Continuous dynamic monitoring of bridges and special structures: ongoing research at ViBest/FEUP

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ABSTRACT: This paper makes a brief characterisation of the extensive research activity of the Laboratory of Vibrations and Structural Monitoring (ViBest) of the Faculty of Engineering of the University of Porto (FEUP) in the field of Continuous Dynamic Monitoring of Bridges and Special Structures, selecting four examples where large high quality databases have been created since 2007, namely Infante D. Henrique bridge, Pedro e Inês footbride, FEUP Campus stress-ribbon footbridge and Braga Stadium suspension roof.

1 INTRODUCTION

(a) (b) (c) (d)

Figure 1. Images of (a) Pedro e Inês footbride, (b) Infante D. Henrique bridge, (c) FEUP Campus stress-ribbon footbridge and (d) Braga Stadium suspension roof.

During the last ten years, the Laboratory of Vibrations and Structural Monitoring (ViBest, www.fe.up.pt/vibest) of the Faculty of Engineering of the University of Porto (FEUP) has developed a remarkable effort in the field of Continuous Dynamic Monitoring of Bridges and Special Structures, having presently ten long-term monitoring applications installed in several kinds of large structures, with different typologies and materials, and aiming four levels of objectives:

- The implementation of alert systems used for the safety checking of vibration serviceability limits and the verification of the efficiency of vibration control devices;
- The implementation of Structural Health Monitoring (SHM) systems enabling the vibration based damage detection;
- The investigation of Wind Engineering problems based on continuous measurements on the prototypes;
- The experimental assessment of local vibration fatigue in old metallic railway bridges.

This paper attempts to make a brief characterisation of this extensive research activity, selecting in particular four examples where large high quality databases have been created since 2007, namely Pedro e Inês footbridge, Infante D. Henrique bridge, FEUP Campus stress-ribbon footbridge and Braga Stadium suspension roof (Figure 1).

2 SAFETY CHECKING OF VIBRATION SERVICEABILITY LIMITS

This situation can be well illustrated by the continuous dynamic monitoring of Pedro e Inês footbridge, in Coimbra, Portugal (Figure 1a). This bridge is a slender structure 275m in length and 4m wide, except in the central square with dimensions of 8mx8m. The metallic arch spans 110m and rises 9m and has a rectangular box cross-section with 1.35m x 1.80m. The deck has a L-shaped box cross-section, the top flange being formed by a composite steel-concrete slab 0.11m thick. In the central part of the bridge, each L-shaped box cross-section and corresponding arch “meet” to form a rectangular box cross-section 8m x 0.90m. In the lateral spans, arch and deck generate a rectangular box cross-section 4m x 0.90m. The significant slenderness of the bridge and the geometric characteristics lead to a complex structural behaviour.

According to preliminary studies developed at design stage by the ViBest/FEUP, this bridge would be prone to excessive lateral and vertical human induced vibrations, and so the structure was built with the necessary precautions in order to enable the subsequent installation of several tuned mass dampers (TMDs).

The ambient vibration test performed close to the end of the bridge construction (Caetano *et al* 2010a) permitted to identify a significant number of natural frequencies in the range 0-4.5 Hz, several of them easily excited by pedestrians, namely 0.91, 2.05 and 2.88 Hz, with dominant lateral component, and 1.54, 1.88, 1.95, 2.54, 3.36, 3.57, 3.83, 4.28 and 4.44 Hz, with dominant vertical component. These results were the basis for the subsequent construction of a new and more sophisticated shell finite element model, whose stiffening constants of the springs at the foundations of the arches were iteratively adjusted in order to achieve a very good matching between calculated and measured values. The excellent tuning of the finite element modelling was essential for the numerical evaluation of modal masses and subsequent final design of required TMDs.

On the other hand, free vibration tests permitted to identify, in particular, a modal damping ratio of 0.55% for the critical fundamental lateral mode (lower than the value of 1% assumed at design stage), which enhanced the dynamic effects induced by pedestrians.

At last, the crowd tests indicated that the increase of acceleration with the number of pedestrians on the bridge is not linear, but instead exhibits a “jump” precisely for values of the number of pedestrians close to 70. This value is in perfect agreement with the formula developed in the context of the studies of the Millennium Bridge, in London, to estimate the critical number of pedestrians N_L above which significant lateral oscillations may occur, which led to an estimate of $N_{L=}73$ pedestrians. Extreme values of acceleration of ±1.2 m/s^2 (about 12 times higher the limit value suggested in the recent SETRA (2006) and HIVOSS (2008) guidelines) were measured at the mid-span section, corresponding to a dynamic displacement of ±4 cm, occurring when 145 pedestrians were walking on the bridge.

The lively behaviour of this footbridge was particularly critical for lateral vibrations associated to the fundamental lateral mode, which motivated a special concern with the design of the corresponding TMD (Caetano *et al* 2010b). In this case, assuming the validity of the formula developed by Dallard *et al*, it was possible to fix the number of pedestrians for which lock-in should not occur, which led to an estimate of damping that the control solution should introduce. Following that procedure and the studies of (Bachmann et al 1995), the value of 6% was adopted as the value of damping ratio to achieve after installation of a TMD tuned for this lateral mode of vibration. This implied the specification of a mass of 15000 Kg that was accommodated inside the bridge deck by means of 6 TMD units, each with a mass of 2500 Kg.

The efficiency of this lateral vibration control device was checked, in a first instance, performing a forced vibration test applying a servo-hydraulic shaker at mid-span and measuring the lateral response of the bridge and of each of the TMD units. This test revealed that the activation of all TMD units was achieved for accelerations below the comfort limit of about 0.1m/s^2, which is essential to avoid lock-in phenomena. However, due to different friction characteristics in the corresponding rods, inducing different damping properties, it was noticed that the equivalent damping introduced by the TMD is lower than the required at the corresponding design. Therefore, and taking into account the complex character of the bridge and the scarce experience with mechanical control devices, the Owner of the footbridge required the permanent observation of its structural behaviour during a period of 5 years after construction. In this context, the dynamic behaviour is being permanently monitored with remote control from the ViBest / FEUP, by recording vertical and lateral accelerations in 6 points along the deck.

The monitoring system includes six piezoelectric accelerometers, a signal conditioner, a UPS and a digital computer, stored inside one of the bridge abutments and transmitting measured records of vibration every 20 minutes through the Internet to a central computer at FEUP. The signals are then organized in a main database which can be accessed at any time by post-processing tools in LabView. In order to satisfy the main objective of this monitoring system, a web site was developed which allows the visualization of the time signals of the six accelerometers and subsequently the monitoring of the vibration levels of the structure.

Figure 2 (left) shows a plot with the maximum daily lateral accelerations measured during 3 years (from June 2007 to May 2010), whereas the corresponding histogram is shown in Figure 2 (right). It could be observed that the maximum lateral acceleration was 0.099m/s^2, whereas the maximum vertical acceleration was 0.849m/s^2. Considering the comfort limits recommended by SÉTRA (2006) and HIVOSS (2008) guidelines, it can be concluded that all maximum lateral accelerations and nearly all vertical counterparts fall in the range of maximum comfort levels, so no serviceability problem has occurred in this footbridge under normal operational conditions during 3 years. This is consequence of the implementation and tuning of the passive control devices. Furthermore, it is worth noting that maximum lateral acceleration of about 0.1m/s^2 amplitude was just recorded before on the opening day, when a large number of pedestrians crossed the bridge, corresponding to an amplitude of displacement of 3mm.

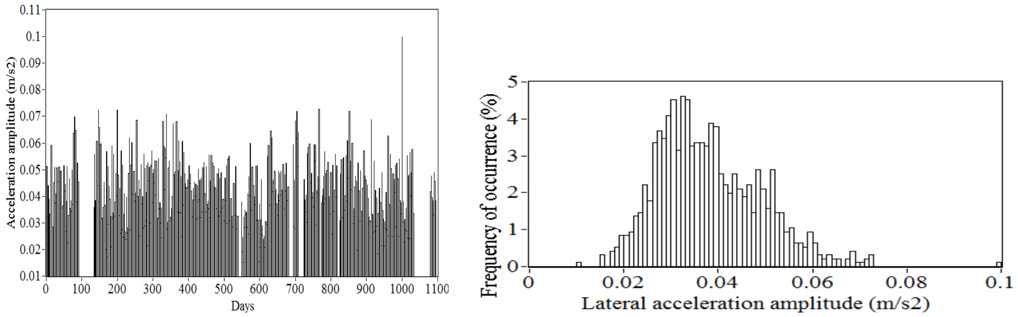


Figure 2. Maximum daily lateral accelerations recorded between June 2007 and May 2010: (left) Time variation; (right) Histogram.

3 VIBRATION BASED STRUCTURAL DAMAGE DETECTION

The Infante D. Henrique Bridge (Figure 1b), over Douro river at Porto, is formed by two mutually interacting fundamental elements: a very rigid prestressed reinforced concrete box beam, 4.50 m in height, supported by a very flexible reinforced concrete arch, 1.50 m thick. The arch spans 280 m between abutments and rises 25 m until the crown, thus exhibiting a shallowness ratio greater than 11/1. In the 70 m central segment, arch and deck join and define a box-beam 6 m in height. The arch has constant thickness and a linearly varying width from 10 m in the central span up to 20 m at the abutments.

ViBest/FEUP implemented a permanent dynamic monitoring system in this bridge, which has been in continuous operation since September 2007. This system, with high reliability over time, is formed by 12 force-balance accelerometers (in 4 sections of one half of the deck, 2 vertical upsetram-downstream and 1 lateral in each section), two 6-channel digitisers and one computer installed inside the box-girder. The acceleration time series collected every 30 minutes at 50 Hz are immediately transferred to FEUP via Internet and processed using appropriate software (DynaMo) specifically developed for this purpose (Magalhães, 2010). This software enables the automatic identification of modal parameters based on EFDD, SSI and p-LSCF methods, and the application of statistical tools (multivariate linear regression or principal components analysis (PCA)) to remove the effects of environmental/ operational factors on the natural frequencies estimates (Magalhães *et al*. 2012).

A first inspection of the time evolution of the 12 first natural frequencies identified in the frequency range 0.5-4.5 Hz on the basis of the p-LSCF method in the period 13/09/2007 to 12/09/2008 may suggest that these frequency estimates are very stable. However, a zoom of individual frequency plots evidences the occurrence of seasonal effects, as shown in Figure 3 for the first four natural frequencies during a period of observation of three years.

Frequency variations with time stem from environmental (e.g. temperature) and operational (e.g. traffic intensity) factors, and though they are relatively small, they can seriously disturb any attempt of damage detection based on natural frequency shifts. Therefore, it is important to mitigate the effect of these influences using appropriate statistical models and building suitable control charts using a novelty index that might flag the occurrence of slight structural damage (Magalhães 2010, Hu 2011). Figure 4(a) presents, for instance, the time evolution of the first natural frequency estimates before and after the removal of the environmental and operational effects. Inspection of this figure shows the efficiency of that correction, most of the estimates becoming then located inside a very narrow frequency range with an amplitude of about 0.005 Hz. This means that relatively small damage can be detected in the future, provided that the consequent frequency shifts are higher than that order of magnitude (possible trigger level for alert). This conclusion was confirmed analysing different slight damage scenarios idealized numerically and applying appropriate control charts, as indicated in Figure 4(b) (Magalhães 2010).

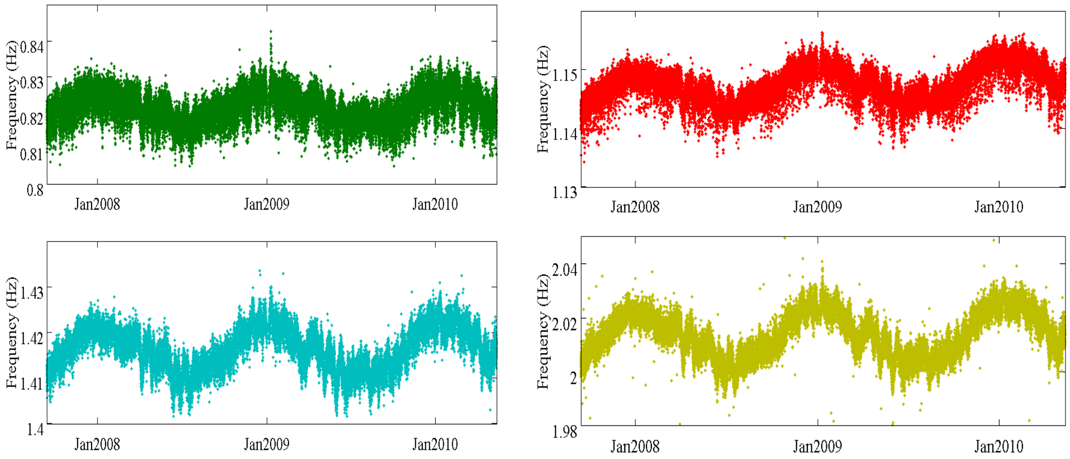
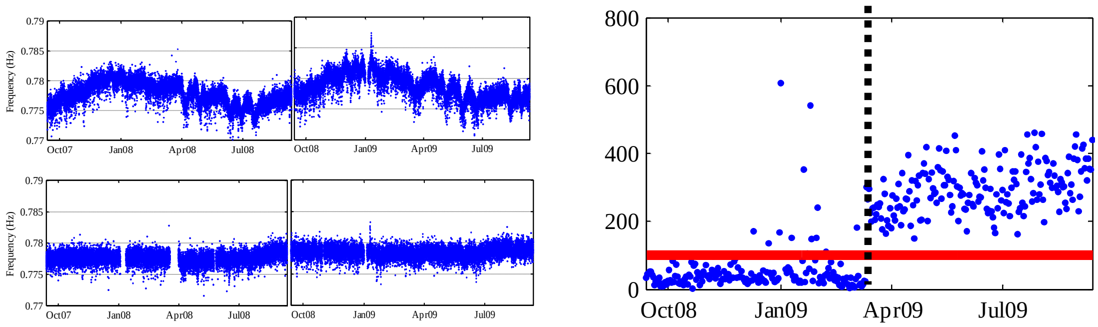


Figure 3. Evolution of the estimates of the first four vertical bending natural frequencies along three years, using the p-LSCF method.

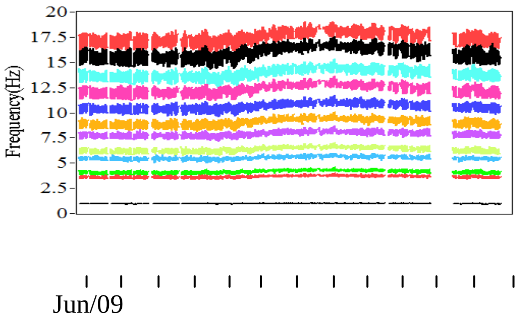


(a) (b)

Figure 4. (a) Time evolution of the first natural frequency estimates before and after the removal of environmental / operational effects; (b) Control chart used for damage detection.

Further research on vibration based damage detection has also been developed on the FEUP stress-ribbon footbridge (Figure 1c). This bridge establishes a link between the main buildings of FEUP and the students' canteen. The two spans 28m and 30m long and the 2m rise from the abutments to the intermediate pier are the starting points for the definition of the bridge structural geometry. A continuous concrete cast-in-situ slab embedding four pre-stressing cables takes a catenary shape over the two spans, with a circular transition over the intermediate support, which is made of four steel pipes forming an inverted pyramid hinged at the base. The constant cross-section is approximately rectangular with external design dimensions of 3.80mx0.15m.

The continuous dynamic monitoring system implemented in FEUP stress-ribbon footbridge includes four sensor units. They are mounted separately on the lower surface of the bridge deck at both 1/2 and 1/3 of each span. Each unit comprises a vertical accelerometer, a signal conditioner and a thermal sensor. Acceleration signal conditioners and thermal sensors are connected via cable with National Instruments Ethernet data acquisition (DAQ) devices, which are incorporated in a steel box installed beneath the deck at the intermediate support. The NI Ethernet DAQ device is driven by a signal acquisition toolkit, generating a nearly real-time zipped acceleration and temperature files every 10 or 30 minutes, respectively, continuously. Signal files acquired under operational conditions are conveniently accessed via Internet. The continuous dynamic monitoring system has been operating from the 1st of June 2009 up to now, except for occasional stops due to small technical problems.



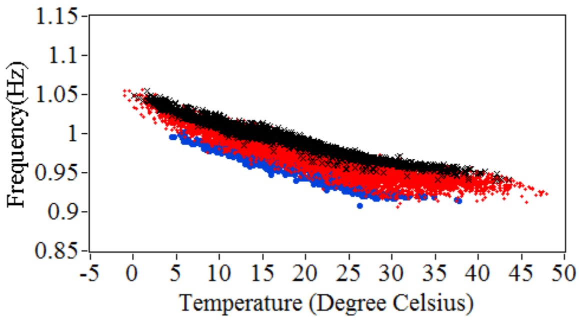


Figure 5. (Left) Identified frequencies from 1/6/09 to 31/5/10; (Right) Frequency of first mode vs temperature (black, red and blue dots: low, medium and high vibration levels, respectively).

The identified frequencies of 12 modes in the range of 0-20Hz estimated by the SSI-COV method are shown in Figure 5 (this corresponds to all the first vibration modes, except the ones associated to frequencies of about 2.1 and 2.4 Hz, due to the perturbation resulting from the strong interaction with pedestrians). The variations of frequency estimates of higher order modes are quite clear. The tendency of increasing in winter time and decreasing in summer time reflects the seasonal environmental effects. Maximum relative variations of frequency estimates are in the range 15.3%-21.4%, which is significantly higher than in the two previous applications.

A detailed correlation analysis between natural frequency estimates, temperature and vibration levels can be found in (Hu 2010), showing that temperature is again the main factor inducing frequency changes. However, in this case, the influence of traffic intensity is slightly higher than in Pedro e Inês footbridge, as stressed by Figure 5 (right). Moreover, the trend of decrease of frequency estimates with the increase of temperature tends to vanish above 30ºC. Still, the existence of approximately linear correlation between the variations of frequency estimates of the several modes of vibration allowed the application of Principal Component Analysis (PCA) to remove the influence of environmental factors on the frequency estimates, enabling the reliable detection of small simulated structural damages motivated by the slight release of the clamped connection of the deck at the abutments achieved by the addition of springs with controlled constants (Hu 2010).

4 WIND ENGINEERING INVESTIGATION BASED ON IN-SITU MEASUREMENTS

The Braga Stadium (Figure 1d), in Portugal, is one of the football stadia constructed to host some of the matches of the 2004 European Football Championship. It was designed by the architect Eduardo Souto Moura who, recently, was awarded the Pritzker architecture prize, in collaboration with the engineering design office AFassociados. The most interesting element of the stadium, from the structural engineering point of view, is, undoubtedly, the suspension roof, which consists of two concrete slabs supported by 34 pairs of full locked coil cables 0.35m distant. These pairs of cables span 202 m, with diameters ranging from 80 to 88mm and are spaced by 3.75m (Caetano *et al*. 2010c).

A complete characterization of the modal parameters of the suspended roof in the frequency range of 0-1.2 Hz was achieved by an ambient vibration test performed by ViBest/FEUP in 2007 (Magalhães *et al.* 2008) on both slabs of the roof. In May 2011, another ambient vibration test with higher spatial resolution was developed (Amador *et al*. 2012) to assess higher modes of vibration, as these modes seem to be more sensitive to the environmental factors. This allowed the accurate identification of 20 modes of vibration in the range 0.2-1.9 Hz.

The outstanding characteristics of the structure and the need of a tight control of the corresponding behaviour and geometry during the construction justified the installation of a static monitoring system. Subsequently, ViBest/FEUP implemented a dynamic and wind characterisation monitoring system at the west roof slab, comprehending 6 force-balance accelerometers (A1 to A6) and 2 ultra-sonic anemometers (WS1, WS2) and thermometers installed in one side of the roof. The main purpose of this monitoring component is focused on the characterisation of the wind model and analysis of the correlation of temperature and wind conditions with the modal properties estimates and the dynamic response.

Figure 6 and 7 characterise the type of information obtained in terms of variation of mean wind speed and turbulence intensity, enabling the identification of dominant wind directions. On the other hand Figure 8 shows the non-linear increase of the dynamic structural response with the mean wind speed, which also led to the clear increase of modal damping ratio estimates (Amador *et al*. 2012).

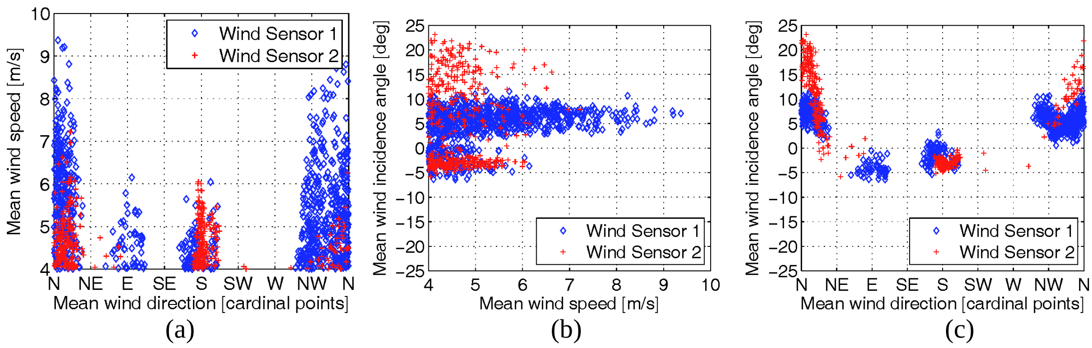


Figure 6. (a) Distribution of 10-min mean wind speed with direction; Distribution of 10-min mean wind incidence angle with: (b) mean wind speed; (c) mean wind direction.

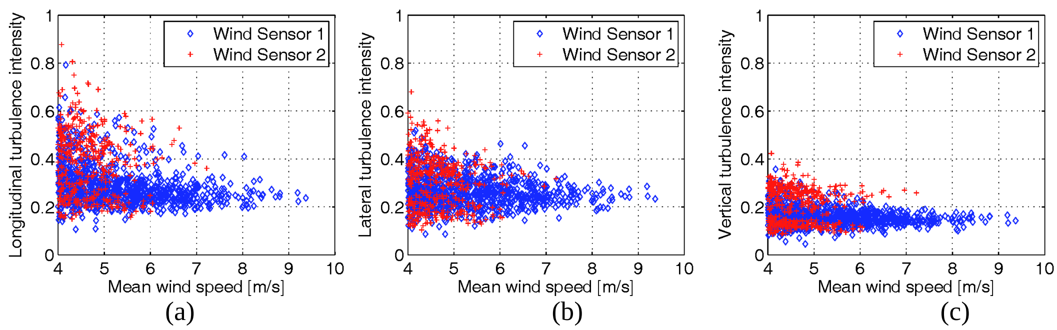
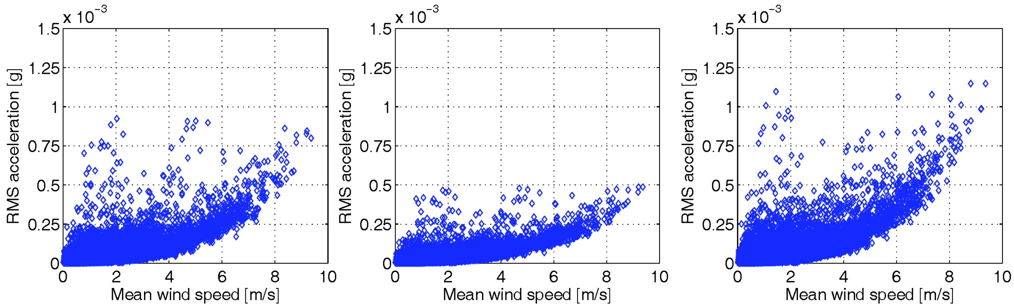


Figure 7. Relationship between the turbulence intensity and 10-min mean wind speed measured by the two sonic anemometers: (a) Longitudinal turbulence intensity; (b) Lateral turbulence intensity; (c) Vertical turbulence intensity.



(a) (b) (c)

Figure 8. Relationship between the 10-min mean wind speeds measured by wind sensor WS1 and the RMS of the vertical acceleration responses for: (a) accelerometer A1; (b) accelerometer A2; (c) accelerometer A3 (distributed along the inner edge of the west roof slab).

5 CONCLUSION

The four case studies from ViBest/FEUP briefly described in this paper clearly show the interest and potential of Continuous Dynamic Monitoring either as alert systems used for the safety checking of vibration serviceability limits and the verification of the efficiency of vibration control devices, or as SHM systems enabling the vibration based damage detection, or even for the investigation of Wind Engineering problems based on continuous measurements on the prototypes.

6 ACKOWLEDGEMENTS

The authors acknowledge all the support provided by FCT to ViBest/CEC in the context of research activities related with Dynamic Testing and Structural Monitoring, particularly within the Projects “Wind analysis of special structures from full scale measurements” and “Advanced Tools for Dynamic Structural Health Monitoring of Bridges and Special Structures”.

7 REFERENCES

Amador, S, Martins, N, Magalhães, F, Caetano, E, and Cunha, A. 2012. One year of continuous dynamic monitoring of a football stadium suspension roof: analysis of data provided by an autonomous monitoring application, Int. Conf. on Noise and Vibration Engineering, ISMA 2012, Leuven, Belgium.

Bachmann H. et al. 1995. *Vibration Problems in Structures: Practical Guidelines*, Birkhäuser Verlag, Basel, 234 pp.

Caetano, E, Cunha, A, Magalhães, F, and Moutinho, C. 2010(a). Studies for controlling human-induced vibration of the Pedro e Inês footbridge, Portugal. Part 1: Assessment of dynamic behaviour, *Engineering Structures*, 32, pp. 1069-1081.

Caetano, E, Cunha, A, Moutinho, C, and Magalhães, F. 2010(b). Studies for controlling human-induced vibration of the Pedro e Inês footbridge, Portugal. Part 2: Implementation of tuned mass dampers, *Engineering Structures*, 32, pp. 1082-1091.

Caetano, E, Cunha, A, and Magalhães, F. 2010(c). Numerical and experimental studies of Braga sports stadium suspended roof. *Journal of Structure and Infrastructure Engineering*. Vol. 6, No. 6, pp. 715-724.

HIVOSS. 2008. Design of footbridges: Guideline.
http://www.stb.rwth-aachen.de/projekte/2007/HIVOSS/download.php

Hu, W.-H. 2011. *Operational Modal Analysis and Continuous Dynamic Monitoring of Footbridges*, Ph.D. Thesis, University of Porto, Faculty of Engineering (FEUP).

Magalhães F. 2010. *Operational Modal Analysis for Testing and Monitoring of Bridges and Special Structures*, Ph.D. Thesis, University of Porto, Faculty of Engineering (FEUP).

Magalhães, F, Cunha, A, and Caetano E. 2012. Vibration based structural health monitoring of an arch bridge: from automated OMA to damage detection, *Mechanical Systems and Signal Processing*, Vol. 28, 2012, pp. 212-228.

Magalhães, F, Caetano, E, and Cunha, A. 2008. Operational modal analysis and finite element model correlation of the Braga Sport Stadium Suspended roof. *Engineering Structures*. Vol. 1, No. 30, pp. 1688-1698.

SETRA/AFGC. 2006. Footbridges – Assessment of dynamic behaviour under the action of pedestrians, Guidelines, Sétra.