CSCE Annual Conference
Growing with youth – Croître avec les jeunes

Laval (Greater Montreal)
June 12 - 15, 2019

Evaluation and Calibration of Pedestrian Bridge Design Standards for Vibration Serviceability of Lightweight Bridges

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Abstract: Rapid advancements in material technology have paved the way for lightweight yet highly durable materials such as aluminum, providing fascinating opportunities to build lightweight pedestrian bridges. This has resulted in lively bridges, which often suffer excessive vibrations leading to serviceability problems under pedestrian-induced loads. Various standards for serviceability design have been developed, primarily based on low-frequency bridges. These standards have overlooked the altered mass-stiffness relationship for lightweight structures, which often induce high-frequency responses. Another central issue in their design is proper consideration of the uncertainties in the pedestrian loading. This study underscores the deficiencies in current standards by comparing predictions with measurements from aluminum pedestrian bridges. Experimental results from two full-scale bridges show significant differences in the predictions by the design models as compared to the measurements. Accordingly, modifications have been recommended to better align predictions with experimental observations, which also harmonize these standards amongst each other. In addition, a reliability-based evaluation is carried out on code-compliant bridges by incorporating the uncertainties associated with the various parameters in the design process. Based on the evaluation results, the design equations are calibrated for higher reliability indices and partial factors for the calibrated design equation are estimated. For economic designs, user comfort limits based on the frequency of occurrence of the traffic event and the class of pedestrian bridge are adopted during the calibration process. The calibrated design standards ensure acceptable performance during both design and non-frequent heavy traffic loading events, while at the same time yielding economic designs.

1 INTRODUCTION

With the advancements in material technology, there is a growing trend in constructing architecturally appealing pedestrian bridges with various lightweight yet highly durable materials, e.g., aluminum, with relatively low maintenance cost. However, due to their lightweight, these bridges are often associated with relatively high frequencies, i.e. their fundamental frequency is outside the range of normal walking frequencies (1.6 Hz to 2.4 Hz in the vertical direction). Hence, these bridges have thus far not attracted much attention in the literature in terms of vibration serviceability issues, since most of the literature focuses on low frequency bridges resonating with the first harmonic of the walking frequency. However, resonance with the higher harmonics of walking frequency could result in excessive bridge vibrations, potentially leading to serviceability problems under pedestrian-induced walking loads.

Early research into vibration serviceability of pedestrian bridges dates back to the seventies, with the work of Blanchard et al. (Blanchard et al. 1977) to determine the vertical vibrations of pedestrian bridges subjected to walking forces induced by a single pedestrian. The aforementioned work was later incorporated into several international bridge design codes, such as the Canadian standard (CAN/CSA-S6 2011). In the last decade, research on vibration serviceability of pedestrian bridges has been advanced from a deterministic approach towards a more comprehensive probabilistic approach focussing on groups of pedestrians (Živanovic 2006). Subsequently, these approaches have found their place in recent design standards (CEN 2004, BSI 2003, SÉTRA 2006), which employ an equivalent deterministic approach to estimate the crowd-induced bridge response using the resonant response from a single pedestrian.

Primarily, current pedestrian bridge standards are based on observations from low frequency bridges and they have overlooked the altered mass-stiffness relationship for lightweight structures. The light weight produces high frequency bridges outside the critical frequency ranges by the standards, while at the same time resulting in excessive bridge vibration under operational loads. Hence, it is essential to evaluate these standards for lightweight bridges. Moreover, none of the standards has been evaluated in a reliability-based framework incorporating uncertainties arising either from the pedestrians or from the structure. The authors have underscored such discrepancies in the bridge standards through a coupled experimental and numerical investigation on aluminum pedestrian bridges under walking loads (Dey et al. 2016a, 2016b, 2017, 2018). Key findings from these studies on performance assessment of design standards for serviceability assessment of pedestrian bridges are summarized in the current paper. For the sake of brevity, the results reported here are limited to the SÉTRA guide and UK National Annex to Eurocode 1 for the vertical vibration of aluminum pedestrian bridges.

2 VIBRATION SERVICEABILITY DESIGN CRITERIA

Pedestrian bridge standards follow a two-step approach for vibration serviceability design. In the first step, the standards ensure serviceability through restricting the structural frequency outside the critical frequency ranges, which are 0-8 Hz and 1-5 Hz, prescribed by the UK National Annex and SÉTRA, respectively. However, if a pedestrian bridge falls within these critical frequency ranges, they should satisfy the serviceability design equation for the natural mode under consideration:

$$[1]\ a_{ln} \geq a_N$$

where, a_{ln} is the acceleration limit as reported in Table 1 and a_N is the resonant response of the bridge under N pedestrians or crowd density of d P/m^2. With the single degree of freedom approximation for the resonant mode under consideration, a_N is estimated as:

$$[2]\ a_N = \frac{S_N G_n \alpha_{mn}}{\pi M_n \xi_n}$$

Here, M_n *and* ξ_n are respectively the modal mass and damping for the structural mode under consideration. G_n and α_{mn} are respectively the pedestrian weight and dynamic load factor for the m^{th} resonating harmonic of walking frequency and S_N is the crowd factor. The values of these design parameters by the standards are listed in Table 1, where k_v and ψ are the reduction coefficients depending on the structural frequencies and can be found in the respective standards. γ is a factor that allows for the unsynchronized combination of actions in a pedestrian group and is a function of damping of the structure (ξ_n) and the group size (BSI 2003). In the current paper, the key findings from the performance evaluation of these two standards based on measurements from two full-scale aluminum pedestrian bridges are discussed first. This is followed by summarizing the results from the evaluation of these standards in a reliability-based framework incorporating the primary sources of uncertainties in the design equation.

Table 1: Design parameters by UK National Annex and SÉTRA

Codes	a_{ln} (m/s^2)	G_n(N)	α_{mn}	S_N
UK National Annex	0.5-2.0 depending on routes, usage etc.	*700*	$\alpha_1 = 0.4$, $\alpha_2 = 0.14$, $\alpha_3 = 0.051$	$1.8k_v\sqrt{[\gamma N/\lambda]}$
SÉTRA	0.5-2.5 depending on comfort level	*700*	$\alpha_1 = 0.4$, $\alpha_2 = 0.1$	$10.8\sqrt{(\xi_n N)}\psi$ for $d < 1.0$ $1.85\sqrt{(N)}\psi$ for $d \geq 1.0$

3 PERFORMANCE ASSESSMENT OF THE DESIGN STANDARDS

3.1 Deterministic Framework

Dey et al. (2016b) evaluated existing design standards for aluminum pedestrian bridges based on an experimental study on two full-scale aluminum pedestrian bridges under various groups of pedestrians. Based on the outcomes of the evaluation study, the authors proposed key recommendations aimed at reconciling the design methodologies with the experimental observations. The following section summarizes the experimental study and the results from the performance evaluation of UK National Annex and SÉTRA.

3.1.1 Brief Overview of the Experimental Study

The authors of this paper conducted a comprehensive experimental program in the laboratory on two modular aluminum pedestrian bridges of spans 12.2 m and 22.9 m, constructed by the MAADI Group in Montreal, Quebec. Several walking tests were carried out with varying crowd densities ranging from 0.2-1.0 P/m^2. Fig. 1 presents the 22.9 m bridge specimen and crowd tests on the bridge. To measure the acceleration response, the bridges were instrumented with twelve low frequency, high-sensitivity accelerometers on the bottom chords at quarter and mid-points along the length, in both lateral and vertical directions. However, this paper reports the measurements from the vertical direction only. A detailed description of the experimental study and data collection is provided in Dey et al. 2016. The first vertical frequencies of the 12.2 m and 22.9 m bridge specimens are 11.81 Hz and 4.58 Hz and the modal damping ratios are experimentally determined to be 1.0% and 0.8%, respectively. The experimental observations from these two bridges are compared with the serviceability predictions by the standards in the following section.

(a) (b)

Figure 1: (a) 22.9 m bridge specimen, and (b) crowd tests on the bridge

3.1.2 Results of Serviceability Assessment

The first step in the serviceability assessment of the two pedestrian bridges requires a frequency evaluation by comparing the structural frequencies with the critical frequency ranges in Table 1. Both standards do not recommend a dynamic analysis for the 12.2 m bridge specimen with a frequency of 11.81 Hz, while a vibration evaluation needs to be performed for the 22.9 m bridge. The measured and simulated peak accelerations by the design standards are shown in Fig. 2 along with the average acceleration limits as reported in Table 1 for the two bridge specimens. As shown in Fig.2 (a), despite satisfying the frequency criteria by the standards, the measured vibration on the 12.2 m bridge exceeds the average comfort limit, even for low traffic densities. This observation justifies the need for accounting for higher harmonics of walking frequency within the design provisions. On the other hand, the predicted responses by the standards are overly conservative for the 22.9 m bridge specimen. Moreover, despite employing a basic load model with different multiplication factors, the predicted values by the two standards are inconsistent. Based on an in-depth investigation of the design standards, Dey et al. 2016b proposed recommendations to reconcile predictions with the measurements. These recommendations include considering higher harmonics of walking frequency, traffic dependent average walking speed, effect of added mass due to the presence of the pedestrians and a multiplication factor by the UK National Annex with response reduction factor of 1.0 to the existing design provisions. By incorporating these modifications to the standards, it is observed in Fig. 2 that the existing design provisions can be improved substantially in terms of predicted acceleration response and serviceability assessment. These modifications also harmonize the design standards across one another by simply incorporating the aforementioned recommendations into the existing design guidelines.

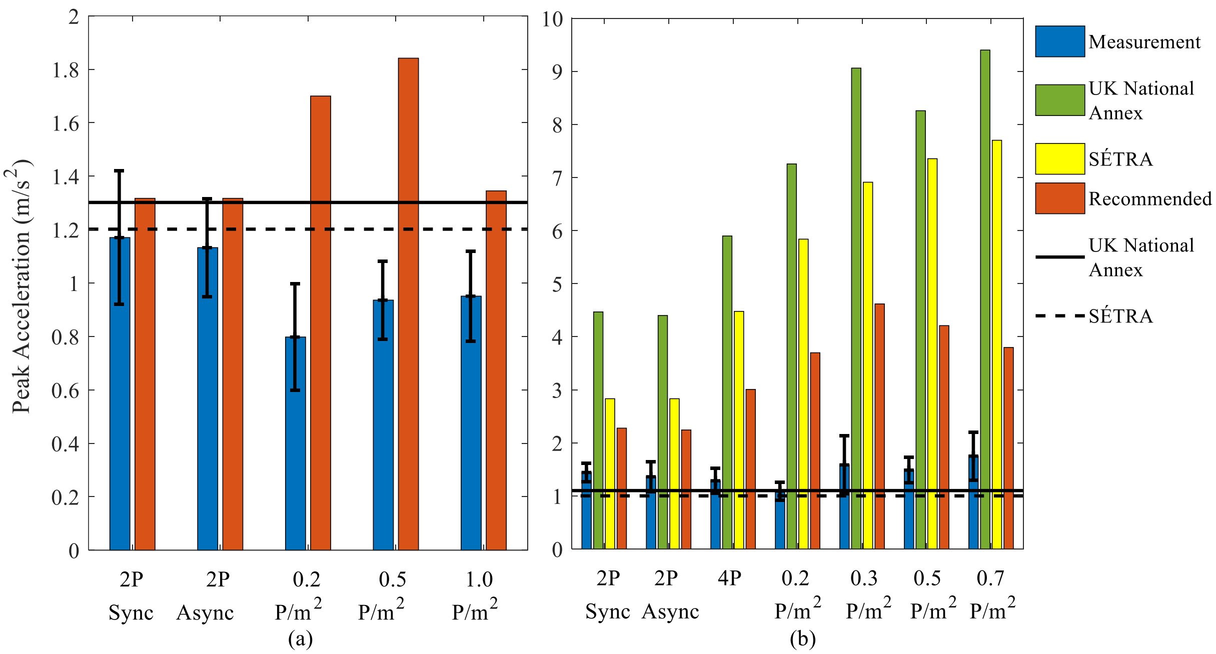

Figure 2: Comparison of measured and predicted peak accelerations along with the average comfort limits as proposed by the standards in the vertical direction for (a) 12.2 m and (b) 22.9 m bridge specimens (*P* stands for pedestrians)

3.2 Probabilistic Framework

This section presents the results from the evaluation and calibration of the pedestrian bridge standards in a reliability-based framework. For this purpose, 3,160 structural configurations of pony truss bridges were analyzed and only those optimally designed configurations passing the ultimate limit state design check for bending (CAN/CSA-S6 2011) as well as the serviceability checks according to Eq. (1) were retained for the reliability analysis. A detailed description of the structural configurations is provided in Dey et. al 2017.

3.2.1 Reliability-based Evaluation

Reliability-based evaluation of the design standards requires formulation of the limit state function *g(x)* identifying the random variable space as follows:

$$[3]\quad g(x) = a_l - \frac{S_N G \alpha_m}{\pi M_n \xi_n}$$

where, the acceleration limit (a_l), pedestrian's weight (G) and dynamic load factors (α_m) are considered as the primary sources of uncertainties in Eq. (1) and design variables related to the structural properties are assumed deterministic. According to Dey et al. 2018, a_l is assumed normally distributed with coefficient of variation (COV) of 0.20 and bias of 0.79. G is assumed to be log-normally distributed with COV of 0.17 (Portier and Roberts 2007), while α_m is normally distributed with COV of 0.17 and 0.40 in case of m = 1 and m > 1, respectively (Willford and Young 2006). In order to cover all possible design applications by the standards, Dey et al. 2017 adopted four bridge classes as listed in Table 2. Besides frequently occurring design traffic, the bridges are also evaluated for rare or non-frequent heavy traffic, such as those that might be experienced during inauguration with a traffic density of 1.5 P/m^2.

The modified Hasofer and Lind reliability index (β) using the advanced first order second moment (AFOSM) reliability method (Hasofer and Lind 1974) is calculated for all of the optimally designed configurations employing Eq. (3) for the design as well as rare traffic densities. The mean reliability indices are reported in Table 2. The sufficiency of the design standards is examined by comparing these mean β values under both design and rare events with the target reliability, β_t = 0 in accordance with ISO 2934 (ISO 1998). The comparison shows that the designed configurations do not satisfy the sufficiency criteria under both design and rare events, while the deviation from the target value is very high under rare loading scenarios. In general, designs belonging to Classes I to III are deemed insufficient during a rare event due to their lower design densities. This issue has been addressed by Dey et al. 2018 through calibrating the design equation, which results in the overdesign of Class I to III pedestrian bridges under design traffic densities, while at the same time achieving better serviceability performance during rare events. In addition, the authors recommended adopting traffic and bridging class dependent comfort classes that will ensure economic design by the calibrated design equation. A summary of the calibration results is presented in the next section.

Table 2. Mean reliability index under different traffic events

Bridge class	Design density (P/m^2)	Design density (rare density)	
		UK National Annex	SÉTRA
I	*0.2*	-0.47 (-2.93)	-0.45 (-3.93)
II	0.5	-0.46 (-1.85)	-0.44 (-3.43)
III	0.8	-0.44 (-1.22)	-0.42 (-2.81)
IV	1.0	-0.41 (-0.91)	-0.33 (-0.70)

3.2.2 Results of Calibration

The calibrated design equation corresponding to Eq. (1) is represented by:

$$[4] \quad (\gamma_{a_l} a_{ln}) - \frac{S_N(\gamma_G G_n)(\gamma_{\alpha_m} \alpha_m)}{\pi M_n \zeta_n} \geq 0$$

where, γ_{a_l}, γ_G, and γ_{α_m} are the partial factors corresponding to the acceleration limit (a_l), pedestrian's weight (G), and the dynamic load factor (α_m). The first step towards estimating these factors involves determining the reliability index required at the design event for calibrating Eq. (1), known as the *desired reliability index*, β_d. This is determined iteratively so that the mean reliability indices achieved by all the designs during both the design and the rare traffic events are more than or equal to the code-prescribed target reliability index of 0 (ISO 2934). The steps to estimate β_d and the partial factors γ_{a_l}, γ_G, and γ_{α_m} are described in Dey et al. 2018.

To demonstrate the effect of traffic and bridge class dependent comfort limits on the calibrated design, the authors created two sets of design cases with varying levels of comforts during the loading events. This paper presents the results of three design cases: [D_i, DR_{i1}, DR_{i2}]. Here, the subscript "*i*" takes on the values of 1 or 2 denoting the first and second sets, which are created by assigning class-based design acceleration limits. While the first set is based on the acceleration limit set by the standards for maximum comfort, the second set allows bridges corresponding to Class III and IV to be designed for a mean comfort limit of 1.0 m/s^2. Here, *D* represents the case when the reliability analysis is performed under only the design loading event. On the other hand, *DR* considers both the design and rare events in calculating the reliability indices and the minimum β_{mean} value during these two events is retained to ensure sufficiency. Case DR_{i1} is developed by assigning the same acceleration limit as Case D_i for the reliability analysis, while DR_{i2} takes on acceleration limits based on the minimum comforts for pedestrians as prescribed by SÉTRA (Table 1). However, if this limit is less than the design acceleration limit, the reliability analysis is performed based on the design limit.

The estimated desired reliability index (β_d) through the iterative steps are summarized in Table 3 for all the cases. As the estimation of β_d is based on Class I of pedestrian bridges for uniform reliability index across all bridge classes, the values of β_d are the same for the two sets. Moreover, β_d is the same for both the standards for the cases of D_i, since they have reliability indices in the range of -0.33 to -0.47 under the design event as listed in Table 2. On the other hand, cases DR_{11} and DR_{12} are developed to meet sufficiency for both the design and rare loading events. In such cases, estimates of β_d are not uniform, which is mainly attributed to the inconsistent reliability indices implied in the design guidelines during rare traffic due to different acceleration limits.

Table 3: Summary of desired reliability indices and corresponding partial factors

Cases	UK National Annex				SÉTRA			
	β_d	γ_{a_l}	γ_G	γ_{α_m}	β_d	γ_{a_l}	γ_G	γ_{α_m}
D_i	0.25	0.77 (0.78)	1.01 (1.00)	1.02 (1.08)	0.25	0.77 (0.78)	1.01 (1.00)	1.02 (1.08)
DR_{i1}	2.75	0.42 (0.47)	1.18 (1.17)	1.16 (1.60)	4.25	0.13 (0.15)	1.08 (1.10)	1.09 (1.42)
DR_{i2}	1.50	0.62 (0.65)	1.12 (1.09)	1.11 (1.41)	1.00	0.68 (0.71)	1.07 (1.05)	1.08 (1.30)

Fig. 3 compares the reliability indices embodied in the standards before and after calibration for the design cases corresponding to bridges of Class I and Class III. In the case of Class I, β_{mean} values estimated for the UK National Annex and SÉTRA are -0.47 and -0.45 under the design event D_1 and the resulting β_{mean} values are 0.38 and 0.39 after calibrating the guidelines uniformly with β_d = 0.25. As sets 1 and 2 are identical for Class I, the reliability indices embodied in the standards before and after calibration are the same as shown in Fig. 3. However, Classes III and IV are designed for a lower comfort or higher acceleration limit (1.0 m/s^2) in the case of Set 2, i.e., D_2 to DR_{22} in order to yield economic design while at the same time achieving sufficiency. For instance, for SÉTRA, while the design limit for Set 1 is 0.5 m/s^2 (maximum comfort) this is 1.0 m/s^2 (mean comfort) for Set 2, thus generating different sets of cases for Classes III and IV. As shown in Fig. 3(b), β_{mean} for SÉTRA Class III is 1.91 after calibrating it for β_d = 1.00 in the case of DR_{12}, while designing this class with mean comfort of 1.0 m/s^2 results in β_{mean} of 0.08 in the case of DR_{22}, which is better from an economic perspective. It should be noted that these calibration results are based on an assumed COV of the acceleration limit, which is 0.20, and a different COV of a_l may yield a different set of desired reliability and calibration factors.

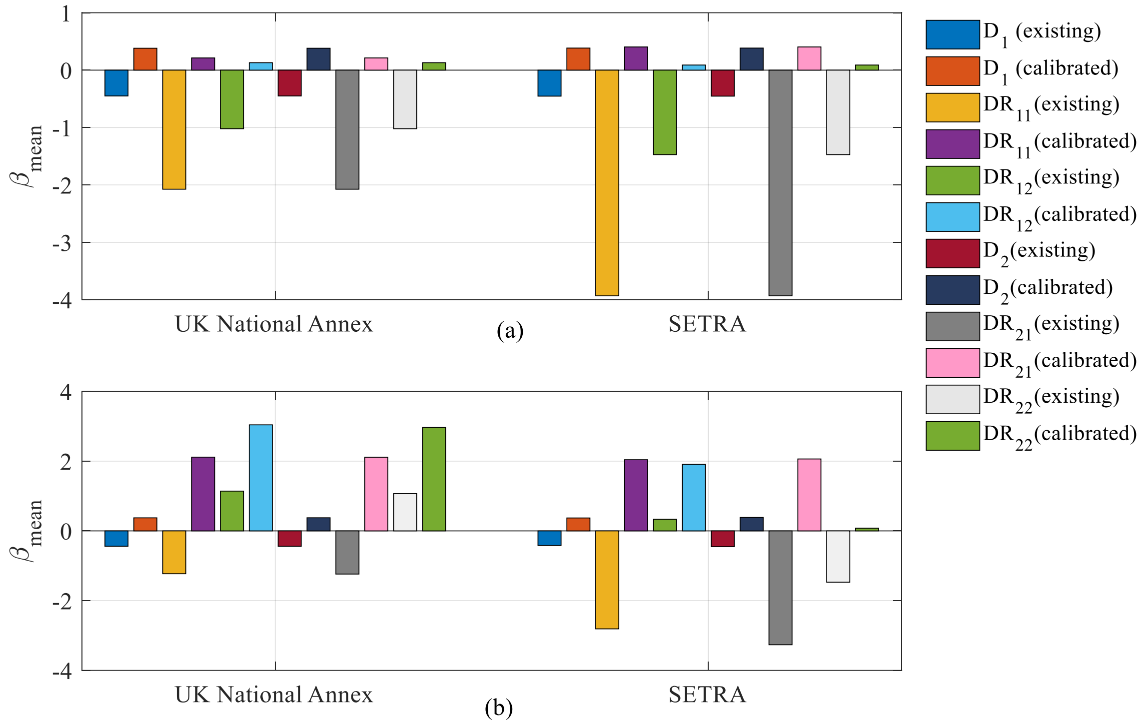

Figure 3: Mean reliability indices implied in the standards before and after calibration for pedestrian

4 CONCLUSIONS

This paper summarizes the key findings from the evaluation and calibration study conducted by the authors on design standards, specifically the UK National Annex and SÉTRA guide, for vibration serviceability of pedestrian bridges. Evaluation of the standards in the deterministic framework is based on a comprehensive experimental program, which was undertaken by the authors on two full-scale aluminum pedestrian bridges. These bridges were instrumented and subjected to a range of modal and pedestrian walking tests of varying traffic sizes. The comparison of measured and predicted vertical responses in general shows that the standards overestimate the measurements in the vertical direction while they fail to capture the serviceability outcomes of high frequency aluminum bridges having the possibility of resonance with higher harmonics of walking frequency. Some of the main conclusions of this comparison study are that the dynamic load factors used by these standards need to be revisited with a sufficient number of harmonics, and appropriate resonating harmonics should be considered through the traffic dependent average walking speed in a group. Hence, modification in the design provisions are performed through adopting revised values of the dynamic load factors and the multiplication factor. In addition, appropriate harmonic for resonance through traffic dependent walking speed and the effect of added mass from pedestrians are accounted for predicting the bridge response. Upon including these modifications into the existing design provisions, it is observed that they can be improved substantially in predicting the observed vibration behavior of lightweight pedestrian bridges.

In addition, the design standards are evaluated in a reliability-based framework through incorporating the primary sources of uncertainty associated with pedestrian loads and the acceleration limits. The reliability indices estimated for code-compliant bridges reveal that the design standards do not satisfy the target reliability index value of 0 (for this serviceability limit state) under the design and rare traffic densities, while the deviation from the target value is very high under rare events. Hence, the standards are calibrated to achieve sufficient reliability under both the design and rare loading events by iteratively estimating the desired reliability index for calibrating the standards. The general conclusion from the code calibration is that it is possible to achieve a sufficient reliability index across all of the code-compliant bridges under both design and rare loading events, while ensuring economic designs by adopting acceleration limits depending on the occurrence of the traffic event and bridge class.

5 ACKNOWLEDGEMENTS

The authors would like to acknowledge the funding provided to perform this work by the MAADI Group, the Aluminum Association of Canada, and the Natural Sciences Engineering Research Council of Canada through their collaborative research and development grants (CRD) program.

6 REFERENCES

Blanchard, J., Davies, B. L. Davies, and Smith J. W. 1977. Design Criteria and Analysis for Dynamic Loading of Footbridges. *In Proceeding of a Symposium on Dynamic Behaviour of Bridges at the Transport and Road Research Laboratory,* Crowthorne, Berkshire, England.

CSA (Canadian Standards Association). 2011. Canadian highway bridge design code. CAN/CSA S6. Toronto: CSA.

BSI (British Standards Institution). 2003. UK National Annex to Eurocode 1. Actions on structures. Traffic loads on bridges. BS NA EN 1991-2. London: British Standards Institution.

CEN (European Committee of Standardization). 2004. Design of timber structures—Part 2.

Dey, P., Sychterz, A., Narasimhan, S. and Walbridge, S., 2016. Performance of pedestrian-load models through experimental studies on lightweight aluminum bridges. *Journal of Bridge Engineering*, *21*(8), p.C4015005.

Dey, P., Narasimhan, S. and Walbridge, S., 2016. Evaluation of design guidelines for the serviceability assessment of aluminum pedestrian bridges. *Journal of Bridge Engineering*, *22*(1), p.04016109.

Dey, P., Walbridge, S. and Narasimhan, S., 2017. Evaluation of design provisions for pedestrian bridges using a structural reliability framework. *Journal of Bridge Engineering*, *23*(2), p.04017132.

Dey, P., Narasimhan, S. and Walbridge, S., 2018. Calibrating Pedestrian-Bridge Standards for Vibration Serviceability. *Journal of Bridge Engineering*, *23*(10), p.04018072.

Hasofer, A., and Lind, N. 1974. “Exact and invariant second-moment code format (for reliability analysis in multivariate problems).” J. Eng. Mech. Div. 100 (1): 111–121.

ISO. 1998. General principles on reliability for structures. ISO 2394. Geneva: ISO.

Portier, K. K. T. J., and Roberts, S.M. 2007. “Body weight distributions for risk assessment.” Risk Anal. 27 (1): 11–26.

SÉTRA (Service d’Études techniques des routes et autoroutes). 2006. Assessment of vibrational behaviour of footbridges under pedestrian loading. Technical guide. Paris: SÉTRA.

Willford, M. R., and Young, P. M. 2006. A design guide for footfall induced vibration of structures. Surrey, UK: Concrete Society.

Živanovic, S. 2006. “Probability-based estimation of vibration for pedestrian structures due to walking.” Ph.D. thesis, Univ. of Sheffield.