Dynamic Investigation of a Suspension Timber Footbridge

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Abstract

Presented are results of a dynamic numerical analysis of a suspension timber footbridge of approximately 78–m span over the river Piracicaba in the city of Piracicaba, Brazil. The study of the effects caused by human excitation on structures has gained a significant evolution during the last few years. The increased understanding that followed the first studies is difficult to codify and, therefore, is not yet clearly stated in regulatory guidance on dynamic design of pedestrian bridges. In the course of this study, which involved the dynamic analysis of a numerical model of the three–dimensional structure developed using the SAP2000 software, vibration modes of the deck, towers and cables were determined. The dynamic behavior of suspension footbridges under walking dynamic loads was simulated by resonant vibration caused by synchronous excitations. After completing this study, guidelines for vibrations performance are suggested, focusing on the definition of the pedestrian load, frequency ranges of interest, criteria that can evaluate dynamic behavior and human comfort on footbridges with acceptable limits of vibration. Established measurement criteria allow designers to calculate new structures, and also evaluate the need of repairs of existing ones.

Keywords: vertical and lateral vibrations, human comfort, numerical analysis.

Introduction

Timber and timber–based materials have been successfully used in pedestrian or low-traffic short to medium span bridges. However, the low density and modulus of elasticity of these materials may result in excessive vibration caused by dynamic loading, particularly in the case of synchronized action of pedestrians. The vibration serviceability limit state (SLS) criteria stated in guidelines consists of limiting the maximum value of the acceleration in the deck, so that fatigue risk in structural elements is minimized and an appropriate level of comfort may be ensured. This study compares the values provided by the simplified method with those calculated from a detailed step–by–step dynamic analysis numeric. References to prescriptions of other standards on this topic are made whenever relevant. Prior to proceeding with the step-by-step analysis, the generation of dynamic loadings corresponding to the crossing of only one or a stream of pedestrians had to be undertaken. The procedure used for this purpose is described in section following.

Human–induced Vibrations

During walking on a structure, pedestrians induce dynamic time varying forces on the surface of the structure. Several studies have been performed in order to quantify pedestrian walking forces. These forces have components in all three directions, vertical, lateral and longitudinal and they depend on parameters such as pacing frequency, walking speed and step length, Wheeler (1982).

The typical pacing frequency for walking is around 2 steps per second, which gives a vertical forcing frequency of 2 Hz. This has been confirmed with several experiments, for example by Matsumoto et al. ((1972) e (1978)) who investigated a sample of 505 persons. He concluded that the pacing frequencies followed a normal distribution with a mean of 2,0 Hz and a standard deviation of 0,173 Hz. Slow walking is in the region of 1,4–1,7 Hz and fast walking in the range of 2,2–2,4 Hz. This means that the total range of vertical forcing frequency is 1,4–2,4 Hz with a rough mean of 2 Hz. Since the lateral component of the force is applied at half the footfall frequency, the lateral forcing frequencies are in the region of 0,7–1,2 Hz. Many footbridges have natural vertical and lateral frequencies within the limits mentioned above (1,4–2,4 Hz vertical and 0,7–1,2 Hz lateral), Bachmann et al. (1995).

Fujino et al. (1993), when walking on a structure, pedestrians produce lateral dynamic forces on the surface of the structure. Although it is widely known that people walk with a frequency of about 2 Hz, it is not commonly known but about 10% of the vertical loading works laterally when people walk. These forces are a consequence of a lateral oscillation of the gravity center of the body and the lateral oscillations are a consequence of body movements when persons step with their right and left foot in turn. The amplitudes of these lateral oscillations are, in general, of about 1–2 cm. It should be noted that the lateral loading parameters are not well quantified. Few measurements of the magnitude of lateral loading due to walking have been made and, in addition, they have almost all been made on unmoving surfaces.

Pedestrian–Induced Actions and Relevant Model

Pedestrian induced loads may be due to walking, running, or jumping, the so–called vandalism loading. Each of these types of loading produces a different loading curve over time as well as frequencies in which the oscillations can occur. The vertical load component is larger than the lateral components, but the lateral and longitudinal components can also cause vibrations problems of slender bridges especially if a pedestrian–bridge–interaction develops. The loading due to walking and running is represented by the summation term, which is a Fourier series with coefficients at the discrete frequencies. The forcing function due to a person's rhythmical body motion can be mathematical described by a Fourier series of the form:

$$F_s(t) = G + \sum_{i=1}^{n} G \cdot \alpha_i \cdot \sin(2\pi i f_s t - \phi_i)$$

where $G =$ dead load of the pedestrian $(800\ \mathrm{N})$, $\alpha_i =$ Fourier coefficient of the i–th harmonic, $f_s =$ activity rate (Hz), $t =$ time (s), $\phi_i =$ phase angle of the i–th harmonic, $i =$ number of the i–th harmonic and $n =$ total number of contributing harmonic.
In Table 1 pertinent values for the Fourier coefficients and phase angle proposed by Bachmann et al. (1995) to the representative type of activity. Bachmann e Ammann (1987) pointed out that dynamic pavement load is dominated by the pacing frequency, see Table 2.

Table 1 – Fourier coefficient and phase lag considering three harmonics and directions vertical, Bachmann et al. (1995)

Representative type of activity	Activity rate (Hz)		α_1	α_2	ϕ_2	α_3	ϕ_3
"walking"	vertical	2.0	0.4	0.1	$\pi/2$	0.1	$\pi/2$
"running"	vertical	2.0 to 3.0	1.6	0.7	-	0.2	-

Table 2 – Typical values for step frequency, velocity and step length, Bachmann e Ammann (1987)

	$f_s\ (\mathrm{Hz})$	$v_s\ (\mathrm{m/s})$	$l_s\ (\mathrm{m})$
slow walking	1.7	1.0	0.60
normal walking	2.0	1.5	0.75
fast walking	2.3	2.3	1.00
normal running	2.5	3.1	1.25
fast running	> 3.2	5.5	1.75

Footbridge Structural Details

The Piracicaba footbridge, see Figure 1, provides access to Museum in Piracicaba, São Paulo, Brazil, across the upper reaches of the River Piracicaba. The suspended deck is 78–m long, see Figure 2. There are side spans and the equal cable back stays are straight with anchorages 21–m from the towers. The wooden deck provides a footway 400–cm wide. Longitudinal bending strength is provided by two beams of longitudinal stiffness truss of 150–cm height. The wooden deck is made up of two layer of boards 200–mm x 25–mm crossed perpendicular, forming an angle of 45º and supported on longitudinal by two beams of stiffness truss. At every 150–cm there is also truss forming rigid connections between the longitudinal beams, see Figure 3. The wooden deck is fixed in position at the end and is free to move under expansion and live loading at the other. The towers comprise twin welded boxes steel (Young's modulus 210.000 MPa and density 7.850 kg/m^3), 15–m high and 419–mm x 380–mm section, with contravener. At the tower top curved machined blocks form the saddles where friction forces are sufficient to prevent cable slippage. The wooden deck is connected to the cables by hangers at 3–m intervals where crossbeams are also located. The hangers are 25.40–mm diameter 1020 steel (Young's modulus 210.000 MPa and density 8.030 kg/m^3), screwed at each end and pinned and clamped to the cable using high strength friction grip bolts. The suspension is composed by four Filler 19 x 25 steel (Young's modulus 205.000 MPa and density 7.850 kg/m^3) cables and each main cable consists of a 44.45–mm diameter galvanized locked coil wire rope with a sag of 12–m. Eucalyptus citriodora wood (Young's modulus 18.421 MPa and density 999 kg/m^3) is chosen for the beams of longitudinal stiffness truss, transverse and contravener. Eucalyptus tereticornis wood (Young's modulus 17.198 MPa and density 899 kg/m^3) is chosen for the wooden deck.

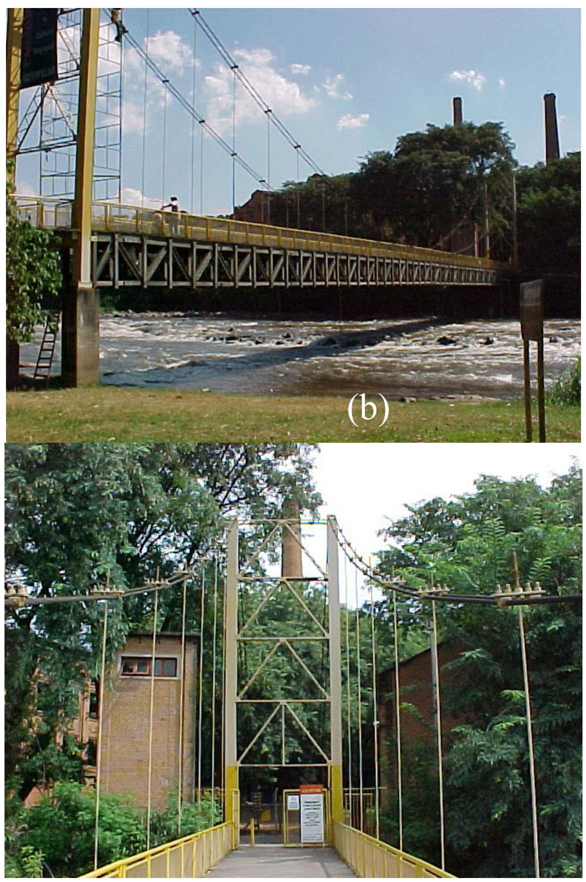

Figure 1 – Footbridge of Piracicaba (a) lateral view e (b) tower

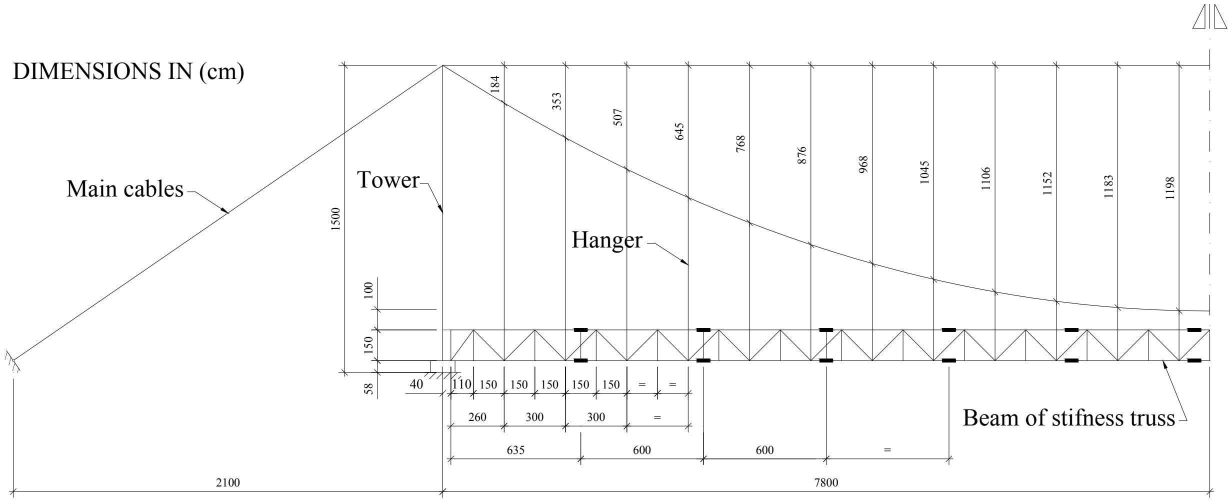

Figure 2 – Schematic diagram of footbridge of Piracicaba, Carlito et al. (1993)

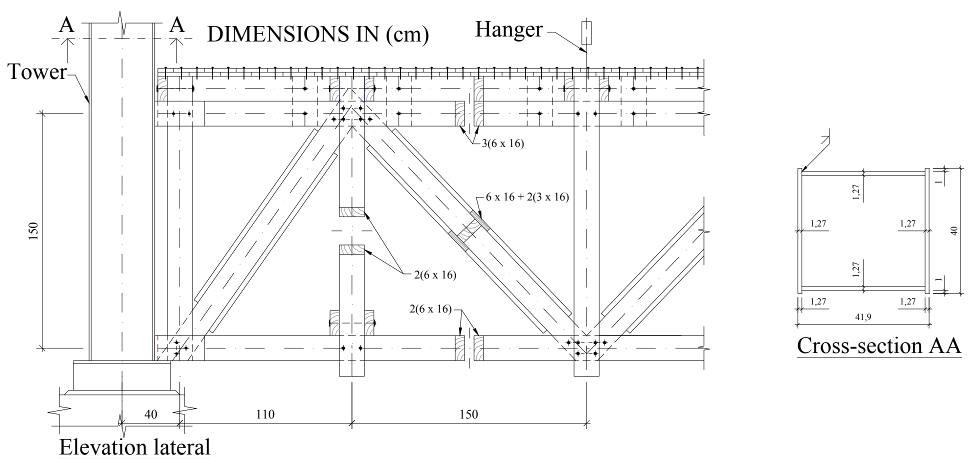

Figure 3 – Footbridge structural details – Elevation lateral, Carlito et al. (1993)

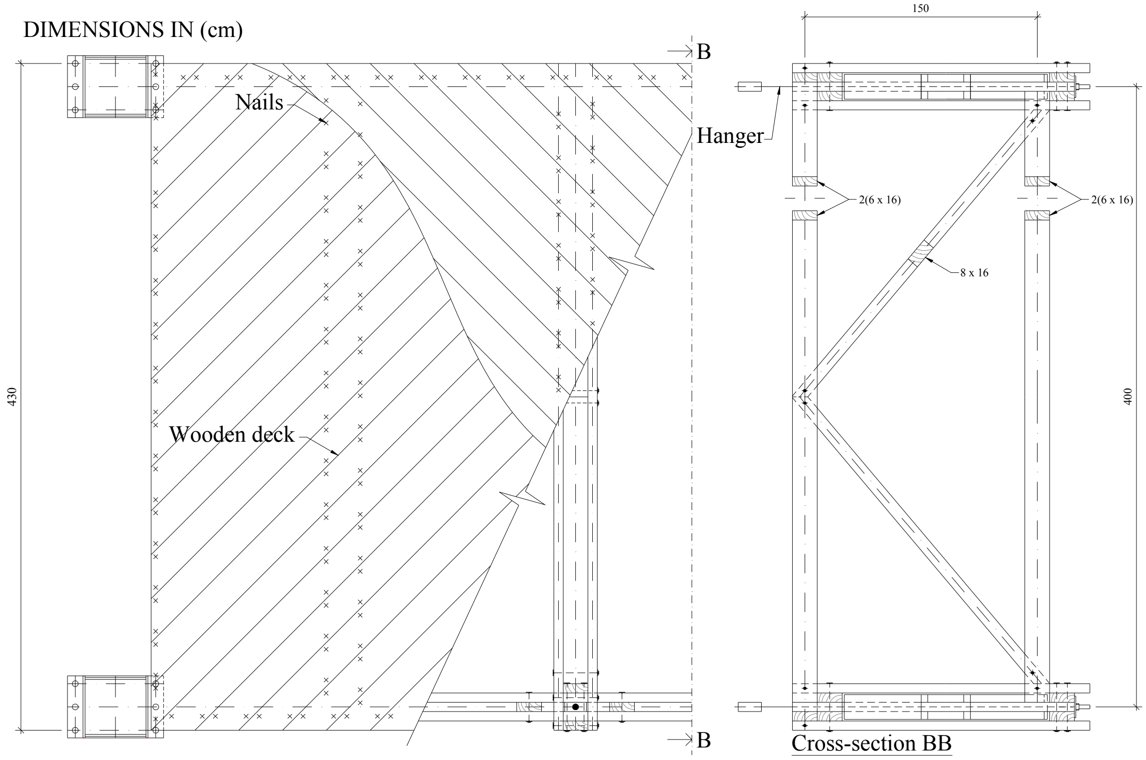

Figure 4 – Footbridge structural details – Plan view, Carlito et al. (1993)

Finite Element (FE) Modelling and Characteristic of the Footbridge

A complete 3D Finite Element (FE) model for the Piracicaba footbridge was developed using the structural analysis package SAP2000 and adopted to carry out numerical analyses, see Figure 5. With advances in numerical modelling, it is often expected that FE models based on technical design data and best engineering judgement can reliably simulate both the static and dynamic behaviour of the bridge. The aim was to construct a detailed model which would be able to simulate the dynamic behaviour of the structure as well as possible. This was based on the limited technical data available and best engineering judgement.

The key finite element modelling assumptions were as follows: The wooden deck units are assumed to be simply supported by the supporting beams and are modelled as shell elements with end releases: the two ends are supposed to have the same pin connections to the supporting beams and can carry torques and axial forces in order to keep the structure symmetric about the bridge centre line. The beams of longitudinal stiffness truss of the footbridge were modelled using 3D beam elements with corresponding bending, shear and axial stiffness and distributed mass. The anchorage and suspension cables were modelled using 3D beam elements by suitably restrained with properly computed characteristics. The steel box towers were modelled using 3D beam as fully fixed considering solid rock foundations. The handrail s weren't modelled in design process.

A beam element has two end nodes and each end node has six degrees of freedom: three translations along the local axes and three rotations about its axes. The deflection of the structural model is governed by the displacements of the joints, and different connections, supports and boundary conditions are simulated by applying corresponding end releases and joint restraints. To simulate the flexible behaviour of cables, each cable element is divided into 20 segments. The material properties are the same as those of footbridge frames and cables.

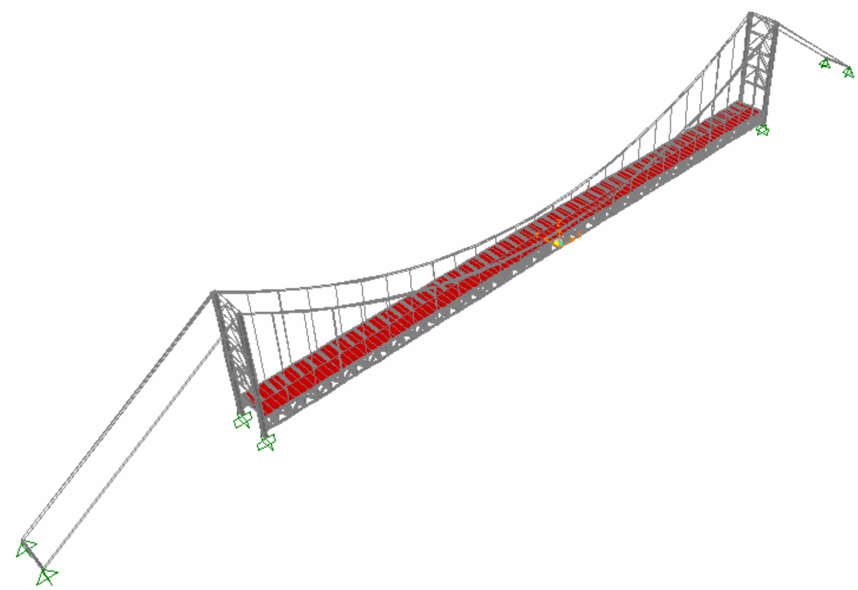

Figure 5 – Finite element model

Limit Values for Acceleration According to International Standards

Limit values for acceleration in the international codes are directly linked to pedestrian comfort. Due to the plethora of studies on such a subjective matter as pedestrian comfort, there are many different acceleration limits in the international codes. An overview of the acceleration limits in the codes and relevant literature is provided in Figure 6. Some of these limit accelerations are dependence on the natural frequency while others are constant for the whole range of pedestrian induced loading frequencies.

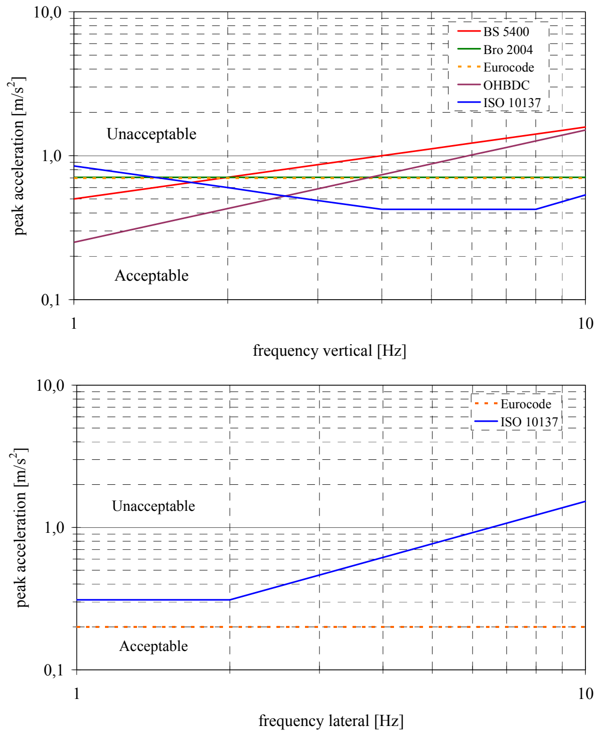

Figure 6 – Acceptability of vertical and lateral vibration

Results and discussion

At this stage the comparison is made between the maximum acceleration values obtained from the load numeric model presented before and those show by the Eurocode (1993), the BS 5400 (1978), the OHBDC (1991), the ISO 10137 (1992) and the Bro (2004). Following the rules commended by recent standards the load model can be applied in three different loading situations concerning pedestrian activity, namely one single pedestrian, one group of 10 pedestrians and finally a constant stream of pedestrians crossing the bridge with a pre–established density over the deck. But, this paper was applied loading situations for one single pedestrian. The given values refer to maximum acceleration situations, so that the resonance of one harmonic of the load, the first or the second, with the first eigenfrequency is the main loading situation. However, pedestrian bridges are mostly used by “normal walking” people at pace rates of about 2 Hz and “normal running” people at pace rates of about 2,5 Hz. The results from the one–span deck loaded with one pedestrian crossing the bridge with a pace rate can be coincident with the eigenfrequency, gives a substantially lower maximum acceleration than the one computed. However, the value obtained from this code for the effect of a continuous stream of pedestrians acting in resonance with the structure compares well with that one obtained from the load model. The first antisymmetric and first symmetric theoretical modes of vibration in the direction vertical are plotted in Figures 7 and 8.

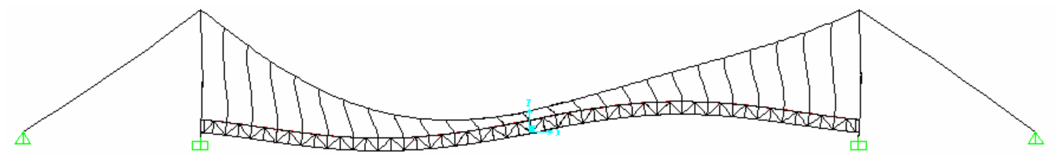

Figure 7 – First simulated antisymmetric modes in the direction vertical – f = 2,0411 Hz

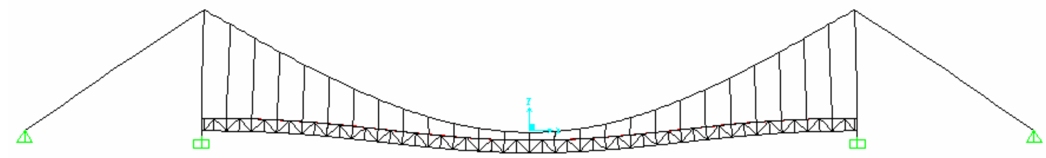

Figure 8 – First simulated symmetric modes in the direction vertical – f = 2,2646 Hz

The simulated modes of vibration in the direction lateral were not easily excited and therefore not a critical concern. The response to a moving pedestrian was simulated first using model normal walking and second using model normal running. The following peak acceleration values (m/s^2) due to the passage of a pedestrian normal walking and normal running were showed in Figure 9.

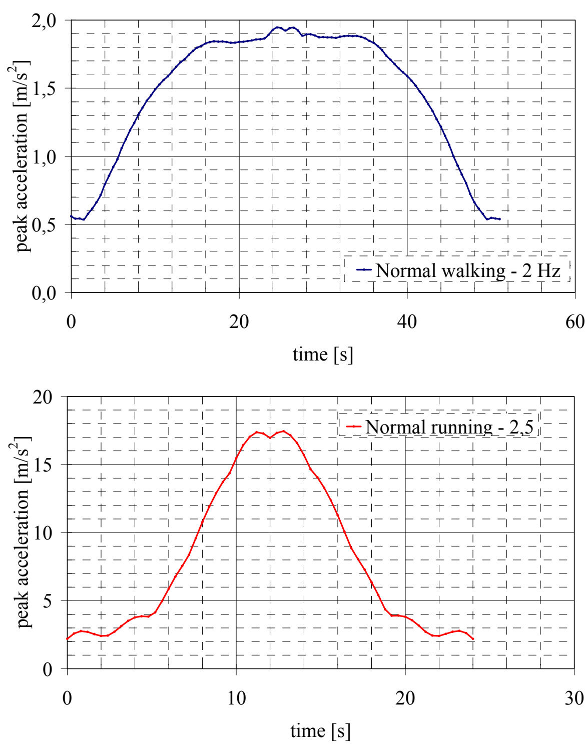

Figure 9 – Peak acceleration values due to the passage of a pedestrian normal walking and normal running

Conclusions

Natural frequencies which are in range coinciding with frequencies typical for human–induced dynamic loading can be avoided by increasing structural stiffness. Increasing stiffness can be an expensive measurement and will almost always have negative effects on the aesthetics of the structure. This, together with the coincidence of the structure's main natural frequencies and the range of frequencies for maximum load from footfalls, makes it susceptible to vibrations induced by pedestrians. This is good for testing but undesirable for other users (and in extreme cases structural integrity). Existing vibration limits presented in standards aren't most probably sufficient to prevent vertical synchronisation between structure and pedestrians. However, observations indicate that lateral synchronisation can't start when the amplitude of the footbridge vibration is only a few millimetres.

Pedestrian induced vibrations are a subject of serviceability. It was therefore assumed, that structures respond linearly to applied loads and that dynamic response can be found by solving the numerical. The loads models proposed by the above mentioned standards are all based on the assumptions that pedestrian loads can be approximated as periodic loads. Dynamic analysis indicated that the footbridge is sensitive to large vertical vibrations due to pedestrian normal walking and normal running, and this resulted in accelerations that exceeded the criteria proposed by the standards. Large bridge structures and lower damping ratios result in higher response to dynamic actions.

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