Experimental and analytical vibration serviceability assessment of an in-service footbridge

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Abstract

This paper discusses vibration serviceability assessment of a highly trafficked local footbridge based on the experimental tests and analytical studies. The selected bridge is an approximately 60 m (196 ft) long multi-span steel structure with a continuous reinforced concrete slab supported on two longitudinal steel girders. The experimental study consists of ambient vibration and pedestrian interaction tests to describe the dynamic characteristics of the selected bridge structure. The fundamental frequency of the bridge in the vertical direction obtained through ambient vibration tests was within the critical range described by available design guidelines. This required further analysis to assess the performance of the bridge relative to the maximum acceleration thresholds. In addition to the peak dynamic response obtained from the pedestrian interaction tests, peak acceleration values were calculated analytically based on current design guidelines and compared to the comfort limits. Results from both experimental and analytical studies suggest that the footbridge possesses satisfactory serviceability performance under low and dense traffic conditions, but the comfort level under very dense traffic loads was classified as minimum according to the results of the analytical calculations.

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1. Introduction

Contrary to highway or railroad bridges, footbridges represent a unique class of structures that are not typically subjected to heavy traffic loads. The nature of their design and construction, which often utilize lighter and more slender structural members, is able to fulfill the lower loading demands while also satisfying architectural concerns that lean more towards longer spans and aesthetically pleasing configurations. As a consequence, these geometrical characteristics often make the footbridges susceptible to human-induced vibrations, as they tend to have natural frequencies congruent with those from pedestrian traffic [1-4]. Although current design methodologies enforce constraints for strength and stability of the lightweight footbridge structures, it is their serviceability, in terms of level of comfort and safety, that has been the major concern affiliated with this specific types of structures [5,6].
The footbridge selected is a multi-span steel girder bridge with a concrete deck. Twin girders support the concrete deck with diaphragms at the pier intersections and cross-bracings at regular intervals along the length of the structure. Other important features of the bridge include its cantilever portions on both ends and parapets with railings. Fig. 1(a) and (b) show this selected footbridge from different angles.

The total length of the bridge is 57.9 m (190 ft) with five spans lengths as shown in Fig. 1(c). The total width of the bridge is 4.9 m (16 ft) including 0.3 m (1 ft) parapets on each side. The dimensions of structural members are given in Fig. 1(c). The center span girders have a thin cover plate on the bottom flange for added strength and stiffness. The unit
weight of the concrete is given as 2322 kg/m³ (145 pcf). The thickness of the slab is 26.7 cm (10.5 in). Based on measured geometry and available plans, the mass of the superstructure per unit length is calculated as 3537.0 kg/m (2376.7 lb/ft).

3. Ambient vibration tests

3.1. Test set-up

The test setup for the bridge included the installation of nine Bridge Diagnostics Inc. [17] accelerometers with a range of \( \pm 2 \text{g} \) and a differential sensitivity of 1 V/g. These sensors were connected by cables to BDI STS-WiFi nodes, which were then connected wirelessly to the BDI mobile base station and data acquisition system. The data acquisition software package used in the testing program was BDI WinSTS configured with a sampling frequency of 100 Hz. Data obtained from the WinSTS was then post-processed in MATLAB [18]. Fig. 2 illustrates the test setup on the selected bridge.

3.2. Modal identification

An ambient vibration test was conducted to determine the natural frequencies of the bridge. The ambient excitation consisted of wind load effects on the structure, vehicles passing underneath the bridge, and light pedestrian traffic on the bridge during the test. The accelerations from the transverse and vertical sensors were obtained for an extended time period of 900 seconds at a sampling frequency of 100 Hz.

In the field test, recorded acceleration time history data was digitally filtered using eight-order high pass and low pass infinite impulse response (IIR) filters with cut-off frequencies of 0.5 Hz and 30 Hz, respectively. Then, the Frequency Domain Decomposition (FDD) method was implemented to identify the first three natural frequencies for the transverse and vertical directions from the singular values as shown in Fig. 3. The frequencies identified from the singular values plot of the...
vertical power spectral density matrix are 4.02, 5.84, and 9.12 Hz, respectively. Also, the frequencies identified from the singular values plot of the transverse power spectral density matrix are 2.79, 4.03, and 6.32 Hz, respectively.

3.3. Evaluation of critical range of natural frequencies

The current guidelines that incorporate the pedestrian effect on the serviceability of footbridges employ a two-stage evaluation method. In the first stage, the fundamental frequencies of the bridge are compared with a critical frequency range. If the frequency limits suggested by the code is not fulfilled, a more detailed investigation is required as a second stage evaluation. In this case, the vertical and transverse accelerations obtained from a dynamic analysis should be in accordance with the maximum allowable values provided in the corresponding codes or provisions. Table 1 provides the critical frequency limits in the vertical and transverse directions according to the available design guidelines. The comparison of the fundamental frequency in the transverse direction obtained from the experimental tests (2.79 Hz) with the limit values specified by the selected guidelines indicate that no further dynamic investigation is needed in this direction. However, the measured fundamental frequency in the vertical direction (4.02 Hz) indicates that a dynamic assessment is necessary to satisfying all European guidelines. Although the natural frequency of the footbridge is above the critical natural frequency defined by the AASHTO specifications, the AASHTO specification also requires an evaluation of the dynamic performance when the second harmonic is a concern. A harmonic is defined as an integer multiple of step frequency for human activities. Noting that the range of normal walking frequencies is 1.6 Hz to 2.4 Hz, the second harmonic of these step rates might match with the fundamental natural frequency of the bridge in the vertical direction (4.02 Hz) and cause a resonant response. Therefore, further dynamic evaluation is needed in the vertical direction according to all selected guidelines.

4. Dynamic response assessment

The measurement of the bridge dynamic response under human excitation is needed to assess the comfort criteria as the natural frequency of the selected footbridge in the vertical direction falls within the critical frequency range. Therefore, experimental tests were conducted to observe the dynamic behavior of the footbridge. Although the footbridge automatically satisfies the maximum comfort level in the transverse direction as its fundamental transverse frequency (2.79 Hz) is outside the critical range, the dynamic response of the bridge was collected in both directions for completeness.

In addition to the experimental tests, a dynamic response analysis was also performed using existing models to estimate the maximum acceleration for the selected footbridge. In particular, analytical approaches for the prediction of human-induced vibration response proposed by HIVOSS [9], SÉTRA [10] and AISC Design Guide 11 [11] were employed. In the following subsections, the methodologies used in these guidelines for evaluation of the maximum expected vibration level are first discussed and then the experimental pedestrian interaction tests are described.

4.1. HIVOSS guidelines

The first step in the dynamic response assessment of footbridges, according to HIVOSS guidelines, is to define a set of physical conditions representing the real conditions that can occur during service time of the footbridge. These design situations are defined by specifying a traffic class and a chosen comfort level. The HIVOSS guidelines define five traffic classes based on pedestrian densities \( d \) [pedestrians/m\(^2\)] as shown in Table 2 and four comfort levels, ranging from unacceptable vibration levels to maximum comfort as shown in Table 3.

Once the design situations are defined, the next step is to calculate the maximum acceleration that the footbridge will experience under each design situation. To predict acceleration response, a harmonic load model, that simplifies the
random pedestrian load corresponding to a pedestrian density \( d \), is uniformly distributed on the bridge deck as an equivalent deterministic load. The model determines an equivalent number of perfectly synchronized pedestrians \( (n') \) that corresponds to a stream of \( n \) random pedestrians, as follows:

\[
n' = 10.8\sqrt{\frac{\xi}{n}} \quad (d < 1.0 \ \text{p/m}^2) \tag{1}
\]

\[
n' = 1.85\sqrt{n} \quad (d \geq 1.0 \ \text{p/m}^2) \tag{2}
\]

where \( \xi \) is the structural damping ratio. A uniformly distributed harmonic load that corresponds to the equivalent pedestrian stream can then be defined as follows:

\[
p(t) = P \cos(2\pi f_s t) \frac{n'}{5} \tag{3}
\]

where \( P \) is the component of the force due to a single pedestrian and given as 280 N for vertical vibrations and 35 N for transverse vibrations, \( S \) is the area of the loaded surface (note that \( n = S \times d \)), \( f_s \) is the walking frequency of the pedestrian or its harmonic multiple, and \( \psi \) is the reduction coefficient that accounts for the probability that the step frequency or its second harmonic approaches the considered natural frequency of the footbridge. Then, a finite element model or an equivalent SDOF system of the footbridge is used to compute the associated maximum acceleration for given dynamic loading. If SDOF approach is used, the maximum acceleration under the above described load model can be estimated as:

\[
a_{\text{max}} = \frac{p^*}{m^*} \frac{1}{2\xi}
\]

where \( p^* \) is the generalized load and calculated as \( 2pL/\pi \), where \( p \) is the distributed load per unit length and \( L \) is the length of the bridge; and \( m^* \) is the generalized mass and calculated as \( m_tL/2 \) for given distributed mass \( m_t \) per unit length.

### 4.2. SÉTRA guidelines

SÉTRA guidelines define four footbridge classes: Class I – urban footbridges with very heavy traffic; Class II – urban footbridges with heavy traffic; Class III – footbridges for standard use; and Class IV – seldom used footbridges. The comfort levels are defined in the same manner with HIVOSS guidelines as given in Table 3 except that the limiting acceleration value for maximum comfort for transverse vibrations is 0.15 m/s². For Class I to III footbridges, a dynamic analysis might be required if their natural frequency is below 5 Hz for vertical vibrations and below 2.5 Hz for transverse vibrations. Table 4 provides the load cases that need to be considered in the dynamic analysis depending on the footbridge’s class and natural frequency. Case I corresponds to a loading due to sparse or dense crowds and is considered for Class II and Class III footbridges. Case II represents a very dense crowd loading and is considered only for Class I footbridges. Case III considers the second harmonic of the vibrations caused by pedestrian loads and is taken into account for Class I and II footbridges. For all loading cases, the loads for dynamic analysis are calculated using the equation (3) given above. The pedestrian density \( d \) and the force due to walking of single pedestrian \( P \) are given in Table 5 for each loading case. The maximum acceleration of the footbridge is calculated using a SDOF approach and given as follows:

\[
a_{\text{max}} = \frac{1}{2\xi} \frac{4|p(t)|w_d}{\rho_s\pi}
\]

### Table 3

Comfort levels defined in HIVOSS.

<table>
<thead>
<tr>
<th>Comfort level</th>
<th>Description</th>
<th>Vertical ( a_{\text{limit}} ) (m/s²)</th>
<th>Transverse ( a_{\text{limit}} ) (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL 1</td>
<td>Maximum</td>
<td>&lt;0.5</td>
<td>&lt;0.1 (0.15')</td>
</tr>
<tr>
<td>CL 2</td>
<td>Medium</td>
<td>0.5–1.0</td>
<td>0.1 (0.15’–0.3)</td>
</tr>
<tr>
<td>CL 3</td>
<td>Minimum</td>
<td>1.0–2.5</td>
<td>0.3–0.8</td>
</tr>
<tr>
<td>CL 4</td>
<td>Unacceptable</td>
<td>&gt;2.5</td>
<td>&gt;0.8</td>
</tr>
</tbody>
</table>

* Limit value defined in SÉTRA guidelines.

### Table 4

Load cases to select for dynamic analysis.

<table>
<thead>
<tr>
<th>Footbridge class</th>
<th>Frequency range (V: Vertical; T: Transverse)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range 1</td>
</tr>
<tr>
<td></td>
<td>( V = 1.7–2.1 \ \text{Hz} )</td>
</tr>
<tr>
<td></td>
<td>( T = 0.5–1.1 \ \text{Hz} )</td>
</tr>
<tr>
<td>Class I</td>
<td>Case 2</td>
</tr>
<tr>
<td>Class II</td>
<td>Case 1</td>
</tr>
<tr>
<td>Class III</td>
<td>Case 1</td>
</tr>
<tr>
<td></td>
<td>Range 2</td>
</tr>
<tr>
<td></td>
<td>( V = 1.0–1.7 \ \text{Hz} ) and 2.1–2.6 Hz</td>
</tr>
<tr>
<td></td>
<td>( T = 0.3–0.5 \ \text{Hz} ) and 1.1–1.3 Hz</td>
</tr>
<tr>
<td>Class I</td>
<td>Case 2</td>
</tr>
<tr>
<td>Class II</td>
<td>Case 1</td>
</tr>
<tr>
<td>Class III</td>
<td>Case 1</td>
</tr>
<tr>
<td></td>
<td>Range 3</td>
</tr>
<tr>
<td></td>
<td>( V = 2.6–5.0 \ \text{Hz} )</td>
</tr>
<tr>
<td></td>
<td>( T = 1.3–2.5 \ \text{Hz} )</td>
</tr>
<tr>
<td>Class I</td>
<td>Case 3</td>
</tr>
<tr>
<td>Class II</td>
<td>Case 3</td>
</tr>
<tr>
<td>Class III</td>
<td>Case 3</td>
</tr>
</tbody>
</table>


where \( |p(t)| \) is the magnitude of the applied load \( p(t) \); \( w_d \) is the available width of the deck for pedestrian walking; and \( \rho_s \) is the total linear density, which is calculated as the sum of the linear density of deck and that of the pedestrians.

4.3. AISC Design Guide 11

AISC Design Guide 11 defines peak acceleration limits by adjusting base line curve given in ISO 2631–2 [19] based on the intended occupancy. The acceleration limits are provided as a function of structure frequency for both indoor and outdoor footbridges. For outdoor footbridges, the recommended tolerance limit is specified as 0.49 m/s² (0.5%g) for structures with fundamental frequencies between 4 Hz and 8 Hz.

Then, a time-dependent harmonic function that matches the fundamental frequency of the footbridge is defined and the resonance response is given as follows:

\[
a_{\max} = \frac{RaP}{\zeta W/g}
\]

(6)

where \( P \) is the pedestrian weight, \( R \) is the reduction factor, \( \zeta \) is the damping ratio, \( W \) is the effective weight of structure, \( g \) is the gravity and \( \alpha \) is the dynamic coefficient, which is given as

\[
\alpha = 0.83 \exp(-0.35f_n)
\]

(7)

where \( f_n \) is the natural frequency of the footbridge. If the peak acceleration estimated with the equation above does not exceed the acceleration limit, the footbridge satisfies the vibration serviceability requirements.

4.4. Pedestrian interaction tests

To assess the serviceability performance of the selected footbridge under real service-loading conditions, dynamic tests were performed under pedestrian walking excitation and the acceleration response of the bridge was collected from the sensors installed at different locations of the bridge as shown in Fig. 2. First, the response of the bridge associated with the walking of a synchronized group of pedestrians was measured. In particular, the tests were conducted with 9 people walking in a 3 × 3 formation, which is the test condition described in previous studies [3], at three frequencies (1.2 Hz, 1.5 Hz and 2 Hz). The walking pace frequencies were guided using a metronome. Note that the second harmonic multiple of 2 Hz step frequency is 4 Hz, which is almost the same with the fundamental frequency of the footbridge in the vertical direction (4.02 Hz) and can produce resonant response. The other two step frequencies represent a slow and normal walking speed. Each test was repeated three times and the peak response was selected as the maximum of peak responses from three tests. Then, the response of the bridge induced by a continuous flow of pedestrians was recorded. In that case, about 40 people walked on the bridge without an intention to synchronize their steps.

5. Evaluation of vibration levels

5.1. Analytical results

To estimate the maximum accelerations using HIVOSS guideline, two design situations were considered. The first design situation (D1) corresponds to the daily usage with a traffic class of TC2 and a target comfort level of CL1. For this design situation, the pedestrian density was given as \( d = 0.2 \) p/m², and the equivalent number of synchronized pedestrian and corresponding harmonic load were calculated as \( n' = 4.8 \) and \( p(t) = 1.360 \cos(2\pi(4.02) t) \). Note that the second harmonic of the walking frequency is considered in the harmonic load and assumed to be the same with the fundamental frequency of the footbridge \( (f_n = 4.02 \text{ Hz}) \). The second situation (D2) resembles the case where a high traffic occurs such as during a sporting event. The traffic class assigned to D2 is TC4 and the target comfort level is CL2. For the second design situation, the pedestrian density was given as \( d = 1 \) p/m², and the equivalent number of synchronized pedestrian and corresponding harmonic load were calculated as \( n' = 29.1 \) and \( p(t) = 8.238 \cos(2\pi(4.02) t) \). The reduction coefficient \( \psi \) was taken as 0.25 from the guidelines. The peak acceleration was calculated as 0.26 m/s² and 1.46 m/s² for the first and second design situation, respectively.

<table>
<thead>
<tr>
<th>Loading case</th>
<th>Pedestrian density, ( d ) (p/m²)</th>
<th>Single pedestrian force, ( P ) (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class I</td>
<td>Class II</td>
</tr>
<tr>
<td>Case 1</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td>Case 2</td>
<td>1.0</td>
<td>–</td>
</tr>
<tr>
<td>Case 3</td>
<td>1.0</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Table 6
Analytical results for peak acceleration response.

<table>
<thead>
<tr>
<th></th>
<th>HIVOSS – D1</th>
<th>HIVOSS – D2</th>
<th>SÉTRA</th>
<th>AISC DG 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_{\text{max}}$ (m/s²)</td>
<td>0.26</td>
<td>1.46</td>
<td>0.49</td>
<td>0.09</td>
</tr>
</tbody>
</table>

Fig. 4. Vertical (V3) and transverse (T3) accelerations due to pedestrian walking at different frequencies.

To estimate the maximum acceleration using SÉTRA guideline, the selected footbridge was considered as Class II footbridge. According to Table 4, the dynamic response of the footbridge was calculated under Case 3 loading conditions since its vertical natural frequency falls into Range 3. The corresponding pedestrian density for this loading case is given as $d = 0.8$ p/m² in Table 5. The equivalent number of synchronized pedestrian and harmonic load are calculated as $n' = 9.6$ and $p(t) = 2.721 \cos(2\pi (4.02) t)$. The reduction coefficient $\psi$ is taken as 1.0 from SÉTRA guidelines. The damping ratio was selected as 0.4%, which is the suggested average value by both HIVOSS and SÉTRA guidelines for the steel footbridges. For this design case, the peak acceleration was calculated to be 0.49 m/s².

The equation (6) was used to estimate the maximum acceleration due to walking excitation according to AISC specifications. Note that no traffic class or design situation are defined in AISC Design Guide 11 and the equation (6) predicts the peak acceleration assuming there is only one pedestrian. The guide provides $P$ as 700 N and recommends that $R$ be taken as 0.7 and the damping ratio be taken as 1% for footbridges. Using these values in the equation (6), the peak acceleration was calculated as 0.9 m/s². Table 6 summarizes predicted maximum acceleration response according to each design guideline.

5.2. Experimental results

Acceleration time histories in vertical (V) and transverse (T) directions were collected from the pedestrian interaction tests. The collected data included acceleration records over the course of the tests for all of the four loading scenarios (9 people at 3 different walking frequencies and random flow of pedestrians) and their corresponding repetitive runs. Representative sets of raw data collected from the synchronized tests from the accelerometer V3 and T3 are illustrated in Fig. 4, which highlight the sensitivity of the bridge dynamic response to the imposed loading frequency. As discussed earlier, for the walking (step) frequency of 2 Hz, the second harmonic of the step frequency is almost equal to the fundamental
frequency of the footbridge in the vertical direction. Therefore, a resonant response was observed in the vertical acceleration history at the 2 Hz walking frequency.

The acceleration time histories obtained from the accelerometer V3 during the passage of about 40 people on the footbridge for three different tests are shown in Fig. 5. It can be seen that the amplitude of the accelerations in these random pedestrian flow tests was lower than those obtained during the synchronized tests at 2 Hz.

The raw data was post-processed to define the peak acceleration values obtained from each test run. Table 7 summarizes the peak acceleration values at different locations according to the mounted instrumentations. The maximum acceleration responses in the vertical direction in all tests was 0.40 m/s². This peak value was compared to the allowable limits provided by current design guidelines in the next section.

5.3. Serviceability evaluation

Fig. 6 illustrates the comparison between critical acceleration values obtained from experimental and analytical approaches, to the limits provided in current guidelines. As demonstrated, the maximum critical acceleration response obtained from the pedestrian interaction tests (0.40 m/s²), satisfies the limits of the codes, which indicates acceptable vibration serviceability. The peak acceleration estimated for the design situation D1 using SDOF approach according to HIVOSS guideline (0.26 m/s²) is also below the target comfort level (maximum comfort or $a_{\text{max}} < 0.50$ m/s²) selected for this situation. In the design situation D2, the analytical values obtained using HIVOSS guidelines (1.46 m/s²) for critical dynamic responses of the footbridge exceed the target comfort limit (medium comfort or $a_{\text{max}} < 1$ m/s²). The peak acceleration estimated by SÉTRA guideline (0.49 m/s²) also indicates a maximum comfort level for the footbridge, but the predicted acceleration is very close to limiting acceleration for the maximum comfort. Finally, the peak acceleration estimated by AISC Design Guide 11 (0.09 m/s²) is considerably below the limiting value of the maximum acceleration response described by the same code (0.49 m/s²).

Note that the selected number of people for the pedestrian interaction tests corresponds to an equivalent pedestrian density of 0.7 p/m² for this footbridge. The SÉTRA guidelines classify the bridge as Class III for such a traffic level and do not require a dynamic analysis for that footbridge. For a pedestrian density $d = 0.7$ p/m², the HIVOSS guidelines predict the peak acceleration as 0.46 m/s², which is close to the peak measured acceleration of 0.40 m/s² during experimental testing. AISC Design Guide 11 refers the study conducted by Backmann and Ammann [20] to relate the peak acceleration calculated by equation (6) for one walker to the peak acceleration due to a group of walker. For $n$ synchronized pedestrians, the peak acceleration is $n$ times that for a single pedestrian. Therefore, for 9 synchronized pedestrians, the peak acceleration is calculated as $9 \times 0.09 = 0.81$ m/s². Therefore, it can be concluded that the analytical expressions provided in HIVOSS, SÉTRA, and AISC Design Guide 11 tend to be conservative for estimating the peak acceleration response of the studied footbridge. Nevertheless, they are considered as safe margins (upper bound on the behavior) that can help engineers control the vibration vulnerability of the pedestrian bridges during the design procedure.

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**Table 7**  
Peak acceleration responses (m/s²).

<table>
<thead>
<tr>
<th>Test</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>V5</th>
<th>V6</th>
<th>T1</th>
<th>T2</th>
<th>T3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2 Hz</td>
<td>0.098</td>
<td>0.097</td>
<td>0.145</td>
<td>0.146</td>
<td>0.059</td>
<td>0.066</td>
<td>0.015</td>
<td>0.021</td>
<td>0.010</td>
</tr>
<tr>
<td>1.5 Hz</td>
<td>0.160</td>
<td>0.154</td>
<td>0.169</td>
<td>0.153</td>
<td>0.119</td>
<td>0.100</td>
<td>0.033</td>
<td>0.036</td>
<td>0.016</td>
</tr>
<tr>
<td>2 Hz</td>
<td>0.207</td>
<td>0.200</td>
<td>0.400</td>
<td>0.396</td>
<td>0.147</td>
<td>0.129</td>
<td>0.028</td>
<td>0.031</td>
<td>0.024</td>
</tr>
<tr>
<td>Random</td>
<td>0.194</td>
<td>0.189</td>
<td>0.192</td>
<td>0.169</td>
<td>0.163</td>
<td>0.161</td>
<td>0.044</td>
<td>0.046</td>
<td>0.028</td>
</tr>
</tbody>
</table>

---

**Fig. 5.** Vertical acceleration time histories at V3 during random flow of pedestrian tests.
6. Conclusions

This paper examines the serviceability of an in-service footbridge through experimental and analytical studies. Ambient vibration tests were conducted to determine the natural frequencies in the vertical and transverse directions. The data obtained from the ambient vibration tests were processed using the frequency domain decomposition method. The fundamental frequency of the bridge in the vertical direction was found to be within the critical range described by various design guidelines, requiring further analyses. As a result, dynamic tests were conducted to evaluate the maximum acceleration response of the footbridge when different groups of people walked along the whole structure at three different frequencies. The induced peak accelerations were below the recommended limits for all loading conditions. It was noted that the second harmonic of the step frequency amplified the acceleration response due to resonant effects. The peak acceleration response was also calculated using the equations provided by different guidelines. The analytical results suggest that the footbridge possesses satisfactory serviceability performance under low and dense traffic conditions. However, under very dense traffic loads, the comfort level was classified as minimum according to HIVOSS guidelines. Further experimental studies are needed to assess the performance of the footbridge under very dense pedestrian loading.

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References


