Chapter 18
Vibration Performance of Bridges Made of Fibre Reinforced Polymer

S. Živanović, G. Feltrin, J.T. Mottram, and J.M.W. Brownjohn

Abstract Due to favourable mechanical and physical properties, and the potential to provide a resilient and low-carbon infrastructure, fibre-reinforced polymer (FRP) material has increasingly been used for construction of highway and pedestrian bridges. Relative low mass, low damping and low stiffness make these bridges sensitive to dynamic excitation, which may lead to discomfort of human occupants and larger dynamic amplification of stress and deformation than is encountered in structures made of traditional structural materials. Consequently, design might be governed by a vibration serviceability state. Lack of data on vibration performance of FRP structures and non-existence of a state-of-the-art vibration serviceability design guideline means that current practice is conservative, often meaning only short-span FRP bridge solutions are executed. To fully exploit the benefits of using FRP material and to extend its use beyond current practice requires a better understanding of dynamic behaviour.

The objective of this paper is to study vibration performance of bridges made with FRP components. Two FRP footbridges having main spans of 15.6 and 63.0 m are used as case studies, and vibration behaviour is critically evaluated against steel/concrete structures of comparable span lengths. Both dynamic properties of the FRP and non-FRP bridges, as well as the vibration response under dynamic excitation by pedestrians, are reported. It is shown that the FRP footbridges could exhibit one order of magnitude larger vibration response under nominally the same dynamic loads. This finding highlights the need for timely research on the in-service vibration performance of FRP bridges with the aim of developing design guidance tailored for this newest structural material.

Keywords Fibre reinforced polymer • Footbridge • Vibration

18.1 Introduction

To fully exploit advantages associated with FRP bridges, such as high strength-to-weight ratio, low maintenance costs, rapid installation and reduction in site waste, detailed knowledge of their behaviour under both static and dynamic loads is required. Several decades of research and experience with FRP material in construction has led to the first (pre)standard for design of FRP structures in the USA; its scope is not necessary for bridge engineering [1]. This is followed by the on-going efforts of Working Group 4 of CEN Technical Committee 250 to prepare a technical report within framework of Structural Eurocodes. These efforts mainly include evaluation of structural performance under static loading. However, light weight (density $<$2,000 kg/m^3), low stiffness (modulus of elasticity in range 20–40 GPa) and low damping (damping ratio often below 1–2 %) of FRP structures make them vulnerable to dynamic loading [2]. As a result, vibration serviceability can govern design. In 2014 there is a serious lack of data on vibration performance, which is reflected in use of non-FRP specific design guidance for assessing vibration serviceability in design practices.

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F.N. Catbas (ed.), *Dynamics of Civil Structures, Volume 4: Proceedings of the 32nd IMAC, A Conference and Exposition on Structural Dynamics, 2014*, Conference Proceedings of the Society for Experimental Mechanics Series,
DOI 10.1007/978-3-319-04546-7_18, © The Society for Experimental Mechanics, Inc. 2014

Footbridges made of traditional structural materials that are exposed to human-induced dynamic loading (such as walking, jumping, bouncing or running) are more likely to fail a vibration serviceability check if at least one of their natural frequencies lies in the range of the first (and sometimes second) harmonic of the human activity. The first harmonic of the dynamic force induced by humans can have a frequency in the range from 1.0 to 4.0 Hz, with second harmonic being between 2.0 and 8.0 Hz [3]. Among all activities, walking is the most frequent action type for footbridges. Pacing rate in a human population typically ranges from 1.4 to 2.5 Hz, resulting in second harmonic of the force having frequency up to 5.0 Hz. As a consequence, first design guideline for footbridges developed in the late 1970s [4] states that vibration problems are unlikely for structures having fundamental natural frequency of at least 5 Hz. To avoid vibration serviceability problems the simple method of frequency tuning has been applied for many years. The introduction of higher-strength materials over several decades has led to lighter and more slender footbridge structures that frequently have one or more vibration modes in the undesirable range up to 5 Hz. This inability to meet the minimum natural frequency requirement in modern design, and the fact that some bridges with undesirable frequencies are still serviceable, has led to abandonment of the frequency limit in modern vibration serviceability guidelines and its replacement by calculation and evaluation of the structural vibration response from human comfort perspective [5, 6]. Unfortunately, current design for FRP bridges is either still relying on outdated requirements for minimum natural frequency or is lacking the details on how to perform the required vibration serviceability checks.

Whereas the UK publication BD 90/05 [7] from the Highways Agency prescribes a minimum frequency of 5 Hz for bridges exposed to pedestrian excitation, the AASHTO guide specifications [8] allows the natural frequency to be as low as 3 Hz. Only for (dynamically) more severe load actions induced by running and jumping is the limit of 5 Hz preserved. These frequency requirements are not only unlikely to be achieved for contemporary structural forms, but also, being originally developed for much heavier structures of, say, steel and concrete, are unlikely to guarantee satisfactory vibration performance of extremely lightweight FRP bridges. To improve the guidelines, more research on vibration behaviour is essential.

The lack of data on in-service vibration performance of FRP structures results in low confidence in design and use of conservative span lengths (that rarely exceed 30 m [9]). This slow development is understandable; since it is well-known that the introduction of a new structural material usually requires 30–40 years to develop confidence and to push the boundaries in bridge design [10]. Given that the first all-FRP bridge superstructure was constructed in 1982 in China [9], it is just a question of time when there will be a "push" for longer spans. To be ready for this next step change in application, a better understanding of dynamic performance of FRP structures is of crucial importance.

The aim of this paper is to gain an insight into the dynamic properties of two FRP footbridges and to compare their vibration performance with three bridges made of traditional structural materials. All five footbridges vibrate perceptibly when excited by pedestrians. One of the two FRP bridges is prone to excitation by the first harmonic of the walking-induced dynamic force, while the second is prone to excitation by the second harmonic. The three non-FRP bridges were chosen because of their similarity with at least one of the two FRP bridges in terms of span length and/or natural frequency.

Due to the availability of relevant experimental data, this study concentrates on the performance of pedestrian bridges under human-induced excitation. Performance investigations to understand effect of wind loading and train buffeting, as well as fatigue performance, will deserve studies too.

Section 18.2 of the paper gives a description of five footbridge structures. This is followed in Sect. 18.3 by a presentation for the comparison of the most relevant dynamic properties. To evaluate and compare vibration response Sect. 18.4 deals with computer simulations of structural response under pedestrian loading. Finally Sect. 18.5 presents a discussion and conclusions.

18.2 Description of Footbridge Structures

Five footbridges investigated in this paper are:

- Aberfeldy bridge (AB): a cable stayed all-FRP bridge with main span of 63 m, and total length of 113 m [11].
- EMPA bridge (EB): an ultra-light cable stayed FRP laboratory bridge with main span of 15.6 m. Total length is 19.2 m [12]. This bridge is composed of FRP girders and deck, and steel pylons and cable strands.
- Podgorica bridge (PB): a steel box-girder structure, with main span of 78 m and total length of 104 m [13].
- Sheffield bridge (SB): a pre-stressed simple beam made of reinforced concrete spanning 10.8 m [14].
- Warwick bridge (WB): a steel-concrete composite laboratory structure with total length of 19.9 m and simple span of 16.2 m [15].

Fig. 18.1 (**a**) Aberfeldy bridge (AB), (**b**) EMPA bridge (EB), (**c**) Podgorica bridge (PB), (**d**) Sheffield bridge (SB), and (**e**) Warwick bridge (WB)

The references quoted in relation with each bridge [11–15] can be consulted for more detailed description of each structure. Photographs showing the five bridges are given in Fig. 18.1a–e. EB and WB were designed to be responsive to human excitation to allow for detailed vibration studies in laboratory environment.

18.3 Dynamic Properties

Dynamic properties of several vibration modes (i.e. mode shapes, natural frequencies, damping ratios and modal masses) for four structures were determined experimentally and are described in detail elsewhere (AB and PB in [13], SB in [14] and WB in [15]). In each case, a vibration mode that was most responsive to pedestrian induced excitation was identified, and the information about these modes is provided herein. This is followed by a description of dynamic testing and the identification of dynamic properties of the EB.

18.3.1 Properties of AB, PB, SB and WB

Constructed in 1992, AB (Fig. 18.1a) was the world's first all-FRP footbridge [11]. More than 20 years later, the 63 m long main span of the bridge remains the longest span for all-FRP bridges. First mode of vibration at 1.52 Hz is extremely easy to excite by a pedestrian walking at a slow pacing rate equal or close to 1.52 Hz. Maximum recorded acceleration under a single person walking is very high at 2.2 m/s^2 [11] due to a low damping ratio of 0.4 % and a low modal mass of 2,750 kg. The vibration mode at 1.52 Hz is shown in Fig. 18.2a, with most movement occurring at the mid-span [13].

PB (Fig. 18.1c) is a three-span steel box girder footbridge. Its main span of 78 is 15 m longer than the main span of AB. Because the frequency of the first mode is 2.04 Hz (Fig. 18.2b), it is excitable by first harmonic of the walking force. This makes the PB comparable to the AB in the dynamic sense. Although the maximum acceleration response recorded at the mid-span due to single pedestrian was about 0.8 m/s^2 and therefore much lower than that recorded on AB, it is still perceptible by a walker. This bridge has extremely low damping ratio of 0.3 % and modal mass of 58,000 kg [13].

SB (Fig. 18.1d) is a simple beam structure having a fundamental natural frequency at 4.5 Hz, modal mass of 6,500 kg, damping ratio of about 0.6 % and the mode shape as shown in Fig. 18.3a [14]. The structure can be excited perceptibly by the second harmonic (similar to EB that is presented in Sect. 18.3.2) when a person is walking at a pacing rate of 2.25 Hz. Maximum measured acceleration response due to single pedestrian is around 0.6 m/s^2.

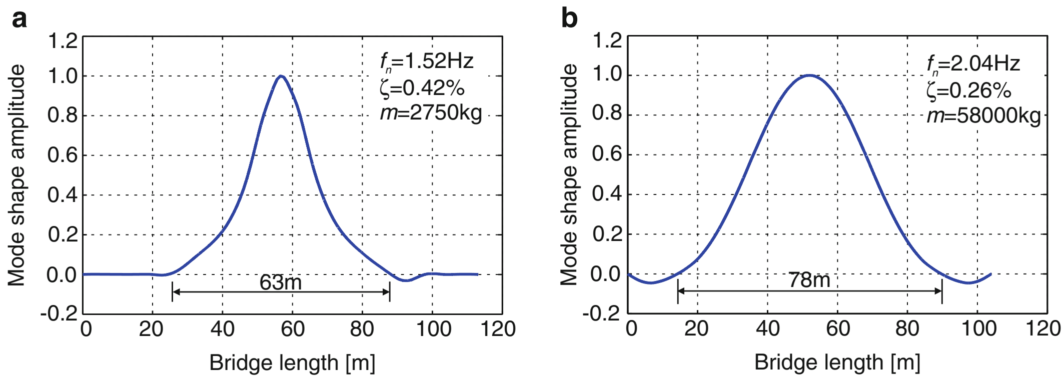

Fig. 18.2 Fundamental vertical bending vibration mode for (**a**) Aberfeldy bridge and (**b**) Podgorica bridge [13]

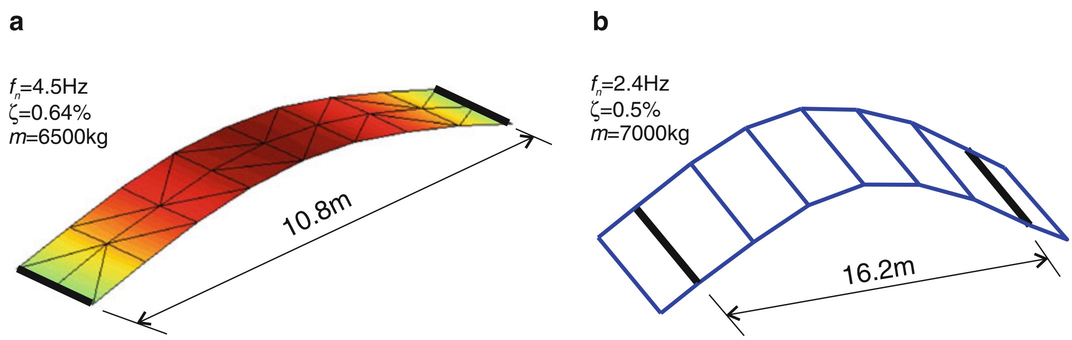

Fig. 18.3 Fundamental vertical bending vibration mode for (**a**) Sheffield bridge [14] and (**b**) Warwick bridge [15]

Finally, the fundamental flexural mode of the WB (Figs. 18.1a and 18.3b) has natural frequency of 2.4 Hz, damping ratio of about 0.5 % and modal mass of around 7,000 kg [15]. The maximum acceleration recorded due to single pedestrian is about 2 m/s^2 [15]. This bridge is similar in span length to EB and it is excitable by first harmonic of the walking-induced force, similar to AB.

18.3.2 Properties of EB

This bridge has two spans (3.6 and 15.6 m) supported by a FRP pylon, end supports and a set of cables. It has been designed so that the main span can be supported by up to three pairs of cable strands. During testing in 2009, only the cable set at the mid-span was used in order to lower the natural frequencies of the vibration modes so to be comparable with other structures studied here. The bridge was exposed to random excitation induced by an electrodynamics shaker (type APS113) in the frequency band of 0–20 Hz. The shaker was placed at a 3/8th of the main span. The excitation force was measured using an Endevco 7754-1000 piezoelectric accelerometer (nominal sensitivity 1,000 mV/g) attached to the shaker inertial mass, while the acceleration response was measured at 22 test points on the FRP bridge deck using force balanced Honeywell QA750 accelerometers (nominal sensitivity 1,300 mV/g). Accelerance Frequency Response Functions (AFRFs) were calculated from 21 data blocks, each lasting 40s, with 75 % overlapping between neighbouring blocks. Hanning window was applied to reduce leakage effects in the measured signals.

Figure 18.4 shows magnitude and phase of 4 out of 22 measured AFRFs (these four being AFRFs due to excitation at test point 6 and the response at points 6, 9, 20, and 5, with locations of test points presented in Fig. 18.5). It can be seen that there exist a number of vibration modes having a natural frequency below 10 Hz. Among these, the two modes with frequency at 4.62 and 5.28 Hz are most responsive to pedestrian induced excitation. The characteristics of these two modes are shown in Fig. 18.5a, b. The liveliness of the two modes primarily stems from their extremely low modal masses of about 300 kg at 4.62 Hz and of about 400 kg at 5.28 Hz.

To investigate if the properties of these modes change with the change in the excitation level, a new set of tests was performed. The shaker was placed at the mid-distance between test points 5 and 16 that are shown in Fig. 18.5 and a chirp

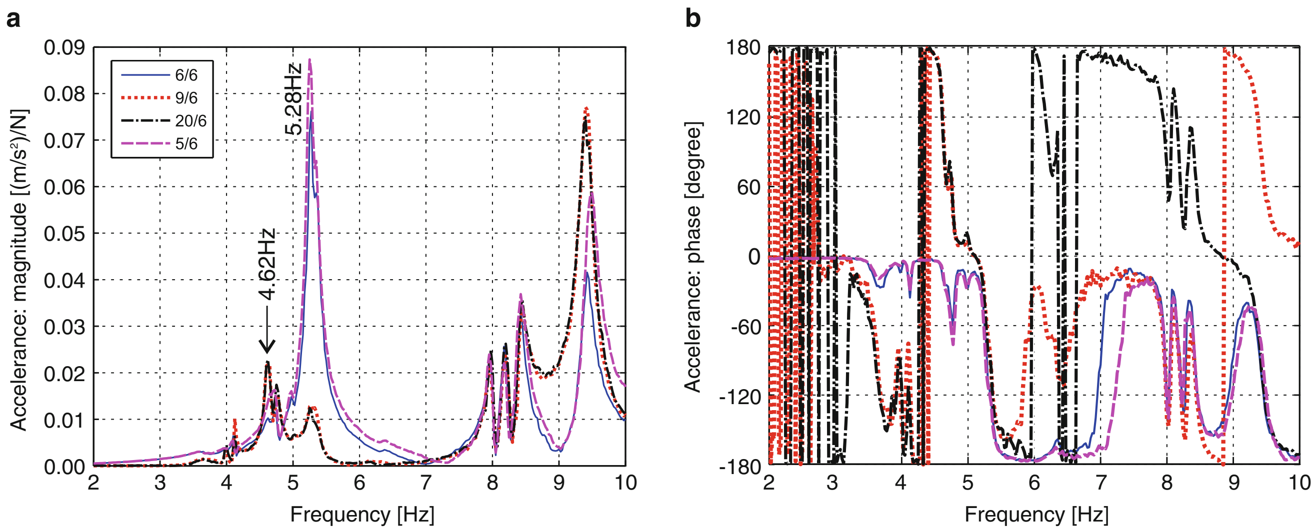

Fig. 18.4 Measured accelerance FRF on EMPA bridge: (**a**) magnitude and (**b**) phase

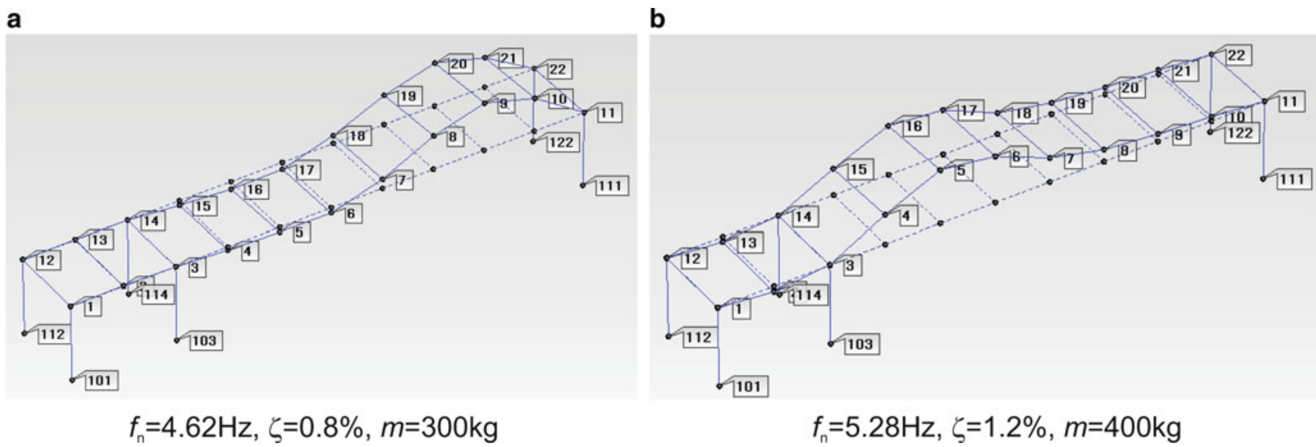

Fig. 18.5 Experimentally identified vibration modes for EB at: (**a**) 4.62 Hz and (**b**) 5.28 Hz

excitation with frequency content between 4 and 6 Hz was generated. Three blocks of response data were acquired, each 256s long. The excitation was applied during the first 230s in each data block. Six levels of the excitation force were used, ranging from approximately 11–105 N. Magnitude and phase of measured AFRFs (for the response measured at test point 8) are plotted in Fig. 18.6. The phenomenon of decreasing natural frequency with increasing excitation level is noticeable in the figure. Assuming that the mode shape and the modal mass are independent from the force level, the decrease in frequency found (from 4.72 to 4.60 Hz and from 5.22 to 5.04 Hz) represents the softening of EB under increasing level of the excitation. At the same time the peak response has also decreased, implying that the damping ratio increased with increase in the force level. Precise quantification of the damping of these modes is difficult due to presence of closely spaced modes (around 4.76 and 5.44 Hz) at some excitation levels.

The tests on EB revealed that the mode at 4.62 Hz is livelier than the mode at 5.28 Hz, due to lower mass and damping (Fig. 18.5). Besides, the mode at 4.62 Hz can be excited by both second and third harmonic of the walking force, while the mode at 5.28 Hz is mainly excitable by the third harmonic. This was the reason to select the lower mode for further analysis and the comparison with the other structures. Although the identified nonlinearity should be accounted for in a detailed analysis of vibration response of the structure, a simplified linear model is adopted for the preliminary analysis performed in this paper.

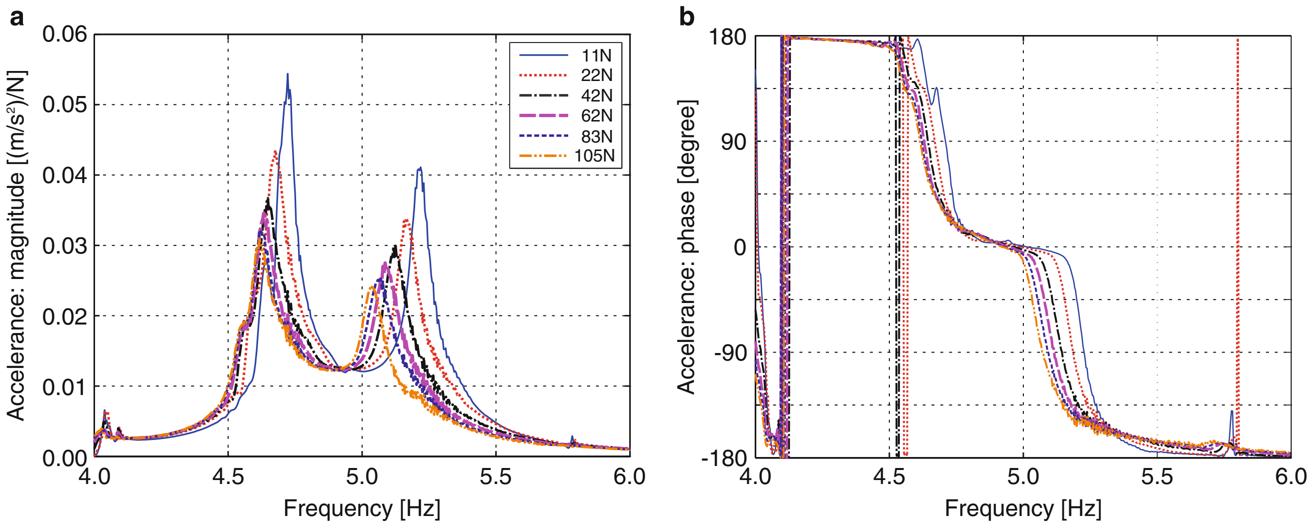

Fig. 18.6 AFRFs for EB under different chirp excitation levels: (**a**) magnitude and (**b**) phase

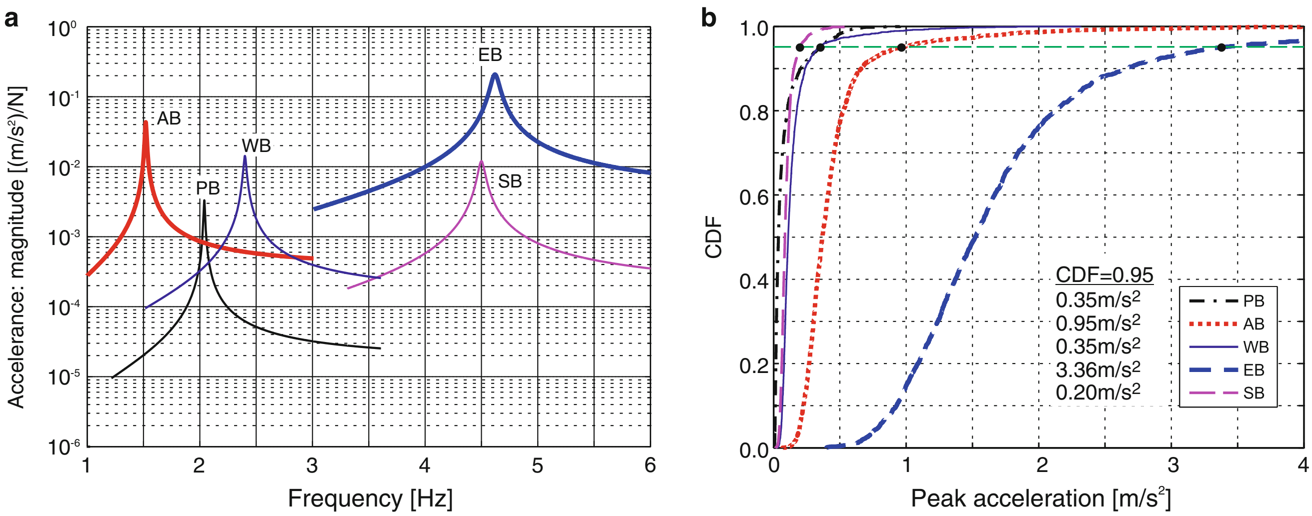

Fig. 18.7 (**a**) AFRFs for the five structures; (**b**) cumulative distribution functions of peak acceleration response under single person excitation scenario (*thick lines* represent FRP bridges)

18.3.3 Comparison of Dynamic Properties

Figure 18.7a shows direct point AFRFs at the antinodes of the five structures. It can be seen that the response of AB at resonance is about one order of magnitude larger than that of PB, despite the two bridges having relatively similar span lengths and natural frequencies, and despite PB being less damped. Even structure WB, which was intentionally constructed to be lively, is found to be less responsive than AB. Similar conclusions follow from comparison of EB and SB. Namely the vibration response of EB is about an order of magnitude larger than that of SB, this is despite the two bridges possessing similar span length and natural frequency. Apart from lower stiffness and mass, non-negligible shear deformation and rotary inertia effects in FRP structures are also likely to contribute to their less favourable dynamic behaviour. It is interesting that the damping ratio ζ for the two FRP structures, laying in the range 0.4–1.2 %, is within the ranges known for footbridges made of traditional structural materials [3].

Table 18.1 Parameters of Gaussian distributions used for modelling pedestrian population

Parameter	Mean	COV
Pacing frequency	1.87 Hz	0.10
Step length	0.74 m	0.11
Weight	750 N	0.20
Amplitude of first harmonic	Function of pacing rate (see [18])	0.16

18.4 Response to Pedestrian Excitation

To compare vibration performance of the five bridges under single pedestrian loading scenario, a set of simulations was performed. Pedestrian-induced dynamic force was modelled using a probabilistic time domain force model that accounts for frequency content up to the fifth harmonic [16]. Response calculations were performed using a set of 2,000 pedestrians, whose properties are drawn randomly from appropriate Gaussian distributions. Parameters of these distributions represent a pedestrian population observed walking over PB [17]. Mean value and coefficient of variation (COV) for the four key parameters of pacing frequency, step length, pedestrian weight and amplitude of the first harmonic are listed in Table 18.1. Živanović et al. [16] presents a detailed description of the model. In all simulations, the structure is represented by a single-degree-of-freedom system whose modelling properties reflect those measured for the vibration mode studied (see Figs. 18.2, 18.3 and 18.5a). The influence of the mode shape is accounted for by weighting the pedestrian force by mode shape ordinates. Peak acceleration response was extracted in each of 2,000 simulated bridge crossings. Plotted in Fig. 18.7b are the cumulative distribution functions (CDFs) for peak vibration response.

From the curves in Fig. 18.7b it can be seen that 95 % of pedestrians crossing AB generate the peak acceleration response up to 0.95 m/s^2. This is almost three times higher than the 95 percentile acceleration response of 0.35 m/s^2 calculated on PB and WB. The response of 0.95 m/s^2 can be seen as an improvement from the expected difference of one order of magnitude (between AB and PB) suggested by the AFRFs in Fig. 18.7a. The better than expected performance for AB is due to its shorter span and more localised mode shape compared with PB. In addition, the high-level vibration response of AB is caused by slow walking, which is characterised by about two times lower force amplitude than the normal walking causing resonance in PB. Finally, the choice of a pedestrian population having mean pacing frequency of 1.87 Hz is influential since it offers higher incidence of resonance on PB than on AB. Had these two bridges had exactly the same natural frequency, mode shape and span length then the response of AB would be one order of magnitude higher. Comparison between the responses calculated on EB and SB reveals that the acceleration level is about 17 times higher on the FRP bridge (cf. 3.36 m/s^2 with 0.20 m/s^2).

Evaluating the 95 percentile acceleration level against vibration limits defined in Setra guideline [6], it can be concluded that the AB provides "mean" comfort level to its users, while the vibrations of the EB would be considered unacceptable. On the other hand, all three non-FRP bridges would be considered to provide maximum comfort. This is only an indication of the level of responsiveness of the five bridges to pedestrian excitation, and a study under stationary multi-person pedestrian traffic could provide a more detailed insight into their vibration serviceability.

18.5 Discussion and Conclusions

FRP bridges are likely to be slender and lightweight structures that are sensitive to dynamic excitation. Vibration serviceability may be the limit state that governs the design, and directly influences the cost. Currently there is lack of a state-of-the-art guidance on vibration serviceability design for structures with FRP components. More broadly, the knowledge of their dynamic properties, as well as their in-service vibration performance under different types of dynamic load, is very scarce. This paper contributes to developing this knowledge by investigating vibration performance of two FRP footbridges and comparing their behaviour with three bridges made of traditional structural materials.

Although the comparison has been made using a small sample of bridges, it indicates that the two FRP bridges can be much livelier than the other three bridges that have similar span length and natural frequency; with relatively low modal mass of FRP bridges being main contributor to this behaviour. This suggests that currently used recommendations for designing FRP bridges to avoid natural frequencies in the frequency range that are historically considered problematic (such as below 5 Hz) might be inappropriate since it incorrectly implies that the FRP and non-FRP structures having similar natural frequencies exhibit similar vibration behaviour. Future design guidelines for FRP structures will need to limit the vibration response (and not the natural frequency) and they might need to recommend efficient vibration suppression measures for such extremely light structures. It is going to be important to develop vibration control measures that will not compromise the benefits associated with the FRP material. In addition, there is a need to understand the extent of non-linear behaviour of FRP structures at different excitation/response levels, and its influence on the vibration performance, as well as on efficiency of vibration suppression devices potentially added to the structure.

To provide a similar positive impact of FRP materials in bridge engineering to that recognised in aerospace and automotive engineering, it is absolutely necessary to better understand dynamic properties and in-service vibration behaviour. In addition, the development of vibration serviceability design guidance tailored to FRP material and components is a necessary goal to empower clients, structural engineers, and other parties, to choose this new bridge material.

Acknowledgements We would like to acknowledge financial support from the UK Engineering and Physical Sciences Research Council (Ref no. EP/I03839X/1: *Pedestrian Interaction with Lively Low-Frequency Structures* and Ref no. EP/E018734/1: *Human Walking and Running Forces: Novel Experimental Characterisation and Application in Civil Engineering Dynamics*). The Empa bridge was built with the support of the Gebert Rüf Stiftung, the ETH Council, and the companies Fiberline S/A and Maagtechnic AG.

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