Vibration damping in steel footbridges

Marek Pańtak¹, Bogusław Jarek¹, Kinga Marecik¹
¹Cracow University of Technology, Institute of Building Materials and Engineering Structures, Warszawska 24, Cracow 31-155, Poland

E-mail: mpantak@pk.edu.pl

Abstract. The subject of the paper concerns the problem of vibrations and vibration damping values in steel footbridges with various structural systems (truss, beam and slab, cable-stayed, suspension). In the paper values of vibration damping in steel footbridges determined on the basis of the results of numerous dynamic field tests of the footbridges are presented. The vibration damping values are given as a logarithmic decrement \( \delta \). Presented information may constitute a database useful at the stage of designing a new or analyzing existing structures.

1. Introduction

In the analyses of the forced vibration of the structures the value of the vibration damping plays an important role. Proper estimation of the vibration damping is important to determine the correct value of the dynamic response of the structure. In general, the steel footbridges are the structures characterized by low damping [1]. This can lead to excessive vibration amplitudes and significant reduction of the comfort of use of the structures.

It is well known that in the case of resonant vibrations the amplitudes of vibrations strongly depend on the value of damping (Fig. 1a).

The situation in which the excitation frequency is equal to the natural vibration frequency of the system (resonance effect) the amplitudes of vibrations are inversely proportional to the damping \( C \) (eq. 1) [1]. In other words, in the case of resonant vibrations decreasing of the vibration amplitudes \( A \) requires increasing the damping of the system.

\[
A = \frac{P}{2\pi f_p C}
\]

where:
- \( A \) – amplitude of forced vibration,
- \( P \) – amplitude of the harmonic excitation force,
- \( f_p \) – frequency of the harmonic excitation force,
- \( C \) – damping coefficient.

Damping in a vibrating structures can be defined as the mechanism of dissipation of the mechanical energy of the system i.e. irreversible absorption of a part of the work of external forces. This energy-loss mechanism due to its complexity is not fully explained. For this reason the damping value in most structural systems is evaluated by experimental methods using time or frequency domain analysis and for simplicity the damping mechanism is modeled as viscous damping.
One of the simplest and most frequently used method for estimating the value of logarithmic decrement $\delta$ (or damping ratio $\zeta$) is a free vibration decay method performed in the time domain (Fig 1b).

To estimate the logarithmic decrement value by using the free vibration decay method the vibrations can be initiated by any convenient method and only the amplitudes of vibrations (displacement or acceleration) must be measured [3]. However a “convenient method” of vibration excitation can not change the dynamic parameters of the structure. It is necessary to perform the dynamic tests in appropriate conditions and to eliminate all factors that can affect the free vibration decay (e.g. unnecessary accessories, machine, devices, people staying on the structure as well as dynamic impact of environment including strong wind and machines or cars working in the vicinity of tested structure or passing under the structure).

In the free vibration decay method the logarithmic decrement $\delta$ can be estimated by means of equation (2):

$$\delta = \ln \frac{A_n}{A_{n+1}} = \frac{1}{m} \ln \frac{A_n}{A_{n+m}}$$

where:

- $A_n$ – amplitude of $n$-th cycle of vibrations, $n = 0, 1, 2,...$,
- $A_{n+m}$ – amplitude of $n+m$-th cycle of vibrations,
- $m$ – number of full vibration cycles (periods) counting from the $n$ cycle, $m = 1, 2, 3,...$

Determined value of damping can be verified by fitting exponential curve to decaying vibration amplitudes (Fig 1b) using equation (3):

$$A(t) = A_0 e^{-\delta \cdot f \cdot t}$$

where:

- $A_0$ – initial amplitude of the analysed part of the free vibration decay,
- $\delta$ – logarithmic decrement,
- $f$ – vibration frequency,
- $t$ – time steep.

The damping ratio value can be determined using equation (4):

$$\zeta = \frac{\delta}{\sqrt{4\pi^2 + \delta^2}} = \frac{\delta}{2\pi}$$
Further part of the paper contains tabular summary of the vibration damping values determined for numerous steel footbridges with various structural systems based on the results of the dynamic field tests of the footbridges.

2. Vibration damping in steel footbridges

Generally the footbridges characterised by low damping. The logarithmic decrement $\delta$ for footbridges is in the range of low damping ($\delta < 0.10$). As set forth in [2] on the basis of the dynamic field tests of 35 footbridges: 50% of tested footbridges had $\delta = 0.05 – 0.10$; 30% of footbridges had $\delta = 0.03 – 0.05$ and only 11% footbridges had $\delta = 0.10 – 0.15$. Characteristic for the footbridges is also lack of the structures with a large damping $\delta > 0.20$. These results do not contain informations about static scheme, structural scheme, frequency, mode shape, span of the structures and should be considered as illustrative. Nonetheless, these results confirm the general thesis of greater dynamic susceptibility of footbridges compared with other types of bridge structures.

Recommendations presented in [4, 5, 6, 7, 8, 9] regarding the vibration damping value for steel footbridges with welded joints define the logarithmic decrement value $\delta = 0.01 – 0.03$ for vibrations of small and medium amplitudes in a frequency range 1.0 – 3.5 Hz. In the case of vibrations of large amplitudes $\delta = 0.04 – 0.06$ is recommended.

In Tab. 1 the logarithmic decrement values $\delta$ established on the basis of the dynamic field tests of numerous steel footbridges with various structural systems carried out by authors during several years of research are presented. The logarithmic decrement values $\delta$ were determined using a data (vibration acceleration signals) acquired during dynamic field tests of the footbridges during vibrations excited by one walking or running person. The person inducing the vibration was passing the footbridge and leaving the structure (the person was not remaining on the footbridge deck). The logarithmic decrement values $\delta$ were determined using free vibration decay method taking into account 10-40 vibration periods. Examples of determination $\delta$ using free vibration decay are presented in Fig. 3.

![Figure 2](image-url) Distribution of logarithmic decrement $\delta$ a) for bridges and viaducts, b) for footbridges [2]

![Figure 3](image-url) Examples of determination of the logarithmic decrement $\delta$ by free vibration decay method
### Table 1. Natural vibration frequencies and logarithmic decrements $\delta$ for 32 various footbridges

| No. | Footbridge location | General view of the footbridge | General informations | $\delta$ [%] |
|-----|---------------------|--------------------------------|----------------------|-------------|
| 1.  | Brno, Czechia       | three span $L = 8.5+27.5+8.5$ m, lightweight orthotropic deck, anchored in abutments $f = 5.83$ Hz - vertical. | 4.6-5.4             |
| 2.  | Beroun, Czechia     | three span $L = 21+42+21$ m, rectangular box girders, lightweight openwork deck, $f = 2.88$ Hz - vertical. | 4.0-4.8             |
| 3.  | Kraków, Poland      | two span $L = 35+32.5$ m, box-girder deck, thin epoxy resin pavement, $f = 2.68$ Hz - vertical. | 9.0-12.0            |
| 4.  | Łódźmierz, Poland   | three span $L = 16+36+16$ m, heavy asphalt pavement $f = 3.19$ Hz - vertical. | 2.9-4.8             |
| 5.  | Eberswald, Germany  | three span $L = 13.5+30+13.5$ m, lightweight orthotropic deck, $f = 4.98$ Hz - vertical. | 5.7-6.6             |
| 6.  | Kassel, Germany     | steel-UHPC concrete composite footbridge, six spans $L = 19.2+24+21+36+21+12$ m, $f = 3.47$ Hz - vertical. | 2.2-3.2             |
| 7.  | Kraków, Poland      | single span $L = 35$ m, lightweight orthotropic deck, thin epoxy resin pavement, $f = 2.71$ Hz - vertical. | 6.8-9.2             |
| 8.  | Trstená, Slovakia   | single span $L = 35$ m, light deck (thin steel plate without pavement), $f = 2.98$ Hz - vertical. | 3.6-4.1             |
| 9.  | Svatava, Czechia    | single span $L = 26$ m, wooden deck, $f = 5.22$ Hz - vertical. | 24.4-26.3            |
| 10. | Roztoky, Czechia    | single span $L = 30$ m, lightweight steel openwork deck, $f = 4.20$ Hz - vertical. | 14.2-15.1            |
11. Frydek Mistek, Czechia
   
   single span \( L = 45 \) m,
   heavy asphalt pavement
   \( f = 3.30 \) Hz - vertical.

12. Svatava, Czechia
   
   single span \( L = 34 \) m,
   wooden deck, structure anchored in abutments
   \( f = 4.88 \) Hz - vertical.

13. Rożnov pod Radhoštěm, Czechia
   
   single span \( L = 42 \) m,
   concrete deck,
   \( f = 3.76 \) Hz - vertical.

14. Brno-Obtany, Czechia
   
   single span \( L = 33 \) m,
   concrete deck,
   \( f = 4.30 \) Hz - vertical.

15. Biecz, Poland
   
   single span \( L = 47 \) m,
   heavy orthotropic deck,
   thin epoxy resin pavement,
   \( f = 2.38 \) Hz - vertical.

16. Sławięcice, Poland
   
   single span \( L = 44.8 \) m,
   heavy orthotropic deck,
   heavy asphalt pavement,
   \( f = 2.50 \) Hz - vertical.

17. Osjaków, Poland
   
   single span \( L = 50 \) m,
   heavy orthotropic deck,
   thin epoxy resin pavement,
   \( f = 2.39 \) Hz - vertical.

18. Kraków, Poland
   
   single span \( L = 30 \) m,
   lightweight orthotropic deck,
   thin epoxy resin pavement,
   \( f = 4.21 \) Hz - vertical.

19. Jaroměř, Czechia
   
   single span \( L = 60.4 \) m,
   prestressed steel structure,
   lightweight openwork deck,
   \( f = 0.98 \) Hz - horizontal,
   \( f = 2.48 \) Hz - torsional.

   Węgierska Górka, Poland
   single span \( L = 51.3 \) m,
   wooden deck,
   \( f = 1.44 \) Hz - vertical.

20. Wola Wieruszycka, Poland
   single span \( L = 63 \) m,
   concrete prefabricated deck,
   \( f = 2.68 \) Hz - vertical.

21. Ojcow, Poland
   single span \( L = 60.4 \) m,
22. **Korwinów, Poland**
   - Single span $L = 32.0$ m, lightweight orthotropic deck, thin epoxy resin pavement, $f = 3.11$ Hz - vertical.
   - Frequency range: 7.1-10.5

23. **Beroun, Czechia**
   - Single span $L = 37.5$ m, wooden deck, crossing inclined hangers, $f = 4.88$ Hz - vertical.
   - Frequency range: 17.6-23.3

24. **Włocławek, Poland**
   - Single span $L = 41.8$ m, prestressed steel structure, wooden deck, $f = 2.44$ Hz - vertical.
   - Frequency range: 17.4-19.8

25. **Cieszyn - Český Těšín (Polish - Czechia border)**
   - Four spans $L = 17+45+18+13$ m, box-girder deck, $f = 2.85$ Hz - vertical.
   - Frequency range: 2.4-3.5

26. **Łapanów, Poland**
   - Single span $L = 45$ m, wooden deck, $f = 2.14$ Hz - vertical.
   - Frequency range: 4.4-5.8

27. **Plzeň, Czechia**
   - Single span $L = 46$ m, lightweight orthotropic deck, thin asphalt pavement, $f = 3.98$ Hz - vertical.
   - Frequency range: 4.8-6.4

28. **Řež, Czechia**
   - Three spans $L = 37.3+62.3+37.3$ m, wooden deck, $f = 3.32$ Hz - vertical.
   - Frequency range: 1.7-2.4

29. **Tvrdošín, Slovakia**
   - Single span $L = 65$ m, wooden deck, $f = 1.60$ Hz - vertical.
   - Frequency range: 1.3-1.6

30. **Kielce, Poland**
   - Two spans $L = 12.7+29$ m, heavy orthotropic deck, thin epoxy resin pavement, $f = 2.52$ Hz - vertical.
   - Frequency range: 3.7-4.5

31. **Piwniczna Zdrój, Poland**
   - Single span $L = 102$ m, suspension footbridge, lightweight orthotropic deck $f = 2.18$ - vertical (three extremes).
   - Frequency range: 2.5-3.2

32. **Beroun, Czechia**
   - Three spans $L = 31.2+61.4+31.2$ m, suspension footbridge, wooden deck, wooden railings, $f = 1.90$ Hz - torsional.
   - Frequency range: 3.0-5.8
The results presented in Tab. 1 refer mainly to vertical vibrations in the frequency range 1.0-6.0 Hz. Additionally, in Fig. 4 the mean values of the logarithmic decrements $\delta$ from Tab. 1 are presented.

![Figure 4](image1.png)  
**Figure 4.** Mean values of the logarithmic decrements $\delta$ of the footbridges presented in Tab. 1 in relation to fundamental vibration frequency and main span length

The analysis of the results shows that 80% of the examined footbridges have $\delta \leq 10\%$, 55% of the footbridges have $\delta \leq 5\%$. Moreover, natural vibration frequency of 70% tested footbridges is in the frequency range of vertical forces generated during walking and running (1.4 – 3.4 Hz). This increases the risk of excitation of large vibration amplitudes during the daily use of these footbridges.

3. Summary

The results presented in the paper indicate the low vibration damping ability of footbridges compared with other types of bridge structures. Relatively low damping can result in high vibrations caused by dynamic action of users (walking or running person or crowd) or by dynamic environmental influences (e.g. wind action).

Knowledge of the damping value is necessary to perform forced vibration analysis and to determine the dynamic response of the structure. Data presented in the paper allows to estimate the damping value in constructions with structural systems similar to constructions presented in Tab. 1.

It should be remembered that in situations of resonant (or near-resonant) dynamic action the value of damping has a significant effect on the vibration amplitude. When designing steel footbridges the possibility of increasing the damping value by installing the damping devices (e.g. passive, semi-active or active TMD device) should be taken into account at the design stage.

References

[1] Lewandowski R 2006 Dynamic of building structures (Poznan: Publishing House of Poznan University of Technology)

[2] Salamak M 2003 Experimental methods for determining the level of vibration damping in footbridges. (Silesian University of Technology, Faculty of Civil Engineering)

[3] Clough R Penzien J 1993 *Dynamic of Structures* (New York: McGraw-Hill)

[4] Heinemeyer Ch Butz Ch et al. 2009 *Design of lightweight footbridges for human induced vibrations*. JRC Scientific and Technical Reports (Luxemburg: JRC-ECCS)

[5] Bachman H Anman W Deischl F et al. 1995 *Vibration problems in structures – practical guidelines* (Basel: Springer Birkhäuser Verlag AG)

[6] Tilly G P Cullington D W Eyre R 1984 Dynamic behaviour of footbridges, IABSE Surveys S-26/84 pp 13-24

[7] Butz C Feldmann M Sedlacek G et al. 2008 *Advanced load models for synchronous pedestrian excitation and optimised design guidelines for steel foot bridges* (Luxembourg: European Commission CORDIS)

[8] Grundmann H Kreuzinger H Schneider M 1993 Schwingungsuntersuchungen für Fußgängerbrücken, *Bauingenieur* 68 pp 215-225

[9] Schlaich M Brownlie K Conzett J et al. 2005 *fib Bulletin 32: Guidelines for the design of footbridges* (Stuttgart: Sprint - Digital – Druck and fib)