Acta Polytechnica CTU Proceedings **49**:1–7, 2024
https://doi.org/10.14311/APP.2024.49.0001

EXPERIMENTAL DYNAMIC ANALYSIS OF THE FOOTBRIDGE ACROSS JIZERA RIVER IN MLADÁ BOLESLAV

Miroslav Čáp[a,c], Vladimír Šána[b,*], Michal Polák[a], Tomáš Plachý[a]

[a] *Czech Technical University in Prague, Faculty of Civil Engineering, Department of Mechanics, Thákurova 7, 160 00 Prague, Czech Republic*

[b] *Czech Technical University in Prague, Faculty of Civil Engineering, Experimental Centre, Thákurova 7, 160 00 Prague, Czech Republic*

[c] *Pontex Consulting Engineers, Ltd., Bezová 1658, 147 14 Prague, Czech Republic*

[*] corresponding author: vladimir.sana@fsv.cvut.cz

Abstract. The text of this submitted paper is devoted to the experimental dynamic analysis of the newly designed footbridge across the Jizera River in Mladá Boleslav. Theoretical modal analysis has shown potential risk that some of the natural frequencies of the bridge deck will belong to the range which is typical for pacing frequencies induced by pedestrians. The resonance behaviour of this structure should be reduced by Tuned Mass Dampers (TMD), which would be tuned for separate natural frequencies of this structure. Therefore, the experimental dynamic analysis was performed on the footbridge in order to assess the effectiveness of installed TMDs. The experiment was divided into two stages, the first one was realized at the footbridge when TMDs were not yet installed, and the second one was carried out on the footbridge with installed and activated TMDs. Moreover, the authors have performed an experimental modal analysis in order to verify the aptness of the computational model and its results.

Keywords: Experimental dynamic analysis, dynamic load test, experimental modal analysis, human-induced vibration, vandalism.

1. Introduction

As can be seen from the number of contributions and participants, who regularly presents their papers at the international conferences dealing with the structural dynamics, such as EURODYN, IMAC, etc., and in technical journals, dynamic behaviour (obtained by either numerical predictions or in-situ experiments) of the footbridges is still at the foreground of international researchers community interest. The newly built structures are designed still more and more subtle thanks to the advanced computational procedures and modern materials. The effort to create more slender and subtle structures is caused in particular by a desire of architects to not break the view of the surrounding landscape. The combination of slenderness, low value of damping, physical properties of used materials, such as mass and stiffness, and static system of the superstructure lead very often to the fact, that some of the natural frequencies are scattered very closely in the region of the pacing frequencies induced by pedestrians. On the other hand, the artistic efforts of the architects create a natural pressure on designers (in the sense of numerical calculations and choice of a correct mathematical model of pedestrian, vandal, etc.), who must properly and with a reasonable measure of the accuracy determine the response of the individual members of the footbridge load-bearing structure. The final maximal level of vibration in the dimension of acceleration is used for structural assessment with respect to pedestrians' comfort. The maximal values are usually less suitable for a footbridge experimental assessment since their value can be significantly affected by sudden impulse loading, such as stamping in the close vicinity of the used accelerometers, etc. Therefore, some guidelines and standards recommend RMS (Root Mean Square) values of acceleration for assessing a comfort level of a footbridge structure with respect to pedestrians.

In literature, one can find a lot of sources, where authors designed TMD on existing or newly built structures. In [1], authors designed and assessed a TMD on the cable-stayed footbridge. Ferreira et al. in [2] proposed a design of a TMD and SATMD (semi-active tuned mass damper) on the Infinity bridge in Stockton, Great Britain, which is an arch footbridge across River Tees. The SATMD was used as an efficient tool to prevent the lock-in effect, which is in this case lateral instability of the footbridge induced by a critical number of synchronous pedestrians in the lateral direction. A numerical and experimental study of simple structure with friction TMD was addressed by Eliecer et al. in [3] in order to reduce the vibrations of the floor. The bridge deck vibration of the lively footbridge across Motlawa River (Gdansk, Poland) was reduced by high coefficient TMD, see [4]. The suspended footbridge Żabia kładka in Wrocław and cable-stayed footbridge in Poznań, which have been enriched by TMDs, were studied in [5]. Optimal parameters of TMDs, such as their mass, stiffness, and

Figure 1. A view of the investigated footbridge in the direction of stationing.

damping properties, can be found e.g. in [6] or [7]. Sometimes, the situation requires and permits the usage of TLD (Tuned Liquid Damper), where the kinetic energy of the vibrating structure is absorbed by the inertia of moving liquid, see e.g. [8].

2. Description of the investigated structure

For the purposes of the presented paper, we have chosen the footbridge, which is located in Mladá Boleslav (Czech Republic) and serves to pedestrian and cycling traffic as a part of the cycling route A1. A view of the structure in the direction of stationing is depicted in Figure 1 and Figure 2. The experimentally investigated footbridge is steel trussed structure with an orthotropic bridge deck, see Figure 3, and with two simply-supported spans, which is mounted on two massive abutments and a reinforced-concrete pillar. The length of individual spans are $L_1 = 68.40$ m and $L_2 = 23.68$ m, see Figure 4. Width of the superstructure of 4.60 m is constant along the entire structure, see Figure 2.

The footbridge centreline is direct in the horizontal direction and curved in the vertical plane with a radius of $R = 811.541$ m, see Figure 1 and Figure 2.

The main beams have been constructed from two trussed beams made of S355J2+N and S355J2H category steel. Top and bottom box-sectional chords with outer dimensions of 250 × 200 mm are welded to an arc. The thickness of the flanges and webs at the bottom chord is 15 mm. The top chord has a thickness of webs 15 mm and flanges of 15 mm, 20 mm and 25 mm. Both truss beams are rigidly connected by horizontal box-sectional bracing with dimensions of 250 × 200 mm. The height of the main beams is variable with respect to the longitudinal axis. The webs of the trussed beams are a combination of rigid tubes, box-sectional rods, providing the shape stability of the top chords, and pair of prestressed rectifiable rods.

The bridge deck was designed from a 10 mm plate reinforced with a system of longitudinal and transversal stiffeners with plate profiles and inverted T profiles. The profiles were added to the places with the premised locations of Tuned Mass Dampers (TMDs).

Figure 2. A view of the bridge deck in the direction of stationing.

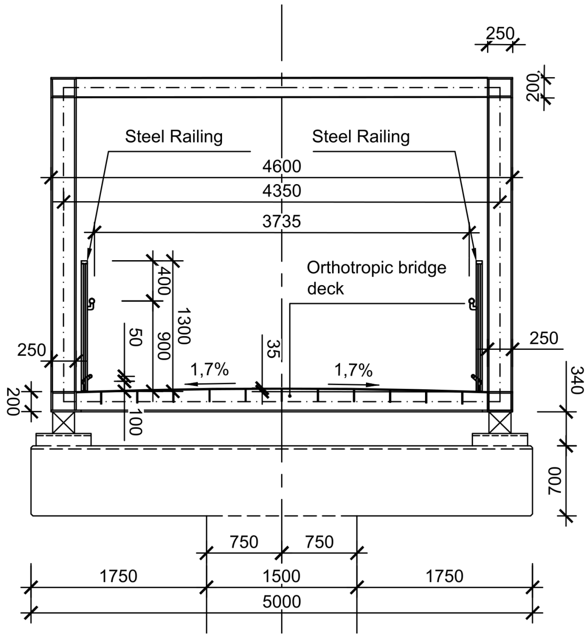

Figure 3. A cross-section of the investigated footbridge structure – support [10].

Generally, 8 TMDs were designed in the theoretical dynamic analysis, see [9]. The distance between separate cross-girders with inverted T profiles is 3000 mm and the mutual spacing of longitudinal plate stiffeners is 360 mm. The distance between inverted T profiles and plate stiffeners at the places, where the inverted T profiles were added, is 270 mm.

3. Experiment

The in-situ experiment was divided into two phases. While the first stage was focused on the determination of the natural frequencies, relevant mode shapes of the superstructure (experimental modal analysis, informative dynamic test), and the response of the superstructure without installed TMDs, the second stage was primarily aimed at verification of the effectiveness of the Tuned Mass Dampers (TMDs) after

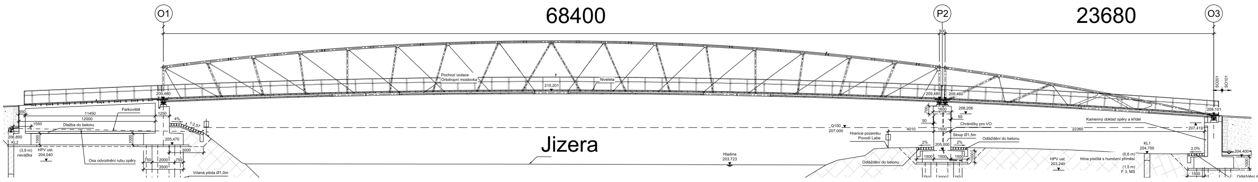

Figure 4. A longitudinal section of the investigated footbridge structure [10].

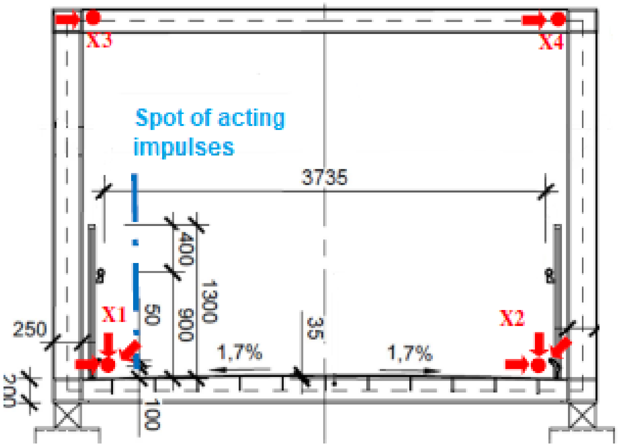

Figure 5. A view at sensor placement.

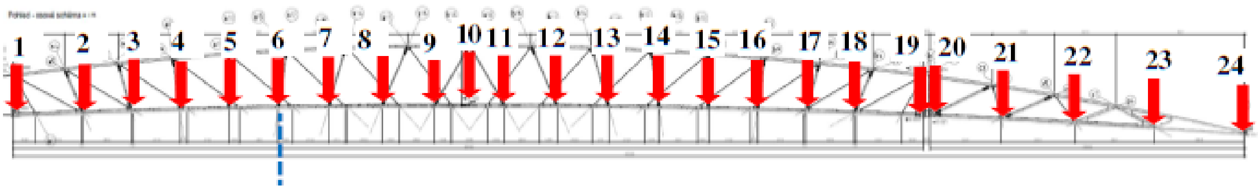

Figure 6. A view at sensor placement in a longitudinal section.

their installation and activation for a purpose of decreasing a level of vibration of the bridge deck. The dynamic forces were produced by diversely formed groups of pedestrians. The groups of pedestrians used in the second stage corresponded to the groups from the first stage. Thus the results from both stages were mutually comparable and we were able to determine the effect of activated TMDs.

Response of the structure was observed in the preselected mesh of points by accelerometers, which were adjusted by steel weights and placed in the correct position. These used accelerometers were piezoelectric seismic sensors of type 8344 (Brüel&Kjær). The sensors were adjusted to the top chord and on the steel weights directly by neodymium magnets, see Figure 5. Sensitivity of the accelerometers is 2500 mV g^{-1} with a frequency range of 0.2 Hz–3 kHz.

Eight-channel vibration control stations SIRIUS 6ACC – 2ACC+ and SIRIUS 8ACC have been used to collect seismic sensor data.

3.1. Experimental modal analysis

Experimental modal analysis (dynamic informative test within the meaning of standard ČSN 73 6209 [11]) was focused on the determination of natural vibration characteristics of the empty structure. The logarithmic decrement, dominant natural frequencies, and corresponding global mode shapes belong among these characteristics. The structure was excited by regularly jumping person – Ambient Vibration Test (AVT) in a predefined spot to excite all desired mode shapes.

The dash-and-dot line, depicted in Figure 5, represents the approximate spot of the jumping person. The eccentricity of this spot permitted the excitation of potential torsion mode shapes. Figure 6 represents the placement of individual measured sections in the longitudinal direction. Reference sensors were placed in points 62 and 102 (measurement in y and z directions) throughout the experimental modal analysis. Besides this, the next reference sensor was placed in point 104 (measurement in the y direction). During the experimental modal analysis of the second span, the reference sensors were located in point 222 (measurement in the y and the z directions).

Experimentally obtained data of the time behaviour of the oscillating bridge deck were stored on the hard drive and subsequently processed by Fast Fourier Transform (FFT), which transforms the original signal from the time domain to the frequency domain, and natural frequencies are depicted as local peaks. The width of the peak at a specific height is related to the damping connected with this natural frequency.

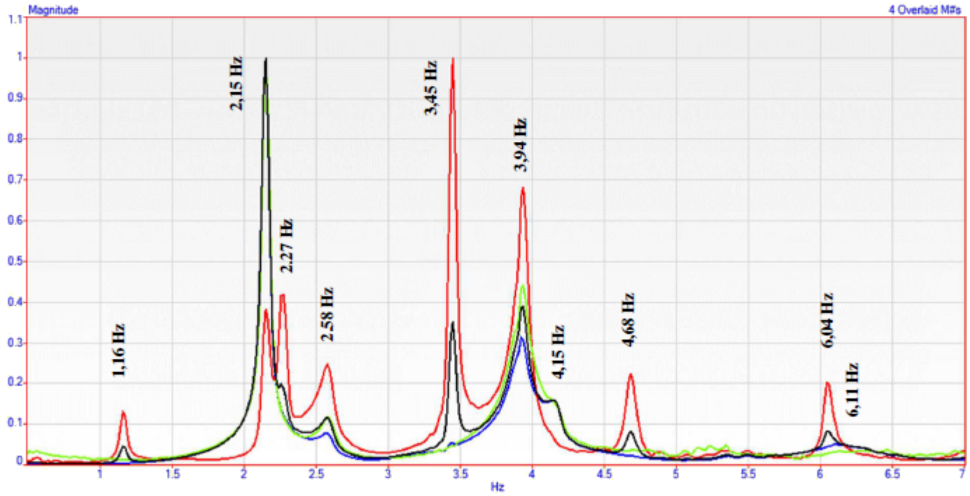

FIGURE 7. The example of Complex Mode Indicator Function (the black line – evaluated across all measured points in x, y, and z directions, the green curve in the x direction, the red line in the y direction, and the blue curve for the z direction).

Thanks to the fact, that the force of excitation was produced by a jumping person, Frequency Response Spectra in individual points on the bridge deck have been used for the determination of Operating Deflection Shapes Frequency Response Functions (ODSFRF or $\mathrm{ODSH}_{kR}(if)$). $\mathrm{ODSH}_{kR}(if)$ is generally a complex function consisting of real and imaginary part. Since the individual points have been fitted by sensors at different times and therefore the level of excitation force was not constant, the transmissibility functions $\mathrm{T}_{kR}(if)$ were determined as well. Transmissibility functions permit precise evaluation of the mode shapes. These functions were determined by the equation:

$$\mathrm{T}_{kR} = \frac{\ddot{w}_k(if)}{\ddot{w}_R(if)}, \tag{1}$$

where

i means an imaginary unit,

$\ddot{w}_k(if)$ stands for the response (acceleration) of the structure in point k in the frequency domain,

$\ddot{w}_R(if)$ denotes the response (acceleration) of the structure in reference point R in the frequency domain.

The natural frequencies and mode shapes were evaluated in the software ME'scope VES. The example of evaluated Complex Mode Indicator Function with depicted natural frequencies is presented in Figure 7.

3.2. DYNAMIC LOAD TEST

According to the standard ČSN 73 6209 [11], we have tested such an arrangement of pedestrians, which was in accordance with the requirements of this standard. An ordinary traffic is usually simulated by:

- Random footbridge crossing with a pedestrian flow density of the same order as the density during standard use of the structure,
- excitation of torsional or bending vibration; two synchronized pedestrians stepping on the same foot at the same time; pacing frequency according to natural frequencies of empty structure,
- excitation of lateral vibration; two synchronized pedestrians stepping on the same foot at the same time; pacing frequency according to natural frequencies of empty structure.

In addition to the ordinary traffic, other form of pedestrian excitation of the footbridge (for example running joggers, swaying and bobbing vandals) were also investigated. The walking pedestrians, running joggers and swaying/bobbing vandals have been synchronized by digital metronome.

3.2.1. THE FIRST STAGE – INACTIVATED TMD

On the basis of the previous experimental modal analysis, we have chosen the following pacing frequencies, which were the same as some of the natural frequencies:

The first span $\mathbf{L_1 = 68.40\,m}$

- $f_{(2)} = 2.15\,\mathrm{Hz}$ – the first shape of vertical bending vibration; $f_p = 2.15\,\mathrm{Hz}$, $f_v = 2.15\,\mathrm{Hz}$,
- $f_{(4)} = 2.58\,\mathrm{Hz}$ – the first shape of torsional vibration; $f_p = 2.58\,\mathrm{Hz}$,
- $f_{(5)} = 3.44\,\mathrm{Hz}$ – the third shape of lateral bending vibration (top chords of main beams); $f_p = 1.73\,\mathrm{Hz}$, $f_p = 3.44\,\mathrm{Hz}$ $f_v = 1.72\,\mathrm{Hz}$, $f_v = 3.44\,\mathrm{Hz}$,
- $f_{(6)} = 3.94\,\mathrm{Hz}$ – the shape of lateral bending vibration; $f_p = 1.97\,\mathrm{Hz}$, $f_v = 1.97\,\mathrm{Hz}$, $f_v = 1.95\,\mathrm{Hz}$,
- $f_{(7)} = 4.16\,\mathrm{Hz}$ – the second shape of vertical bending vibration (top chords of main beams); $f_p = 2.08\,\mathrm{Hz}$, $f_v = 2.08\,\mathrm{Hz}$.

The following compositions of loading groups have been used:

j	1^{st} span $f_{(j)}$ [Hz]	2^{nd} span $f_{(j)}$ [Hz]
1	1.16	5.27
2	2.15	5.92
3	2.27	7.31
4	2.58	8.42
5	3.45	11.46
6	3.94	11.92
7	4.15	13.87
8	4.68	15.59
9	6.04	17.21
10	6.11	-

TABLE 1. Natural frequencies of the first and the second spans during the first stage (inactivated TMD).

- Simulation of an ordinary traffic: 4 pedestrians (random crossing),
- synchronized walking/running: 2 pedestrians side by side,
- vandalism: 4 qualified vandals.

The second span $\mathbf{L_2 = 23.68\,m}$

- $f_{(1)} = 5.28\,\text{Hz}$ – the first shape of vertical bending vibration; $f_p = 1.76\,\text{Hz}$, $f_p = 2.64\,\text{Hz}$, $f_v = 1.76\,\text{Hz}$, $f_v = 2.64\,\text{Hz}$.

The following compositions of loading groups have been used:

- Simulation of ordinary traffic: 5 pedestrians (random crossing),
- synchronized walking/running: 2 pedestrians side by side,
- vandalism: 4 qualified vandals.

In the previous list, $f_{(k)}$ denotes k-th natural frequency, f_p stands for exciting frequency produced by pedestrians, and finally, f_v means exciting frequency produced by vandals.

3.2.2. The second stage – activated TMD

The first span $\mathbf{L_1 = 68.40\,m}$

- $f_{(2)} = 2.15\,\text{Hz}$ – the first shape of vertical bending vibration; $f_p = 2.15\,\text{Hz}$, $f_v = 2.15\,\text{Hz}$,
- $f_{(4)} = 2.47\,\text{Hz}$ – the first shape of torsional vibration; $f_p = 2.47\,\text{Hz}$,
- $f_{(5)} = 3.42\,\text{Hz}$ – the third shape of lateral bending vibration (top chords of main beams); $f_p = 1.71\,\text{Hz}$, $f_p = 3.42\,\text{Hz}$ $f_v = 1.71\,\text{Hz}$, $f_v = 3.42\,\text{Hz}$,
- $f_{(6)} = 3.94\,\text{Hz}$ – the shape of lateral bending vibration; $f_p = 1.95\,\text{Hz}$, $f_p = 1.98\,\text{Hz}$, $f_v = 1.95\,\text{Hz}$, $f_v = 1.98\,\text{Hz}$,
- $f_{(7)} = 4.16\,\text{Hz}$ – the second shape of vertical bending vibration (top chords of main beams); $f_p = 2.08\,\text{Hz}$, $f_v = 2.08\,\text{Hz}$.

The following compositions of loading pedestrian groups have been used:

- Simulation of ordinary traffic: 8–9 pedestrians (random crossing),
- synchronized walking/running: 2 pedestrians side by side,
- vandalism: 4 qualified vandals.

The second span $\mathbf{L_2 = 23.68\,m}$

- $f_{(1)} = 5.27\,\text{Hz}$ – the first shape of vertical bending vibration; $f_p = 1.76\,\text{Hz}$, $f_p = 2.64\,\text{Hz}$, $f_v = 1.76\,\text{Hz}$, $f_v = 2.64\,\text{Hz}$,
- $f_{(2)} = 5.92\,\text{Hz}$ – the first shape of vertical bending vibration; $f_p = 1.98\,\text{Hz}$, $f_p = 2.97\,\text{Hz}$, $f_v = 1.98\,\text{Hz}$, $f_v = 2.97\,\text{Hz}$,
- $f_p = 2.30\,\text{Hz}$, $f_v = 2.30\,\text{Hz}$.

The following compositions of loading groups have been used:

- Simulation of ordinary traffic: 4 pedestrians (random crossing),
- synchronized walking/running: 2 pedestrians side by side,
- vandalism: 3 qualified vandals.

4. Results

This text summarizes obtained and evaluated results from the in-situ experiment, which was focused on experimental modal analysis and forced vibration (dynamic load test). The Table 1 and Table 2 present evaluated natural frequencies of the footbridge during the first stage.

An example of the second mode shape of the first span is graphically depicted in Figure 8.

Table 3 summarizes the important values of damping. These values were determined for the first vertical bending frequency at the first span and for the first vertical bending frequency at the second span.

The following tables (Table 4, Table 5) present the results of maximal RMS values of acceleration at the first span because dominant values of measured dynamic response were observed right there.

j	1st span	2nd span
1	Lateral bending – top chords	Vertical bending – bridge deck
2	Vertical bending – bridge deck	Lateral bending – top chords
3	Lateral bending – top chords	Torsional – bridge deck
4	Torsional – bridge deck	Torsional – bridge deck
5	Lateral bending – top chords	Lateral bending – top chords
6	Lateral bending – bridge deck	Vertical bending – bridge deck
7	Vertical bending – bridge deck	Lateral bending – top chords
8	Lateral bending – top chords	Vertical bending – bridge deck
9	Lateral bending – top chords	Torsional – bridge deck
10	Vertical bending – bridge deck	-

TABLE 2. Description of the individual evaluated mode shapes (inactivated TMD).

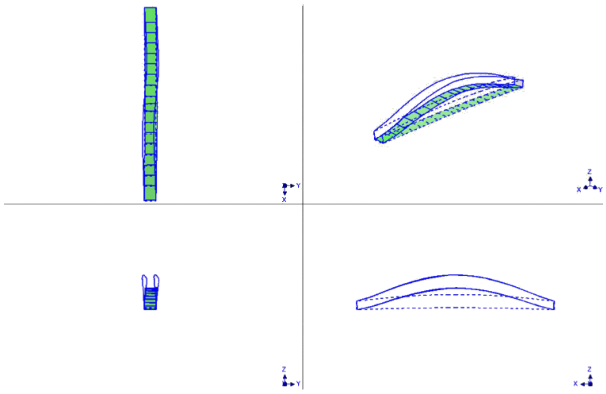

FIGURE 8. The second mode shape $f_{(2)} = 2.15$ [Hz] – The first span (inactivated TMD).

Span	f [Hz]	ϑ [-]	ξ [%]
1st	2.15	0.031	0.49
2nd	5.27	0.061	0.97

TABLE 3. Values of logarithmic damping decrement ϑ.

5. Conclusion

The submitted paper presents the results of the experimental dynamic analysis of the newly built footbridge across the Jizera River in Mladá Boleslav. The calculations, in the design phase, have shown that the structure must be supplemented by TMDs to decrease vibrations induced by pedestrians. The experiment should prove whether the installed absorbers are effective or not. As can be seen from Table 4 and Table 5, the absorbers helped significantly to decrease the level of vibration, which is the most noticeably evident for vandalism at a frequency of 1.95 Hz and synchronized running pedestrians at a frequency of 3.44 Hz.

Acknowledgements

The financial support has been provided by grant No. SGS22/089/OHK1/2T/11 of the Czech Technical University in Prague, which is gratefully acknowledged.

Description	Freq. [Hz]	Spot	Max [m s^{-2}]
Ordinary traffic	-	102-Z	0.170
Syn. ped.	1.73	102-Z	0.089
Syn. ped.	1.97	102-Z	0.123
Syn. ped.	2.08	102-Z	**0.258**
Syn. ped.	2.15	102-Z	0.245
Syn. ped.	2.58	102-Z	0.250
Syn. ped.	3.44	102-Z	**0.572**
Vandals	1.72	102-Z	0.117
Vandals	1.95	102-Z	**2.123**
Vandals	1.97	102-Z	1.376
Vandals	2.08	102-Z	0.340
Vandals	2.15	102-Z	0.491
Vandals	3.44	102-Z	0.205

TABLE 4. Maximal RMS values for individual exciting frequencies – the first stage (inactivated TMD).

Description	Freq. [Hz]	Spot	Max [m s^{-2}]
Ordinary traffic	-	102-Z	0.111
Syn. ped.	1.71	102-Z	0.062
Syn. ped.	1.79	102-Z	0.078
Syn. ped.	1.95	102-Z	0.086
Syn. ped.	1.98	102-Z	0.072
Syn. ped.	2.08	102-Z	**0.085**
Syn. ped.	2.15	102-Z	0.111
Syn. ped.	2.47	102-Z	0.158
Syn. ped.	3.42	102-Z	**0.276**
Vandals	1.71	102-Z	0.141
Vandals	1.79	102-Z	0.270
Vandals	1.95	102-Z	**0.193**
Vandals	1.98	102-Z	0.153
Vandals	2.08	102-Z	0.114
Vandals	2.15	102-Z	0.143
Vandals	2.47	102-Z	0.211
Vandals	3.42	102-Z	0.470

TABLE 5. Maximal RMS values for individual exciting frequencies – the second stage (activated TMD).

References

[1] D. Makovička, M. Studničková, D. Makovička, J. Korbelář. Dynamický tlumič na lávce pro chodce [In Czech; Dynamic absorber on a footbridge]. *Stavební obzor* **12**(5):149–155, 2003.

[2] F. Ferreira, C. Moutinho, Á. Cunha, E. Caetano. Use of semi-active tuned mass dampers to control footbridges subjected to synchronous lateral excitation. *Journal of Sound and Vibration* **446**:176–194, 2019. https://doi.org/10.1016/j.jsv.2019.01.026

[3] J. E. C. Carmona, S. M. Avila, G. Doz. Proposal of a tuned mass damper with friction damping to control excessive floor vibrations. *Engineering Structures* **148**:81–100, 2017. https://doi.org/10.1016/j.engstruct.2017.06.022

[4] K. Zoltowski, A. Banas, M. Binczyk, P. Kalitowski. Control of the bridge span vibration with high coefficient passive damper. Theoretical consideration and application. *Engineering Structures* **254**:113781, 2022. https://doi.org/10.1016/j.engstruct.2021.113781

[5] W. Fiebig. Reduction of vibrations of pedestrian bridges using tuned mass dampers (TMD). *Archives of Acoustics* **35**(2):165–174, 2010. https://doi.org/10.2478/v10168-010-0015-3

[6] J. Máca. Dynamic response of footbridges with tuned mass dampers. *IOP Conference Series: Materials Science and Engineering* **236**:012060, 2017. https://doi.org/10.1088/1757-899X/236/1/012060

[7] H. Bachmann, W. J. Ammann, F. Deischl, et al. *Vibration Problems in Structures: Practical Guidelines*. 1. Birkhäuser Basel, 2011. https://doi.org/10.1007/978-3-0348-9231-5

[8] M. Pirner, S. Urushadze. Liquid damper for suppressing horizontal and vertical motions – parametric study. *Journal of Wind Engineering and Industrial Aerodynamics* **95**(9–11):1329–1349, 2007. https://doi.org/10.1016/j.jweia.2007.02.010

[9] S. Hračov, D. Gregor, V. Janata, P. Nehasil. *Dynamické posouzení lávky [In Czech; Dynamic assessment of a footbridge]*. Excon a.s., 2021.

[10] T. Lindtner, D. Šindler, O. Dědek, V. Hvízdal. *SO201 Lávka přes Jizeru (000 – Přehledné výkresy) [In Czech; Plans (Cross sections and Lontitudinal sections)]*. Pontex, Ltd., 2022.

[11] Český normalizační institut. ČSN 73 6209. Zatěžovací zkoušky mostních objektů [In Czech; Loading tests on bridges], 2019.