien EJO., "Recent developments in bridge weigh in motion (B-WIM).", Journal of Civil Structural Health Monitoring, 2016, 6(1): 69-81.
[10]. Sekiya H., Kosaku K., Chitoshi M., "Simplified Portable Bridge Weigh-in-Motion System Using Accelerometers.", Journal of Bridge Engineering, 2017, 23(1): 1-15.
[11]. Richardson J., Steven J., Brown A., O'Brien E., Hajializadeh D., "On the use of bridge weigh-in-motion for overweight truck enforcement.", International Journal of Heavy Vehicle Systems, 2014, 21(2): 83-104.
[12]. Liu Y., Zhang H., Yang D., Jiang N., "Effect of live load on simply supported bridges under a random traffic flow based on weigh-in-motion data.", Advances in Structural Engineering, 2017, 20(5): 722-736.
[13]. Zolghadri N., Halling M.W, Johnson N., Barr P.J., "Field verification of simplified bridge weigh-in-motion techniques.", Journal of Bridge Engineering, 2016, 21(10): 1-17.
[14]. Soto G., Hernández-Martínez A., Valdés-Vázquez J.G., "Probabilistic assessment of a design truck model and live load factor from weigh-in-motion data for Mexican Highway bridge design.", Canadian Journal of Civil Engineering, 2015, 42(11): 970-974.
[15]. John L. "Reliability evaluation of the Eurocode model for fatigue assessment of steel bridges.", Journal of Constructional Steel Research, 2018, 141:1-8.
[16]. Han W., Yuan Y., Xie Q., Chen X., Huang P., "Reliability-Based Truck Weight Regulation of Small-to Medium-Span Bridges.", Journal of Bridge Engineering, 2017, 23(1): 04017109.
[17]. Ghasemi S.H., Nowak A.S., "Reliability analysis for serviceability limit state of bridges concerning deflection criteria.", Structural Engineering International, 2016, 26(2):168-7
[18]. Yanik, A., Higgins, C., Borello, D. "Development of live load factors for rating of Oregon bridges using WIM load effects and statistical bridge models.", Technical Report for Oregon Department of Transportation, (2015).
[19]. AASHTO. "The Manual for Bridge Evaluation First Edition", American Association of State Highway and Transportation Officials, Washington, D.C, (2008).
[20]. Haldar A., Mahadevan S., "Probability, Reliability, and Statistical Methods in Engineering Design.", John Wiley, New York, 2000.
[21]. Mlynarski M., Wassef W.G., Nowak A.S., "A Comparison of AASHTO Bridge Load Rating Methods.", NCHRP Report 700, Transportation Research Board, National Research Council, Washington, D.C, 2011.
[22]. Kim S.J., Nowak A.S., "Load Distribution and Impact Factors for I-Girder Bridges.", Journal of Bridge Engineering, 1997, 2(3), 97-104.
[23]. American Association of State Highway and Transportation Officials (AASHTO), "The Manual For Bridge Evaluation First Edition.", Washington, D.C, 2008.
[24]. Kim S.J., Nowak A.S. Load Distribution and Impact Factors for I-Girder Bridges. Journal of Bridge Engineering 1997; 2(3), 97-104.

[25]. Eom J., Nowak A.S., "Live Load Distribution for Steel Girder Bridges.", Journal of Bridge Engineering, 2001, 6(6), 489-497.
[26]. American Association of State Highway and Transportation Officials (AASHTO), "LRFD Bridge Design Specifications.", Washington, D.C., (2012).