HUMAN INDUCED VIBRATIONS OF ALUMINIUM PEDESTRIAN BRIDGES

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Abstract: This paper reviews the modeling of human induced vibrations of pedestrian bridges. In particular, the load models of Bachmann and Seiler/Hüttner are discussed and compared to others. Measurements of human induced vibrations were performed on an aluminium bridge, which was excited in the second harmonic of the load function. When varying the step frequency, a sharp resonance peak can be observed at half of the eigenfrequency of the bridge. The results obtained numerically by a finite element analysis are in close agreement with the measured values if loading is modeled as a series of load time histories measured for a single footstep. However, the models of Bachmann and especially of Seiler/Hüttner are found to possess values too large for the Fourier coefficient of the second harmonic. In cases where the second harmonic of the load function is predominant, a realistic modeling of its size is highly important. Recommendations are given for the second Fourier term of the load, in order to obtain results which are in accordance with the measurements on real bridges.

Keywords: Human induced vibrations, Pedestrian bridges, Field tests, Computer simulations, Structural dynamics

1 INTRODUCTION

Aluminium bridges are beneficial due to their light weight and their low maintenance requirements. Aluminium is especially suited for prefabricated pedestrian bridges. They are fabricated with spans of 12 m to 45 m in a plant and can be economically transported over distances up to several hundred kilometres at the construction site. Whereas aluminium has a bearing capacity similar to steel, its stiffness is considerably lower. The Young's modulus of aluminium is only 1/3 of that of steel. Therefore aluminium pedestrian bridges are prone to the excitation of vibrations induced by humans passing over the bridge.

In order to assure the serviceability of footbridges, a dynamic analysis should be part of the design process at an early stage. The fundamentals of the excitation and the analytical methods have been known since the 80's of the 20th century. However in many countries codes including serviceability requirements considering vibrations in the design of bridges were introduced only in the last decade. The guidelines and requirements among different national or international codes are not comprehensive and indicate the need for further research. In some cases even new bridges have suffered excessive vibrations under dynamic loads induced by pedestrians, requiring later major retrofitting measurements to ensure their serviceability.

In order to assess the vibrations of aluminium bridges, extensive numerical and experimental investigations have been performed on a pedestrian bridge (Werkle et al., 2009). Tests have been undertaken on the bridge in the plant as well as on the site where the bridge was later built.

2 LOAD MODELS

Humans may excite bridge vibrations by different actions, such as walking, running, jumping and vandalism. The loading of the bridge can be described by load models. Here it is presumed that there is no interaction between the load and the movement of the bridge, also called “lock-in effect”. A review of different models is given by Zivanovic, 2005 and Butz et. al., 2008.

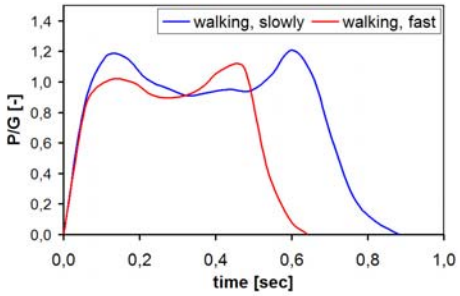

Figure 1. Time histories of the vertical load for a single footstep (after Wheeler 1982)

The walking of a single person excites forces on the ground in the vertical as well as in the horizontal direction. In this paper only vertical excitations are considered. Fig. 1 shows the force time history of a single footstep. The steps of the left and the right foot can be added up to a load time history for walking. This results in a single load *F(t)* varying in time and propagating with the velocity *c* as

$$c = f_S \cdot l_S \tag{1}$$

where f_s denotes the step frequency and l_S the step length. The step length depends on the step frequency and can be assumed to be 0.75 m in the frequency range of about 2.0 Hz considered here. The load time history is expressed by a Fourier series as

$$F(t) = G \cdot (1 + \sum_{j=1}^{4} \alpha_j \cdot \sin(2 \cdot \pi \cdot j \cdot f_S \cdot t - \varphi_j)) \quad , (2)$$

where *G* denotes the weight of the person and f_s the step frequency. The Fourier coefficients α_j, φ_j have been determined by different authors as given in Table 1. Frequency dependent coefficients have also been given by Rainer et al., 1988 (Figure 2). Typical step frequencies for walking are between 1.7 and 2.3 Hz. In a dynamic analysis the step frequency has to be assumed in an unfavourable fashion, i.e. in resonance with the structure, if the structure possesses an eigenfrequency in that range.

Table 1. Fourier coefficients for walking

Author	α_j
Bachmann, 1988, 1997	$\alpha_1 = 0.4 - 0.5$ $\alpha_2 = 0.1$ $\alpha_3 = 0.1$
Seiler et al, 2004	$\alpha_1 = 0.4$ $\alpha_2 = 0.15$ $\alpha_3 = 0.1$
Young, 2001 (after Zivanovic, 2005)	50% exceedance probability: $\alpha_1 = 0.37 \cdot (f_S - 0.95) \leq 0.5$ $\alpha_2 = 0.054 + 0.0044 \cdot f_S$ $\alpha_3 = 0.026 + 0.0050 \cdot f_S$ $\alpha_4 = 0.010 + 0.0051 \cdot f_S$ 25% exceedance probability: $\alpha_1 = 0.41 \cdot (f_S - 0.95) \leq 0.56 \quad for \quad f_S = 1 \; to \; 2.8 \; Hz$ $\alpha_2 = 0.069 + 0.0056 \cdot f_S \quad for \quad f_S = 2 \; to \; 5.6 \; Hz$ $\alpha_3 = 0.033 + 0.0064 \cdot f_S \quad for \quad f_S = 3 \; to \; 8.4 \; Hz$ $\alpha_4 = 0.013 + 0.0065 \cdot f_S \quad for \quad f_S = 4 \; to \; 11.2 \; Hz$

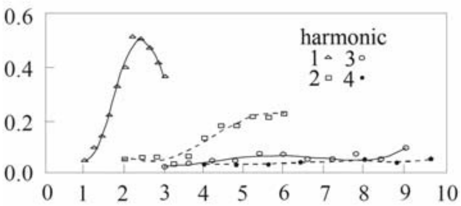

Figure 2. Fourier coefficients after Rainer et. al., 1988

3 STRUCTURAL MODEL

The bridge investigated is a through truss bridge with a span of 29.60 m and a width of 2.6 m (Fig. 2). It possesses a slight curvature in the vertical direction. In the horizontal direction the bridge is stiffened by a horizontal truss under the paving slab and the slab itself.

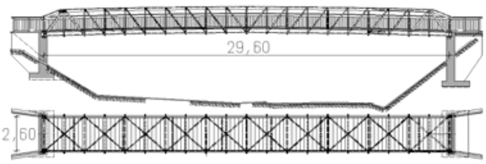

Figure 3. Aluminium bridge investigated

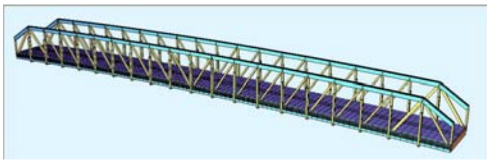

Figure 4. Finite element model

4 COMPUTATIONAL SIMULATION

The computational simulation of the bridge has been performed using a very detailed finite element model with 2060 nodes and 1750 elements, Figure 4 (Püschel, 2006). The first eigenfrequency has been determined to be f_1=4.0 Hz. The corresponding mode shape is shown in Figure 5a. All other frequencies are above 5 Hz and hence not relevant for the vibration excitation by pedestrians.

(a) f_1 = 4.0 Hz

(b) f_2 = 13.0 Hz

Figure 5. Mode shapes for vertical vibrations

When solving the finite element equations

$$\underline{K} \cdot \underline{u}(t) + \underline{C} \cdot \underline{\dot{u}}(t) + \underline{M} \cdot \underline{\ddot{u}}(t) = \underline{F}(t) \qquad (3)$$

the load vector $\underline{F}(t)$ varies in time and space (Werkle, 2007).

The computational simulation of the vibrations induced by pedestrians has been performed using different load models:

- Single footsteps with load time histories acc. to Figure 1 (Wheeler 1982),
- load model of Bachmann (Bachmann, 1988 and 1997),
- load model of Seiler/Hüttner (Seiler et. al., 2004).

Time histories of the accelerations have been obtained by a modal analysis with 25 modes and a modal damping of 1% in the first mode and 0.5% in the higher modes. The damping values have been derived from field tests of the bridge with impulse loading.

Figure 6 shows the accelerations in the middle of the bridge for a step frequency of 2.0 Hz when the load is applied by single footsteps. The maximum vertical acceleration obtained is 0.5 m/s².

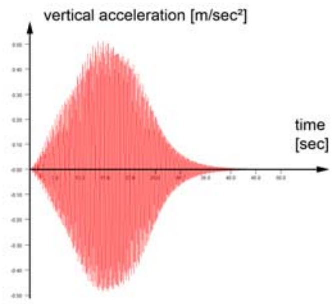

Figure 6. Computed vertical acceleration time history, f_s=2.0Hz, G=0.8 kN

For load models with a continuously propagating load, the maximum accelerations in the middle of the bridge are given in Figure 7, independent of the step frequency. They show a pronounced resonance peak at half of the first eigenfrequency of the structure. At the same frequency the footstep model with time histories according to Wheeler (Wheeler 1982) shows a considerably lower acceleration.

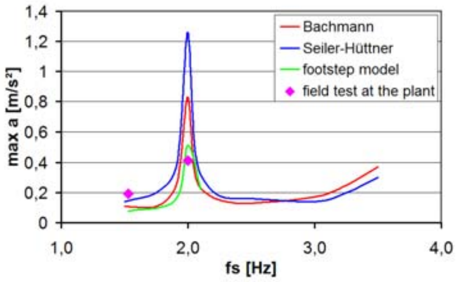

Figure 7. Maxium acceleration vs. step frequency, G=0.80 kN

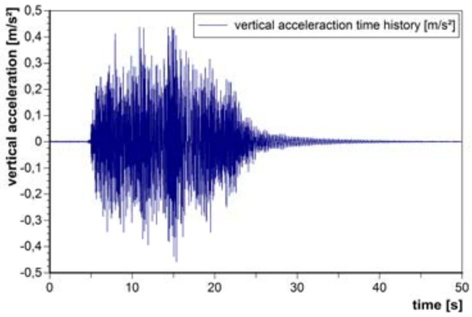

Figure 8. Measured vertical acceleration time history, f_s=2.0Hz, G=0.95 kN

5 FIELD TESTS

Tests of the bridge have been performed at the plant. A second series of tests has been undertaken at the finished bridge in its final location.

The natural frequencies of the bridge have been derived from the FFT spectrum of the bridge response due to impulse excitation as well as by ambient vibration measurements. Values of the first eigenfrequency are given in Table 2. The eigenfrequency of the FE model agrees closely with the frequency measured at the plant. However there is a significant difference between the eigenfrequencies measured at the plant and at the site. It originates from an arc effect, resulting from the horizontal fixing of the bearings of the bridge when the bridge was built in at its final location.

Table 2. First natural frequency of the bridge

Computational simulation	3.98 Hz
Measurement at the plant	3.85 Hz
Measurement at the final location	4.50 Hz

Field tests with a person passing the bridge have been performed with different step frequencies. Figure 8 shows a typical acceleration time history as measured at the plant with a step frequency f_s*=2.0Hz*. The maximum acceleration compares closely with the single-footstep model, (see Figure 7).

Figure 9. Field tests

Field tests with different step frequencies have been performed at the bridge at the final location (Figure 9). The maximum acceleration is strongly dependent on the step velocity, see Figure 10.

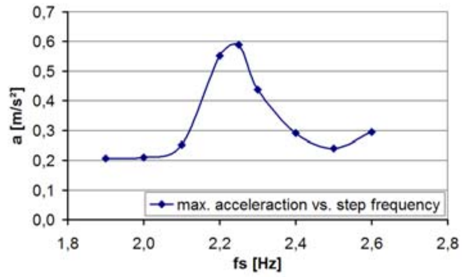

Figure 10. Maxium acceleration vs. step frequency, G=0.80

The shift of the peak from 2.20 to 2.25 Hz results from the different natural frequencies in the plant and at the site, due to the different bearing conditions of the structure. The peak accelerations are about twice the values measured at the plant because here the step frequency at the peak (f_s=2.25Hz) matches better with half of the eigenfrequency of the structure. However it should be noted that the resonance peak is very sharp. The difference between two measuring points of the step frequencies at the peak is 2.25-2.00=0.05 Hz, which corresponds to a difference in the time period between two steps of 0.01 s. To get the peak acceleration value presumes that the bridge is passed exactly with that frequency. A slight change of e.g. 0.01s in the time period between two steps reduces the acceleration by half. In the tests, special measurements were necessary in order to keep the step frequency constant at a predefined value. In practice these conditions are not given and hence it is very unlikely to obtain the peak values of the accelerations measured.

6 ASSESSMENT OF THE RESULTS

In the present study the first eigenfrequency of the structure is about twice the typical step frequency of 2.0 Hz. Therefore, in Eq. (2) the second Fourier term with j=2 is significant for the structural response. Table 3 summarizes the Fourier coefficients α_2 obtained by different methods for f_s=2.0 and 2.25 Hz, respectively.

Table 3. Second Fourier coefficient α_2

Numerical Simulation:	
- Bachmann, 1988, 1997	0.10
- Seiler et al, 2004	0.15
- Young, 2001 / 50% exceed. prob.	0.064
- Young, 2001 / 25% exceed. prob.	0.082
- Rainer et. al., 1988	0.065
Field tests:	
- Measurement at the plant	0.06
- Measurement at the final location	0.10

The Fourier coefficient varies between 0.06 and 0.15 according to the model chosen. The results of the tests presented here indicate that a Fourier coefficient $\alpha_2 \approx 0.06 - 0.07$ will be an appropriate choice. This takes into account that the probability of reaching the resonance peak is low for naturally passing persons. Here it is presumed that the lock-in effect can be excluded. The sharpness of the resonance peak (Figures 7 and 9) is related to the damping of the system. Therefore also a small damping ratio as in the case of aluminium structures is pressumed.

This paper deals with the loading of the bridge by a single person. For the design of bridges the vibrations due to several persons have to be considered. Limit values for the admissible maximum accelerations have to be observed (Werkle et. al. 2009). In cases where these limits cannot be kept, Tuned Mass Dampers are an appropriate means to reduce the vibrations.

7 CONCLUSIONS

The Fourier coefficients for the induced load given in the literature differ in a wide range. When comparing measured accelerations on pedestrian bridges and values obtained by numerical modeling often remarkable differences are reported. Accelerations measured in tests are often lower than as predicted by the computational model. This is true in the bridge investigated here too. One reason can be found in the difference between the natural walking load varying slightly in frequency and the loading in the model which possesses a very constant frequency, causing pronounced resonance effects. Therefore in a numerical model the Fourier terms for the loading should be chosen carefully in order not to overestimate the bridge vibrations.

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