DYNAMIC ANALYSIS OF THE ROTTERDAM CENTRAL STATION FOOTBRIDGE

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Summary

The dynamic performance of the Rotterdam Central Station footbridge (The Netherlands) is analysed. Two experiments are performed to study the dynamic behaviour. The first experiment involves an ambient test to study the dynamic behaviour during normal load conditions. The second experiment involves a controlled test with groups of pedestrians to study the effect of a small scale provisional tuned mass damper (TMD). The results of the first test show that unacceptable vibrations are expected for a pedestrian density exceeding 0.3 ped./m². Also, excessive vibrations are observed for a single pedestrian jumping at a resonant frequency. The results of the second test show that a TMD can be a useful measure for reducing vibrations. An average reduction factor of 1.3 is observed during the controlled test with a small scale provisional TMD.

Keywords:  dynamic response; pedestrians; footbridge; tuned mass damper; group effect; vibrations.

1. Introduction

In 2007, a new footbridge was built at the central train station in the city centre of Rotterdam (The Netherlands). This footbridge is designed as part of a large scale redevelopment project which includes the train station and surrounding areas. Modifications to the train station are required because of the growing number of passengers in the near future. It is expected that the number of passengers will increase from 110,000 per day in 2010 up to 323,000 per day by 2025 caused by new connections to the regional and international railway network.

Fig. 1 Footbridge at Rotterdam Central Station (The Netherlands)

Since the footbridge opened in 2007, several complaints are reported concerning its dynamic behaviour. Some pedestrians are surprised by the lively character of the bridge, others even feel uncomfortable. It is expected that these vibrations are caused by the dynamic load imposed by pedestrians crossing the bridge. This phenomenon is often observed for slender footbridges where a natural frequency of the structure is close to the main excitation frequency of walking pedestrians (approximately 2 Hz).

The purpose of this investigation is to analyse the dynamic performance of the bridge in order to verify whether acceptable vibration limits are exceeded and if so, for which load scenarios. Secondly, this study aims at verifying whether the installation of a TMD can be a valuable solution to reduce vibrations.
The research topics are analysed by two on-site experiments. The first experiment is performed to assess the dynamic performance in the current situation. A second experiment is performed with controlled groups of pedestrians and a small scale provisional Tuned Mass Damper (TMD) in order to analyse the effect of this damping device. The experiments and their analysis as presented in this paper are an addition to a previous numerical study on footbridge dynamics as described in [1].

2. Footbridge Design

The bridge has a total length of 136 meter and is subdivided into 6 simply supported sections, each spanning two or three railroad tracks. The structural principle of the bridge is shown in Fig. 2. The bridge deck is supported by a single column on each platform. These columns provide stability in both horizontal directions by a fixed connection at the foundation. Additional stiffness is provided parallel to the railway tracks by the stairs attached to the top of the column.

![Fig. 2 Structural principle of the footbridge parallel to railway tracks (left) and perpendicular to railway tracks (right)](image)

A typical cross section of the bridge is shown in Fig. 3. The structure consists of a box girder in 15mm steel plates. A layer of approximately 70 mm concrete is added on top of the deck to provide a rigid surface. Other typical features include a balustrade consisting of steel banisters with glass infill panels positioned on both sides of the deck and a catenary suspension system which is attached to the bridge.

![Fig. 3 Typical cross section above railway tracks](image)

The bending stiffness (EI) and mass per unit length (m) equal approximately $2.0 \times 10^9$ Nm² and 2500 kg/m respectively. The bending stiffness is based on the steel box girder only. The mass includes the steel box girder as well as the concrete top layer and balustrade. These properties are derived from structural drawings and used to estimate the dynamic characteristics. Assuming a mode shape of half a sine wave, the fundamental frequency and modal mass are calculated using well known formulas based on bending theory [2]:

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Footbridge Dynamics
$$f_{n=1} = \frac{(n\pi)^2}{2mL^2} \sqrt{\frac{EI}{m}} = \frac{(1\pi)^2}{2\pi^2 \times 27^2} \sqrt{\frac{2.0 \times 10^9}{2500}} = 1.9 \text{ [Hz]}$$ (1)

$$M_{n=1} = \frac{1}{2} mL = \frac{1}{2} \times 2500 \times 27 = 34 \times 10^3 \text{ [kg]}$$ (2)

3. Dynamic Testing

3.1 Description of Experiments

Two separate experiments are performed and analysed. Experiment 1 is performed during rush hour on a weekday. This experiment is used to record dynamic response during normal load conditions without any additional damping devices. The results are used to assess the dynamic performance of the bridge in the current situation and to verify whether vibration limits are exceeded. Experiment 2 involves a controlled test with small groups of pedestrians walking simultaneously at a predefined step frequency matching the fundamental frequency of the bridge. The step frequency is monitored using a metronome. The second test is carried out with and without a small scale provisional TMD. By using controlled groups of pedestrians, a reliable comparison can be made between response data with and without TMD. The results for the second experiment are used to verify whether a TMD is a possible solution to reduce vibrations.

3.2 Equipment Set-up

The dynamic tests presented in this paper are restricted to the south section of the bridge which has the largest span of 27 meter. As a result, the fundamental frequency of this section is the lowest and considered the most critical. Two sensors are used and placed on the locations as indicated in Fig. 4. Both sensors are placed in the middle of the span, sensor 1 is placed in the middle of the bridge, sensor 2 is placed alongside the balustrade. The sensors are connected to a computer which records continuously at a sampling rate of 1000 Hz. In experiment 2, the TMD is positioned in the middle of the bridge, next to sensor 1. At this location, the TMD is most effective with respect to the fundamental mode of vibration. The sensors record vertical as well as horizontal vibration velocities. For analytical purposes, the response is transformed into acceleration. Finally, peak-peak accelerations are derived for further analysis.

**Fig. 4 Top view of south section with sensor layout**
3.3 Bridge Natural Frequencies

The natural frequencies of the bridge are derived on site by introducing a pulse load in the middle of the bridge. In Fig. 5, a response spectrum is shown of the vertical response observed at the middle of the bridge. The spectrum shows a clear peak at 2.09 Hz which corresponds with the fundamental mode of vibration. On site observations as well as a Finite Element analysis confirmed that this frequency corresponds to the mode shape of half a sine wave. The fundamental frequency according to Fig. 5 ($f = 2.09$ Hz) is greater than the frequency found using eq. (1) ($f = 1.9$ Hz). This can be explained by the fact that the bending stiffness in eq. (1) is based solely on the steel part of the cross section. It is expected that the concrete top layer provides additional stiffness to the system, even though it has no primary structural function. Higher modes of vibration are summarised in Table 1. The dynamic analysis of the bridge focuses on the fundamental frequency considering the relatively high values for the other modes of vibration.

![Fig. 5 Vertical response spectrum at the middle of the bridge](image)

**Table 1 Dynamic characteristics bridge**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency ($f_i$) [Hz]</th>
<th>Damping ratio [-]</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.09</td>
<td>0.017</td>
<td>vertical</td>
</tr>
<tr>
<td>2</td>
<td>4.20</td>
<td></td>
<td>horizontal</td>
</tr>
<tr>
<td>3</td>
<td>6.29</td>
<td></td>
<td>horizontal</td>
</tr>
<tr>
<td>4</td>
<td>8.38</td>
<td></td>
<td>vertical</td>
</tr>
</tbody>
</table>

3.4 Damping

Fig. 6 shows the vertical response in the time domain recorded in the middle of the bridge after introducing a pulse load. In a period of approximately 5.0 seconds, the response curve shows 10.5 amplitudes. Thus, the fundamental frequency equals 2.1 Hz which corresponds with the frequency spectrum shown in Fig. 5. The curve fit function shown in Fig. 6 is used to obtain the modal damping ratio ($\zeta$) for the fundamental mode of vibration:

$$a = Ce^{-\omega t} = Ce^{-0.220t} \quad \rightarrow \quad \zeta = \frac{-0.220t}{-\omega t} = \frac{-0.220xt}{-2\pi\times2.09xt} = 0.017 \ [\ ]$$

A damping ratio of 0.017 is relatively high compared to damping ratios found in literature. The Dutch code for steel bridges, NEN6788 [3] and Fib Bulletin 32 [4] for example advice a damping ratio of 0.002 and 0.004 respectively for welded steel structures. In this specific case however, additional damping is provided by deformation and friction in the concrete layer as well as the rubber fittings in the glass balustrade.

![Fig. 6 Decay curve for response in the middle of the bridge after introducing a pulse load](image)
3.5 TMD

An impression of the provisional TMD is presented in Fig. 7. The design of the TMD consists of a mass supported by four 10 mm thick polycarbonate sheets which provide the required stiffness of the system. For practical reasons, the total participating mass of the damper ($M_{\text{TMD}}$) was limited to a total of 105 which includes a main mass of 100 kg and 5 kg of polycarbonate sheets. The choice for polycarbonate is based on the low Modulus of Elasticity and high bending strength of this material. As a result, the dimensions of the damper are kept to a minimum. The span and width of the polycarbonate sheets equal 830 mm and 250 mm respectively. The sheets of polycarbonate are separated by aluminium tubes in order to avoid friction between the sheets.

![Fig. 7 TMD principle (left) and experimental set-up on bridge (right)](image)

The span of the TMD is tuned to match the fundamental frequency of the bridge. The natural frequency of the TMD is crucial in order to obtain maximum effectiveness. Based on the fundamental frequency of the bridge and the mass of the TMD, the optimal natural frequency and stiffness of the TMD according to well known formulas equal [5]:

$$f_{\text{TMD}} = \frac{f_1}{(1 + (M_{\text{TMD}}/M_1))} = \frac{2.09}{(1 + (105/34000))} = 2.08 \text{ [Hz]} \quad (4)$$

$$k_{\text{TMD}} = (2\pi f_1)^2 M_{\text{TMD}} = (2\pi\times2.08)^2 \times 105 = 18 \times 10^3 \text{ [N/m]} \quad (5)$$

The effect of the TMD is tested by a single pedestrian jumping for approximately 10 seconds at a constant frequency at the middle of the bridge. The test is repeated for five different jump frequencies in a range from 2.0 Hz up to 2.2 Hz and performed with and without the TMD. For each load case the maximum peak-peak acceleration is derived from the measurements and shown in Fig. 8.

The response in Fig. 8 shows a clear peak in the response pattern for jump frequencies close to the fundamental frequency of the bridge (2.09Hz). The effect of the TMD can be observed for the whole frequency range. The response without the TMD is significantly higher compared to the response with the TMD for all jump frequencies. Because the TMD is relatively light, it is easily excited by frequencies besides the fundamental frequency.

![Fig. 8 Peak-peak accelerations observed at the middle of the bridge for a single pedestrian jumping at different frequencies close to the fundamental frequency of the bridge with and without TMD](image)
3.6 Acceleration Limit

The response limit according to British Standard BS5400 is frequently used in the dynamic assessment of footbridges. This vibration limit equals: $a_{\text{limit}} = \frac{1}{2}\sqrt{f_1}$, where $f_1$ represents the fundamental frequency of the bridge. In this case, $a_{\text{limit}} = \frac{1}{2}\sqrt{2.09} \approx 723 \text{ mm/s}^2$. This response limit corresponds well with the Dutch code for steel bridges NEN6788 [3]: $a_{\text{limit}} = 700 \text{ mm/s}^2$ and Eurocode EN1990 [6]: $a_{\text{limit}} = 700 \text{ mm/s}^2$. This limit corresponds to comfort class 2 (500 – 1000 mm/s$^2$) in the HiVoSS guideline [7] and is described as a ‘medium degree of comfort’. For the analysis described in this paper, the vibration limit according to BS5400 is used in accordance with previous research presented in [1].

4. Results

4.1 Response Examples

In Fig. 9, three examples are given of vertical response patterns for different load cases. The typical response is shown for a single pedestrian jumping at 2.1 jumps per second at the middle of the bridge. Also, a stream of 7 up to 13 pedestrians walking over the bridge at random step frequencies is presented as well as a single pedestrian walking over the bridge at a resonant frequency of 2.1 steps per second. In addition, the response limit as introduced in section 3.6 is added.

Fig. 9 shows that all three load cases result in a significant response. This response is dominated by the fundamental resonance frequency of 2.1 Hz. The maximum response for a pedestrian walking and jumping at a resonant frequency equals approximately 210 mm/s$^2$ and 890 mm/s$^2$ respectively. In this case, the response for single pedestrian shows a similar pattern as the response for the stream of multiple pedestrians. The response of a group or stream however depends strongly on the number of pedestrians within the group or stream walking at a resonant frequency. Fig. 9 shows that the response limit is exceeded by the jumping pedestrian.

![Fig. 9 Examples of typical vertical response at the middle of the bridge for different load cases](image_url)

4.2 Results Experiment 1: Random Rush Hour Walking without TMD

For experiment 1, the response of the bridge during rush hour is recorded continuously for 1.5 hours. For analytical purposes, the response signal is divided into sections of one second. For a total duration of 1.5 hours, this procedure yields 90 minutes x 60 seconds = 5400 separate response samples. For each sample, the peak-peak acceleration and the number of pedestrians present on the bridge are derived. The results are presented in Fig. 10. For each sensor three graphs are shown representing the maximum, average and minimum peak-peak acceleration relative to the number of pedestrians present on the bridge.
During the first experiment, a maximum of 17 pedestrians are recorded on the bridge at the same time. This number corresponds with a pedestrian density of 0.11 ped./m². The graphs representing minimum and average peak acceleration show a clear increase in response up to 13 pedestrians. The decrease in response for more than 13 pedestrians can be explained by the limited results available. The graphs for both sensors show a similar pattern as they are both positioned on the critical location on the bridge with respect to the fundamental mode of vibration.

![Fig. 10 Vertical response for experiment 1: random walking during rush hour without TMD](image)

The results for sensor 1 are reproduced in Table 2. For each number of pedestrians a population of peak-peak accelerations is determined. For these populations, four separate parameters are derived which include the maximum and minimum accelerations as well as the average value \( p \) and 95th percentile \( q \). The 95th percentile of the response ignores the relatively few high peaks that might occur. This parameter is used in the analysis to compare the response with acceleration limits. In addition, a factor \( r \) is calculated, which represents the ratio \( q / p \). This ratio is used for further interpretation of the results in section 5.

<table>
<thead>
<tr>
<th>No. ped. (n)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (d)</td>
<td>0.007</td>
<td>0.013</td>
<td>0.020</td>
<td>0.027</td>
<td>0.034</td>
<td>0.040</td>
<td>0.047</td>
<td>0.054</td>
<td>0.061</td>
<td>0.067</td>
<td>0.074</td>
<td>0.081</td>
<td>0.088</td>
<td>0.095</td>
<td>0.102</td>
<td>0.109</td>
<td>0.116</td>
</tr>
<tr>
<td>[ped./m²]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. Samples</td>
<td>848</td>
<td>513</td>
<td>359</td>
<td>214</td>
<td>173</td>
<td>184</td>
<td>104</td>
<td>94</td>
<td>57</td>
<td>62</td>
<td>46</td>
<td>24</td>
<td>14</td>
<td>6</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max [mm/s²]</td>
<td>269</td>
<td>288</td>
<td>309</td>
<td>366</td>
<td>372</td>
<td>395</td>
<td>351</td>
<td>351</td>
<td>419</td>
<td>375</td>
<td>324</td>
<td>433</td>
<td>397</td>
<td>412</td>
<td>287</td>
<td>234</td>
<td>205</td>
</tr>
<tr>
<td>q (95%) [mm/s²]</td>
<td>173</td>
<td>180</td>
<td>236</td>
<td>233</td>
<td>289</td>
<td>290</td>
<td>304</td>
<td>310</td>
<td>361</td>
<td>348</td>
<td>292</td>
<td>352</td>
<td>356</td>
<td>&quot;*&quot;</td>
<td>&quot;*&quot;</td>
<td>&quot;*&quot;</td>
<td>&quot;*&quot;</td>
</tr>
<tr>
<td>p (50%) [mm/s²]</td>
<td>60</td>
<td>79</td>
<td>103</td>
<td>122</td>
<td>133</td>
<td>154</td>
<td>169</td>
<td>160</td>
<td>159</td>
<td>157</td>
<td>153</td>
<td>211</td>
<td>242</td>
<td>173</td>
<td>153</td>
<td>144</td>
<td>139</td>
</tr>
<tr>
<td>Min [mm/s²]</td>
<td>4</td>
<td>11</td>
<td>20</td>
<td>28</td>
<td>31</td>
<td>46</td>
<td>28</td>
<td>34</td>
<td>48</td>
<td>51</td>
<td>54</td>
<td>98</td>
<td>100</td>
<td>61</td>
<td>82</td>
<td>83</td>
<td>73</td>
</tr>
<tr>
<td>r (q/p) [-]</td>
<td>2.89</td>
<td>2.27</td>
<td>2.30</td>
<td>1.92</td>
<td>2.17</td>
<td>1.89</td>
<td>1.79</td>
<td>1.94</td>
<td>2.27</td>
<td>2.21</td>
<td>1.91</td>
<td>1.67</td>
<td>1.47</td>
<td>&quot;*&quot;</td>
<td>&quot;*&quot;</td>
<td>&quot;*&quot;</td>
<td></td>
</tr>
</tbody>
</table>

* for \( q \) (95th percentile), a minimum of 20 samples is required

4.3 Results Experiment 2: Controlled Group Walking with and without TMD

During the second experiment, groups of three up to six pedestrians are asked to walk over the bridge at a resonant frequency of 2.1 steps per second. The configuration of the groups consists of two rows of pedestrians walking next two each other with a spacing distance of approximately two meters. This experiment is performed with and without the TMD. In total, 43 crossings are recorded with groups of different sizes. For each crossing, a single peak-peak acceleration is derived and shown in Fig. 11. Two graphs are added which represent the average peak acceleration with and without the TMD. Furthermore, the acceleration limit as specified in section 3.6 is added.

In Fig. 11, it can be seen that the average response for groups without TMD is higher compared to the average response for groups with the TMD. In contrary to this general observation, in some specific cases the results with the TMD are higher compared to the response without the TMD.

The average response for groups without TMD exceeds the acceleration limit for groups of five or more pedestrians. In one case, the response of a group of four pedestrians also reaches the acceleration limit. For the average response with
the TMD, unacceptable vibrations are observed for groups of six pedestrians. In one case unacceptable vibrations are also observed for a group of five pedestrians.

Fig. 11 Vertical response for sensor 1, experiment 2: controlled group walking with and without the TMD

The average accelerations in Fig. 11 are reproduced in Table 3. In addition, the effect of the TMD is presented, defined as the average peak acceleration without the TMD divided by the average peak acceleration with the TMD. Table 3 shows that the average effect of the TMD equals 1.3. This effect however depends strongly on the group size and varies between approximately 1.1 for a group of four pedestrians and 1.8 for groups of three pedestrians.

Table 3 Average vertical peak-peak acceleration for experiment 2: controlled group walking with and without the TMD

<table>
<thead>
<tr>
<th>Group size (n) [ped.]</th>
<th>Without TMD [mm/s²]</th>
<th>With TMD [mm/s²]</th>
<th>Effect TMD (= Without TMD / With TMD) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>590</td>
<td>322</td>
<td>1.83</td>
</tr>
<tr>
<td>4</td>
<td>638</td>
<td>595</td>
<td>1.07</td>
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<tr>
<td>5</td>
<td>739</td>
<td>659</td>
<td>1.12</td>
</tr>
<tr>
<td>6</td>
<td>824</td>
<td>719</td>
<td>1.15</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>1.29</td>
</tr>
</tbody>
</table>

5. Discussion

During the first experiment, the maximum number of pedestrians on the bridge at the same time was limited to 17. This corresponds to a density (d) of 0.11 ped./m². For a bridge on a train station, higher densities are expected on a regular basis. The dynamic performance of the bridge is assessed for higher densities by extrapolating the average response (p) and 95th percentile (q) of the response, as presented in Table 2. The results for p and q are extrapolated using a curve fit function derived for p, taken into account the results up to 13 pedestrians because of the limited data available for higher number of pedestrians. The curve fit function for p is given by:

$$ p(d) = 656e^{0.48d} \text{[mm/s}^2] $$

Eq. (6) shows that for p, the group effect is given by a factor $e^{0.48}$. This factor is close to the group factor $\sqrt{d}$ found in literature [1], [8]. In [1] for p as well as for q, a linear relation was found with respect to $\sqrt{d}$ based on numerical Monte Carlo simulations. As a result, the ratio $r = q / p$ is given by a constant. In [1], it was shown that the value of this constant depends on the bridge characteristics. For experiment 1, the ratio $r$ is given in Table 2. The value for q is obtained by multiplying p with the average factor for r. For up to 13 pedestrians, the average factor for r equals 2.05:
The graphical representation of \( p \) and \( q \) as well as the acceleration limit is shown in Fig. 12 together with measurement data. It is shown that the acceleration limit is exceeded by \( q \) and \( p \) for a pedestrian density of approximately more than 0.3 ped./m\(^2\) and 1.2 ped./m\(^2\) respectively.

The result for \( q \) is compared with the response calculated using the response spectra method according to the HiVoSS guideline [7]. This approach is also based on a 95\(^{th}\) percentile response calculation. For a pedestrian density of 0.3 ped./m\(^2\), a fundamental frequency (\( f_1 \)) of 2.09 Hz and a damping ratio (\( \zeta \)) of 0.017, the response equals 1270 mm/s\(^2\). This response is significantly greater than the response for a density of 0.3 ped./m\(^2\) shown in Fig. 12. This can be explained by the fact that the HiVoSS guideline is based on the assumption that the fundamental frequency of the bridge equals the average step frequency of the pedestrians. This is not the case for this specific bridge. According to [7], for a pedestrian density of 0.3 ped./m\(^2\), an average step frequency of 1.8 Hz is taken into account. This step frequency deviates from the fundamental frequency (2.09 Hz) of the bridge.

The results for the second experiment show that the TMD introduces an average reduction factor of approximately 1.3. This effect is considered significant given the small TMD dimensions and its limited mass. This effect however depends strongly on the group size. This phenomenon can be explained by the limited data available as well as the variation in dynamic force imposed by the group of pedestrians. Despite the assistance of a metronome, it is difficult to maintain a constant pace and a synchronous step pattern. Synchronisation becomes more difficult for increasing group size.

Due to its limited mass, the TMD is sensitive to errors. Therefore, a heavier TMD is required to study the effect for large groups of pedestrians. However, based on the effect observed for experiment 2, a TMD is considered as a valuable solution to reduce the dynamic response. In this case, a TMD is specifically interesting because the excessive vibrations are caused by a single mode of vibration. Therefore, a single TMD in the middle of the bridge installed inside the box girder can be a practical solution. A permanent damper would require a traditional design with springs and viscous dampers. Furthermore, the mass of the TMD should be increased to provide sufficient damping for all design scenarios.

6. Conclusion

Two different experiments are performed to and analysed to assess the dynamic behaviour of the Rotterdam Central Station footbridge and to study the effect of a TMD. Based on the results presented in this paper, it can be concluded that the vibration limit is exceeded in the following scenarios:
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- A pedestrian density exceeding 0.3 ped./m²;
- A single pedestrian jumping in the middle of the bridge at a resonant frequency of 2.1 jumps per second;
- A group of five or more pedestrians walking over the bridge simultaneously at a resonant frequency of 2.1 steps per second.

The latter case can be seen as exceptional and therefore not relevant. However, the first two scenarios are realistic load conditions which occur frequently. The risk for unacceptable vibrations will increase considering the expected increase in the number of pedestrians in the future. The results for experiment 2 show that a TMD can provide a solution to reduce the dynamic response. Based on the provisional TMD with a mass of 105 kg, an average reduction factor of 1.3 is observed for groups walking at predefined step frequencies. Additional TMD mass is required for a permanent solution to increase damping and to make the system less sensitive to errors.

7. References


