Vibration serviceability limit state of pedestrian bridges*

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ABSTRACT

Modern constructions of pedestrian bridges must satisfy functional and increasingly complex architectural requirements. In order to achieve the attractive design of pedestrian bridges, modern constructions generally differ from older, conventional solutions. The vibration problem of such structures shows that the dynamic response of the structure is governing for the design. This paper presents the comparative analysis of several dynamic models of pedestrian loads, described in various standards, guidelines, and recommendations. On the example of an arched pedestrian bridge, comparative analysis of the structure behavior due to the effect of pedestrian load was performed using numerical simulations in the SOFISTIK software, with the results obtained by analytical calculation procedures.

1 Introduction

The development of building materials, construction technologies as well as various design methods result in the construction of attractive, slender structures of a large span. Because of the reduction of the rigidity of the structure, the natural frequencies of the bridge decrease, which increases the risk of uncomfortable vibrations, for example vibrations caused by human walk. The modern approach of checking functionality and comfort as the serviceability limit state implies the analysis of the dynamic response of the structure under the action of the design load, with a frequency corresponding to the natural frequency of the bridge, which causes the most unfavorable response of the structure. Human sensitivity to vibrations is most often expressed by the acceleration of the deck and the time of exposure to vibrations. The criterion of vibrations acceptability of structures is basically a function of frequency and is mainly expressed in units of acceleration [3]. This article was created as a result of a master's thesis defended at the University of Belgrade - Faculty of Civil Engineering.

2 Dynamic loads of pedestrian bridges

The movement of pedestrians on the bridge causes a dynamic force that is variable in time and space, and which has components in three different directions: a vertical component and two components in the horizontal direction - transverse and longitudinal. The force component in the vertical direction is the subject of most studies for the simple reason that it has the largest load magnitude. In recent years, more detailed studies have been done, showing that the lateral component of the force (in the transverse direction)

can also cause problems regarding the serviceability limit state of the structure.

In general, the load caused by the pedestrian movement on the bridge can occur due to various activities such as walking, running or jumping. Each of the possible pedestrian activities on the bridge can be represented by a load curve that includes the intensity and frequency of the pedestrian load.

The intensity of the vertical component of the force due to the pedestrian movement is determined primarily by the weight of the pedestrian, the length of the steps as well as the frequency of walking. The most common frequency range that occurs due to pedestrian movement is 1.6-2.4 Hz.

The horizontal components of the force are much less intense than the vertical component but cannot be ignored. The main reason for transverse vibrations control is the so-called "lock in" effect, when a group of pedestrians, moving at different frequencies, begins to gradually adjust the walking frequency to the natural frequency of the bridge which leads to pedestrian-induced forces resonate with the structure. Small values of transverse vibrations may be sufficient to throw the pedestrian out of balance [3].

Eurocode, EN 1990: 2002 [1] in Annex A2.4.3.2, defines comfort criteria and criteria in which dynamic analysis of the structure is required. The comfort criterion is defined by the maximum allowed structure accelerations. Dynamic analysis is required to prove comfort in cases where the vertical natural frequency of the structure is less than 5 Hz and the horizontal or torsional frequency is less than 2.5 Hz. Vertical accelerations are limited to 0.7 m/s², while horizontal accelerations are limited to 0.2 m/s² under normal operating conditions and 0.4 m/s² for exceptional crowd conditions.

Dynamic load models which should be used in the analysis are not given, it is left to the National Annexes to propose the dynamic load models.

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2.1 Dynamic load models

2.1.1 Eurocode 1 Prestandard

Prestandard of Eurocode 1 (pr EN 1991-2:2003) [4] proposed Annex C which provides recommendations for determining the natural frequencies of the structure, damping as well as dynamic load models to be applied in verifying the serviceability limit state. Three load models have been proposed that simulate the movement of a single pedestrian (DLM1), a group of pedestrians (DLM2), and a continuous stream of pedestrians (DLM3). DLM1 and DLM2 are defined as a stationary pulsating force that has two components to be considered separately. Dynamic load model DLM3 consists of a uniformly distributed pulsating surface load $[N/m^2]$ with two components also to be considered separately. In the following expressions: f_v , f_h – are vertical and horizontal natural frequency of the bridge, $k(f_v)$, $k(f_h)$ – pedestrian synchronization factors depending on the frequency [4].

2.1.2 UK National annex to EN 1991-2

The UK National Annex to Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges [8] defines dynamic load models to be applied in the design of pedestrian bridges, as well as comfort criteria in the form of limitations of vertical accelerations of the structure. In Chapter NA.2.44.2 bridges are classified into four categories, depending on their usage and location, on the basis of which the number of pedestrians moving on the bridge is determined, as well as the density $[ped/m^2]$ in case of crowd loading. Two load models are given that represent the passage of single pedestrian or pedestrian groups as well as the steady state modeling of pedestrians in crowded conditions.

The maximum vertical accelerations that result from single pedestrian or group of pedestrians, should be calculated assuming that these loads are represented by application of a vertical pulsating load $F(N)$ moving across the span of the bridge at a constant speed $v(t)$:

$$F = F_0 \cdot k(f_v) \cdot \sqrt{1 + \gamma \cdot (N - 1)} \cdot \sin(2\pi f_v t)$$

The design maximum vertical accelerations that result from pedestrians in crowded conditions may be calculated by assuming that these are represented by a vertical pulsating distributed load $w [N/m^2]$, applied to the deck for a sufficient time so that steady state conditions are achieved as follows:

$$w = 1.8 \cdot \frac{F_0}{A} \cdot k(f_v) \cdot \sqrt{\gamma \cdot \frac{N}{\lambda}} \cdot \sin(2\pi f_v t)$$

In the previous expressions: N – is the number of pedestrians determined based on the defined class of the bridge, F_0 – reference amplitude of the pulsating force $[N]$, f_v – natural frequency of the vertical mode under consideration, $k(f_v)$ – combined factor to deal with the effects of a more realistic pedestrian population, harmonic responses and relative weighting of pedestrian sensitivity to response, t – elapsed time $[s]$, γ – reduction factor to allow for the unsynchronized combination of actions in a pedestrian group, is a function of damping and effective span, S_{eff} – effective span (in all cases it is conservative to use $S_{eff} = S$), S – bridge span $[m]$

ρ – required crowd density obtained from, b – width of the bridge subject to pedestrian loading, λ – factor that reduces the effective number of pedestrians when loading from only part of the span contributes to the mode of interest [8].

2.1.3 Sétra Guide (Service d'Études techniques des routes et autoroutes)

The recommendations given in the Sétra Guide [2] aim to summarize current knowledge on the dynamic behavior of pedestrian bridges due to pedestrian loading.

These recommendations also define bridge classification, as a function of traffic level and location, and the required comfort level which directly depends on the acceleration of the structure. Given the subjective nature of the comfort concept and convenience, the range of acceleration is defined, not the individual value. The methodology proposed by Sétra [2] makes it possible to limit the risk of the occurrence of the resonance caused by the pedestrian load.

The recommendations define three cases of dynamic loading depending on the bridge class and the ranges within which its natural frequencies are situated. Load models for different directions should be considered individually, therefore the dynamic response of the structure should be determined for each load direction separately. The load should be applied to the whole surface of the deck which is intended for the pedestrian movement. Also, the load should be such that the amplitude of the force is always of the same sign as the mode shape. In this way, the maximum response of the structure is obtained, see Figure 1 [2].

The load amplitudes of different dynamic models, for the frequency of the second oscillation mode are shown in Table 2.

Table 1. Dynamic Load Models according to Prestandard of Eurocode 1

Single pedestrian – DLM1	Group of pedestrians – DLM2	Continuous pedestrian stream – DLM3

Vertical component of pulsating force		
$Q_{pv} = 280 \sin(2\pi f_v t)$	$Q_{gv} = 280 k_v(f_v) \sin(2\pi f_v t)$	$q_{sv} = 12.6 k_v(f_v) \sin(2\pi f_v t)$
Horizontal component of pulsating force		
$Q_{ph} = 70 \sin(2\pi f_h t)$	$Q_{gh} = 70 k_h(f_h) \sin(2\pi f_h t)$	$q_{sh} = 3.2 k_h(f_h) \sin(2\pi f_h t)$

Figure 1. Sign of the amplitude of the load in the case of a mode with several sags [2]

Table 2. Dynamic load amplitudes according to relevant standards

Standard	Load model	Vertical amplitude	Horizontal amplitude
EC 1	DLM1	280 [N]	70 [N]
	DLM2	840 [N]	210 [N]
	DLM3	37.8 [N/m ²]	9.6 [N/m ²]
UK NA	Walking N = 8	434 [N]	/
	Running N = 2	1097 [N]	/
	Pedestrian Crowd	9.0 [N/m ²]	/
Sètra	Walking N = 1	280 [N]	35 [N]
	Pedestrian crowd	14.87 [N/m ²]	1.96 [N/m ²]

3 Dynamic analysis of an arched pedestrian bridge

On the example of one pedestrian bridge, a parametric analysis was performed. It was checked whether the structure meets the serviceability criteria in terms of vibrations induced by pedestrians according to the stated standards and guidelines. A numerical analysis was performed in the SOFISTIK software [7] using time history analysis by step-by-step integration method.

3.1 Technical description of the structure

The conceptual design of the pedestrian bridge over the river Đetinja at the location of Stari grad in Užice was analyzed. The structure of the bridge is formed by two steel arch girders positioned at a distance of 3 m from each other.

The arch span is 70 m while the total length of the bridge is approximately 100 m. The rise of the arch is 12.52 m so that the ratio of the rise and the span f/L is 0.18 which corresponds to the recommended values, see figure 2.

The deck structure consists of longitudinal and transverse beams that rest on the main arch girders via the columns. Columns are positioned at a distance of 7 m. In order to provide the lateral stability of the bridge, the arches are interconnected by beams at the joints in the zones of columns supports. Vertical bracings are provided between columns and are made in the form of crossed diagonals. The spatial stability is also achieved by a horizontal bracing member at the level of the lower flange of the transverse beam. The longitudinal and transverse beams form a structural system that serves as a support structure for the glass panels that make up the deck for pedestrian movement.

Figure 2. Longitudinal and cross section of the bridge

3.2 Bearing capacity of the glass panel

Glass has been increasingly used as a material in construction in recent years in order to increase the aesthetic value of the buildings. The unique properties of glass, such as unlimited lifespan, transparency, and the ability of letting the light passing through the material lead to the glass being increasingly used not only for non-structural elements but also for structural elements of the buildings. Glass is a brittle material, sensitive to stress concentrations. Laminated glass panels composed of two or more glass panels connected by interlayers of a certain transparent plastic material - glue - are most often used for the glass structural elements. During breakage, cracks appear on the panel constructed in this way and if it disintegrates, pieces of glass will remain attached to the foil layer. The development of standards for the design of glass structures does not keep pace with the dynamic development of the glass industry. The paper analyzes the bearing capacity of the deck in accordance with the draft version of the standard pr EN 13474 "Glass in Buildings" - Part 1 and Part 2 [5] [6].

The behavior of the panel predominantly depends on the properties of the polymer interlayer. The calculation of stresses and deflections is based on the concept of effective thickness [5]. The essence of the concept is to approximate the laminated panel with an equivalent glass panel made of homogeneous material. The structural details of the bond ensure that the panels are loaded only perpendicular to their plane and that the horizontal and vertical loads from pedestrian traffic are transferred to the transverse and longitudinal deck supports. Bearing capacity has been

conducted for the dead load and traffic load of $q=5.0\text{ kN/m}^2$. Rectangular laminated glass panels having dimensions $3.0 \times 2.33\text{ m}$ have been adopted. These dimensions were determined by the spacing of longitudinal and transverse beams of the deck structure. The laminated panels are formed from three panels of tempered glass 19mm thick and two PVB interlayers 1.52mm thick. The total thickness of the panel is 60mm. The effective thicknesses for the calculation of deflection and stress are 27.40 mm and 32.91 mm, respectively. For a panel formed in such a way, the ultimate stress values are 39.32 MPa, and the allowable deflection values according to the recommendations of the standard are 10 mm [6]. Table 3 shows the results of obtained stress and deflection.

3.3 Natural frequencies of the structure

Natural frequencies of the structure are determined using the numerical analysis in the SOFISTIK software [7]. Figure 4 shows the relevant vibration modes with vertical frequencies less than 5 Hz and horizontal frequencies less than 2.5 Hz that require additional dynamic analysis.

A very important input parameter for conducting the dynamic analysis is the relative damping of the structure. In the general case, this value can only be estimated. The damping was determined on the basis of the logarithmic decrement δ . The recommended value of this parameter for steel is 0.03 and it was used as input data in the SOFISTIK software [7].

Table 3. Laminated glass panels analysis results

Calculation method	Stress [MPa]	Deflection [mm]
Numerical analysis - SOFISTIK	$16.98 < f_{g,d}=39.32\text{ MPa}$	$7.08 < 10$

Figure 3. Stress and deflection diagrams of glass panel

Figure 4. Natural frequencies and mode shapes: a) horizontal - $f_h=1.158\text{ Hz}$, b) vertical - $f_{v1}=2.219\text{ Hz}$, c) vertical - $f_{v2}=3.444\text{ Hz}$

3.4 Dynamic analysis results

Prestandard of Eurocode 1 [4] defines a dynamic load that does not move across the bridge but acts at a single point (DLM1 and DLM2) or as a stationary surface load (DLM3). Load duration is an important calculation parameter. The duration of the load was varied iteratively in order to achieve a steady response of the structure due to the dynamic forces and the adopted damping. The values of the obtained accelerations were recorded after 40,0 s of action of the force. The adopted value of the timestep is $\Delta t = 0.02$ s. The load amplitudes were determined in accordance with the expressions provided in table 1. Coefficients $k_v(f_v)$ and $k_h(f_h)$ were determined based on the diagrams provided in [4]. Based on the results presented in table 4 it can be noticed that the vertical acceleration criteria, defined by the standard, was exceeded in case of the DLM3 load model – continuous pedestrian stream. The vertical acceleration diagram and the function of the dynamic surface load are presented in figure 5.

The UK National Annex to Eurocode 1 [8] defines the vertical dynamic loads to be applied to determine the structural response. Loads are defined for various pedestrian activities such as walking and running. The load amplitudes depend on the adopted bridge class and the obtained frequencies of the structure. Class C was adopted for the considered pedestrian bridge, on the basis of which the sizes of groups of pedestrians for walking, running as well as the density of pedestrians on the bridge in case of a crowd loading were determined.

The load of a group of pedestrians walking or running on the bridge is modeled as a concentrated force moving along a line located in the middle of the deck structure. Dynamic load is defined by a sinusoidal function which means that the amplitude of the load changes over time. The concentrated force acts in the nodes whose distance is defined by the speed of pedestrians as defined by the National Annex [8] and amounts 1.7 m/s for walking, or 3.0 m/s for running and with the load frequency that coincides with the natural frequency of the bridge to obtain the most unfavorable response of the structure.

Table 4. Dynamic analysis results according to Prestandard of Eurocode 1 [4]

Load model	Vertical acceleration [m/s ²]	Vertical acceleration criteria [m/s ²]	Horizontal acceleration [m/s ²]	Horizontal acceleration criteria [m/s ²]
DLM1	0.09	0.7	0.02	0.2
DLM2	0.26	0.7	0.05	0.2
DLM3	0.75	0.7	0.36	0.4

Figure 5. Vertical acceleration diagram (left) and DLM3 load model function (right)

Table 5. Dynamic analysis results according to BS EN1991-2 NA.2.44 [8]

Load model	Natural frequency [Hz]	Vertical acceleration [m/s ²]	Acceleration criteria [m/s ²]
Walking N=8	2.219	0.10	0.7
Running N=2	2.219	0.22	0.7
Running N=2	3.444	0.09	0.7
Crowd loading [0.8 ped/m ²]	2.219	0.35	0.7

Figure 6. Vertical acceleration diagram (left) and moving force load function (right) – Running N=2

In the dynamic analysis, in accordance with the recommendations of the Sétra Guide [2], bridge class 2 was adopted, which corresponds to the class C given in the UK National Annex [8]. Based on the adopted bridge class and calculated natural frequencies and in accordance with the calculation methodology, load case to be applied to calculate the vertical and horizontal response of the structure was determined. The recommendations given in these guidelines mainly refer to the amplitudes of dynamic loads caused by pedestrian crowd, since this case is usually governing. The load case applied in the dynamic analysis is Load case 1 to which corresponds the pedestrian density of 0.8 ped/m².

Having in mind the dimensions of the structure, it is expected that the accelerations caused by the movement of single pedestrian, or a group of pedestrians are significantly below the acceleration criteria, and that the case of a pedestrian crowd loading is most relevant to satisfy the serviceability limit state in terms of vibrations, which is proven by dynamic analysis.

Based on the values shown in Table 7, it can be concluded that the criterion of the serviceability limit state in terms of vibrations is met according to the recommendations of UK NA [8] and Sétra Guide [2] for bridge classes C and 2, respectively, while the vertical accelerations criteria is exceeded according to recommendations given in Prestandard of Eurocode 1 [4].

Table 6. Dynamic analysis results according to Sétra [2]

Load model	Vertical acceleration [m/s ²]	Acceleration ranges and comfort criteria [m/s ²]	Horizontal acceleration [m/s ²]	Horizontal acceleration criteria [m/s ²]
Dense crowd Class 2	0.55	0.0 ÷ 0.5 – max comfort	0.06	0.1
Very dense crowd Class 1	1.23	0.5 ÷ 1.0 – mean comfort 1.0 ÷ 2.5 – min comfort	1.14	0.1

Figure 7. Vertical acceleration diagram (left) and dynamic load function (right) - Sétra Load case 1

Table 7. Comparative analysis for pedestrian crowd case

Standard	EC1	BS NA	Sétra Bridge class 2	Sétra Bridge class 1
Load model	DLM3	Pedestrian crowd	Pedestrian crowd	Pedestrian crowd
Vertical acceleration [m/s ²]	0.75	0.35	0.55	1.23
Horizontal acceleration [m/s ²]	0.36	/	0.06	0.14

According to the comfort criteria defined by Sétra Guide [2], the obtained value of vertical acceleration, in case when a bridge is treated as class 2 bridge, is 0.55 m/s² which corresponds to the mean comfort. Taking into consideration the purpose and location of the bridge, the structural response is determined for class 1 bridge too. Such approach to the analysis is justified in certain situations such as the opening day when the bridge is commissioned, when higher densities of pedestrians may occur on the bridge. In this case, vertical acceleration is 1.23 m/s² and it corresponds to the minimum comfort, while the horizontal acceleration criteria, which is limited because of the "lock in" effect, is exceeded.

4 Conclusions

The very fact that the official versions of Eurocode do not define precise recommendations and data for performing dynamic analysis due to pedestrian load indicates that this is a complex engineering problem. It has been shown that there are differences in the results of the dynamic analysis obtained on the basis of different recommendations and guidelines. The synchronization of a large number of pedestrians, defined through dynamic coefficients, has the greatest impact on the obtained differences. By designing structures according to the UK National Annex and the Sétra Guide, footbridge designs having lower stiffness can be obtained, satisfying the serviceability limit state in terms of vibration. The analysis of the structure according to the Prestandard of Eurocode 1 shows that the adopted solution does not meet the minimum comfort criteria. In this case, an increase in the height of the longitudinal deck girder or the introduction of vibration dampers should be considered.

The obtained results indicate that by using advanced numerical methods, serviceability limit state in terms of vibrations of the lower rank bridges can be satisfied, even when the calculated frequencies of the structure are in critical ranges where there is the maximum risk of resonance occurring. It should be emphasized that the calculation results are very sensitive to the bridge span (dead load of the structure). Also, it is necessary to carefully analyze the class/rank of the bridge, with a precise prediction of pedestrian traffic in the future, so that a bridge that is classified in a lower rank would not move to a higher one and thus potentially endanger the comfort on the bridge.

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