Recent developments in vibration analysis of historic and masonry structures: damage detection and wireless sensor networks

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ABSTRACT: Preservation of architectural heritage is considered a fundamental issue in the cultural life of modern societies. This heritage is accumulating damage due to deterioration of materials, repeated loading and exceptional events. This means that conservation, repair and strengthening are often necessary. In this process, monitoring and non-destructive testing play a major role, providing information on the building condition and existing damage, and allowing to define adequate remedial measures. Dynamic based methods are an attractive tool because they are non-destructive and are able to capture the global structural behaviour. The present paper focuses on different aspects related to vibration analysis, namely exploring damage, environmental effects on the dynamic response, a comparison between commercial wireless based platforms and conventional wired based systems, and a novel wireless based system.

1 INTRODUCTION

1.1 Historic and Masonry Structures

The analysis of historic masonry constructions is a complex task that requires specific training. The continuous changes in materials and construction techniques, which swiftly moved away from traditional practice, and the challenging technical and scientific developments, which make new possibilities available for all the agents involved in the conservation of the architectural heritage, are key aspects in the division between the science of construction and the art of conservation and restoration.

The consideration of these aspects is complex and calls for qualified analysts that combine advanced knowledge in the area and engineering reasoning, as well as a careful, humble and, usually, time-consuming approach. Several methods and computational tools are available for the assessment of the mechanical behaviour of historic constructions. The methods resort to different theories or approaches, resulting in: different levels of complexity (from simple graphical methods and hand calculations to complex mathematical formulations and large systems of non-linear equations), different availability for the practitioner (from readily available in any consulting engineer office to scarcely available in a few research oriented institutions and large consulting offices), different time requirements (from a few seconds of computer time to a few days of processing) and, of course, different costs.

The possibilities of structural analysis of historic constructions have been addressed in Lourenço (2002), where it is advocated that most techniques of analysis are adequate, possibly for different applications, if combined with proper engineering reasoning. Recent advances in the homogenization techniques are very powerful from a theoretical and mathematical viewpoints, but their application remains a true challenge, Lourenço *et al.* (2007). Moreover, the difficulties in the selection and usage of the analysis tool remain rather high in the case of seismic applications, as shown in Lourenço *et al.* (2011).

Structures of architectural heritage, by their very nature and history (material and assembly), present a number of challenges in conservation, diagnosis, analysis, monitoring and strengthening that limit the application of modern legal codes and building standards. Recommendations are desirable and necessary to ensure rational methods of analysis and repair methods appropriate to the cultural context. Therefore, the International Scientific Committee for the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) has prepared recommendations (Icomos, 2001), intended to be useful to all those involved in conservation and restoration problems.

A multi-disciplinary approach is required and the peculiarity of heritage structures, with their complex history, requires the organization of studies and analysis in steps: condition survey, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions. Understanding of the structural behaviour and material characteristics is essential for any project related to architectural heritage. Diagnosis is based on historical information and qualitative and quantitative approaches. The qualitative approach is based on direct observation of the structural damage and material decay as well as historical and archaeological research, while the quantitative approach requires material and structural tests, monitoring and structural analysis, see Figure 1.

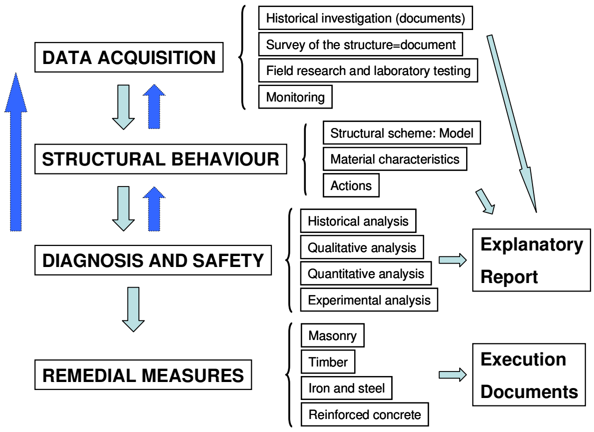


Figure 1: Possible flow-chart for ICOMOS Methodology.

In the process described above, data acquisition is the first step, possibly incorporating different aspects related to vibration analysis, namely sonic testing for characterizing local condition (damage, voids and other), dynamic modal identification as a technique for global validation of the condition of the structure and of the model used in structural monitoring, and damage identification for global damage detection and control of progressive phenomena. The latter two aspects are further detailed next.

1.2 Structural monitoring and damage identification

Structural monitoring and damage identification at the earliest possible stage are issues that receive much attention from scientific community. Damage identification is relevant to all the engineering fields as the service loads and the accidental actions may cause damage to the structural systems, Doebling *et al.* (1996). Regular inspections and condition assessment of engineering structures allow programmed repair works and cost-effective management of the infrastructures. In the case of historical constructions, maintenance is even more relevant because of their cultural importance, the safety of visitors, the potential seismic vulnerability and the accumulation of physical, chemical and mechanical damage through time. Dynamic based damage identification methods are based on changes of dynamic parameters (eigen frequencies, mode shapes and damping coefficients), typically a decrease in stiffness and an increasing in damping. As damage is a local phenomenon and may not influence significantly the lower frequencies or the global response, it is important to estimate higher modes and to have accurate information about modal displacements. For these reasons, the methods based on changes on the mode shapes or modal curvatures (or the combination of all responses, e.g. frequencies, modes shapes and modal curvatures) are usually more successful.

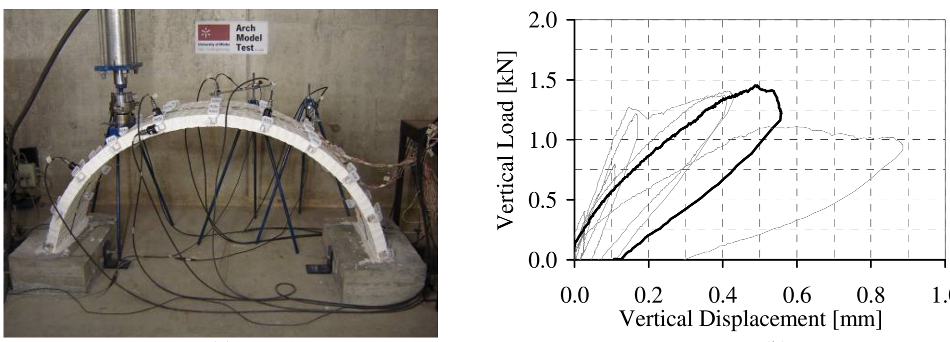
The difficulty is to measure accurately the modal strains (modal curvatures) in civil engineering structures or to derivate accurately the modal curvatures from the modal displacements. In any case, to estimate damage a sufficient number of measuring points is necessary in order to have enough spatial resolution of the mode shapes. Historical masonry structures have a complex geometry and successive past interventions, while the constituting materials tend to exhibit significant variations in properties and internal structure. Therefore, the selection of appropriate models for structural analysis is not easy. Damage in masonry structures are mainly cracks, material deterioration and excessive deformations. When cracks occur, generally they are localized, splitting the structures in macro-blocks. The use of dynamic based methods to identify the damage is an attractive tool to use in this type of structures due to the assumption that damage can be associated with the decrease of stiffness. Many methods are presented in the literature for damage identification based on vibration signatures, but there are relatively few works discussing their application to masonry structures. From the point of view of the applicability of dynamic based identification methods to masonry structures, the aspects under consideration are the possibilities of detection (Level 1), localization (Level 2) and assessment (Level 3). A detailed comparison and a description of different methods are presented elsewhere, Ramos (2007).

The current practices of structural health condition are based mainly on periodic visual inspections or condition surveys but, during the last decade, software and hardware developments made continuous monitoring possible, Chang *et al.* (2003). Typically, one can install hundreds of sensors in a structure and read the data in real time. The attention now seems to be focused on what type of information is important from the structural point of view and how the data should be processed and stored for damage analysis, Maeck (2003). The developments in Micro Electro Mechanical Systems (MEMS) and Wireless Sensor Networks (WSN) are promising technologies in this field, in particular for historical masonry structures, Lynch (2007) and Ramos *et al.* (2010c), which are also addressed here.

2 DAMAGE DETECTION OF ARCHED MASONRY STRUCTURES

2.1 Characterization of the specimen

One replica of an old masonry arch was built in the laboratory and several controlled Damage Scenarios (DS) were induced with static tests, see Figure 2. For each DS, system identification tests were carried out with SSI techniques, Peeters (2000), and the results of the damage identification methods were compared with the internal crack growth. The ambient temperature and humidity were also recorded, to evaluate possible environmental effects on the dynamic response.



(a) (b)

Figure 2: Arch static tests: (a) view of the test apparatus; (b) example of one damage scenario considered.

The arch was built with clay bricks with $100 \times 50 \times 25$ mm^3, handmade in the northern area of Portugal. The clay bricks have low compressive strength and are laid using a Mapei® mortar with low mechanical properties, trying to be representative of the materials used in historical constructions. The arch has a semicircular shape with a radius of 0.77 m. It has a span equal to 1.50 m, a width equal to 0.45 m, and a thickness equal to 0.05 m. The thickness of the joints is about 0.5 cm. The arch rests in two concrete abutments fixed to the ground floor with bolts, see Ramos et *al.* (2010a) for more details.

2.2 *Static tests to induce damage*

The load stages/DSi, being *i* a Roman numeral, were produced with the application of a static point load located at a quarter span, where the lowest safety factor for arches is obtained. The load was applied with increasing magnitude, being removed after each DS. This assumption tries to represent the situation of temporary exceptional loading in the lifetime of the structure (e.g. a heavy truck passing in a bridge or an earthquake) and assumes that cracks can partially close after the extreme event. The assumption makes the task of damage identification more difficult but more challenging, considering that damage can be difficult to detect with a visual inspection.

Figure 2b shows the response of the model during the successive static tests after the first crack, and one selected damage scenario. The tests results in terms of load displacement diagrams indicate a clear loss of stiffness upon reloading, with visible load drops due to crack opening. Figure 3 presents the resulting crack patterns in the arch and the position of the dynamic transducers. Four cracks were found but it was impossible to register the full crack sequence. The first crack to appear (c_1) was located between the positions P3 and P4 (accelerometers A03 and A04) and not below the load application point. The crack appears in the intrados, as expected, but it was only visible during the loading branch of the DSVI, although it is evident that it has occurred on DSV, given the loss of stiffness in the static tests.

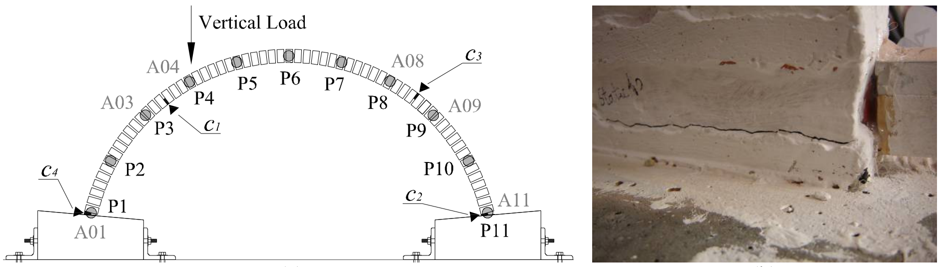


Figure 3: Arch static tests: (a) view of the test apparatus; (b) example of one damage scenario considered.

Crack c_2 was located in position P11 (right support in the measuring point A11) in the intrados, crack c_3 was located between P8 and P9 (measuring point A08 and A09) in the extrados, and crack c_4 was located at position P1 (left support in the measuring point A01) in the extrados, see Figure 3. The load value corresponding to the occurrence of these cracks was impossible to detect. The cracks became visible in the loading range of DSVIII. This can be explained by the fact that the self-weight of the arch is a stabilizing action and almost fully closes the cracks at the equilibrium position. The decision to stop each static test after DSV was based on the sound of a crack that suddenly opens. Therefore, the designation c_1 to c4 presented here does not try to represent the real crack sequence, which is partly unknown.

It should be stressed that the maximum remaining crack opening after the entire testing program was 0.05 mm and the maximum crack depth in the loading branch of the tests was 30 mm (more than half of the arch thickness) for crack c1. Analyzing in detail the stiffness during the several load steps presented in Figure 4, after the occurrence of the crack c_1 it was possible to observe a decrease of stiffness. A loading and unloading stiffness can be defined, as static tests allow measuring the rapidly varying loading stiffness k_l and the dynamic tests only measure the slowly varying unloading stiffness k_u. As a result, dynamic tests are less sensitive than static tests to identify structural damage.

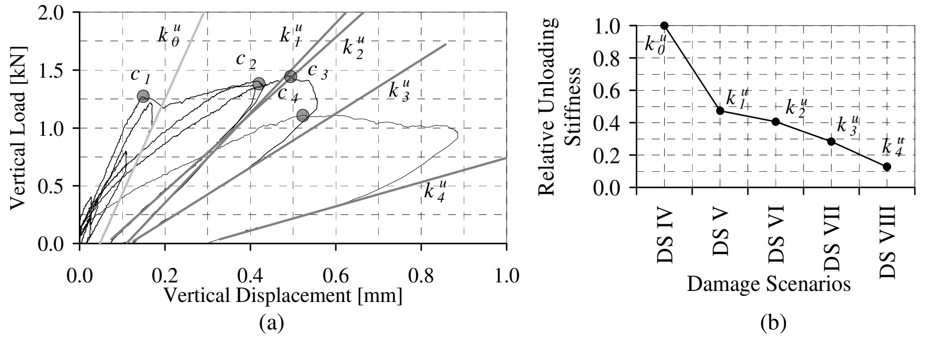


Figure 4: Stiffness decrease: (a) possible sequence of cracks and gradual stiffness decrease; (b) relative unloading stiffness variation.

2.3 *Dynamic identification tests*

The selected sensors for the dynamic tests were accelerometers and strain gauges. It was decided to measure accelerations only in the in-plane direction, and, for convenience, in the normal and tangential directions on each location. It was also decided to measure both edges of the arch (front and back edges) to estimate the torsion modes and to detect any asymmetric behaviour. Figure 5 shows the location of the measuring points where the reference points are given in a grey box. The designation Ai is for accelerometers and Si is for strain gauges. In the case of the strain gauges, as an attempt to study the possibility to measure strains for dynamic modal analysis, it was decided to measure only the middle line of the arch on both sides. This means that the torsion modes will not be present in the strains measurements.

The dynamic acquisition system was composed by 8 uniaxial piezoelectric accelerometers and 22 strain gauges glued in the masonry. Modal identification tests at each load stage/DS were carried out by two different excitation types: (a) natural and ambient noise present in the laboratory; (b) random impact excitation in space and in time (not recorded), induced by a hammer with 2.5 kg of mass. The results in terms of frequency values and damping coefficients for the Reference Scenario (RS) are presented in Table 1 for the ambient and random excitation tests, respectively. Both standard deviation (σ) and Coefficient of Variation (CV) are presented for the same modal parameters: frequency ω and damping ξ. In general, the random impact excitation tests have lower standard deviation values, indicating a possible better modal estimation. The high CV values for the damping in all cases should be stressed as possible less accurate results. The damping values depend on the excitation and incorporate nonlinear phenomena, but one can conclude that an average value of 0.6% can be observed for all modes and all analyses.

The first modes shapes according to the excitation type are presented in Figure 6, see Ramos *et al.* (2010a) for more results. All the modes have torsion components, with the exception of the first mode, which has only components in the arch plan. It is stressed that the modes from the two excitation types are highly coincident, with the exception for the modal configuration of the seventh mode shape at middle span and at the left support.

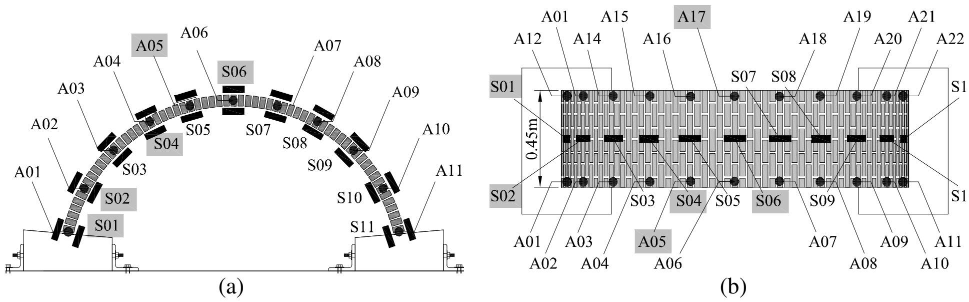
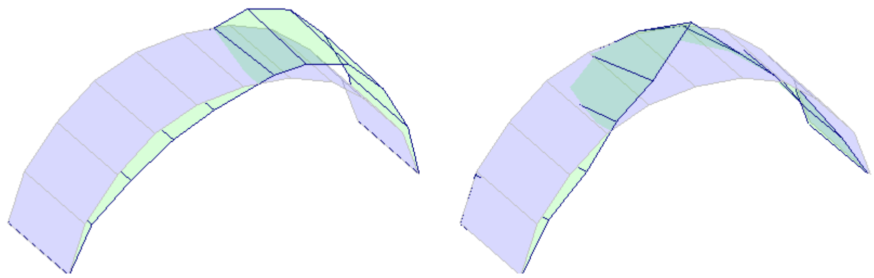


Figure 5: Location of the measuring points for the dynamic tests: (a) front view; and (b) top view. Ai indicates accelerometers and Si indicates strain gauges. Boxed-values in grey are the reference points.

Table 1: Results from the Reference Scenario (RS).

Modes	Ambient Excitation						Random Impact Excitation					
	ω [Hz]	σ_ω [Hz]	CV_ω [%]	ξ [%]	σ_ξ [%]	CV_ξ [%]	ω [Hz]	σ_ω [Hz]	CV_ω [%]	ξ [%]	σ_ξ [%]	CV_ξ [%]
1	35.59	0.20	0.57	0.44	0.20	45.18	35.21	0.12	0.33	0.51	0.08	15.94
2	67.30	0.46	0.69	0.53	0.12	22.50	66.58	0.36	0.55	0.64	0.09	14.68
3	72.11	0.38	0.53	0.88	0.90	101.92	71.16	0.21	0.30	0.72	0.14	19.21
4	125.74	0.65	0.52	0.59	0.10	17.53	124.52	0.74	0.60	0.79	0.10	12.32
5	140.08	0.89	0.63	0.46	0.31	66.76	138.94	0.90	0.65	0.73	0.22	30.11
6	173.38	1.63	0.94	0.83	0.33	39.02	172.54	0.84	0.49	1.11	0.30	27.38
7	199.32	2.91	1.46	1.84	0.65	35.39	196.76	2.13	1.08	2.55	0.94	36.98



(a) (b)

Figure 6: First mode shape configurations for the undamaged scenarios: (a) mode 1; (b) mode 2.

2.4 *Experimental damage identification analysis*

The damage analysis over the eight experimental DS induced by increasing external loading is now described. Table 2 presents for the first four estimated modes and for the case of ambient excitation the frequency results for the consecutive DS. The frequency values are presented together with the value $\pm 2\sigma\omega$ as a 95% confidence interval, and the frequency differences $\Delta\omega$ to the RS. It is stressed that small increases in the frequency values are found before the occurrence of the first crack. This is due to the normal adjustments of the structure with the applied load, mainly at the supports and masonry joints. Considering the first four estimated frequencies, the first significant frequency decrease, i.e. higher that $2\sigma\omega$ (given in a grey box), happens around DSV. In fact, the first significant change in the static stiffness also appears in this scenario. In the subsequent scenarios it is also possible to observe significant frequency decreases.

Table 2: Frequency results for the arch model with ambient excitation.

Damage Scenario	Mode 1			Mode 2			Mode 3			Mode 4		
	ω [Hz]	$2\sigma_\omega$ [Hz]	Δ_ω [Hz]	ω [Hz]	$2\sigma_\omega$ [Hz]	Δ_ω [Hz]	ω [Hz]	$2\sigma_\omega$ [Hz]	Δ_ω [Hz]	ω [Hz]	$2\sigma_\omega$ [Hz]	Δ_ω [Hz]
RS	35.59	0.40	–	67.30	0.93	–	72.11	0.76	–	125.74	1.30	–
DS_I	35.55	0.31	–0.05	67.51	0.83	0.21	71.80	0.38	–0.30	125.69	1.91	–0.05
DS_{II}	35.55	0.24	–0.04	67.39	1.11	0.09	71.83	1.06	–0.28	125.79	2.03	0.05
DS_{III}	35.42	0.31	–0.17	67.47	1.19	0.17	71.66	0.94	–0.45	125.75	2.21	0.01
DS_{IV}	35.15	0.24	–0.44	67.11	0.88	–0.19	71.33	0.58	–0.78	126.01	1.09	0.28
DS_V†	33.72	0.32	–1.87	65.68	0.72	–1.62	69.36	0.60	–2.75	124.48	1.60	–1.25
DS_{VI}	33.19	0.34	–2.40	64.91	1.02	–2.39	68.56	0.58	–3.55	123.58	1.37	–2.16
DS_{VII}	31.49	0.44	–4.10	63.08	1.29	–4.22	65.72	0.69	–6.39	121.97	1.83	–3.77
DS_{VIII}	28.09	0.62	–7.50	58.44	1.40	–8.86	62.61	0.93	–9.50	119.44	1.76	–6.30

† - Damage scenario in which the first crack was detected in the static tests.

Figure 7 presents the variation for the frequency, where it is noticeable that damage was detected in DSV and it is coincident with the significant decrease observed in the static results.

As a Level 1 approach, the results of the USI method presented in Figure 8 shows the comparison with the RS. One conclusion is the difference in the order of values before and after DSV, confirming the significant change that happened in this scenario. The USI values in DSVI are similar to DSV but the last two DS show a significant increase. These results indicate that when the USI is calculated for the several scenarios, the detection of damage (Level 1) is possible. When no information exists about the modal history, the detection of damage with USI might be difficult, because there are no reference values in the undamaged condition.

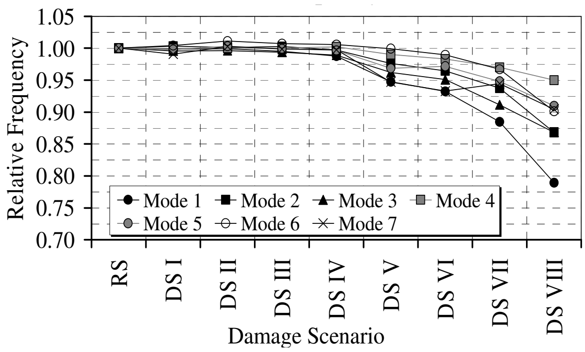


Figure 7 : Relative frequency values for ambient excitation.

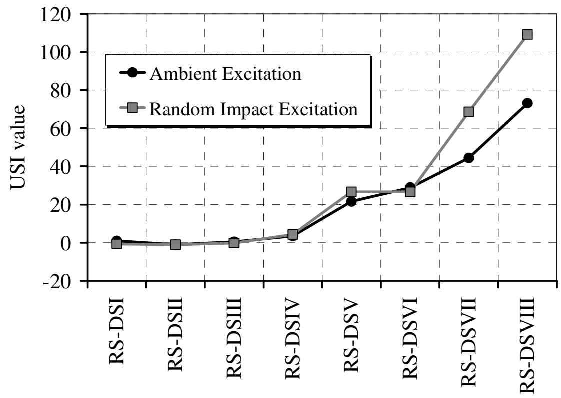


Figure 8: Unified Significance Indicator (USI) results and comparison with the RS.

The analyses carried out with modal displacements were inconclusive, as no damage location was indicated. Using the measured modal curvatures at the middle line of the arch, several methods provided reliable results. The definition of the damage location was based on three criteria with decreasing importance, see Ramos *et al.* (2010a), that allowed locating all the cracks in the vicinity of the observed experimental positions. Finally, with a combination with the Finite Element Model Updating Method, it was possible to conclude that the updating analysis did not provide accurate results, on the contrary to damage analysis using only numerical crack simulation.

3 MONITORING WITH OPERATIONAL MODAL ANALYSIS: TWO CASE STUDIES USING CONVENTIONAL SENSORS

3.1 Mogadouro Clock Tower

The Mogadouro Clock Tower is located inside the castle perimeter of Mogadouro, a small town in the Northeast of Portugal. The tower was built after the year 1559. It has a rectangular cross section of 4.7 × 4.5 m^2 and a height of 20.4 m. Large granite stones were used in the corners and rubble stone with thick lime mortar joints were used in the central part of the walls. The thickness of the walls is about 1.0 m. In 2004, the tower was severely damaged, with large cracks, material deterioration and loss of material in some parts. A geometrical survey of the structure was performed and the existing damage was mapped. Conservation works carried out in 2005 reinstated the tower safety, including lime grout injection for the consolidation of the walls, replacement of material with high level of degradation, filling of voids and losses, and installation of ties (or a steel belt) at two levels.

Dynamic modal identification tests were performed before and after the works, see Figure 9. The same test planning was adopted in the two conditions by using the same measuring points. Table 3 presents the first seven estimated natural frequencies and damping ratios, see Ramos *et al.* (2010b) for more details. An analysis of the two structural conditions shows that, on average, the frequencies increased 50% with the works, while the damping decreased 40%. Concerning the modal displacements, local protuberances could be observed in the areas close to the cracks and in the upper part of the tower, before the works. This is due to the presence of severe damage. On the contrary, the structure behaves monolithically after the conservation works.

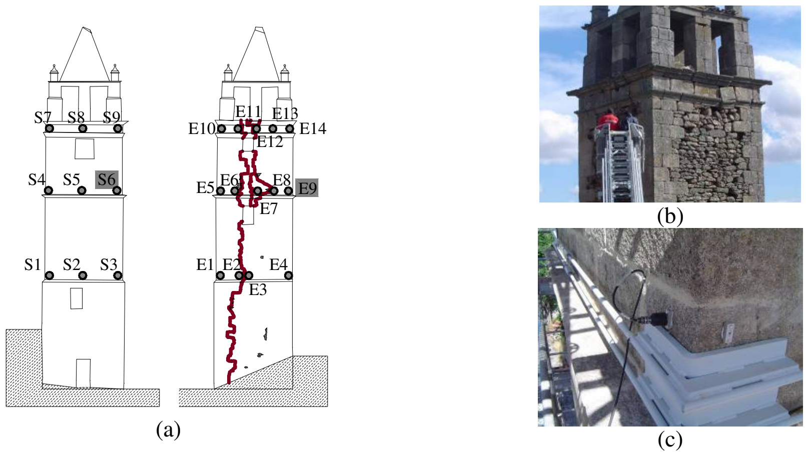


Figure 9: Dynamic tests and examples of sensor locations: (a) crack pattern and measuring points in the South and East façades, respectively; measurements (b) before and (c) after rehabilitation works.

Table 3: Mogadouro Clock Tower: Dynamic response before and after the rehabilitation.

Mode Shape	Before ω [Hz]	Before CV_ω [%]	After ω [Hz]	After CV_ω [%]	Δ_ω [%]	Before ξ [%]	Before CV_ξ [%]	After ξ [%]	After CV_ξ [%]	Δ_ξ [%]
1st	2.15	1.85	2.56	0.21	+19.28	2.68	219.51	1.25	0.13	–53.26
2nd	2.58	1.05	2.76	0.30	+6.70	1.71	94.02	1.35	0.17	–21.00
3rd	4.98	0.69	7.15	0.27	+43.67	2.05	65.33	1.20	0.14	–41.32
4th	5.74	1.56	8.86	0.47	+54.37	2.40	24.27	1.31	0.13	–45.72
5th	6.76	1.13	9.21	0.21	+36.13	2.14	31.74	1.16	0.12	–45.65
6th	7.69	2.94	15.21	2.24	+97.87	2.33	55.98	2.54	0.24	+9.11
7th	8.98	1.21	16.91	1.40	+88.27	2.30	46.39	1.49	0.23	–35.07
Average values	–	1.49	–	0.73	+49.47	2.23	76.75	1.47	0.17	–40.34*

* - Average value calculated only with negative differences

The challenge now is to verify if the cracks were stabilized with the intervention by means of a dynamic monitoring system. During 2006 and 2007, and after the conservation works, a dynamic monitoring system was installed in the tower. Three piezoelectric accelerometers connected to an USB data acquisition card recorded each hour ten minutes of ambient vibrations. In parallel a combined sensor recorded the ambient temperature and relative air humidity. This task was performed in several campaigns (test series), in different periods of the year. The data series were analyzed and they allowed concluding that the environmental effects significantly change the dynamic response of the structure. Mainly, the water absorption of the walls in the beginning of the raining seasons changes the frequencies about 4%. The significant influence of moisture inside the walls on the dynamic response of masonry structures is not reported in literature, as the changes are always attributed to others environmental effects or loading conditions, such as temperature and excitation level. The series presented in Figure 10 were recorded during October and November, 2006, which is linked to the first strong raining event at the site. As no damage was observed in the structure, the humidity influence can be observed by a shift in the linear relation between frequency and temperature by a transition series.

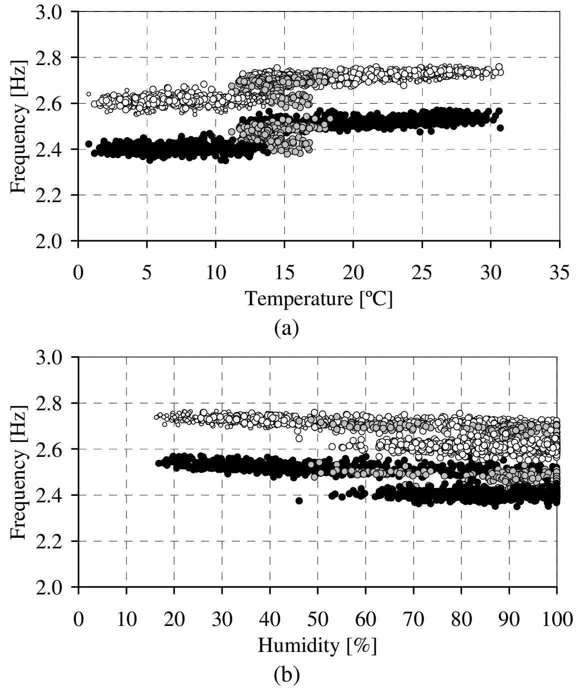


Figure 10: Environmental effects: (a) temperature; (b) relative air humidity.
Note: Black dots correspond to the first frequency, white dots to the second frequency, and grey dots to the transition period of each frequency.

The ambient temperature, the relative air humidity and the excitation level, by means of the Root Mean Square (RMS) of the signal, were correlated to the resonant frequencies. Multiple linear regression models were compared with Auto Regressive outputs with eXogeneous input models, also known as ARX models, Ljung (1999), in order to evaluate the environmental and loading effects. In general, see Ramos *et al.* (2010b), the model is able to replicate the frequency variation, but does not takes into account the water absorption phenomenon, because the relative humidity variation does not totally represent that change. The results indicates that ARX models can simulate the natural frequencies, but sensors to measure the water absorption inside the walls are necessary in this case to better simulate the dynamic response and to detect the presence of damage. Apparently, no damage was observed by global modal parameters changes during the monitoring period reported.

3.2 Church of Monastery of Jerónimos

The Monastery of Jerónimos, located in Lisbon, is one of the most famous Portuguese monuments. The church of the monastery, Santa Maria de Belém church, has considerable dimensions: a length of 70 m, a width of 40 m and a height of 24 m. The main nave of the church was tested using output-only modal identifications techniques, which provided the modal parameters: resonant frequencies, mode shapes and damping coefficients. Two techniques were applied to compare the experimental dynamic parameters obtained and have more accurate results. The Enhanced Frequency Domain Decomposition (EFDD), Brincker *et al.* (2000), and the Stochastic Subspace Identification (SSI) method. Thirty points on the top of the main nave were selected to measure the acceleration response. Ten points were located on the top of the external walls with the purpose of measuring the nave boundaries and also the global dynamic response of the church. The other points are located either on the top of the columns or on the top of the vault keys.

Table 4 summarizes six natural frequencies, damping ratios and MAC values estimated by two different output-only system identification techniques. The natural frequencies range from 3.7 to 12.45 Hz and no significant differences could be found between the two methods. For the damping coefficients, differences up to 140% can be observed. The MAC values are only higher than 0.95 for the first two modes as a consequence of the difficulty in exciting this heavy structure. The modes can be found in Ramos *et al.* (2010b). Due to the high level of complexity of the structure a beam FE model was manually tuned to the dynamic experimental results. The adoption of a relatively "simple" model is due to the fact that the model was used in subsequent non-linear time integration analysis, including a parametric study using a set of mechanical properties, different return period scenarios and strengthening possibilities, see also Roque (2010).

Table 4: Monastery of Jerónimos: Comparison of the estimated modal parameters of the main nave.

Mode Shape	ω [Hz]		ξ [%]		MAC
	EFDD	SSI	EFDD	SSI	
1^{st}	3.69	3.68	2.34	1.26	0.99
2^{nd}	5.12	5.04	1.11	2.68	0.92
3^{rd}	6.29	6.30	1.00	0.82	0.67
4^{th}	7.23	7.29	0.77	1.44	0.67
5^{th}	9.67	9.65	1.10	1.45	0.62
6^{th}	12.45	12.51	1.25	1.19	0.71

Since April of 2005, a dynamic monitoring system was installed in the church within the scope of the Euro-Indian research project "Improving the Seismic Resistance of Cultural Heritage Buildings". The monitoring system is composed of two strong motions recorders with 18 bits AD converters connected to two tri-axial force balance accelerometers. One accelerometer was installed at the base of the structure near the chancel and a second accelerometer was installed at the top of the main nave (extrados), between two consecutive columns, and in the location with higher signal levels obtained in the dynamic modal identification analysis. The two recorders are connected by an enhanced interconnection network, which allows a common trigger and time programmed records. The monitoring task is mainly processed by the master recorder, with a trigger armed for low level signals corresponding to micro tremors occurring at the site. In parallel, dynamic data are registered with selected periodicity. The dynamic monitoring system is complemented by a static one, which measures temperatures and rotation of the columns at several points in the structure. Additional sensors for relative air humidity and wind velocity were recently added to this system. For the study of the environmental and loading effects all data acquired in the strong motion recorders were used. 1300 events were acquired: 28% of which correspond to programmed events with ten minutes of total sampling duration, and 72% correspond to triggered events, with an average sampling duration of one minute and fifteen seconds.

To estimate the modal parameters, the procedure described previously for the Mogadouro Clock Tower was again adopted. The majority of the triggered events occurred during working hours, due to the road traffic, special events inside the church (like mass or concerts), and minor earthquakes. The results indicate that the temperature effect is significant for the nave. A trend for a bilinear relation between temperature and frequency was found, with an apex for a temperature about 18ºC. This trend occurs for all the estimated frequencies. As no continuum series were recorded in the monitoring system, modelling of thermal inertia is difficult and only static regression models were used. From the observed bilinear trend, two linear regressions for temperature values lower and higher than 17.5ºC were adopted. Figure 11a shows the static models for the first estimated frequency by correlating the two quantities, temperature and frequency with the 95% (±2σ) confidence intervals. The results show that the bilinear static model follows the evolution of the frequencies but a significant number of outliers can be observed in Figure 11b, where the residuals between the first natural frequency and the simulated frequency are shown. This indicates that others environmental effects need to be studied to better model the dynamic response.

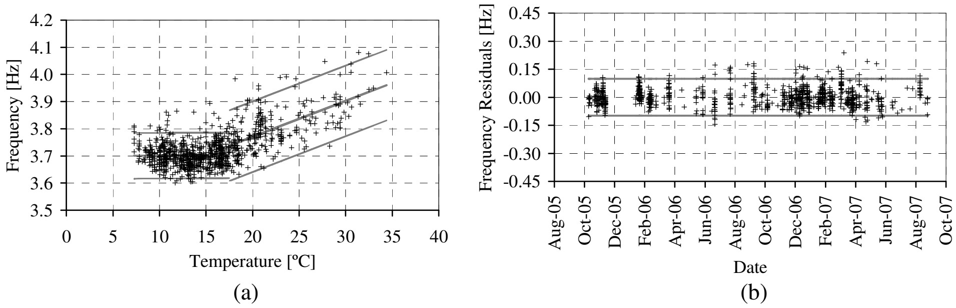


Figure 11: Static regression for the first mode: (a) temperature influence; (b) residuals history.

Finally, it should be stressed that in 12 February, 2007, at 10:35 am a 5.8 magnitude earthquake, with a Modified Mercalli intensity V, occurred in the Southwest of Lisbon. The permanent staff of the monument felt the ground shake. No visitors were inside the church because on Mondays the church is closed to public. The strong motions recorders acquired the signals and the peak of the frequency contents of the elastic response spectra was in the range of the estimated natural frequencies, but the estimated natural frequencies did not suffer any significant shift, as can be observed through Figure 11b. Therefore, no damage in the structure occurred due to this minor earthquake.

4 OPERATIONAL MODAL ANALYSIS USING WIRELESS PLATFORMS

Micro-Electro-Mechanical Systems (MEMS) with low power and high frequency transceivers have been recently integrated in sensor prototypes, called “motes”, to reach four attributes: sensing, processing, communication and actuation. A mote is an autonomous, compact device, and a sensor unit that has the capability of processing and communicating wirelessly. One of the biggest strengths of motes is that they can form networks, known as Wireless Sensor Networks (WSN), which allows the units to cooperate between themselves. The use of wireless technology with embedded MEMS for structural monitoring was first proposed by Kiremijdian *et al.* (1997), aiming at the integration of wireless communications with sensors in order to develop a near real time monitoring system. Due to fact that masonry structures are difficult to excite and due to the low resolutions capabilities of the commercial MEMS, only one application was found in the literature, namely the Aquila Tower in Italy, see Ceriotti *et al.* (2009).

Here, conventional wired based sensors with high sensitivity piezoelectric accelerometers model PCB 393B12 are used as reference. For DAQ purposes, the NI-USB9233 board with an ADC resolution of 24 bits was selected. In the case of the wireless based systems, the Crossbow technology was chosen, as it offers inexpensive solutions with low powering boards and platforms with embedded micro-accelerometers. For comparison purposes, Table 5 presents the characteristics of the micro-accelerometer ADXL202 embedded in the Crossbow platforms and the piezoelectric accelerometer PCB 393B12.

4.1 Dynamic response of an inverted pendulum

A Single Degree-of-Freedom (SDOF) structure represented by an inverted pendulum is one of the simplest examples used by civil engineers to explain the fundamentals of structural dynamics. Here, an inverted timber pendulum with 1.70 m height and with steel plates in its top and base is used. To perform a complete dynamic characterization of the pendulum, three wire-based and three wireless with embedded MEMS platforms accelerometers were used (the

MEMS sensors were programmed to perform measurements only in one axis). For comparison purposes the wired and wireless systems were set to run concurrently. Initially, the performance with respect to an acceleration time series was studied using tests under random excitation and ambient noise. Figure 12 shows the recorded signal by mote 3 and accelerometer 3 in both scenarios. The results demonstrate the poor performance of the micro-accelerometers for measuring low amplitude vibrations. The maximum values and the root mean square (RMS) registered by the wireless platforms are, respectively, 3 to 6 times and 8 to 22 times lower than the values recorded by the conventional platforms with ambient noise. Similar results were obtained in the case of the RMS in random excitation, even if the time series recorded with both systems are rather similar.

Table 5: Characteristics of a MEMS and a conventional piezoelectric accelerometer.

	MEMS microaccelerometer	Piezoelectric accelerometer
Sensor Type	ADXL202JE	PCB 393B12
Channels	X, Y	X
Range (g)	±2.0	±0.5
Sensitivity (mV/g)	167 ±17%	10000
Resolution (g rms)	0.002	0.000008
Size (mm)	5.0 x 5.0 x 2.0	30.2 (diam.) x 55.6 (high)
Weight (gram)	1.6	210.0

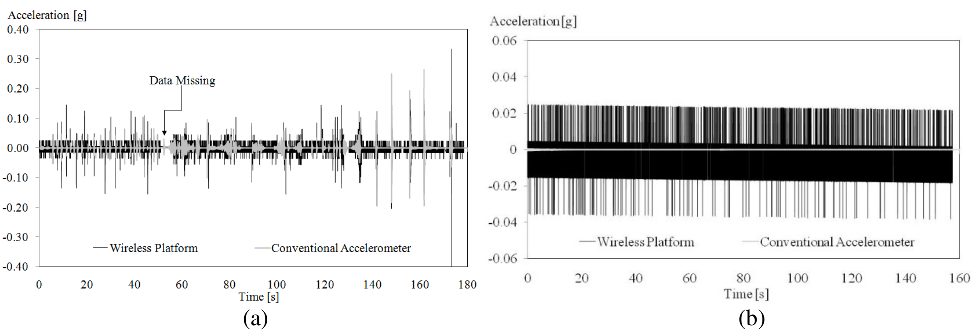
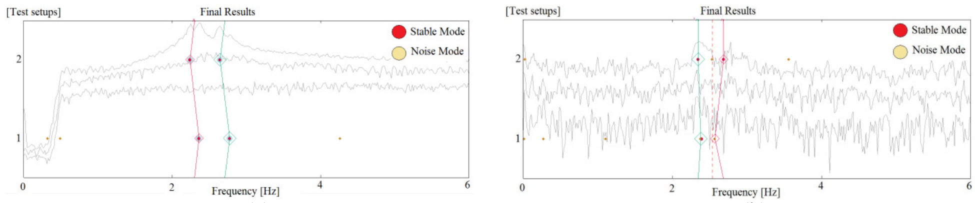


Figure 12: Time series collected by mote 3 and accelerometer 3 in the inverted pendulum tests: (a) response under random excitation in Setup 01; (b) response under ambient noise in Setup 02.

Then, the dynamic characteristics of the system were studied. For this purpose, the SSI-Data method implemented in the ARTeMIS extractor software was used. Figure 13 shows the stabilization diagram corresponding to the random excitation and Table 6 shows a summary of the results accelerometers, where f is the frequency and ξ is the damping. According to the frequency content results, the wireless based platforms give accurate results (errors of about 2% for random excitation and about 7% for ambient vibration). When the structure is lightly and randomly excited, the modal identification is easier because the stable poles are properly aligned in the natural frequencies. In the case of ambient noise the dynamic identification becomes more complicated due to the appearance of noise poles (stabilization diagrams not shown). The results related to damping tend to show a large scatter and are often unreliable. Still, no correlation was found between damping values using conventional and wireless based systems, with extremely large (and incorrect) values found with the wireless based platforms. Due to the lack of synchronization algorithms implemented for the motes, no information can be gathered on the mode shapes.

4.2 The chimneys at Paço dos Duques, Guimarães

The Paço dos Duques (Palace of the Dukes of Bragança) was built between 1422 and 1433 by D. Afonso in Guimarães, north of Portugal. Since 2002, the building suffered some conservation works, mostly related to the roofs and chimneys. The chimneys exhibited considerable damage, with one chimney requiring strengthening. Based on the previous results of the experimental tests, the use of commercial wireless platforms for structural dynamic monitoring was again explored. The dynamic response of one of the four original chimneys was studied using conventional and wireless platforms. Figure 14a shows a general view of the conservation works that were carried out and Figure 14b shows the location of the wireless platforms in the experimental tests.



(a) (b)

Figure 13: Stabilization diagrams for the analysis of the time series recorded under random excitation in the inverted pendulum tests: (a) results of the conventional wired based accelerometers; (b) results of the wireless platforms.

Table 6: Results of the experimental modal analysis of the inverted pendulum study.

		Conventional Accelerometers		Wireless Platforms			
	Mode	f (Hz)	ξ (%)	f (Hz)	ξ (%)	f_{Error} (%)	ξ_{Error} (%)
Random excitation	1	2.30	1.45	2.35	3.57	2.13	59.39
	2	2.71	1.57	2.68	2.94	1.12	46.60
Ambient	1	2.26	0.82	2.41	9.82	6.22	--
	2	2.63	2.12	2.83	10.42	7.07	--

(a) (b)

Figure 14: Chimneys at Paço dos Duques: (a) recent conservation works; (b) sensors location.

The advantages of using wireless platforms were clear in this case study, as their use is much simpler. The DAQ process was also easier allowing safer work in a zone with difficult access. Table 7 shows the results of the identified frequencies using conventional accelerometers and wireless platforms. The results show very small differences in the identified frequencies obtained by using the conventional and the wireless platforms (maximum error is 3.5%). Again, inconclusive results are obtained with respect to damping. When the tests are performed with ambient noise (results not shown) similar frequencies could be identified, again with more difficulties due to the spurious poles appearing in the stabilization diagrams.

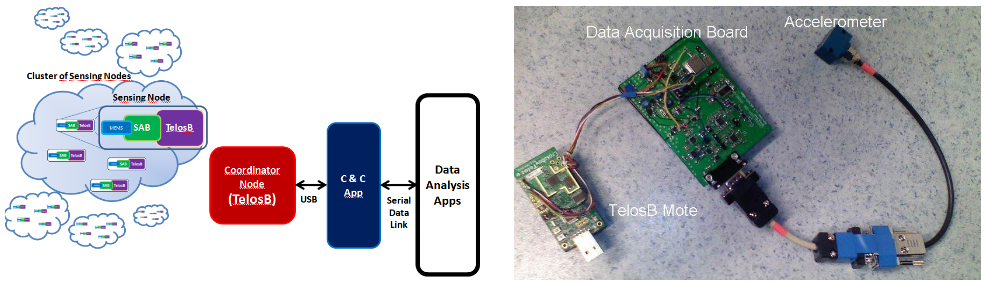
Table 7: Dynamic response of the chimney at Paço dos Duques.

	Conv. Accelerometers		Wireless Platforms			
Mode	f (Hz)	ξ (%)	f (Hz)	ξ (%)	f_{Error} (%)	ξ_{Error} (%)
1	1.69	1.34	1.68	1.61	0.60	16.77
2	1.77	4.22	1.71	0.72	3.51	--

4.3 A novel wireless sensor network (WSN) system

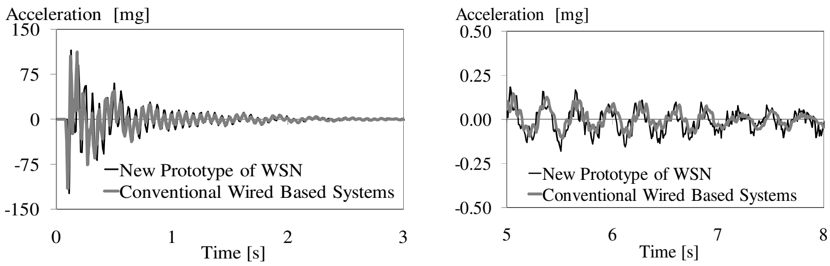
A novel WSN system for Structural Health Monitoring (SHM) that overcomes most of the limitations already identified was developed, see Aguilar (2010) for more details. The WSN: (a) enables high resolution measurements, through a custom designed 24 bits resolution signal acquisition board interfacing a MEMS accelerometer and a TelosB mote; (b) is based on Commercially available Off-The-Shelf (COTS) technologies, in terms of hardware platform, communications protocols and operating system; (c) guarantees tight time synchronization between sensor measurements, through a standard-based, in-network synchronization mechanism; (d) is ready to be scaled up to monitor larger structures in an effective way, i.e. with a consistent time correlation of data samples.

The WSN system architecture is illustrated in Figure 15a. The WSN is composed of several clusters of Sensing Nodes, organized in a cluster-tree topology. Each Sensing Node is composed of a TelosB node, a signal acquisition board and a MEMS acceleration sensor. A small-scale prototype system was conceived with four sensing nodes equal to the one shown in Figure 15b, allowing validation of this solution against a reference wired system. As shown in Figure 16, even for signals with amplitudes below 0.25 mg, the records from the new WSN platform and the conventional wired-based accelerometers presented a remarkable degree of similarity.



(a) (b)

Figure 15: Novel WSN: (a) snapshot of the system architecture; (b) sensing node.

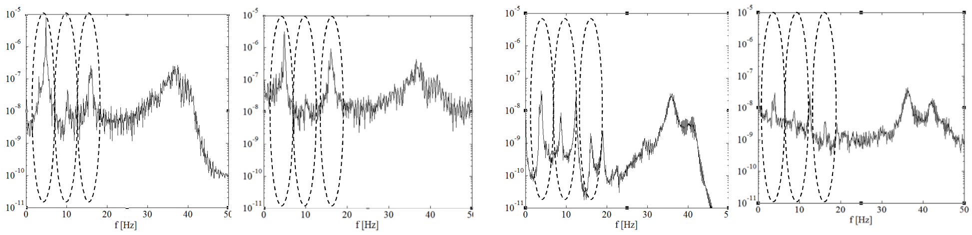


(a) (b)

Figure 16: Time domain series: (a) High amplitude excitation recordings;
(b) low amplitude excitation recordings.

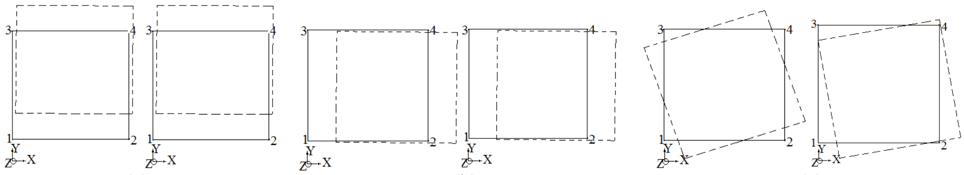
The next stage consisted on verifying the accuracy of the frequency content of the acquired signals. Considering the same pair of sensors located at the top of the pendulum and 30 s of sampling time, experiments were carried out in two excitation scenarios: random impact (vibrations amplitudes below 5 mg) and ambient noise (vibrations amplitudes below 1.5 mg). The Welch Spectrums of the time series records were calculated and are shown in Figure 17. The results evidenced the high accuracy of the resultant frequency domain spectrums calculated from the records of the new developed system. With this respect, even in the case of ambient noise tests, outstanding similarities in the content of frequencies were detected.

The last stage of the experimental operational modal analysis process consists on the estimation of the dynamic properties by means of their natural frequencies, damping coefficients and mode shapes. For this purpose, a more refined data processing method was used which consisted on the evaluation of the time series recordings with 3 conventional (wired) and 3 new (wireless) sensors located at the top of the pendulum using the SSI method. Figure 18 shows the results of this analysis for the case of random excited system. The first two mode shapes of the structure were identified with no uncertainties, with a difference in measured frequencies about 2%. There was a slight difference in the 3rd mode shape, which needs further investigation.



(a) (b)

Figure 17: Frequency domain results – Tests new WSN Platform: (a) excited tests; (b) ambient tests. In each figure, left are the wired system results and right are the WSN prototype results.



(a) (b) (c)

Figure 18: Figure 10. Experimental modal analysis under excited tests: (a) 1st mode; (b) 2nd mode; (c) 3rd mode. In each figure, left are the wired system results and right are the WSN prototype results.

5 CONCLUSIONS

Conservation and rehabilitation of historical structures are still a challenge to modern practitioners even if significant research advances have occurred in the last decades. Time shows that many historical masonry constructions collapsed due to accidental actions such as earthquakes, but not only exceptional events affect historical constructions. Fatigue and strength degradation, accumulated damage due to traffic, wind and temperature loads, soil settlements and the lack of structural understanding of the original constructors are high risk factors for the architectural heritage. Therefore, structural analysis and safety assessment of historical masonry buildings are often necessary. For such purposes, structural monitoring and damage identification at the earliest possible stage play a major role.

This paper firstly introduces a damage identification approach based on vibrations measurements for masonry structures and a selection of methods available in the literature. The methods were combined in a qualitative way with the aim to capture the difficult phenomena and in assisting the decision of damage identification analysis. The methods can be easily applied to masonry structures and they give information about the detection (Level 1), the localization (Level 2) and, possibly, the assessment (Level 3) of damage. Then, a methodology based on operational modal analysis was presented for masonry structures, aiming at detecting damage at an earlier stage. The methodology comprises four phases, namely data collection, simplified health monitoring, detailed health monitoring and local non-destructive testing. As an illustration, the first two phases were applied to two complex Portuguese monuments using conventional sensors. For these monuments, the modal identification results, the development of structural model with model updating techniques, for subsequent assessment with FE models, the installation of monitoring systems and the automatic parameter retrieval were presented. From the experience with the two cases, the proposed methodology for damage identification seems to be useful and applicable to masonry structures, especially to complex historical constructions. In particular, the frequency monitoring seems to be a reliable quantity for damage detection. Finally, a new platform for performing operational modal analysis of structures, based on wireless technology with embedded MEMS sensors, was also tested. Commercial WSN platforms available in the market were chosen for comparison purposes against widely used conventional wired based systems. The results showed that the WSN platforms have poor performance with respect to the acceleration time series recorded, due to the low resolution of the micro-accelerometers and DAQ systems embedded. The wireless platforms showed also very poor performance for the detection of modal shapes due to the lack of synchronization algorithms. In the case of frequency detection, reliable results were obtained especially when the systems were randomly excited. A new prototype system is presented and seems to resolve most of the deficiencies in commercial WSN platforms.

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