



EXPERIMENTAL DYNAMIC EVALUATION OF THE GUARDA FOOTBRIDGE

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Abstract: This paper deals with the methodologies used for footbridges design and their application to a steel footbridge built over a road junction in Guarda, Portugal. The complexity of the bridge behavior and the high uncertainty connected with the damping and natural frequencies estimation supported the decision to carry out vibration measurements included in the commissioning tests. A modal identification was performed and a FE model of the bridge was updated accordingly. Tests with pedestrians were also performed in order to gain as much information as possible towards the final decision to be made concerning the installation or not of the foreseen tuned mass dampers. Tests included continuous streaming of pedestrian groups with increasing size crossing the bridge in synchronized walking at various pace rates.

1. INTRODUCTION

The need for providing convenient and safe pedestrian or light traffic load crossings over highways and railways has led to an increasing number of lightweight overpasses. Most of those structures are made of steel elements or are mixed structures. Whatever the system to be used, however, the lightweight material and the high structural stiffness may raise some concerns towards the dynamic response and vibration amplitude of such bridges in both vertical and transversal directions, namely under the action of synchronous pedestrian excitation.

The current practice of footbridge design regarding dynamics usually includes a preliminary calculation of the dynamic response of the bridge to the various types of pedestrian induced dynamic loads, according to the sensitivity of the location and the possible occupation density. In case of doubt, provisions for tuned mass dampers can be made at the design stage and the final decision about their installation is postponed to the end of the construction when measurements of the real dynamic parameters can be carried out.

This was the methodology followed by the designers of a steel cable stayed footbridge built over a road junction in Guarda, Portugal. The complexity of the bridge behaviour and the high uncertainty connected with the damping and natural frequencies estimation supported the decision to carry out such measurements.

The modal identification using the accelerations measured during ambient vibration and during free vibration induced by initial imposed displacements allowed the dynamic characte-

rization of the structure in terms of mode shapes, natural frequencies and modal damping in the frequency range of interest, that is, up to about 5Hz. A FE model of the bridge was updated according to the measurement results. Tests with pedestrians were also performed in order to gain as much information as possible.

2. 'SÉTRA' METHODOLOGY FOR FOOTBRIDGE DESIGN

Design methodologies (Ramos *et al.*, 2007, Sétra/AFCG, 2006) have been recently proposed for footbridges that can be used by designers and owners in order to define whether a dynamic analysis is needed and the respective dynamic loads to be considered during the design.

One of these methodologies was developed by the French authority for bridges, "SÉTRA – Service d'Études techniques des routes et autoroutes" (Sétra/AFCG, 2006) and is summarized in the flowchart of Figure 1. It relies on the initial classification of the bridge depending on the expected pedestrian traffic and on the comfort level that must be assured for that traffic.

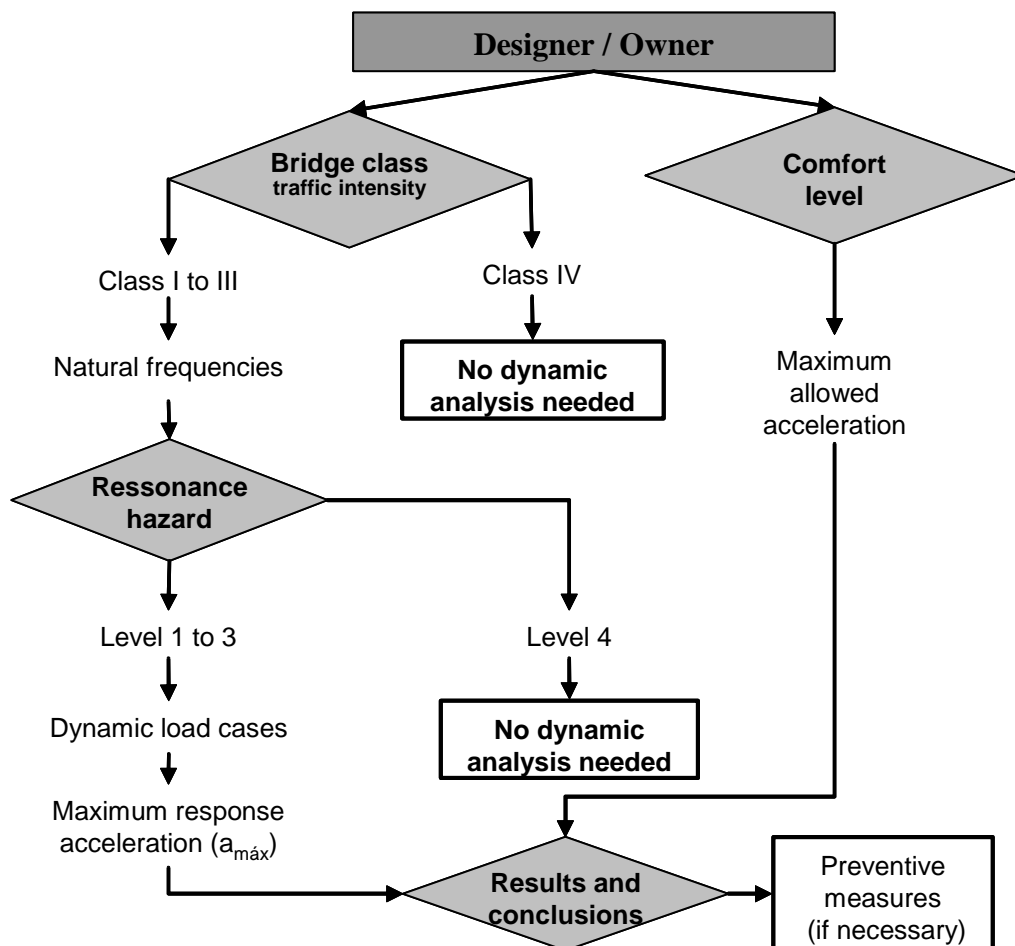


Figure 1: Flowchart for the methodology 'SÉTRA' adapted from Sétra/AFCG (2006)

The classification of the bridges is made according to the type and intensity of the pedestrian traffic and, therefore, strongly conditioned by the geographic localization (see Table 1). The criteria include localization in urban or non urban areas and the expected traffic density. Classification as Class IV implies that the bridge will not be checked concerning its dynamic behaviour. In case of slender structures, Class III should be assumed in order to have, at least,

results from the modal analysis to support the decision concerning the dynamic response analysis.

Table 1: Footbridge classes

Class I	Footbridges linking urban areas where very high density traffic is expected or located nearby infrastructures where high concentration of people is possible (e.g. transportation modal interfaces or sport arenas)
Class II	Footbridges linking urban areas where high density traffic is expected to fulfil the entire bridge deck even if during short periods of time
Class III	Footbridges for normal use which may occasionally be crossed by large groups of people, although there is a low probability that those groups are able to fulfil the entire deck.
Class IV	Footbridges outside urban areas usually used by single pedestrians or, seldom, by small groups of people (e.g. pedestrian bridges over railways or highways)

Concerning the resonance hazard four levels are considered, from Hazard Level 1 corresponding to the maximum probability of occurrence of resonance to Hazard Level 4 corresponding to the lowest probability not requiring dynamic computations. The probability distribution can be visualized in tables 2 and 3 for, respectively, vertical and horizontal vibrations. To compute the natural frequencies for bridges of classes 1 to 3 a modal analysis must be performed considering two situations for the masses: i) only the mass of the bridge is accounted for; ii) the mass of one pedestrian per square meter (70 kg/m²) is added to the mass of the bridge.

Table 2 Resonance hazard levels for vertical vibrations

Frequency (Hz)	0,0	1,0	1,7	2,1	2,6	5,0
Level 1			Maximum			
Level 2		Medium		Medium		
Level 3					Low	
Level 4	Negligible					Negligible

Table 3 Resonance hazard levels for transversal vibrations

Frequency (Hz)	0,0	0,3	0,5	1,1	1,3	2,5
Level 1			Maximum			
Level 2		Medium		Medium		
Level 3					Low	
Level 4	Negligible					Negligible

When a dynamic calculation is necessary, the load cases are defined according to the bridge class and the resonance hazard class as shown in Table 4. Three load cases are defined corresponding to low density traffic, Case 1, high density traffic, Case 2 and second harmonic effect, Case 3. The generic formulation of the load function is

$$F(t) = F_{p,i} \times \cos(2\pi f_i t) \times N_{eq} \times \psi \quad (1)$$

and must be established separately for each direction, vertical and horizontal transversal to the bridge axis. The relevant load harmonic amplitude, $F_{p,i}$, the equivalent number of pedestrians, N_{eq} , and the resonance probability ψ , can be obtained from (Ramos *et al.*, 2007).

Table 4: Identification of the load cases for dynamic analysis

Bridge class	Resonance hazard level		
	1	2	3
I	Case 2	Case 2	Case 3
II	Case 1	Case 1	Case 3
III	Case 1	-----	-----

Comfort levels are established according to acceptable or not acceptable horizontal or vertical acceleration values. Four comfort levels are defined for each direction. The lower level, Level 1, corresponds to the maximum comfort, that is, vibrations are almost imperceptible for the pedestrians. The upper level, Level 4, corresponds to unacceptable vibration amplitudes. To avoid the lock-in effect for horizontal vibrations, that is, the synchronization of groups of pedestrians with the lateral movement of the bridge, the horizontal acceleration should be limited to $0,10\text{m/s}^2$.

Table 5: Comfort levels for vertical accelerations

Vertical acceleration (m/s^2)	0.0	0.5	1.0	1.5	2.0	2.5	higher
Level 1							
Level 2							
Level 3							
Level 4							

Table 6: Comfort levels for horizontal accelerations

Horizontal acceleration (m/s^2)	0.0	0.15	0.30	0.80	higher
Level 1					
Level 2					
Level 3					
Level 4					

3. GUARDA FOOTBRIDGE

The results from the dynamic studies using the available methodologies during the design of a steel footbridge to be built over a road junction in Guarda, Portugal (Ramos *et al.*, 2007) pointed towards the probable need for the installation of TMD's in special to prevent the expected lock-in effect in the lower transversal modes. Commissioning tests including modal identification and acceleration measurements during the crossings of synchronized pedestrian groups were then included as requirements in the tender.

The steel footbridge is shown in Figure 2. It has a total length of 122.22 m and is composed by a central suspended span of 90.0 m length and two additional spans at both sides with lengths of 8.125 m and 5.850 m. The deck is three meters wide and is made of precast concrete panels supported by a steel girder. The deck is suspended from two steel arches in its

central part. The arches are inclined towards the inside of the bridge, connected by transversal bars and linked to the deck at the intersection level. The arches supports allow the rotation in the plane of the arch. The deck is simply supported at the ends and the continuity on both extremities of the walkway is assured by simply supported spans structurally independent from the intermediate suspended part of the deck. Stainless steel bars with 30 mm diameter suspend the deck at points spaced of about 8.20 meters and their tension can be controlled by varying their length up to ± 50 mm.



a) General view



b) Walkway steel girder

Figure 2: Guarda footbridge

4. MODAL IDENTIFICATION

4.1 Ambient vibration

The localization and direction of the measured degrees of freedom of the structure are represented in Figure 3. Five sensors were kept fixed for reference (RefS) and the other five sensors (FreeS) were sequentially placed on the several measurement points. For each of the fourteen possible configurations the acceleration time history was recorded during about twelve minutes.

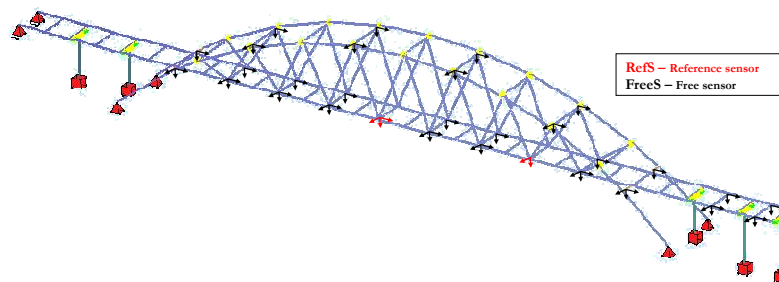


Figure 3: Accelerometers location for ambient vibration measurements

Using the enhanced frequency domain decomposition (EFDD) technique the modal parameters shown in Table 7 were extracted from the measured time series. The corresponding average of the normalized singular values of the spectral density matrices of all test setups are shown in Figure 4. In this figure the marked peaks correspond to the global modes of the bridge and the peaks in the region 2-4 Hz include global modes and local modes corresponding to some of the inclined suspension bars used to suspend the bridge deck. To identify their first natural frequencies these suspension bars were also instrumented separately and the re-

spective results are given in Table 8 for the bars on the southern side and on the northern side of the bridge deck.

Table 7 – Natural frequencies and modal damping using EFD technique

Mode	Frequency [Hz]		Damping [%]		Mode shape
	Mean	St. dev	Mean	St. dev.	
Mode 1	0.65	0.0042	1.27	0.717	1st horizontal
Mode 2	1.07	0.0051	0.88	0.335	2nd horizontal
Mode 3	1.48	0.0143	1.13	0.474	3rd horizontal
Mode 4	2.19	0.0092	0.34	0.148	1st vertical
Mode 5	3.65	0.0145	0.18	0.064	2nd vertical
Mode 6	3.69	0.0344	0.29	0.190	3rd vertical

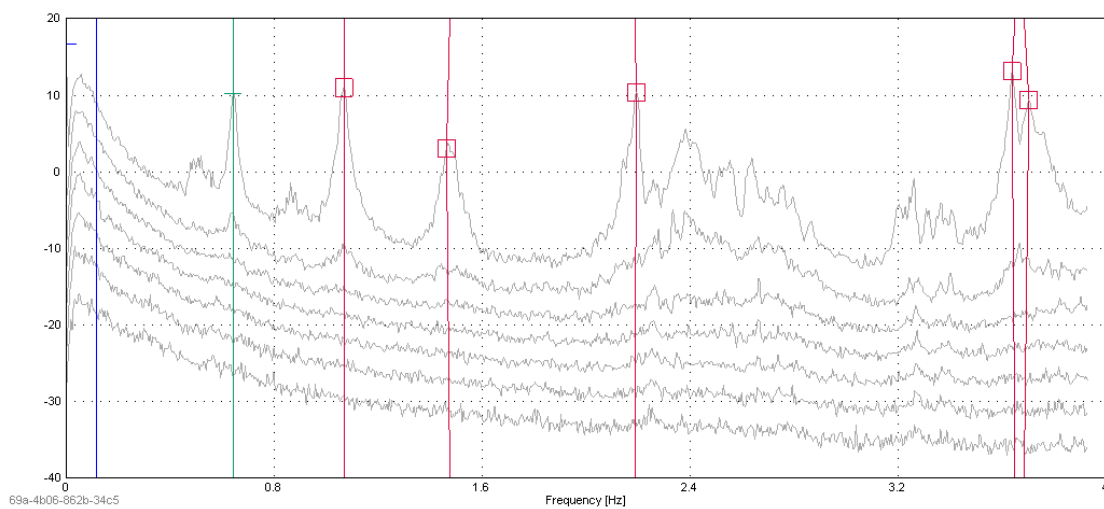


Figure 4 – Peak picking of the singular values of the spectral density matrices calculated using all setups

Table 8 – First natural frequencies (Hz) of the suspension bars

Bar number	4	5	6	7	8	9	10	11	12	13
Southern side	3.21	5.35	3.24	3.41	3.98	2.48	2.83	3.25	2.75	4.48
Northern side	2.23	7.40	3.20	3.53	3.84	2.61	2.31	3.99	2.29	4.16

When analysing the results given in Table 8, it becomes clear that the expected symmetry concerning the tension loads of the suspension bars on both sides of the deck does not exist. Also for the longitudinal direction, there should be an almost symmetry of the values, since there is no significant loss of structural and load distribution symmetry due to the longitudinal inclination of the deck. That is not the case, as it can be concluded when comparing, for instance, the results for bars 5 and 12.

Although the tension force installed in the bars was corrected after the measurements, the results concerning the vertical modal shapes were influenced and it was the reason for the difficulties experienced in trying to obtain a smooth longitudinal line for the modal displacements along the deck (see Figures 5-c and 5-d).

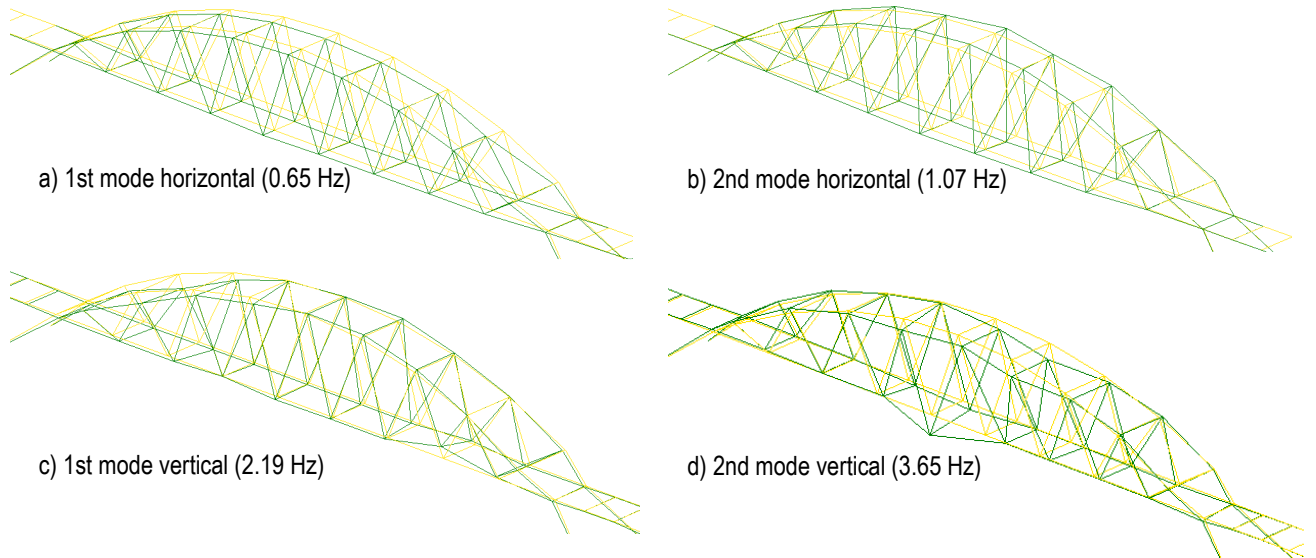


Figure 5 – Mode shapes – first and second horizontal and vertical modes

4.2 Free vibration with initial displacements

When trying to foresee the dynamic response of the bridge under pedestrian traffic the damping of the lower modes characterized by horizontal displacements of the deck are key values for the computations. In order to accurately estimate damping for larger amplitude vibrations the free acceleration response of the bridge was measured when initial displacements were imposed in the horizontal and vertical directions at the centre and quarter span of the bridge.

To perform the modal identification in time domain, the free vibration was firstly filtered using a Chebyshev type II band-pass filter covering, separately, the first and second natural frequency of the bridge corresponding to horizontal displacements of the deck. The filtered signal was then fitted in the time domain using the fitting function given by the uncoupled free vibration acceleration response $\ddot{y}(t)$ of a one degree of freedom (1DoF) linear oscillator (Humar, 2005), that is,

$$\ddot{y}_i(t) = A_i \omega_i^2 \left[(\xi_i^2 - 1) \cos(\omega_i t - \phi_i) + 2\xi_i \sin(\omega_i t - \phi_i) \right] e^{-\xi_i \omega_i t} \quad (2)$$

where A_i , ω_i , ξ_i and ϕ_i are the regression variables for each of the modes considered.

A non linear least squares based regression estimation algorithm implemented in statistics software SPSS (SPSS, 2006) was used to optimize the regression parameters, being the most important the natural frequencies ω_i and respective modal damping ξ_i . In Figure 6 is represented one example of the curve fitting. The results concerning the natural frequencies and damping are summarized in Table 9.

This procedure was not applied to the vertical modes because it was not possible to obtain filtered responses containing only the contribution of the mode of interest, due to the interference of the vibration of the suspension bars, with natural frequencies very close to the pertinent vertical modes. To overcome this, the modal identification was also performed in the frequency domain using the EFDD technique (see Figure 7). The results are also included in Table 9.

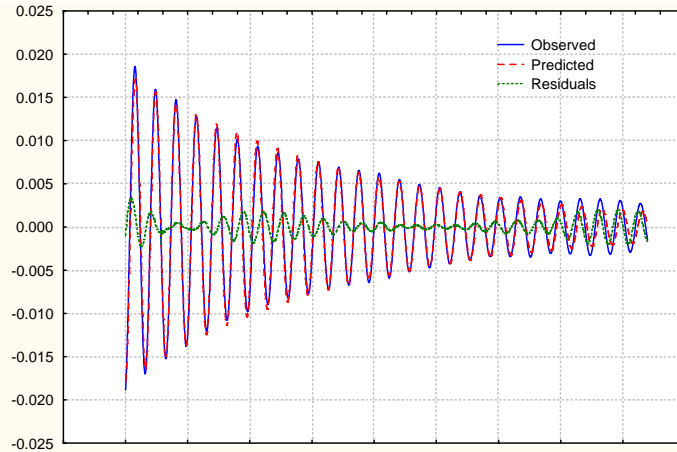


Figure 6 – Curve fitting of the free response acceleration time series band-pass filtered between 0.4 and 0.8 Hz.

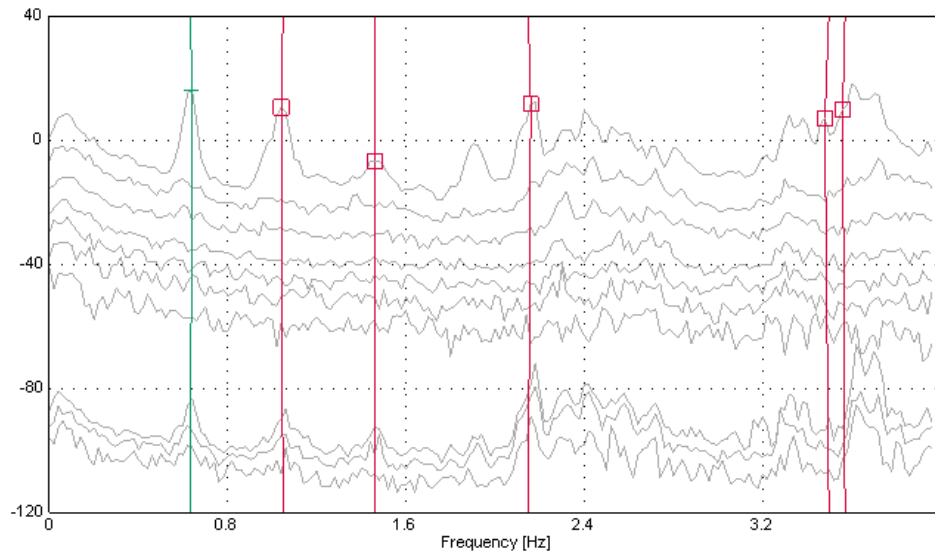


Figure 7 – Frequency domain identification (EFDD) using the free response time series from imposed initial displacements

Table 9: Modal results from free vibration with initial displacements

	<i>Time domain curve fitting (eq. 2)</i>				<i>EFDD (Artemis)</i>			
	Frequency [Hz]		Damping [%]		Frequency [Hz]		Damping [%]	
	Mean	St.Dev.	Mean	St.Dev.	Mean	St.Dev.	Mean	St.Dev.
1st mode horizontal	0.624	0.0065	2.21	0.5	0.63	0.005	2.21	0.268
2nd mode horizontal	1.019	0.0072	2.48	0.7	1.05	0.016	1.74	0.847
3rd mode horizontal	-	-	-	-	1.46	0.021	1.43	0.452
1st mode vertical	-	-	-	-	2.15	0.025	0.79	0.522
2nd mode vertical	-	-	-	-	3.50	0.112	0.36	0.156
3rd mode vertical	-	-	-	-	3.57	0.049	0.30	0.099

5. PEDESTRIAN INDUCED VIBRATIONS

Among the several experiments carried out with pedestrians crossing the bridge, the most interesting were those that intended to induce the lock-in effect in the first and second natural modes. To perform these experiments the pedestrians were requested to cross the bridge in synchronized walking, trying to keep the cadence given by a metronome. The number of pedestrians on the bridge was increased in groups of ten to a maximum of about 80 pedestrians on the bridge deck.

During the experiments the horizontal and vertical accelerations were monitored at 1/2 and 1/3 of the bridge span. The evolution of the maximum values of the horizontal accelerations with the number of pedestrians walking on the bridge is given in Figure 8 separately for the situations when pedestrians crossed the bridge in a cadence of (a) 1.3 steps/s and (b) 2.1 steps/s.

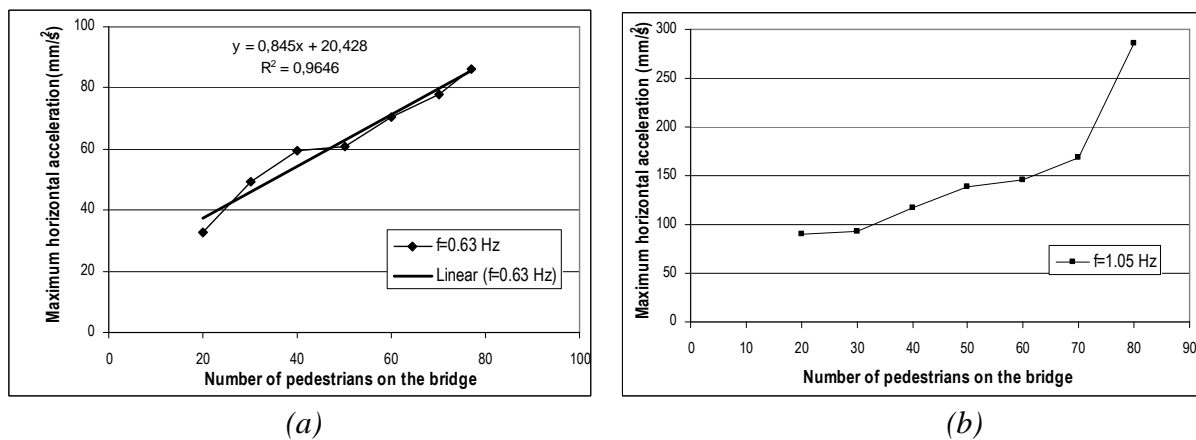


Figure 8 – Variation of the maximum horizontal acceleration with the number of pedestrians crossing the bridge; a) resonance in the first mode and linear fit; b) resonance in the second mode.

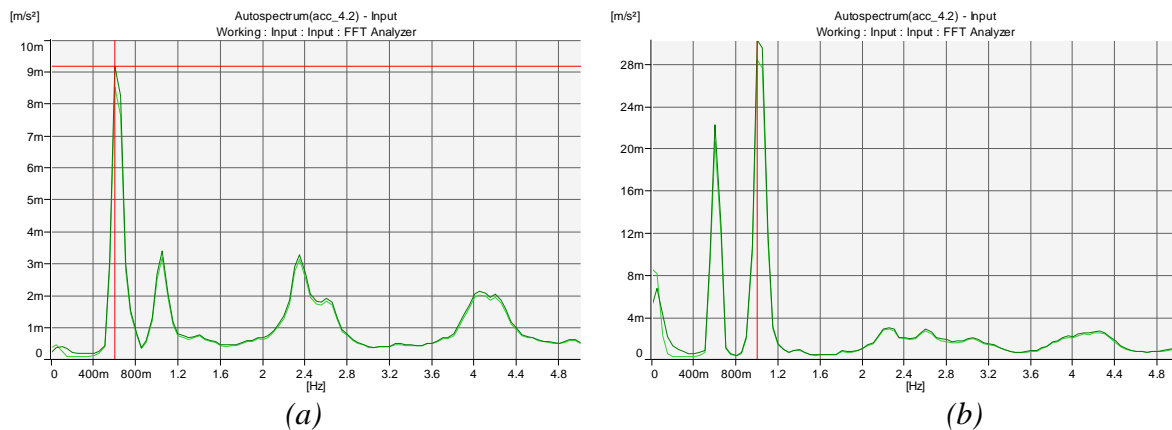


Figure 9 – Envelope of the auto spectra (RMS) of the horizontal acceleration measured at 1/2 and 1/3 of the span when pedestrians crossed the bridge in a cadence of (a) 1.3 steps/s and (b) 2.1 steps/s.

In the first case the first natural frequency was clearly excited but no lock-in effect was detected, that is, there is a steady increase of acceleration with the number of pedestrians. When the experiment was carried out with a faster walking, trying to excite the second natural frequency there was a rapid increase of the acceleration for the last group of ten people entering

the bridge, indicating that the number of pedestrians necessary to develop the lock-in effect could have been reached.

During the second experiment the synchronization was poorer than during the first one. This was visually observed during the measurements and also in the power spectra of the signal (Figure 9-b), which reveal that both the first and the second frequencies were excited.

6. FINITE ELEMENT ANALYSIS

A Finite Element model (Fig. 2-b) was developed using the software SOFISTIK (Sofistik, 2007) in order to fit the modal information obtained from the measurements. Beam and shell elements were used to model, respectively, the steel structure and the concrete panels. Table 10 shows the natural frequencies and modal masses concerning the mode shapes with transversal horizontal displacements (Figure 10). The accuracy of the model was considered to be sufficient to reproduce the time response acceleration for prescribed dynamic loading.

Table 10- Frequencies and modal masses for the transversal modes

<i>Mode</i>	<i>Frequency [Hz]</i>	<i>Modal Mass [kg]</i>
1 – transversal	0.65	77168
2 – transversal	1.05	100498
3 – transversal	1.39	83574

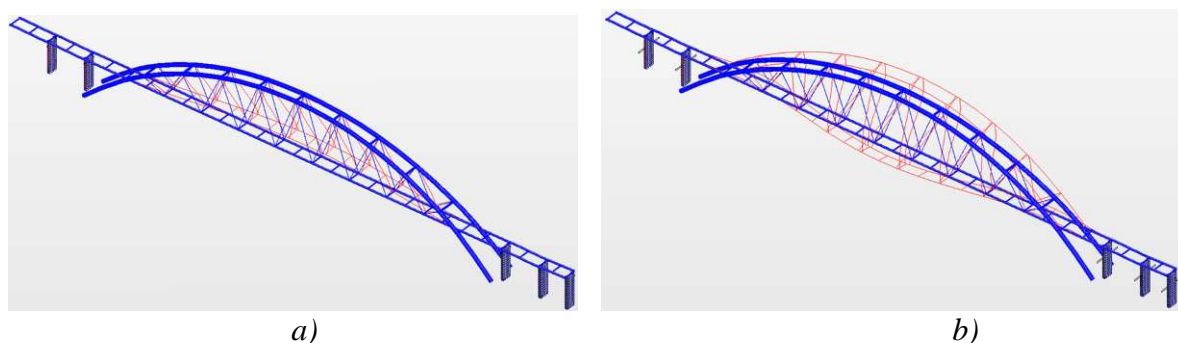


Figure 10 – computed mode shapes – first (a) and second (b) horizontal modes

Following the methodology summarized in Figure 1 this footbridge is classified as Class II. Concerning the resonance hazard, modes 1 and 2 are classified as Level 1, mode 4 as Level 2 and the other modes as Level 3.

Considering only the first mode for the dynamic calculations the density of 0.8 pedestrians/m² shall be considered (CEB-FIB, 2005) corresponding to 184 pedestrians simultaneously on the deck. Using the measured damping of 2.21% and the load time function

$$F_1(t) = 3.708 \cos(2\pi t) \quad [\text{N/m}^2] \quad (3)$$

the maximum response acceleration of 183 mm/s² was obtained. According to Table 6 this value is classified as Level 2 with possible lock-in effect.

Previous calculations during the design phase of the bridge (Ramos, 2007) using several methods lead to maximum accelerations in the range 156-200 mm/s² for the steady state response in the first natural frequency.

To compare these values with those obtained from the measurements during the pedestrian crossings the linear regression given in Figure 8-a can be used to extrapolate the maximum acceleration for a number of 184 pedestrians on the bridge. That value would be 176 mm/s^2 which compares well with the computed values.

7. CONCLUSIONS

The recently developed methodologies, like the one proposed by SÉTRA provide valuable information for the designers, since a global approach is possible for the evaluation of the risk of excessive vibrations taking into account several indicators concerning the geographic situation of the bridge and the probability of resonance induced by groups of pedestrians. However, the use of this type of methodologies needs to be applied in as many case studies as possible in order to validate the assumptions.

According to the available guidelines for the assessment of the vibration behaviour of the footbridges during the design phase, the computation of the dynamic behaviour under pedestrian loading is compulsive for some footbridges. However these computations rely on assumptions like structural damping and stiffness which can be, for several reasons, significantly different from the real conditions of the built structure. Therefore, for those structures, basic modal identification of the built structure should always be foreseen in the tender specifications.

The case study presented is a good example where this procedure was adopted. The initial computations assumed damping values that were lower than those effectively measured. The higher real damping allowed the designer to decide on a much better knowledge basis, whether the initial foreseen TMD's were required or not.

The experiments with groups of pedestrian crossing the bridge aiming to excite the second mode showed that both first and second natural frequencies were difficult to excite. Because of the high damping and probably also because they are enough closely spaced to be excited in a similar way, what makes difficult the lock-in effect to arise in one of them.

8. REFERENCES

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