Vibration Serviceability Issues of Slender Footbridges

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Abstract: Three important issues related to vibration serviceability were investigated in this study using a slender steel footbridge: (1) the evaluation of footbridge vibrations by passive people (standing still on the footbridge) and active people (pedestrians); (2) the increase in vibrations when a group of people walk/run on a footbridge compared with a single pedestrian crossing the structure; and (3) the modal damping ratios for vibration analysis of footbridges. Using the results of a number of dynamic tests, relationships between various evaluation parameters were established, and more reliable and consistent limits based on the vibration dose values (VDVs) are proposed. In addition, a simplified method of computing the acceptable level of peak acceleration based on average number of footbridge crossings per day for the performance-based (usage-based) serviceability design of footbridges is suggested. The investigation on the group effects showed results consistent with those of past studies when the pedestrians crossed the footbridge at the average normal speed of 2 steps/sec. However, the footbridge vibrations resulting from a group of people increased less than the reported value in the literature when people walked randomly and more than the reported value when they moved at the first-mode resonance frequency of the structure. Based on the results of the dynamic field testing of the footbridge, a modal damping ratio of 0.6–0.8% for steel footbridges with timber decking is recommended. DOI: 10.1061/(ASCE)BE.1943-5592.0000951. © 2016 American Society of Civil Engineers.

Introduction

Excessive vibrations of footbridges resulting from human movements date back to the nineteenth century with the collapse of Broughton footbridge in England and Angers Bridge in France, both of which collapsed when a group of soldiers marched in unison over them. The incidents resulted in over 200 fatalities and many injuries (Dupuit 1850). The sign at the entrance to the Albert Bridge in London reads “All troops must break step when marching over this bridge,” evidence of the historic importance of the problem.

Another well-known example of vibration serviceability problems in footbridges occurred at the Millennium Bridge over the Thames River in London. Upon the opening of the footbridge in 2000, annoying levels of vibrations were reported when a group of pedestrians crossed the footbridge. The structure was closed down to the public for more than 18 months while it underwent major structural modifications to correct the problem.

This study used the results of the dynamic field testing of a lively footbridge structure to investigate the following three important issues as related to the vibration serviceability of this class of structures: (1) footbridge vibration evaluation and assessment for human acceptance; (2) group effects related to footbridge vibrations; and (3) damping ratios for steel footbridges with timber decking.

A brief description of the footbridge used in this study is provided next, followed by the details of the aforementioned studies.

Description of the Clifton Forge (CF) Footbridge

The footbridge used in this study is located in Clifton Forge, Virginia. The total length of the structure, including a main span of 13.6 m (44 ft 8 in.) and a ramp of 11.2 m (36 ft 6 in.), is 28.4 m (93 ft 1 in.). Fig. 1 shows the elevation and transverse section of the structure. As shown in the figure, the footbridge is made of three main segments: the bridge, the hub, and the ramp. The floor is made of wood decking supported by wood joists, which are connected to the main structural steel sections [W200 × 41.7 (W8 × 28), W200 × 35.9 (W8 × 24) and W150 × 29.8 (W6 × 20)] along the two sides of the footbridge, as shown in Fig. 1. Details on the design of this structure were given by Setareh et al. (2014). The footbridge is slender; the bridge segment has a span/depth ratio of 66, which has resulted in the structure being susceptible to excessive vibrations caused by human movements.

Evaluation and Assessment of Footbridge Vibrations

Background

Griffin (2007) distinguishes the difference between the evaluation and assessment of measured vibrations for human acceptance. According to him, evaluation takes into account various important parameters of vibration related to human response, such as magnitude (acceleration, velocity, or displacement), frequencies, directions, and duration, to provide a larger value when conditions are worse. The assessment, however, involves a consideration of the vibration and judgment about it, and it indicates the consequences of different values predicted by the evaluation procedure. For a particular evaluation, the assessment may vary as a result of such factors as age, gender (intersubject variability), the person’s activity and surroundings (intrsubect variability), expectations, cultural differences, and so forth. For the specific case of footbridges, Mackenzie et al. (2005) identified factors, such as the height of the structure and bridge parameter and route redundancy, that can influence vibration assessments.

Vibration evaluation has been conducted in both time and frequency domains.

Time-Domain-Based Evaluation Techniques

In the time-domain-based method, the peak or RMS of acceleration is compared with a prespecified limit to assess the acceptability of
vibration. Use of peak acceleration ($a_p$) for vibration evaluation has two main problems: (1) $a_p$ is susceptible to errors as a result of variations in signal-processing parameters, such as filtering; and (2) vibration duration and frequency content can significantly affect humans’ reaction (Griffin 1990), which are clearly neglected when simply the peak acceleration is used.

However, most standards and design guides for footbridges have adopted this approach. Bachmann et al. (1995) recommend a limit of $a_p < 5\%g$, and the Canadian Standards Association (CSA 2006) limits $a_p$ as a function of the natural frequency of the footbridge, $f_s$ (e.g., $a_p < 6\%g$ for a footbridge with $f_s = 3.35\,\text{Hz}$, such as the one used in this study).

The British standard BS 5400 (BSI 2006) also sets the vibration acceleration limit as a function of $f_s$ (e.g., $a_p < 9\%g$ for $f_s = 3.35\,\text{Hz}$).

The French code (SETRA 2006) recommends a range of footbridge peak accelerations for various comfort levels: (1) $a_p < 5\%g$—maximum comfort; (2) $5\%g < a_p < 10\%g$—medium comfort; (3) $10\%g < a_p < 25\%g$—minimum comfort; and (4) $25\%g < a_p$—unacceptable.

The United Kingdom National Annex to Eurocode No. 1 (BSI 2008b) recommends $a_p < 3.1\%g$ for footbridges with a height of less than 8 m that are the sole access to hospitals and other sensitive sites. It limits $a_p$ to $23\%g$ for footbridges with a height of less than 4 m situated in rural environments and with alternative routes.

**Frequency-Domain-Based Evaluation Techniques**

The frequency-domain based approach uses rating or weighting techniques. In the rating method, a frequency spectrum of the
measured vibrations is computed, and the maximum component is compared with the acceptable limits. These limits are computed from the base curves (representing the absolute lowest perceptible vibrations). This is not a very general method, however, and it has been shown that it is only valid if the dominant frequencies of the vibrations are within a bandwidth of one-third octave (Shoenberger 1976). Fig. 2 shows the base curve recommended by ISO 10137 (ISO 2007) for the evaluation and assessment of vertical vibrations in footbridges. This standard recommends the use of a base curve multiplier of 30 to compute the RMS acceleration limit for the assessment of vibrations by passive people (footbridge bystanders) and a factor of 60 for active people (pedestrians).

The weighting method is a more general approach than the rating method for vibration evaluation. It recommends the use of a frequency-weighting function based on the vibration direction and expected effects on the subject because the sensitivity of human body to vibration is frequency and direction dependent. The frequency-weighting functions are the reciprocals of the equivalent perception/comfort base curves and are used to scale accelerations based on the level of sensitivity of the human body to different vibration frequencies. Fig. 3 shows the variation of the frequency-weighting functions for vertical vibrations within the 0- to 20-Hz range based on several current standards: $W_k$ [ISO 2631-1 (ISO 1997)], $W_m$ [ISO 2631-2 (ISO 2003)], $W_g$ [BS 6472-1 (BSI 2008a)], and $W_b$ [BS6472-1 (BSI 2008a)]. Note that $W_b$ and $W_k$ have very similar trends.

One important aspect of the weighting method is the introduction of vibration dose value (VDV) for vibration evaluation and assessment (Griffin and Whitham 1980a, b; Griffin 1984, 1986; Howarth and Griffin 1988, 1990, 1991), as follows:

$$\text{VDV} = \left[ \int_{t=0}^{T} a_w^2(t) \, dt \right]^{1/4}$$

where $T$ is the period in seconds during which a person is exposed to vibrations; and $a_w(t)$ is the frequency-weighted acceleration in m/sec$^2$. VDV has the unit of m/sec$^{1.75}$ and has the advantage of accumulating the vibration effects rather than averaging them (as RMS does), and it increases with duration. In addition, it can be used to quantify vibrations of any type and changes with peak acceleration, as humans are sensitive to peak acceleration (Griffin 2007). VDV has been used to evaluate a very large number of widely differing vibration exposures, with reasonable conclusions (Griffin 1998).
Recent Studies on the Evaluation and Assessment of Footbridge Vibrations

Few studies during the past decade focused on the problems associated with the evaluation and assessment of footbridge vibrations. MacKenzie et al. (2005) conducted a study to determine the tolerance of pedestrians to footbridge vibrations. They used four different footbridges with a diverse range of forms and situations and assembled a database of 421 pedestrian responses, collecting their reactions to different issues related to the structures, including their vibration serviceability. They proposed a rational dosage-based model of physical comfort for pedestrian bridges, which includes vibration assessment of the structure based on a number of factors, such as footbridge height, parapet height, and route redundancy. MacKenzie et al. defined acceleration limits as a function of these factors.

Kasperski (2006) studied the dynamic behavior of two footbridges and measured their dynamic properties. He also measured the vibration response of a footbridge when people crossed it. Through observations only, he found that 298 out of a total of 3,951 pedestrians were alarmed by the vibrations when crossing the footbridge. Kasperski found that the ISO 10137 (ISO 2007) requirement regarding the limit on the maximum transient vibration value (MTVV), which represents the maximum 1-s running RMS of frequency-weighted acceleration, does not correspond with the number of people alarmed by the vibrations. He concluded that the ISO 10137 (ISO 2007) recommended multiplier of 60 for active people to compute the acceptable vibration of footbridges is excessive and has to be reduced to 24 (60% reduction). However, in a recent paper, Czwikla and Kasperski (2014) found a perfect agreement between the results of their field tests and the provisions of ISO 10137 (ISO 2007) for acceptability of footbridge vibrations.

Barker (2007) discussed the importance of using vibration doses for footbridge comfort evaluation and assessment. He emphasized the importance of both amplitude and duration for vibration evaluations. Barker recommended the use of root-mean-quad (RMQ) of accelerations for vibration serviceability (RMQ is a variation of VDV, but unlike VDV, it averages the response).

Živanovic and Pavic (2009) interviewed 100 randomly chosen pedestrians about their reactions to vibrations when they crossed a footbridge. They could not find a direct correlation between the pedestrians’ subjective reactions and the measured vibrations. They attributed this to intersubject variability and environmental differences between the pedestrians. They did, however, find a strong correlation between $a_{vp}$, $a_{rms}$ (RMS of acceleration), MTVV, and VDV. From the results of this study, they proposed a probabilistic approach for the evaluation and assessment of footbridge vibrations.

Ingolfsson et al. (2012) studied the evaluation and assessment of footbridge vibrations resulting from pedestrian movements. They found a large difference between the footbridge vibration assessment methods in various design guides in terms of both allowable limits and frequency dependency. They conducted an experimental study using a number of people crossing a footbridge susceptible to lateral vibrations, and they recorded the subjective ratings of the perceived vibrations. The footbridge was laterally and torsionally flexible, with noticeable vibrations. It had a fundamental frequency of 2.11 Hz and 22 modes with frequencies below 9 Hz. They computed the VDV for the measured vibrations (measured accelerations were weighted by the mode shape amplitudes to find the vibration experienced by each pedestrian) and used the recommended limits for residential buildings from BS 6472-1 (BSI 2008a) and ISO 10137 (ISO 2007) of 0.4–0.8 m/sec$^2$ (adverse comments possible) and 0.8–1.6 m/sec$^2$ (adverse comments probable) for 16 h of daily exposure. They found that for most of the conducted tests, the subjective evaluation of vibrations confirmed the results of assessment using VDV, which indicated that for the majority of pedestrians (23 out of 30), the vibrations were in the range that adverse comments may have been possible. This is an important finding because, based on ISO 10137 (ISO 2007), the U.K. National Annex to Eurocode No. 1 (BSI 2008b), and SETRA (2006) limits, the vibrations were in the unacceptable range. The paper also concluded that such issues as earlier experience with vibrations, the person’s expectations, and so forth can affect subjective ratings.

Vibration Evaluation and Assessment of the CF Footbridge

As indicated in the previous sections, most design guides and standards recommend the use of time-domain-based techniques and peak acceleration for the vibration evaluation of footbridges. It was also shown that there are large differences in the recommended acceptable limits. VDV provides a more consistent and reliable approach for vibration evaluation (Griffin 1990). Even though a few attempts have been made to evaluate footbridge vibrations using VDV (Barker 2007; Živanovic and Pavic 2009; Ingolfsson et al. 2012), no limits for vibration assessment have ever been proposed. The following section presents an attempt to establish such limits based on the analysis of tests conducted on the CF footbridge.

Walk Tests

To check the relationship between VDV and other evaluation parameters such as $a_{vp}$, $a_{rms}$ (peak frequency-weighted acceleration), and MTVV, and in attempt to define VDV limits, a number of walk/run tests were conducted with the help of 16 volunteers (9 females and 7 males) with an average age of 20 years. Their movements on the footbridge were synchronized by a metronome. Two observers (one male and one female) stood still along the two edges of the structure, as shown in Fig. 4, to provide their subjective assessment of the vibrations while people crossed the footbridge. Before starting the walk tests, the first-mode resonance frequency of the footbridge while the observers were standing still at their locations was measured at 3.35 Hz. Sixteen accelerometers were placed on the deck of the bridge segment, as shown in Fig. 4, to measure the vibrations in the vertical direction. Fourteen volunteers (8 females and 6 males) crossed the entire footbridge at four different speeds: (1) 101 (3.35/2 × 60) steps per minute (spm) (first subharmonic of the footbridge resonance frequency), (2) 201 (3.35 × 60) spm (resonance frequency), (3) 120 spm (average normal walk speed), and (4) random. For each speed, tests were repeated with one person and three people crossing the entire footbridge, with two exceptions: (1) for 101 and 201 spm, an additional test was conducted in which seven people crossed the entire footbridge; and (2) after the seven-person test at 201 spm, the observers and pedestrians expressed much annoyance about the vibrations. It was decided not to repeat the test with more than seven pedestrians at this speed. The observers used a 7-point Likert-type scale to rate their perceived vibrations as people crossed the footbridge: 1 = not perceptible, 2 = somewhat perceptible, 3 = perceptible, 4 = somewhat uncomfortable, 5 = uncomfortable, 6 = somewhat annoying, and 7 = annoying. Fig. 5 shows the maximum measured accelerations of the footbridge at the points where the observers stood during the walk tests. These vibrations were generated by seven people crossing the footbridge at 201 spm.

Analysis of Recorded Measurements and Proposed Vibration Limits

The weighted accelerations, $a_w(t)$, from the walk tests, using the frequency-weighting functions of three standards, $W_k$ (ISO 2631-1
(ISO 1997), $W_m$ [ISO 2631-2 (ISO 2003)], and $W_b$ and $W_g$ [BS 6472-1 (BSI 2008a)], were computed. Also, the magnitude of peak acceleration, $a_p$; magnitude of peak-weighted acceleration, $a_{w,p}$; maximum 1-s running RMS of weighted acceleration (MTVV), and VDV for each record were found.

Fig. 6 shows the relationships between VDV and $a_p$, $a_{w,p}$, and MTVV, for all 16 channels and 13 different walk tests, using the four frequency-weighting functions.

Linear regressions between each two sets of parameters were conducted using the following equations:

$$VDV = C_{p1}a_p + C_{p2}$$
$$VDV = C_{pw1}a_{w,p} + C_{pw2}$$
$$VDV = C_{M1}MTVV + C_{M2}$$

where $C_{p1}$, $C_{pw1}$, and $C_{M1}$ are the slopes of the regression lines; and $C_{p2}$, $C_{pw2}$, and $C_{M2}$ are their corresponding intercepts, respectively.

The coefficients for each of the lines based on each and all frequency-weighting functions are shown in Table 1. It can be observed from this table that the fit parameters based on different standards are generally close to one another. The $R^2$ goodness of fit for each case is given in Table 2. This table shows that linear approximation provides an accurate fit for the data because the $R^2$ values are very close to 1.0. The residuals of the fit functions had random patterns indicating the adequacy of the linear approximation.

Based on the results of the fit through the entire data using all the frequency-weighting functions (ALL) in Table 1, the following equations to estimate VDV using $a_p$, $a_{w,p}$, and MTVV are proposed:

$$VDV = 0.55a_p$$
$$VDV = 0.75a_{w,p}$$
$$VDV = 1.35MTVV$$

Fig. 4. Locations of accelerometers and human subjects on the bridge portion for modal and walk tests

Fig. 5. Maximum recorded accelerations at the observers' locations during the walk tests
The $R^2$ goodness of fit for Eqs. (3a)–(3c) to represent the measured data are 0.977, 0.983, and 0.995, respectively, showing that these equations can very well represent the VDV in terms of $a_p$, $a_{w,p}$, and MTVV. Also, from Fig. 6 it can be noted that the curve fits of VDV versus MTVV based on different frequency-weighting functions are the closest to one another.

Ellis (2001, 2003) recommended an empirical relationship to compute the peak VDV ($VDV_p$) from the magnitude of peak-weighted acceleration ($a_{w,p}$): $VDV_p = 0.7a_{w,p}$ for building vibrations. This equation, which is similar to Eq. (3b), was developed using the $W_g$ frequency-weighting function. This shows the similarity of building and footbridge vibrations in terms of the relationship between VDV and $a_{w,p}$. Because the current standards do not

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**Fig. 6.** Variations of VDV versus $a_p$, $a_{w,p}$, and MTVV
recommend VDV limits for footbridges, but provide limits based on the maximum 1-s RMS and peak accelerations, Eqs. (3a) and (3c) are used to estimate VDV limits for footbridges.

Based on the lowest acceleration limit of 0.005 m/sec² of ISO 10137 (ISO 2007) as shown in Fig. 2, and using a maximum running 1-s RMS (MTVV) as recommended by this standard, the VDV limits from Eq. (3e) for the passive and active people (multiplier = 30 and 60, respectively, according to ISO 10137) are as follows: VDV = 1.35 × 0.005 × 50 = 0.2 m/sec¹.⁷⁵ for passive bystanders and VDV = 1.35 × 0.005 × 60 = 0.4 m/sec¹.⁷⁵ for active pedestrians. Using Eq. (3a), the equivalent peak (unweighted) accelerations for these limits are as follows: \( a_p = 3.7\% \) and \( 7.5\% \), respectively. These values were considered as the lower limits for the range of adverse comments possible for the passive and active people, and they were doubled to find the lower limits for the adverse comments probable range, as recommended by the ISO 10137 (ISO 2007) and BS 6472-1 (BSI 2008a) standards. Table 3 shows the proposed daily VDV limits along with their equivalent peak accelerations.

The proposed limits for the adverse comments possible and adverse comments probable by active pedestrians are similar to the limits recommended by ISO 10137 (ISO 2007) and BS 6472-1 (BSI 2008a) for residential buildings during 16 h of daily exposure.

The vibration limits recommended by Bachmann et al. (1995) of \( a_p < 5\% \) and the CSA (2006): \( a_p < 6\% \) seem to be conservative for active people when compared with the proposed limits. However, the proposed limits seem to be relatively consistent with BS 5400 (BSI 2006), \( a_p < 9\% \) (for the CF footbridge), even though they are higher for structures with lower natural frequencies. The proposed limits are also relatively consistent with the SETRA (2006) recommendations: \( a_p < 5\% \) (maximum comfort), \( 5\% < a_p < 10\% \) (medium comfort), \( 10\% < a_p < 25\% \) (minimum comfort), and \( 25\% < a_p \) (unacceptable). The proposed VDV limits are also consistent with the results of Ingolfsson et al. (2012).

The suggested VDV limits correctly predicted about 70% of the subjective evaluations of the footbridge vibrations by the two passive pedestrians (observers), and they correctly predicted almost all cases for the active pedestrians. In addition, it was noted that when pedestrians were running (201 spm) compared with walking (101 and 120 spm), they were less sensitive to vibrations. The inconsistency for the passive subjects can be attributed to the intersubject variability.

VDV allows consideration of vibration duration that the pedestrians may experience when crossing a footbridge and also how frequently the structure may be used because VDV is a cumulative evaluation parameter.

Even though VDV is a more consistent parameter to evaluate footbridge vibrations than simply using the peak acceleration, it requires more computational efforts. This may not be desirable to design engineers, and it is not necessary for the evaluation and assessment of footbridge vibrations during the design. In addition, it is logical to expect that the number of times that a footbridge may be used should influence the criterion for its vibration evaluation and assessment. This means that the acceptable vibration limits for a footbridge should depend on its level of use (lower limit for a more heavily used structure), which can result in an expected performance-based or usage-based serviceability design approach. This also means that the designer should be able to compute the acceptable vibration limit for his or her specific structure. As an example, if a footbridge is relatively heavily used (assume 4,000 crossings in 16 h of daily use), the acceptable limit of VDV for each crossing (VDV) using the acceptable total daily VDV, \( \text{VDV}_a = 1.2 \text{ m/sec}^{1.75} \) from Table 3 (average value for the probable adverse comments from active pedestrians) based on ISO 2631-1 (ISO 1997) is

\[
\text{VDV}_a = \frac{\text{VDV}_a}{\sqrt{N}} = \frac{1.2}{\sqrt{4,000}} = 0.15 \text{ m/sec}^{1.75} \tag{4}
\]

and from Eq. (3a), the acceptable peak acceleration for each crossing is

\[
a_p = \frac{\text{VDV}_a}{0.55} = \frac{0.15}{0.55} = 0.274 \text{ m/sec}^2 \quad a_p < 2.8\%
\]

Therefore, for this particular heavily used footbridge, the acceleration for active people should be limited to 2.8\%. This methodology can be applied to footbridges depending on the expected number of crossings using the proposed VDV limits in Table 3 and Eqs. 3(a) and 4. It should be noted that this method of evaluation is appropriate at the design stage when considering the results of analysis. When acceptability of vibrations is being evaluated based on the vibration measurements on an existing structure, use of VDV is more logical and accurate. As mentioned, this is because the measured peak acceleration is susceptible to errors in signal processing, such as filtering. In addition, the vibration duration is not accounted for when using peak acceleration as the main criterion for evaluation.

The CF footbridge is expected to be used by up to about 100 people during the events at the nearby outdoor amphitheater. Therefore, considering an acceptable daily VDV limit of \( \text{VDV}_a = 1.2 \text{ m/sec}^{1.75} \) from Table 3, the VDV limit for each crossing is

### Table 1. Regression Coefficients for Eq. (2)

<table>
<thead>
<tr>
<th>Frequency-weighting function</th>
<th>( a_p )</th>
<th>( a_{np} )</th>
<th>MTVV</th>
</tr>
</thead>
<tbody>
<tr>
<td>( W_b )</td>
<td>0.49</td>
<td>0.07</td>
<td>0.72</td>
</tr>
<tr>
<td>( W_m )</td>
<td>0.55</td>
<td>-0.05</td>
<td>0.78</td>
</tr>
<tr>
<td>( W_f )</td>
<td>0.58</td>
<td>0.00</td>
<td>0.79</td>
</tr>
<tr>
<td>( \text{All} )</td>
<td>0.55</td>
<td>0.02</td>
<td>0.76</td>
</tr>
</tbody>
</table>

### Table 2. \( R^2 \) Goodness of Fit for Fig. 6

<table>
<thead>
<tr>
<th>Frequency-weighting function</th>
<th>( a_p )</th>
<th>( a_{np} )</th>
<th>MTVV</th>
</tr>
</thead>
<tbody>
<tr>
<td>( W_b )</td>
<td>0.980</td>
<td>0.985</td>
<td>0.983</td>
</tr>
<tr>
<td>( W_m )</td>
<td>0.964</td>
<td>0.992</td>
<td>0.990</td>
</tr>
<tr>
<td>( W_f )</td>
<td>0.972</td>
<td>0.984</td>
<td>0.989</td>
</tr>
<tr>
<td>( \text{All} )</td>
<td>0.984</td>
<td>0.984</td>
<td>0.985</td>
</tr>
</tbody>
</table>

### Table 3. Proposed Total Daily VDV Limits and Their Corresponding \( a_p \) Values

<table>
<thead>
<tr>
<th>Pedestrian status</th>
<th>Low possibility of adverse comments, VDV [\text{m/sec}^{1.75} (a_p %)]</th>
<th>Adverse comments possible, VDV [\text{m/sec}^{1.75} (a_p %)]</th>
<th>Adverse comments probable, VDV [\text{m/sec}^{1.75} (a_p %)]</th>
<th>High probability of adverse comments, VDV [\text{m/sec}^{1.75} (a_p %)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passive</td>
<td>(&lt;0.20 (&lt;3.7))</td>
<td>0.2–0.4 (3.7–7.5)</td>
<td>0.4–0.8 (7.5–15)</td>
<td>(&gt;0.8 (&gt;15))</td>
</tr>
<tr>
<td>Active</td>
<td>(&lt;0.4 (&lt;7.5))</td>
<td>0.4–0.8 (7.5–15)</td>
<td>0.8–1.6 (15–30)</td>
<td>(&gt;1.6 (&gt;30))</td>
</tr>
</tbody>
</table>
Group Effects on Footbridge Vibrations

Background
The increase in vibrations when a crowd of people compared with one pedestrian crosses a footbridge was the subject of analytical research by Matsumoto et al. (1978). They proposed an enhancement factor (or multiplier) of $\sqrt{n}$ to estimate the vibrations amplitude when excited by a group of $n$ pedestrians from the vibrations attributable to one person crossing a footbridge, assuming a Poisson distribution for their arrival probability. They also studied 505 pedestrians walking naturally along roads and found that their average gait was about 2 steps per second, with a very small standard deviation.

Bachmann and Ammann (1987) recommended a modified version of the relationship just described for footbridges with a specific range of natural frequencies. They considered $\sqrt{n}$ as the enhancement factor for footbridges with natural frequencies of approximately 1.8 to 2.2 Hz. They suggested that when more than four pedestrians cross a footbridge, the enhancement factor varies linearly to 2.0 for structures with natural frequencies equal to 1.6 and 2.4 Hz. They did not provide any enhancement factors for footbridges with natural frequencies outside these ranges.

Wheeler (1982) conducted an analytical study for which he used loads from groups of randomly walking people. He found that as long as the fundamental frequency of the footbridge was away from the dominant human step frequency of 2.0 Hz, the structural response (nonresonance) for a group of people walking randomly was smaller than that induced by a single pedestrian walking at the footbridge resonance frequency. Barker (2007) conducted an analytical study in which he showed that considering random pedestrian movements, the enhancement factor for the RMQ for a group of $n$ people is about 1.188 $\sqrt{n}$.

Group-Effect Tests on the CF Footbridge
To check the effects of the number of pedestrians on the dynamic response of the footbridge used in this study, the results of the walk tests were used. As mentioned previously, 16 accelerometers were placed on different locations along the two sides of the footbridge, as shown in Fig. 4. Using a metronome, various numbers of people crossed the footbridge at four different speeds. The speeds and number of people in these tests were as follows: 101 spm (1, 3, 7, and 14 people), 120 spm (1, 3, and 14 people), 201 spm (1, 3, and 7 people), and random (1, 3, and 14 people). For the 120-spm and random speeds tests, the group of seven people was not used because the data point did not seem to contribute much to the study and also because of the time constraints. Only a maximum of seven people crossed the footbridge at 201 spm because larger groups of pedestrians would not have been able to keep their pace as a result of the excessive movements of the footbridge.

The MTVV and VDV using the frequency-weighting functions of the three standards—$W_k$ and $W_p$ [BS 6472-1 (BSI 2008a)], $W_a$ [ISO 2631-1 (ISO1997)], and $W_m$ [ISO 2631-2 (ISO 2003)]—along with the unweighted peak acceleration ($a_p$) were computed. It was found that Channel 9 (close to the bridge midspan; see Fig. 4) had the largest levels of vibration for most of the measurements, and therefore this channel was used for the following group-effect studies.

The footbridge group-response enhancement factors representing the increase in the response from a group of people compared with that from a single pedestrian crossing the structure at the same speed were computed. Three enhancement factors were considered: $a_p/a_p$, MTVV/MTVV, and VDV/VDV. MTVV and VDV are the magnitudes of peak unweighted acceleration, maximum transient vibration value, and vibration dose value, respectively, when a group of people (3, 7, and 14 people) crossed the footbridge; $a_p$, MTVV, and VDV are their counterparts when only one person crossed the footbridge at the same speeds. Fig. 7 shows the variation of these enhancement factors as a function of $n$ (number of people crossing the footbridge) along with the $\sqrt{n}$ curve (recommended in the literature) for the largest recorded vibrations of each test.

From Fig. 7(a), it can be noted that, except for the step frequency equal to the first-mode resonance frequency of the structure (3.35 Hz or 201 spm), $\sqrt{n}$ provided a conservative estimate of enhancement factor based on $a_p$. The same trend can be observed in Figs. 7(b and c) for the enhancement factors based on MTVV and VDV when considering groups of 7 and 14 people.

Figs. 7(b and c) show that the enhancement factors using different frequency-weighting functions were close to each other for the case of 201 spm (resonance frequency). This can be attributed to the fact that the bridge response when people walked at this speed was dominated by one frequency of vibration (resonance frequency of the footbridge) and also the fact that the value of the frequency-
weighting functions at 201 spm (3.35 Hz) were close to each other, as can be observed in Fig. 3.

Comparing the responses of Channel 9 using the various frequency-weighting functions, it was found that the results based on $W_w$ and $W_k$ were consistent with each other. In addition, it has been shown that these frequency-weighting functions are more realistic for the evaluation of low vibration levels in structures (Griffin 2007). Therefore, the values of MTVV and VDV using

Fig. 7. Variation of different group enhancement factors with group size ($n$) for the largest footbridge vibrations: (a) $a_p/a_{p1}$ versus $n$; (b) MTVV/MTVV$_1$ versus $n$; (c) VDV/VDV$_1$ versus $n$
enhancement factors for walk speeds and numbers of people in a group. It also includes the analysis of group effects. Table 4 shows the results for different beams, as shown in Fig. 4. A signal analyzer was then used to measure the damping ratios, a complete modal testing and analysis of the CF footbridge was conducted. An important parameter required to compute the footbridge damping ratio is shown because the modal amplitudes in the hub and ramp areas were negligible. The estimated natural frequency and damping ratio for the first mode were 3.48 Hz and 0.8%, respectively, and for the second mode were 4.68 Hz and 0.65%, respectively. As can be noted from this figure, the first mode is a torsional mode and the second mode is a bending mode.

**Footbridge Modal Damping Ratio**

Developing a computer model that can predict footbridge dynamic properties, such as natural frequencies, mode shapes, and footbridge vibration response, with reasonable accuracy is an important task of a design engineer when checking the vibration serviceability of a footbridge. An important parameter required to compute the footbridge response is its modal damping ratio. There are different suggested values for footbridge damping ratios in the available literature. However, there is no recommendation for the particular case of steel footbridges with wood decking. Therefore, to estimate the modal parameters and in particular the damping ratios, a complete modal testing and analysis of the CF footbridge was conducted.

**Modal Tests and Analyses**

An electrodynamic shaker supported by a force plate was placed close to the midspan of the bridge segment on one of the support beams, as shown in Fig. 4. A signal analyzer was then used to measure and record the vibration data. Channels 1 and 2 were connected to the force plate and the accelerometer attached to the shaker armature, respectively, to measure the input force.

A roving accelerometer technique was used, for which accelerometers connected to Channels 3, 5, 6, 8, 9, and 10 were placed on the footbridge main structural steel beams along the edges. Channels 4 and 7 were placed at the center of the deck on the wood flooring. These accelerometers were all oriented vertically. Accelerometers connected to Channels 11–16 were oriented laterally and clamped to the top of the guardrail. Fig. 4 shows the locations of accelerometers on the deck and the guardrail in the bridge segment only. A burst chirp excitation was used for the modal testing of the structure.

A maximum shift of 7% in the measured resonance frequencies during the modal tests was observed. This can be attributed to the temperature variations during the tests because it rose from 65°F to 80°F. From the measurements, it was clear that the resonance frequencies decreased as the structure became more flexible with the temperature rise during the modal tests.

MEscopeVES was used to conduct a modal analysis of the measured data. A local curve-fitting scheme enabled the extraction of modal properties with small variations in the resonance frequencies. Only the first two modes of vibration were identified because the higher modes had natural frequencies greater than 7.5 Hz, which are not susceptible to excitations by human movements on the footbridge. Fig. 8 shows the undeformed shape of the structure and its first two measured mode shapes (for clarity, only the bridge portion of the structure is shown because the modal amplitudes in the hub and ramp areas were negligible). The estimated natural frequency and damping ratio for the first mode were 3.48 Hz and 0.8%, respectively, and for the second mode were 4.68 Hz and 0.65%, respectively.

Based on the results of the CF footbridge modal tests, a damping ratio of 0.6 to 0.8% seems to be reasonable for such structures (steel footbridges with timber decking). This is within the range of recommended footbridge damping ratios found in the literature.

**Conclusions**

This paper presented three studies related to the vibration serviceability of footbridges. The conclusions made for each of the presented investigations are summarized as follows:

1. Footbridge vibration evaluation and assessment: Based on the results of the dynamic tests and analysis of the measured records, relationships between VDV versus \( a_p \), \( a_{wp} \), and MTVV were established. It was found that the relationship between VDV and MTVV using four different frequency-weighting functions was approximately linear and consistent. VDV limits for the vibration evaluation and assessment of the footbridge were established and recommended. The proposed limits were found to be consistent with the results of the subjective evaluation of vibrations conducted by the individuals participating in the tests.

Using the suggested VDV limits, a simplified evaluation and assessment method for performance-based (or usage-based) design was suggested to compute the acceptable peak acceleration for a footbridge based on its expected average daily use.
2. Comparing the proposed VDV limits in this paper with the measured responses of the structure when different numbers of people crossed the footbridge at various speeds demonstrated that, in most cases, the vibrations exceeded the allowable limits. This is consistent with the comments made by the footbridge users.

3. Group effects on footbridge vibrations: From the results of a number of tests conducted on the CF footbridge, it was found that when pedestrians walked slowly or at normal speed in unison, the enhancement factor for group effect increased approximately by the square root of the group size (the number of people in the group), as proposed in the available literature. However, when the group speed approached the first-mode resonance frequency of the footbridge, the enhancement factor became closer to the group size. It was also found that when a group of people walked randomly on the footbridge, the enhancement factors were less than when they walked in unison and the maximum footbridge vibration was smaller than when one pedestrian crossed the footbridge at its first-mode resonance frequency.

4. Footbridge modal damping ratios: Modal testing and analysis of the footbridge were conducted. The measured modal damping ratios were within the range reported in the available literature. Based on the results presented here, a modal damping ratio of 0.6 to 0.8\% for steel footbridges with timber decking is recommended. Designers may conservatively use a damping ratio of 0.6\%.

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Notation

The following symbols are used in this paper:

- \( a_p \) = peak acceleration;
- \( a_{p1}, MVV, VDV_1 \) = peak acceleration, maximum transient vibration value, and vibration dose value resulting from one person crossing the footbridge;
- \( a_{rms} \) = RMS of acceleration;
- \( a_w \) = frequency-weighted acceleration;
- \( a_{pw} \) = peak-weighted acceleration;
- \( C_{p1}, C_{pw1}, C_{M1} \) = slopes of the regression lines for VDV versus the peak acceleration, the peak-weighted acceleration, and the maximum transient vibration value, respectively;
\( C_{\rho Z} \), \( C_{pw2} \), \( C_{M2} \) = intercepts of the regression lines for VDV versus the peak acceleration, the peak-weighted acceleration, and the maximum transient vibration value, respectively; 
\( f_s \) = natural frequency of the structure; 
\( \text{MTVV} \) = maximum transient vibration value; 
\( N \) = total number of daily footbridge crossings; 
\( n \) = number of people in a group; 
\( \gamma \text{VDV} \) = acceptable total daily vibration dose value; 
\( \text{VDV} \) = vibration dose value; 
\( \text{VDV}_n \) = acceptable vibration dose value for one footbridge crossing; 
\( \text{VDV}_p \) = peak vibration dose value; 
\( W_p \) = frequency-weighting function according to BS 6472-1 (BSI 2008a); 
\( W_v \) = frequency-weighting function according to BS 6472-1 (BSI 2008a); 
\( W_i \) = frequency-weighting function according to ISO 2631-1 (ISO 1997); and 
\( W_m \) = frequency-weighting function according to ISO 2631-2 (ISO 2003).


ME’scopeVES 6.0 [Computer software]. Vibrant Technology Inc., Scotts Valley, CA.


References


