SHM-based numerical modelling of masonry constructions in the presence of thermal variations

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Abstract

Long–term ambient vibration monitoring campaigns show that environmental parameters (such as temperature, humidity, wind speed and direction) can influence the structures' static and dynamic behaviour. In particular, thermal variations can affect the modal characteristics of ancient masonry constructions. The present work presents a procedure combining experimental and numerical steps to monitor, assess and model the dynamic behaviour of masonry structures subjected to thermal loads. The procedure is tested and validated through two numerical examples and the simulation of a full-scale masonry structure, the Mogadouro clock tower in Portugal, monitored with accelerometers and temperature and humidity sensors.

1. **Introduction**

The influence of thermal variations on the mechanical behaviour of materials with low tensile strength is a well-known phenomenon that has been observed in many different cases. Practical examples range from masonry bridges and monuments, in which temperature changes typically induce displacement and stress variations [1] and modify the crack distribution [2], [3], to stone arches, whose dynamic properties closely depend on daily and yearly thermal fluctuations [4]. In instances such as the thermo-mechanical process undergone by the refractory structures used in the iron and steel industry [5], the thermal variation is so high that the dependence of the material characteristics on temperature cannot be disregarded [6].

Several Structural Health Monitoring (SHM) campaigns have shown that thermal variations influence not only the static behaviour of masonry structures but also their dynamic features. For instance, slender masonry structures, like towers and bell towers, generally exhibit frequency upshifts for increasing temperature, unlike concrete and steel structures, which usually show an opposite trend. Such behaviour can be attributed to the stiffness increment associated with the micro-cracks closure and the reduction in moisture content induced by higher temperatures. An inverse trend can be observed for low temperatures, typically around 0°C, because of the formation of ice in the micropores and voids of the masonry with temporary increase of the material stiffness and natural frequencies. Although nonlinear frequency-temperature relationships can emerge in case of complicated temperature-driven mechanisms [7], many studies have shown that the typical

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correlation between frequencies and temperature is bilinear, with a vertex close to the freezing point [8], [9], [10], [11], [12], [13].

Thermal variations in masonry structures may be responsible for other phenomena at a local level, such as growing thrusts in arches, increase of tilting movements or loss of tension in metallic tie-rods, thus resulting in global softening effects and frequency downshifts [12], [14]. These phenomena typically affect high-frequency mode shapes; nonetheless, negative frequency-temperature correlations for lower modes have been observed in several cases, such as the cathedral of Milan (Italy) [15], the Consoli Palace in Gubbio (Italy) [7] and the Basilica of Santa Maria di Collemaggio in L'Aquila (Italy) [16]. While in the first two cases the decrease in frequencies as the temperature increases was attributed to the presence of tie rods across the structure, in the last case the inverse frequency-temperature correlation was attributed to the global stiffness reduction induced by the cross-laminated timber roof expansion at higher temperatures.

In addition, it must be stressed that the behaviour of historical masonry structures can also be governed by the interaction of different macro-elements, i.e. portions of masonry macroscopically homogeneous and bounded by lines of weak connection. This means that the thermally induced relative displacements of such elements can modify the connections between masonry portions and significantly affect the modal properties, causing unexpected trends or even abrupt shifts in the natural frequencies. The case study of the Ghirlandina tower in Modena (Italy) represents a relevant example in this regard, as the lowering of temperature values led the tower to move towards the cathedral, with consequent alterations of its boundary conditions as well as its intrinsic modal features [17].

From a numerical point of view, two viable strategies have been developed for simulating, predicting and possibly removing – from the collected field data – the thermal effects in masonry materials, namely data-driven approaches or finite element simulations. The former has been the object of extensive investigations, giving rise to several new statistical ways to address the problem ([18], [19], [20], [21], [22], [23], [24], [25], [26], [27], and [28]). However, these approaches are often unable to explain the evolution of natural frequencies over time by resorting to a unique independent variable (the temperature), as more predictors may influence at the same time the dynamic behaviour of masonry artefacts [29]. Conversely, studies based on Finite Element (FE) analyses aimed at replicating the thermal effects on masonry structures' behaviour own a great potential and are relatively scarce. Noteworthy examples are the uncoupled thermo-mechanical simulation used in [30] to explain the presence of a dominant crack in the South wall of Torre Astesiano (Italy), and the FE model developed in [31] to analyse the structural condition of the Basilica of San Vitale in Ravenna (Italy) taking into account the effects of seasonal thermal variations.

An in-depth investigation on the numerical modelling of the static behaviour of masonry solids subjected to thermal loads was addressed in [32], where the constitutive equation of masonry-like materials [33] is generalized to the non-isothermal case to consider thermal dilatations due to temperature variations. The generalized equation has been implemented in NOSA-ITACA (www.nosaitaca.it/software/), a FE code for the structural analysis of masonry constructions in non-isothermal conditions. This tool has been exploited to analyse the influence of the thermal variations on the dynamic characteristics of masonry constructions in [34], which describes a numerical procedure for considering the effects of thermal stresses on both natural frequencies and mode shapes. The procedure has been recently applied to two numerical case studies consisting of a circular arch and a segmental arch masonry bridge [35].

The present work originates from the necessity to provide a procedure combining both experimental and numerical stages in order to monitor, assess and model the dynamic behaviour of masonry structures subjected to thermal loads. The procedure, grounded on the NOSA-ITACA code, is tested and validated through two numerical examples featuring analogous geometry but different stiffening elements, as well as through the simulation of a full-scale masonry structure, the Mogadouro clock tower in Portugal, monitored with accelerometers, and temperature and humidity sensors.

The paper is organised as follows. Section 2 describes the procedure put into practice for reproducing and evaluating the dynamic behaviour of masonry structures subjected to thermal loads. Section 3 presents a first numerical validation of the procedure by resorting to the numerical case study of a barrel vault on lateral walls, with and without tie-rod constraints. Section 4 analyses and discusses the effects of thermal changes on the dynamic behaviour of a real benchmark masonry tower. Finally, Section 5 summarises the main conclusions of this work.

2. Procedure's outline: experimental and numerical analysis

The modal properties (i.e. frequencies, mode shapes and damping ratios) of old masonry constructions can be affected by environmental parameters (such as temperature, humidity, wind speed and direction, etc.) in different ways depending on various factors: structural typology ([15], [22], [36], [37]), constituent materials and the relative mechanical characteristics ([9], [38]), indoor and outdoor temperature distribution [39], and existence of stiffening structural elements ([7], [15]). Among the various environmental parameters listed above, temperature appears to be the one that most affects the behaviour of masonry structures.

In order to simulate the global dynamic behaviour of a monitored structure, considering the influence of thermal loads, it is necessary to define a proper procedure encompassing both experimental and numerical phases as summarized in the following steps:

a. To execute a survey with a preliminary ambient vibration test (AVT) to estimate the structure's modal properties (reference dynamic properties) and detect the presence of possible cracks;

b. To measure the internal and external structure's temperature during the preliminary AVT (reference temperature ϑ_0);

c. To conduct an experimental long-term monitoring campaign, comprising modal tracking, and continuous temperature measurements;

d. To build and calibrate the FE model of the structure via a nonlinear model updating technique to estimate the constituent materials' mechanical properties at the reference temperature ϑ_0;

e. To calculate the numerical frequencies and mode shapes of the structure for distinct temperature values detected during the experimental campaign.

The first three steps are related to the experimental structural health monitoring; whereas steps d. and e. are related to the numerical counterpart simulations, hence they can be achieved by means of a FE code able to model the nonlinear behaviour of the constituent materials and taking into account the effects of thermal loads on the structure's dynamic behaviour.

In the present work, all the numerical simulations are performed using NOSA-ITACA, a code developed by ISTI-CNR for the analysis and calibration of masonry structures. Here, masonry is modelled as an isotropic homogeneous nonlinear elastic material with zero or low tensile strength σ_t and infinite or bounded compressive strength σ_c [5], [33], and allows to assign thermal loads.

Using NOSA-ITACA, the natural frequencies and mode shapes of a masonry structure can be estimated, considering the crack distribution due to external loads, through a linear perturbation analysis. This numerical procedure, proposed in [40], [41], [34], consists of two steps. In the first step, the equilibrium problem of the structure discretized into finite elements and subjected to assigned loads and boundary conditions is solved through a Newton-Raphson iteration scheme, obtaining its displacements, stress and strain fields. In the second step, the tangent stiffness matrix, K^T, is calculated by using the derivative of the stress with respect to the strain [5] obtained in the previous step, and the generalized eigenvalue problem (modal analysis) is solved using matrix K^T in place of the elastic stiffness matrix. The procedure considers the effects of the stress field and cracks distribution within the structure on its natural frequencies and mode shapes.

An automated nonlinear model updating procedure [42] allows matching the numerical and experimental frequencies and mode shapes, as well as calibrating the model's free parameters, by coupling linear perturbation and modal analysis as described above. Given the structure under examination and the mechanical properties of its constituent materials, together with the kinematic constraints and loads acting on it, the nonlinear model updating consists in solving an optimization problem by assuming that the tangent stiffness K^T and mass M matrices are functions of the parameter vector x, containing Young's moduli, tensile strengths and density masses of the constituent materials. The goal is to determine the optimal value of x, which minimizes the following objective function $\Gamma(x)$ that involves q experimental frequencies to match:

$$\Gamma(x)= \sum_{i=1}^{q} w_i^2 [f_{i,\exp} - f_i(x)]^2$$

The method implemented to find the optimal value of x is based on an adaptive global polynomial interpolation of the objective function, which guarantees good global convergence properties avoiding the convergence to local minima [42].

3. Numerical examples

This section aims to demonstrate the ability of the numerical algorithms implemented to capture the dynamic properties of a masonry structure as a function of the temperature, under varying boundary conditions. For this purpose, two trial examples are considered hereafter, each featuring as a case study a masonry barrel vault on lateral walls subjected to a given thermal load history. The difference between the structures lies in the lack or presence of constraining steel tie rods.

3.1 A barrel vault on lateral walls

The first example is the barrel vault on lateral walls sketched in Figure 1. The structure, fully clamped at the base of the walls, consists of a 4.0 m long vault with a thickness of 0.20 m, a mean radius of 1.5 m and a span of 3.0 m, resting on two lateral walls of 2.5 m height and 0.4 m thickness. The structure is discretized into 1280 thick curved shell elements, element no. 10 of the NOSA-ITACA library [43], resulting into a final model with 1407 nodes and 8442 degrees of freedom.

The structure is subjected to its self-weight and uniform (*case a*) or linear (*case b*) temperature variations through the elements' thickness.
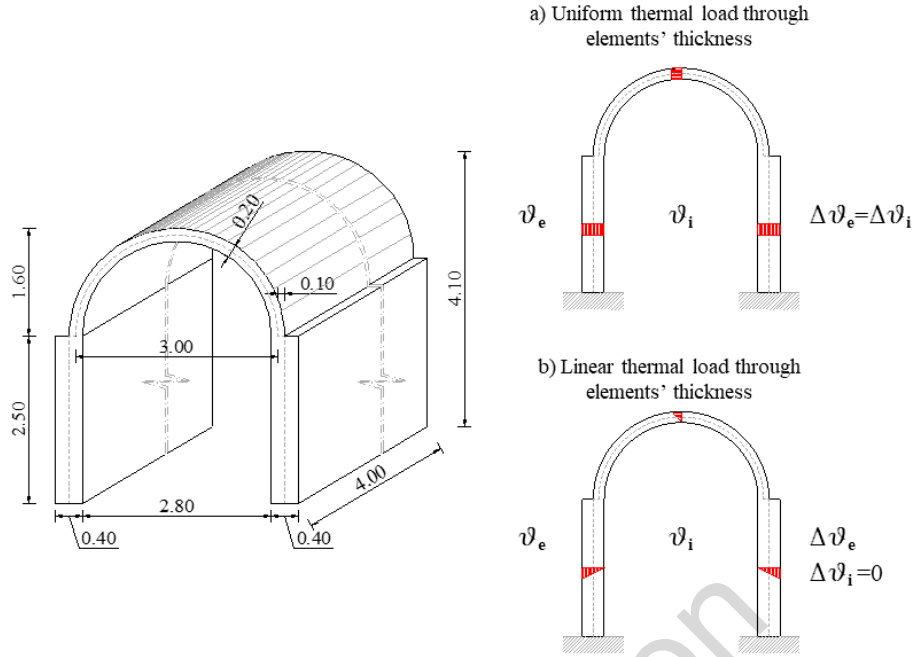
Figure 1- Geometry of the structure (dimensions in meters) with through-thickness uniform (*case a*) and linear (*case b*) temperature variation distributions.

All the analyses are performed assuming that the vault and lateral walls are made of a masonry-like material with Young's modulus $E = 1.74 \cdot 10^9$ Pa, Poisson's ratio $\nu = 0.2$, density $\rho = 2100$ kg/m^3, tensile strength $\sigma_t = 10^5$ Pa, infinite compressive strength and linear thermal expansion coefficient α $=10^{-5}$ (°C)$^{-1}$. The mechanical properties of the masonry are assumed temperature-invariant. The reference temperature ϑ_0 is set equal to 20°C; ϑ is assumed varying in the interval [ϑ_1, ϑ_2] with ϑ_1=0°C and ϑ_2=35°C, respectively. The thermal loads are applied incrementally. Denoting by ϑ_i and ϑ_e, respectively, the internal and external temperature values in the structure, it follows that for *case a*, the thermal variation increments equal each other, i.e., $\Delta\vartheta_e = \Delta\vartheta_i = \Delta\vartheta = \vartheta - \vartheta_0$; whereas for *case b*, the through-thickness temperature variation is not uniform, with $\Delta\vartheta_e = \Delta\vartheta = \vartheta - \vartheta_0$ and $\Delta\vartheta_i$ equal to zero. In both cases, $\Delta\vartheta$ ranges from -20°C to 15°C.

For each temperature increment, the equilibrium problem is solved and the natural frequencies f_i (ϑ) and mode shapes ϕ_i (ϑ) (for i =1,...,4) of the structure are calculated via the linear perturbation analysis implemented in NOSA-ITACA, and the MAC values [44] between the current mode shapes and those corresponding to the reference temperature ϑ_0 are estimated.

Figure 2 summarizes the first four frequencies and mode shapes of the barrel vault without tie rods at the reference temperature ϑ_0. The first three mode shapes can be denoted as global modes as they involve the entire system (vault and walls). They correspond to a bending mode along the *X* direction (mode 1), a vertical bending mode along the *Z*-axis (mode 2) and a torsional mode shape symmetrical with respect to the ZY plane of the vault (mode 3). The fourth mode shape (mode 4) mainly concerns the vaulted part and recalls the first asymmetrical mode shape typical of arched structures [45]. It is worth noting that the frequencies calculated via linear perturbation analysis differ from those obtained in a standard modal analysis because of the microfractures – at the intrados of the keystone section

of the vault and at the extrados of the relative support sections (see also Figure 4, temperature ϑ_0) – resulting from the application of the structure's self-weight. Indeed, masonry buildings may exhibit microcracks even for permanent loads, hence their intrinsic dynamic properties will differ depending on the presence of cracked regions [46].
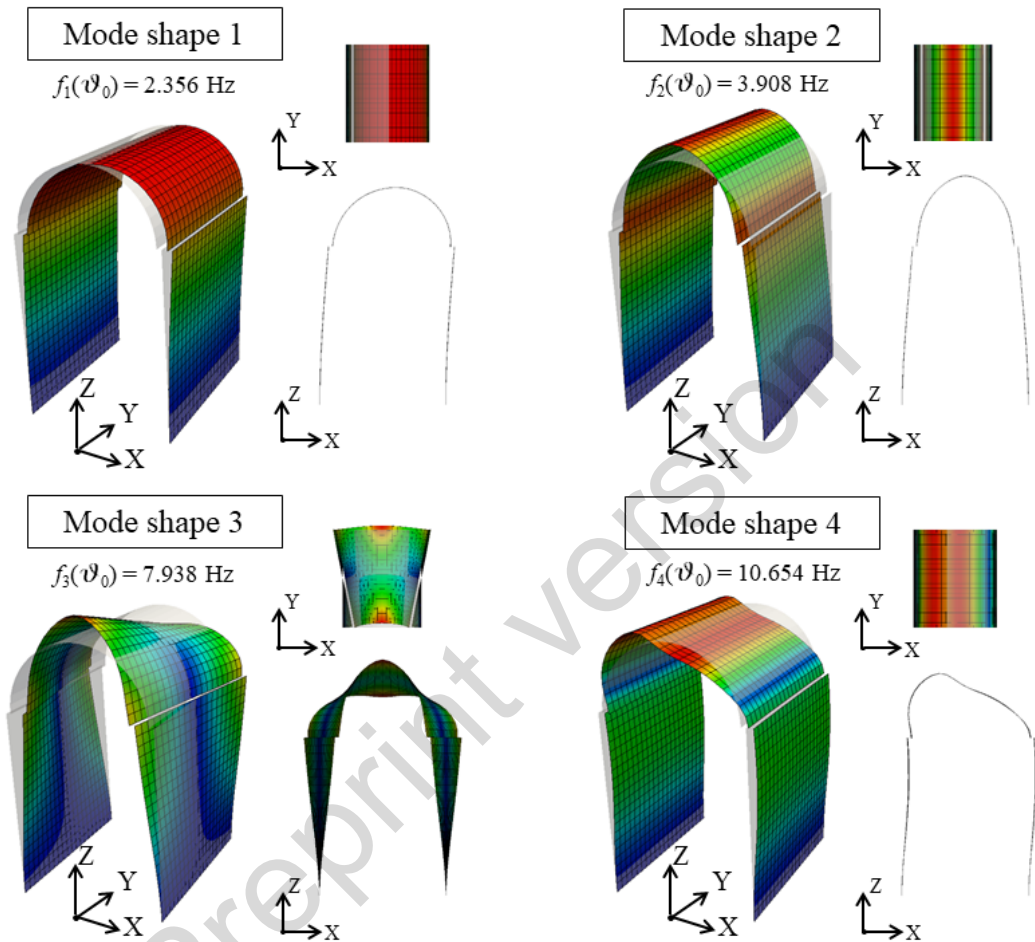

Figure 2- First four frequencies and mode shapes of the unconstrained barrel vault at reference temperature ϑ_0.

Figure 3 shows, for each mode shape and for each temperature value ϑ, the variation of frequencies and MAC values with respect to the modes estimated at the reference temperature ϑ_0 in case of uniform thermal distribution (*case a* - dashed black line) and through-thickness linear thermal distribution (*case b* - solid black line). The figure also shows the frequencies as a function of temperature (down) for *case a* (dashed black line) and *case b* (solid black line).

Figure 4 shows the norm of the fracture strain tensor for both uniform (top) and linear (bottom) thermal loads at the intrados and extrados of the structure. In this regard, it is interesting to highlight the inverse correlation between fracture strain and temperature values, which corroborates the typical behaviour observed in the literature for masonry structures exposed to increasing temperature variations.

Table 1 and Table 2 report, for both cases, the frequencies calculated at the bounds of the temperature variation range, as well as the relative frequency variation against the reference temperature scenario and the MAC values calculated, considering all degrees of freedom, between the mode shapes at ϑ_1 (or ϑ_2) and the corresponding ones at ϑ_0.
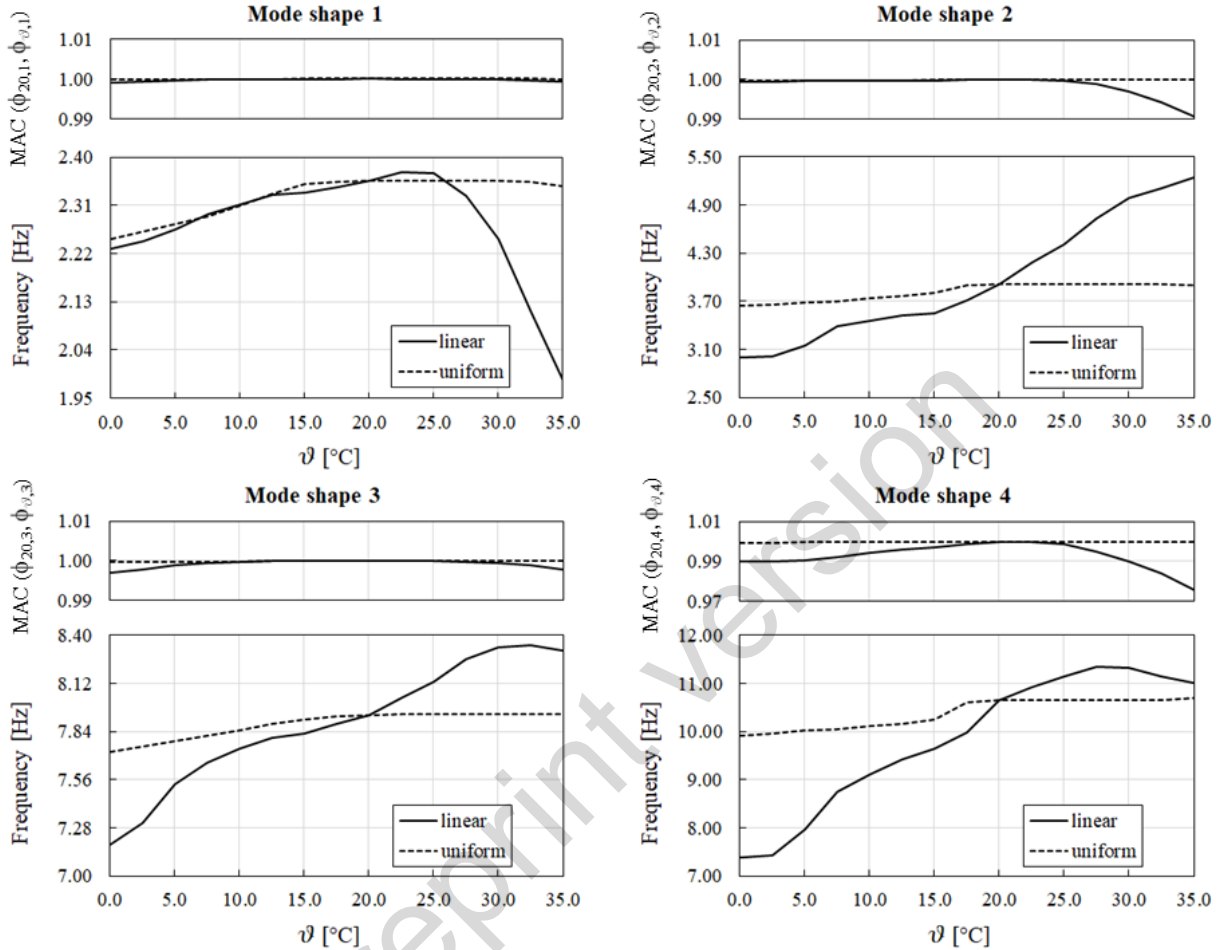

Figure 3- Evolution of the frequencies and MAC values of the system mode shapes with varying temperature in case of uniform (*case a* - dashed black line) and linear (*case b* - solid black line) thermal distribution within the walls. Note the different vertical scales used.

Table 1- *Case a*: natural frequencies of the unconstrained vault at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2); relative frequency variation $\Delta f_1 = (f_1 - f_0)/f_0$ and $\Delta f_2 = (f_2 - f_0)/f_0$; MAC values between the mode shapes $\phi_{0,i}$ (i=1…4) and the corresponding ones at ϑ_1 (MAC$_1$) and ϑ_2 (MAC$_2$)

Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.356	2.248	-4.57	0.999	2.346	-0.40	1.000
ϕ_2	3.908	3.646	-6.70	0.999	3.902	-0.15	1.000
ϕ_3	7.938	7.722	-2.71	0.999	7.944	0.08	1.000
ϕ_4	10.654	9.915	-6.93	0.999	10.706	0.49	1.000

Table 2- *Case b*: natural frequencies of the unconstrained vault at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2); relative frequency variation $\Delta f_1 = (f_1 - f_0)/f_0$ and $\Delta f_2 = (f_2 - f_0)/f_0$; MAC values between the mode shapes $\phi_{0,i}$ (i=1…4) and the corresponding ones at ϑ_1 (MAC$_1$) and ϑ_2 (MAC$_2$)

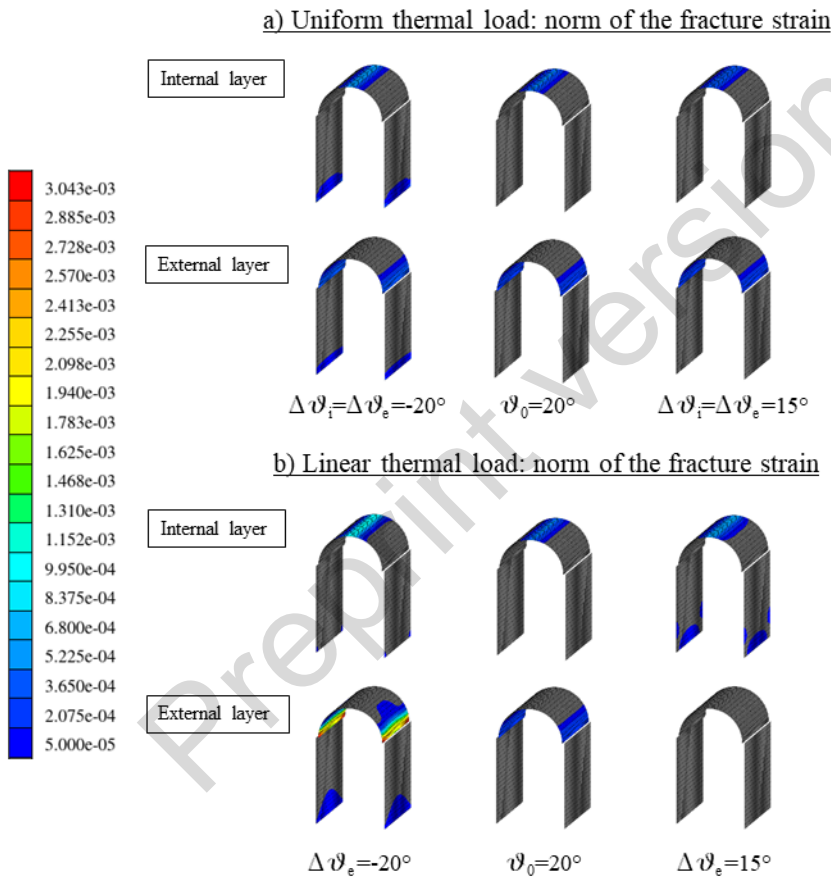| Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.356	2.230	-5.35	0.999	1.984	-15.76	0.999
ϕ_2	3.908	2.993	-23.42	0.999	5.246	34.23	0.990
ϕ_3	7.938	7.181	-9.54	0.996	8.311	4.71	0.997
ϕ_4	10.654	7.381	-30.72	0.989	11.006	3.31	0.976

Figure 4- Crack pattern in the vault under uniform (top) and linear (bottom) thermal load distributions in the configurations corresponding to the reference temperature ϑ_0 and to the lower and upper bounds of the temperature variation range ϑ ([ϑ_1, ϑ_2]). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

The analysis of the previous figures and tables demonstrates that in the case of uniform thermal load, for temperatures greater than 20 °C (ϑ_0), the vault undergoes no significant variation in terms of frequencies and mode shapes (0.40% for the fourth frequency). This outcome can be explained by observing the crack pattern reported in Figure 4 (top plots), showing that the distribution of the cracks

remains substantially unchanged and localized in the same areas as the temperature exceeds the initial reference value. Conversely, when the temperature decreases, the frequencies decrease likewise because of the development of additional cracks at the base of the lateral walls and the growth of fracture areas at the vault's keystone and springing sections.

In the case of linear thermal load, the system's dynamic characteristics differ substantially over the temperature interval. The first frequency shows a linear positive trend like *case a* as the temperature increases up to 25 °C and then undergoes a sudden decay for further increases in the thermal load. The second and third frequencies feature instead an increasing trend up to about 30 °C and are characterized by a very high gradient compared to the corresponding frequencies of the vault with uniform thermal load (Table 2). Finally, the fourth frequency increases until 30°C and only slightly decreases therefrom. This different behaviour likely depends on the fact that the extension of the cracked regions located at the base of lateral walls and at the keystone and springing sections of the vault grows for lower temperatures. In contrast, as the temperature increases, the distribution of thermal loads facilitates the closure of some cracks as the material expands, contributing to stiffer areas (Figure 4).

Regardless of the thermal load distribution, the mode shapes show very little sensitivity to thermal variations, as evidenced by the MAC values, which are close to the unit over the entire temperature interval, thus confirming the experimental results observed in [15], [16], [22].

3.2 A barrel vault on lateral walls constrained by steel tie rods

The second example under investigation is the barrel vault described above reinforced with two steel tie rods having a square section of 0.01 m edge (Figure 5). The FE model of the structure now comprises 1288 thick shell and beam elements (elements no. 10 and 9), respectively, of the NOSA-ITACA library [43], for a total of 1417 nodes and 8502 degrees of freedom. The beams modelling the tie rods are connected to the shell elements via multipoint constraint relations so as to be activated when the masonry structure is in equilibrium under self-weight. The two tie rods are preloaded by a tensile force Np of variable amplitude. As done in the previous section, two different temperature distributions are considered: a uniform thermal load (*case a*) and a linear thermal load (*case b*) through the elements' thickness.
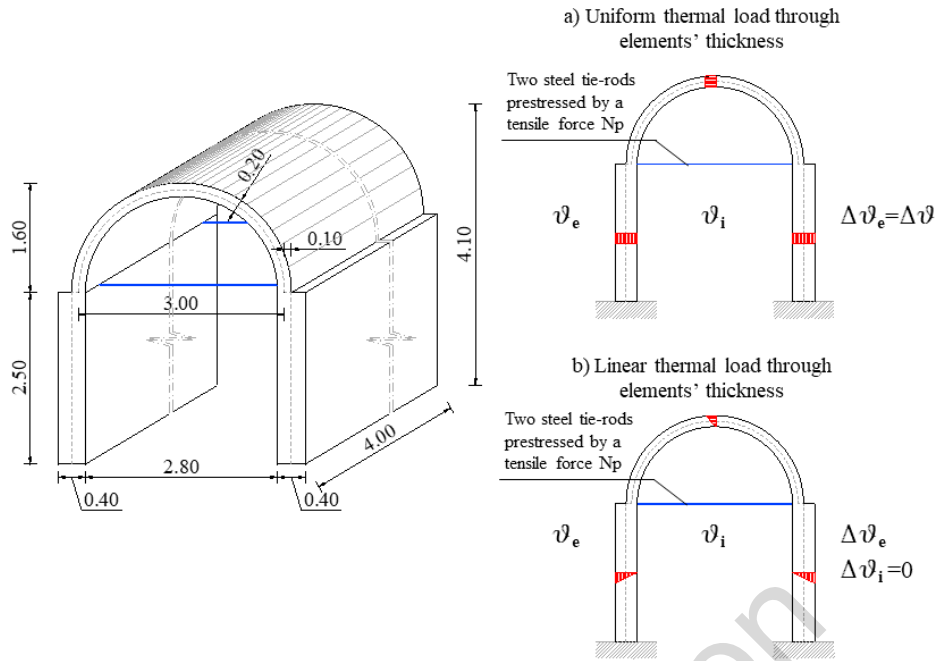
Figure 5- Structure's scheme (dimensions in meters)

Once again, the analyses are performed assuming that both the vault and the lateral walls are made of a masonry-like material while the tie-rods, made of steel, have a linear elastic behaviour. The mechanical properties of the materials, summarized in Table 3, do not vary with temperature ϑ. The reference temperature ϑ_0 is set always equal to 20°C. Each analysis is conducted according to the following sequence: i) application of the structure's self-weight and equilibrium achievement; ii) connection of the tie-rods to the structure and application of beam's self-weight along with a constant tensile preload force Np starting from the stress-strain fields obtained at the previous step; iii) incremental application of the thermal loads (uniform or linear) with ϑ varying in the interval [ϑ_1, ϑ_2] and assuming ϑ_1=0°C and ϑ_2=35°C, respectively; iv) calculation, after each thermal load increment, of natural frequencies $f_i(\vartheta)$, mode shapes (for i =1,...,4) and MAC values between current mode shapes and those corresponding to the reference temperature ϑ_0. The previous steps are repeated for different choices of Np, which is supposed to vary within the interval [0, 15000] N (the upper limit adopted indicates a reasonably high stress level of $1.50 \cdot 10^8$ Pa, still before yielding).

Table 3 - Mechanical properties of the constituent materials of the vault reinforced with tie rods.

	Masonry	Steel
Young's modulus E [Pa]	$1.74 \cdot 10^9$	$2.1 \cdot 10^{11}$
Poisson's ratio ν	0.2	0.3
Mass density ρ [kg/m^3]	2100	7850
Thermal expansion coefficient α [°C^{-1}]	10^{-5}	$1.2 \cdot 10^{-5}$
Tensile strength σ_t [Pa]	10^5	--

Compressive strength σ_c [Pa]	Infinite	--

Figure 6 depicts the first four mode shapes of the system, estimated at the reference temperature, with varying values of preload force applied to the tie-rods. With reference to the same modes and for each value of Np, Figure 7 shows the frequencies and MAC values trends with increasing temperature in case of uniform thermal load distribution, while Figure 8 shows the norm of the fracture strain tensor obtained at the reference temperature and at the bounds of the temperature range. Table 4 lists, for each value of preload force, the structure's natural frequencies at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2); the relative frequency variation against the reference temperature scenario $\Delta f_1 = (f_1 - f_0)/f_0$ and $\Delta f_2 = (f_2 - f_0)/f_0$, as well as the MAC values calculated between the mode shapes ϕ_{0i} at ϑ_0 and the corresponding ones at ϑ_1 (MAC$_1$) and ϑ_2 (MAC$_2$). The analysis of these figures and Table 4 allows to draw the following remarks:

- at the reference temperature, the first two mode shapes of the reinforced vault involve a global translation in X direction and a vertical flexural bending in Z direction, and remain unchanged as the force Np varies; the third mode shape is a bending mode in X direction that mainly involves the vault (recalling the first asymmetrical mode shape typical of arches) when Np is equal to zero, but it becomes a torsional mode around the Z-axis for higher values of preload; whereas, the fourth mode shape varies as a function of the Np applied to tie rods, featuring either a torsional configuration symmetrical with respect to the YZ plane of the vault or a bending configuration in X direction mainly involving the vault. It is noteworthy that variations in the mode shape sequences are not uncommon in this type of structures as already demonstrated in [45];
- overall, mode shapes are less sensitive than frequencies to thermal variations for any value of tie-rods preload;
- for a fixed temperature value, the frequency values increase as the preload force Np rises up to 10000 N and then frequency values decrease. This trend can be explained by observing the evolution of the crack pattern of the vault under uniform thermal load distributions: in fact, the micro-fractures at the intrados section of the vault's keystone and at the extrados of the springing sections progressively close until a preload force equal to 10000 N; for higher values of Np, new micro-cracks open up in a different area;
- regardless of the preload force, all the frequencies increase with rising temperatures up to 15°C, remaining almost constant therefrom (except for modes 3 and 4 in case of Np=0 where frequencies show a decreasing trend after 20°C). This behaviour is likely due to the fact that for temperatures above 15°C all the cracks are closed, the structure has a linear elastic behaviour, and only for some values of Np an increase in temperature corresponds to the opening of new cracks.
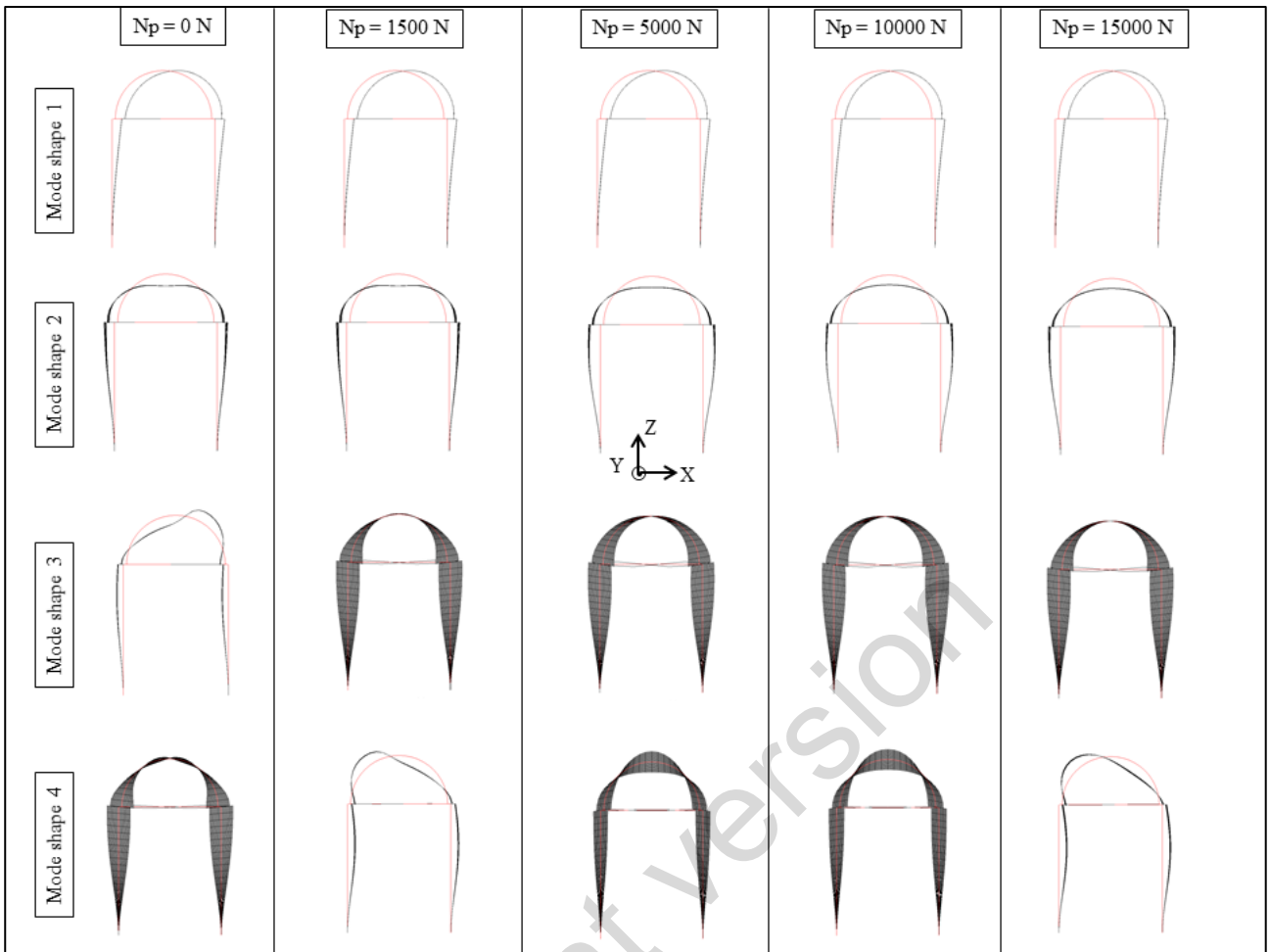
Figure 6- Mode shapes of the vault, at the reference temperature, for varying values of preload force in the tie-rods.
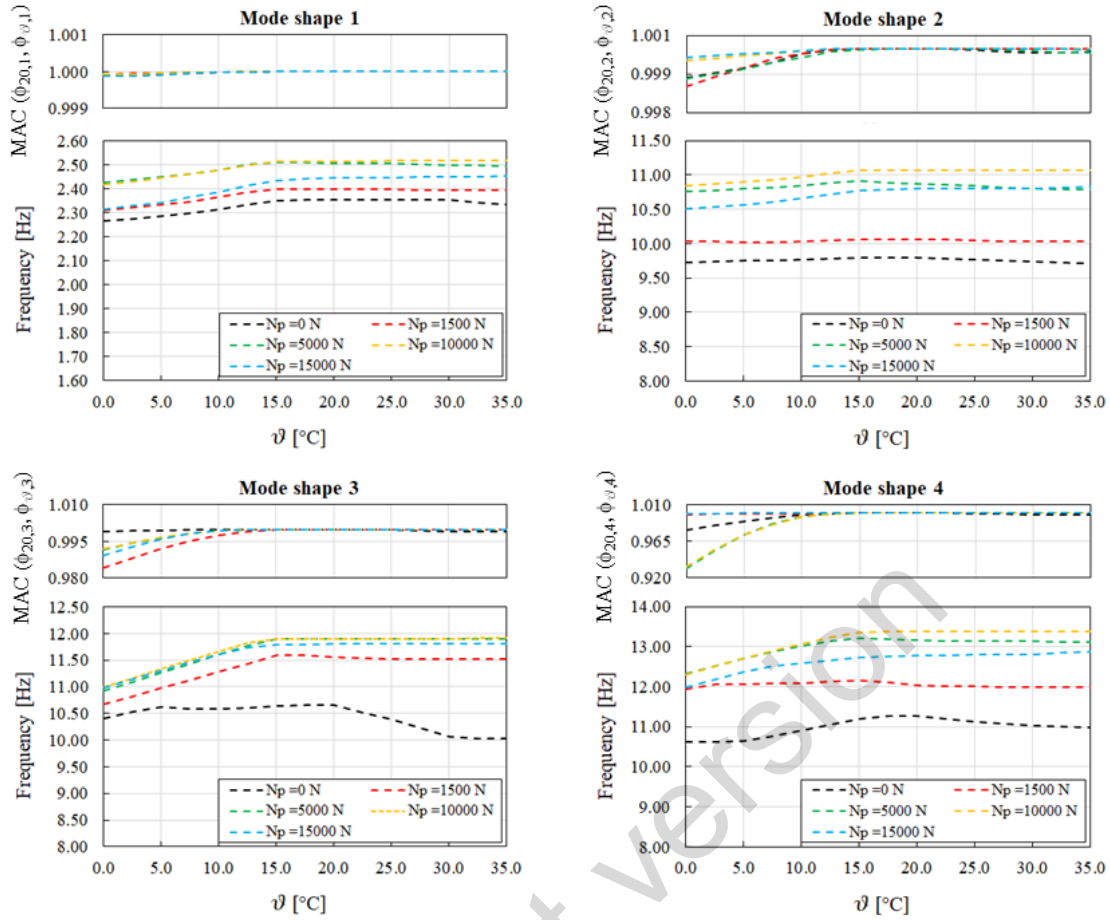
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Figure 7- Evolution of the frequencies and MAC values of the reinforced vault mode shapes with varying the temperature for different values of Np in case of uniform thermal load applied to the structure. Note the different vertical scales used. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

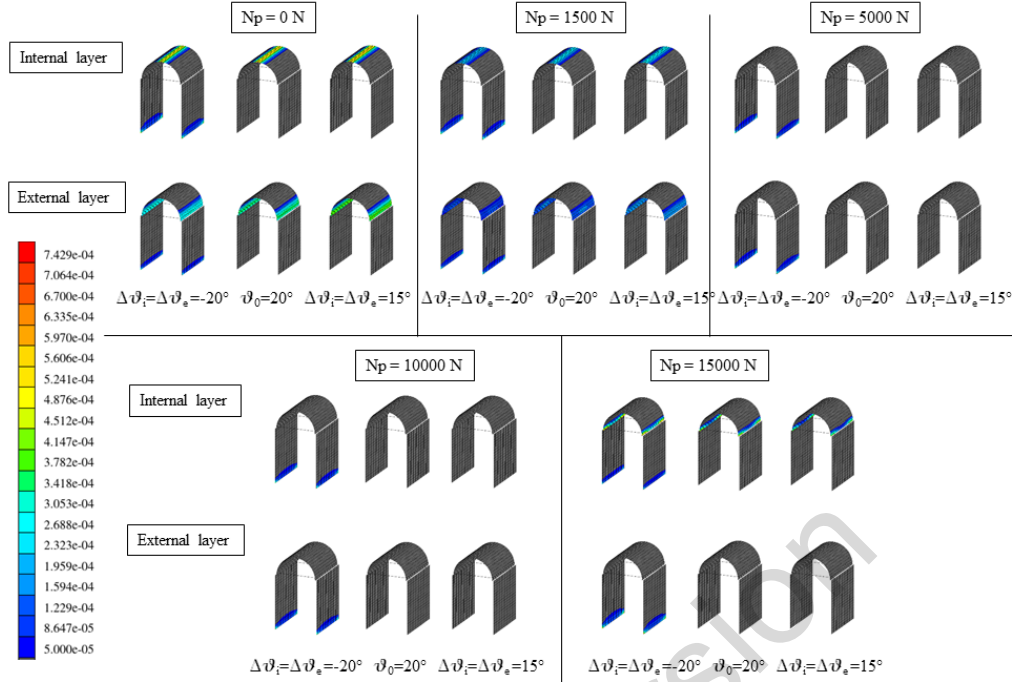a) Uniform thermal load: norm of the fracture strain

Figure 8- Crack pattern in the vault for different values of Np under uniform thermal load distributions in the configurations corresponding to the reference temperature ϑ_0 and to the lower and upper bounds of the temperature variation range ϑ ([ϑ_1, ϑ_2]). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Table 4- *Case a*: natural frequencies of the reinforced vault at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2); relative frequency variation $\Delta f_1=(f_1-f_0)/f_0$ and $\Delta f_2=(f_2-f_0)/f_0$; MAC value between the mode shapes ϕ_{0i} (i=1…4) at ϑ_0 and the corresponding ones at ϑ_1 (MAC$_1$) and ϑ_2 (MAC$_2$) for different values of preload force Np.

Np = 0 N						
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.355	2.266	-3.79	0.999	2.333	-0.93	1.000
ϕ_2	9.800	9.727	-0.74	0.999	9.713	-0.88	1.000
ϕ_3	10.653	10.410	-2.28	0.999	10.020	-5.94	0.999
ϕ_4	11.263	10.625	-5.67	0.979	10.973	-2.57	0.998
Np = 1500 N						
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.398	2.312	-3.61	1.000	2.395	-0.11	1.000
ϕ_2	10.055	10.027	-0.27	0.999	10.036	-0.19	1.000
ϕ_3	11.553	10.673	-7.62	0.984	11.515	-0.33	1.000

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	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_4	12.044	11.928	-0.96	0.998	11.994	-0.42	1.000
Np = 5000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.507	2.426	-3.21	1.000	2.495	-0.46	1.000
ϕ_2	10.868	10.755	-1.04	0.999	10.783	-0.79	1.000
ϕ_3	11.903	10.925	-8.22	0.992	11.896	-0.06	1.000
ϕ_4	13.173	12.327	-6.42	0.931	13.123	-0.38	1.000
Np = 10000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.516	2.418	-3.90	1.000	2.517	0.04	1.000
ϕ_2	11.067	10.840	-2.05	1.000	11.069	0.01	1.000
ϕ_3	11.904	10.994	-7.64	0.992	11.907	0.03	1.000
ϕ_4	13.371	12.301	-8.01	0.934	13.376	0.04	1.000
Np = 15000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.446	2.314	-5.39	1.000	2.454	0.33	1.000
ϕ_2	10.798	10.511	-2.65	1.000	10.830	0.30	1.000
ϕ_3	11.802	10.976	-7.00	0.989	11.814	0.10	1.000
ϕ_4	12.782	11.985	-6.24	1.000	12.874	0.72	1.000

Figure 9 shows, for the first four mode shapes of the structure and for each value of preload force Np, the evolution of frequencies and MAC values with increasing temperature in case of linear thermal load through the elements' thickness. Figure 10 depicts the norm of the fracture tensor, for each value of Np, both at the reference temperature and at the bounds of the considered temperature range. For comparative purposes, Table 5 lists the structure's natural frequencies at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2) for increasing values of preload force, together with the relative frequency variation against the reference temperature scenario $\Delta f_1 = (f_1 - f_0)/f_0$ and $\Delta f_2 = (f_2 - f_0)/f_0$ and the MAC values calculated between the mode shapes ϕ_{0i} at ϑ_0 and the corresponding ones at ϑ_1 (MAC_1) and ϑ_2 (MAC_2). The observation of the figures and Table 5 allows one to underline the following aspects:

- for any value of tie-rods preload, mode shapes are always less sensitive than frequencies to thermal variations even in the case of linear thermal load distribution through the elements' thickness. This outcome corroborates the experimental evidence ([12], [15]);
- for any temperature value, the frequency values increase as the preload force Np rises up to a value of 10000 N and then frequency values decrease. This behaviour can be explained by observing that the evolution of the crack pattern of the vault under linear thermal load distribution: indeed, the micro-fractures of the structure progressively close until the preload force is equal to 10000 N; then, for higher values of Np, new micro-cracks open up;
- particularly, the first frequency shows a greater sensitivity to temperature variations as compared to the case of constant thermal load. If the preload force is less than 5000 N, the

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frequency slowly increases with growing temperatures up to 20-25°C; from that value onwards, it reduces abruptly. This is because the linear thermal variation induces a flexion in the lateral walls, leading to the closure of micro-cracks at the base of the walls and to the reduction of fractures at the vault impost area; yet, for temperatures above 20°C, new cracks open at the vault keystone and the base of the lateral walls. For preload force greater than 5000 N, the turning point between increasing and decreasing frequency moves towards lower temperature values (15°C) and the overall trend changes radically; indeed, for temperatures increasing from 0°C to 15 °C, frequencies quickly rise as a result of some fractures closure at the structure's extrados; whereas, for temperature values higher than 15° C, the natural frequencies of the reinforced vault either remain constant or slowly decrease due to the opening of some new cracks at the intrados section of the vault impost;

- the second frequency, for values of preload Np below 5000 N, does not show a clear trend as a function of temperature; this is because the second mode shape features a vertical bending configuration which mainly involves the vault whose fractures do not vary significantly as a function of the applied thermal load (the keystone is permanently characterized by micro-cracks). Conversely, for preloads higher than 5000 N, at the reference temperature, the structure appears intact (no fracture deformations) due to the beneficial effect of Np. In the range 0°-15°C, the frequency increases with progressive temperature but, once the value of 15° C is exceeded, it remains constant or decreases due to the occurrence of new cracks across the vault and at the base of the lateral walls. An exception is the case with preload Np = 15000 N, which corresponds to a widely diffused crack pattern in the structure at 0 °C, and a more localized crack pattern at the springing of the vault at 35°C; accordingly, the frequency of this mode rapidly increases as the temperature rises up to 15°C and then it stabilizes and remains nearly constant;

- the considerations made hitherto can be also extended to the higher frequencies of the vault (i.e., the frequencies of modes 3 and 4).
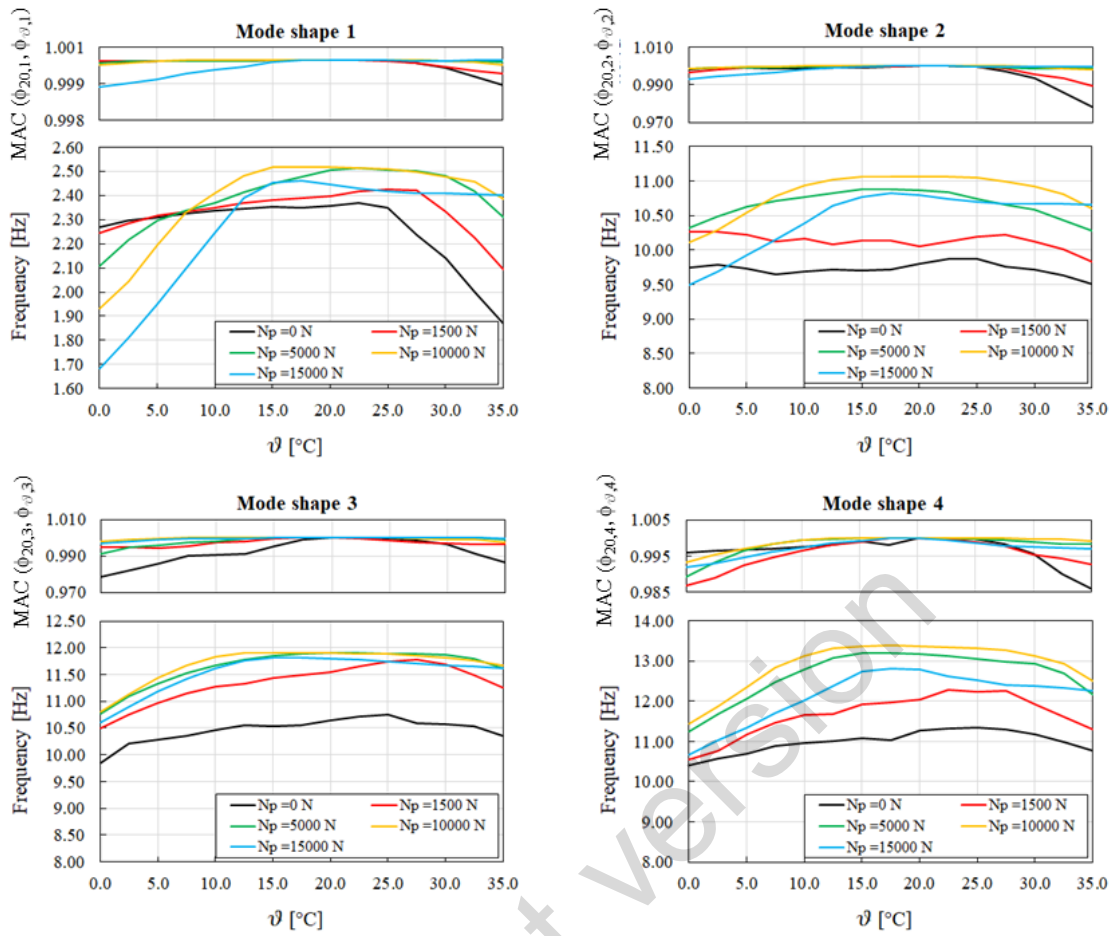
Figure 9- Evolution of frequencies and MAC values of the reinforced vault mode shapes with varying temperature for different values of Np in case of linear thermal load applied to the structure. Note the different vertical scales used. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

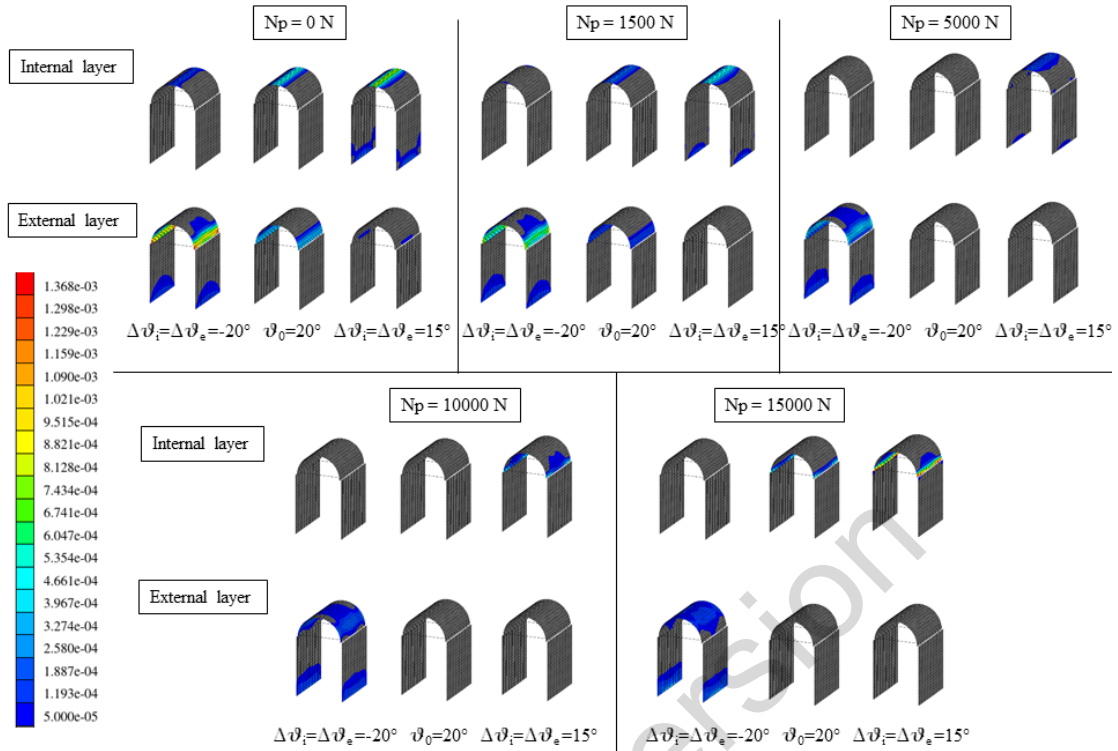b) Linear thermal load: norm of the fracture strain

Figure 10- Crack pattern in the vault for different values of Np under linear thermal load distributions in the configurations corresponding to the reference temperature ϑ_0 (20°C) and to the lower and upper bounds of the temperature variation range ϑ ([ϑ_1, ϑ_2]). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Table 5- *Case b*: natural frequencies of the reinforced vault at ϑ_0 (f_0), ϑ_1 (f_1) and ϑ_2 (f_2); relative frequency variation $\Delta f_1=(f_1- f_0)/ f_0$ and $\Delta f_2=(f_2- f_0)/ f_0$; MAC value between the mode shapes ϕ_{0i} (i=1,…,4) at ϑ_0 and the corresponding ones at ϑ_1 (MAC$_1$) and ϑ_2 (MAC$_2$) for different values of Np.

Np = 0 N						
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.355	2.271	-3.59	1.000	1.867	-20.74	0.999
ϕ_2	9.800	9.740	-0.61	0.998	9.507	-2.99	0.978
ϕ_3	10.653	9.850	-7.53	0.978	10.351	-2.84	0.986
ϕ_4	11.263	10.395	-7.71	0.996	10.759	-4.47	0.986
Np = 1500 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC$_1$ [-]	f_2 [Hz]	Δf_2 [%]	MAC$_2$ [-]
ϕ_1	2.398	2.244	-6.42	1.000	2.094	-12.67	0.999
ϕ_2	10.055	10.266	2.10	0.997	9.834	-2.20	0.990

Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_3	11.553	10.494	-9.17	0.995	11.263	-2.51	0.996
ϕ_4	12.044	10.554	-12.37	0.987	11.295	-6.22	0.993
Np = 5000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.507	2.110	-15.81	1.000	2.309	-7.91	1.000
ϕ_2	10.868	10.314	-5.10	0.999	10.274	-5.47	0.999
ϕ_3	11.903	10.766	-9.56	0.991	11.617	-2.41	0.999
ϕ_4	13.173	11.255	-14.56	0.989	12.156	-7.72	0.998
Np = 10000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.516	1.934	-23.15	1.000	2.384	-5.27	1.000
ϕ_2	11.067	10.109	-8.66	0.998	10.602	-4.20	0.998
ϕ_3	11.904	10.811	-9.18	0.998	11.665	-2.01	0.998
ϕ_4	13.371	11.441	-14.44	0.993	12.489	-6.59	0.999
Np = 15000 N							
Mode shape	f_0 [Hz]	f_1 [Hz]	Δf_1 [%]	MAC_1 [-]	f_2 [Hz]	Δf_2 [%]	MAC_2 [-]
ϕ_1	2.446	1.685	-31.10	0.999	2.402	-1.79	1.000
ϕ_2	10.798	9.489	-12.13	0.993	10.655	-1.33	0.999
ϕ_3	11.802	10.602	-10.17	0.997	11.614	-1.59	1.000
ϕ_4	12.782	10.663	-16.58	0.992	12.262	-4.07	0.997

4. Application to the Mogadouro clock tower, in Portugal

The full five step procedure outlined in Section 2 for the simulation of the dynamic behaviour of a monitored masonry structure subjected to temperature variation is here applied to a real benchmark case study: the Mogadouro clock tower in Portugal (Figure 11). The 20.4 m high tower, built after 1559, has a rectangular cross section, 4.7×4.5 m², and is made of three-leaf large block granite masonry walls with rubblestone inner core and a total thickness of about 1 m. Due to its severe state of decay, repair and strengthening works were carried out in 2005, comprising mostly lime-based injections and installation of an upper steel ring using external profiles. Steps a., b. and c. of the procedure presented in this work, which are concerned with the preliminary dynamic identification tests and the long-term acquisition for modal and temperature tracking, have been thoroughly discussed in [29]. Monitoring was carried out between April 2006 and December 2007 in the strengthened structure. Figure 12 shows the experimental trend of the frequency values for the first four modes of the towers, hereafter used for calibration and comparison purposes, corresponding to the first bending modes along two horizontal directions (modes 1 and 2), torsion (mode 3) and second bending mode along one direction (mode 4). An evident shift of about 4% in the frequency trend emerges in the records of modes 1 and 2, giving rise to a double cluster in their frequency-temperature correlation plots. This peculiar behaviour has been explained as the effect of humidity and water

absorption from the tower's walls at the beginning of the rainy season [29] and is further discussed in the following analyses.

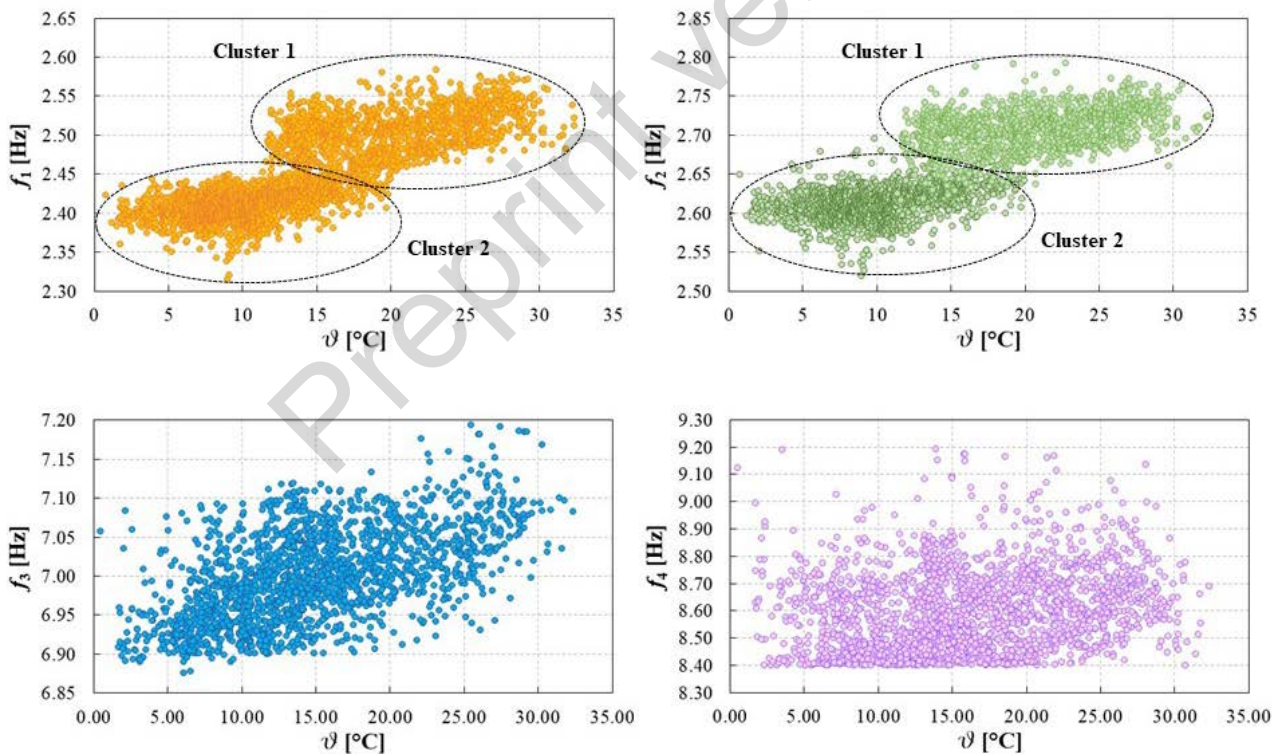Figure 11- Mogadouro castle (left) and the clock tower (right)

Figure 12 - Experimental trend of the first four modes of the Mogadouro clock tower with the identification of the two clusters in frequency 1 and 2. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

As concerns steps d. and e. of the procedure, a FE model of the structure is built using 18024 isoparametric 8-node brick elements to discretize the walls and 352 thick shell elements to discretize the roof, counting a total of 23467 nodes (Figure 13). The tower is assumed to be clamped at the base

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(no information related to the foundations and the soil are available, so the soil structure interaction is neglected) and constituted by linear elastic materials as far as the roof (material no. 5) and top pillars (material no. 4) are concerned, and masonry-like materials in what concerns South and North façades (material no. 1), East and West façades (material no. 2), and walls' corners (material no. 3). The (optimal) mechanical properties of the materials 1, 2 and 3 are estimated via the nonlinear model updating procedure recalled in Section 2 and described in [42]. The choice of linear elastic materials to model pillars and roof stems from the observation that these elements do not affect the overall dynamic behaviour of the tower. Indeed, the low elastic modulus adopted for the roof allows the tower cross section to deform freely within its own plane in agreement with the experimental evidence [46].
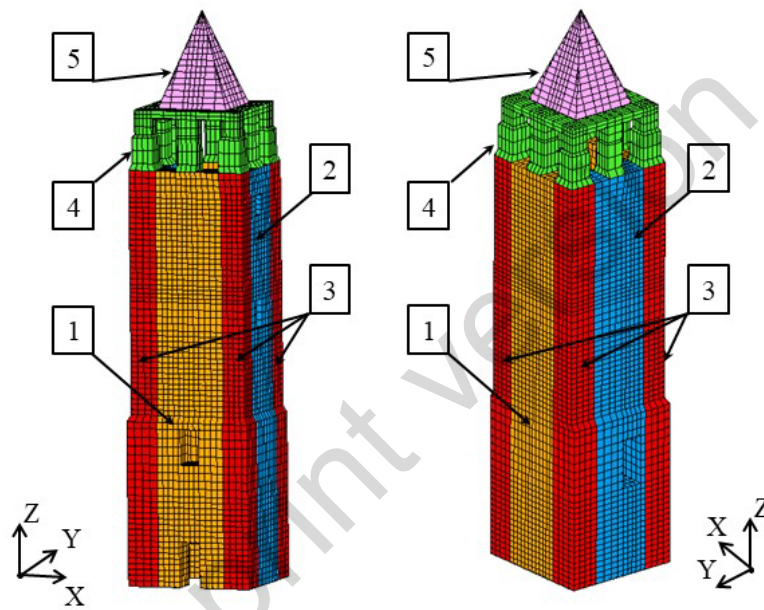
Figure 13- FE model of the Mogadouro tower with indication of the constituent materials (different colours and label number).

For the present case study, the reference temperature ϑ_0 is assumed equal to 17°C, corresponding to the average temperature value of the monitoring period and a thermal expansion coefficient α equal to $5 \cdot 10^{-6}$ is assumed for the whole structure, as suggested in pertinent literature works about historical masonry [47], [48].

The nonlinear model updating assumes that the Poisson's ratio ν of all materials as well as the mechanical characteristics of materials 4 and 5 are fixed and the tower is subjected solely to the self-weight load.

Table 6 - Optimal values of the material mechanical properties

Material	Tower's portion	ν [-]	ρ [kg/m³]	$E_0{}^1$ [Pa]	σ_t [Pa]	α [°C⁻¹]
1	South and North Façades	0.2	2200	$1.760 \cdot 10^9$	11000	$5 \cdot 10^{-6}$
2	East and West Façades	0.2	2200	$0.810 \cdot 10^9$	11000	$5 \cdot 10^{-6}$
3	Walls' Corners	0.2	2400	$4.960 \cdot 10^9$	11000	$5 \cdot 10^{-6}$
4	Top Pillars	0.2	2200	$1.210 \cdot 10^9$	--	$5 \cdot 10^{-6}$

| 5 | Roof | 0.2 | 2000 | $0.195 \cdot 10^9$ | -- | $5 \cdot 10^{-6}$ |

In order to obtain a numerical model close to reality, the experimental frequencies $f_{0,exp}^1$ (or reference dynamic properties) used for the calibration are estimated considering the mean of the values at the reference temperature ϑ_0 in cluster 1. The tensile strength σ_t (the same for the whole structure) and the Young's modulus $E_0^1{}_{,i}$ (for i=1…3) of masonry materials are assumed unknown, and varying within the ranges defined by their lower and upper bounds as follows:

$$0.0 \le \sigma_t \le 5 \cdot 10^5 \text{ Pa}, \ 0.5 \cdot 10^9 \le E_0^1{}_{,i} \le 5.5 \cdot 10^9 \text{ Pa (for i=1…3)}$$

Table 6 reports the optimal values of the mechanical characteristics obtained by nonlinear model updating; Table 7 summarizes the results in terms of experimental ($f_{0,exp}^1$) and numerical ($f_{0,N}^1$) frequencies, relative frequency percentage error, and MAC values between experimental and numerical mode shapes. The first two frequencies of the tower and associated mode shapes are very well approximated, with relative frequency errors close to zero and a high degree of consistency between the modal displacements of the fundamental modes. The approximation for the higher modes is slightly less accurate, particularly for the torsional mode (mode 3), with a maximum relative frequency error of 5.4%. Given the greater degree of complexity characterizing experimental high-frequency modes [49], the outcome of the model updating procedure is more than satisfactory. Yet, for the purpose of this work, the following analysis will focus mostly on the two first fundamental modes of the tower.

Table 7- Cluster 1: Comparison between experimental ($f_{0,exp}^1$) and numerical frequencies ($f_{0,N}^1$); relative frequency error $\Delta f_0^1 = (f_{0,exp}^1 - f_{0,N}^1)/ f_{0,exp}^1$ and MAC^1 values between corresponding mode shapes.

Mode shape	$f_{0,exp}^1$ [Hz]	$f_{0,N}^1$ [Hz]	Δf_0^1 [%]	MAC^1
1	2.503	2.502	0.04	0.98
2	2.706	2.716	-0.37	0.98
3	7.001	6.624	5.38	0.97
4	8.581	8.311	3.15	0.74

Once the optimal values of the constituent materials are estimated, the dynamic properties of the tower are calculated as a function of the temperature variations recorded experimentally in cluster 1. Incremental uniform thermal loads are applied to the FE model up to a maximum temperature ϑ_{max}=29°C (extreme values are disregarded because less representative of the considered temperature range). After each increment, the first two natural frequencies and mode shapes of the tower are calculated via the linear perturbation analysis implemented in NOSA-ITACA and the MAC values between the current mode shapes and those corresponding to the reference temperature are estimated.
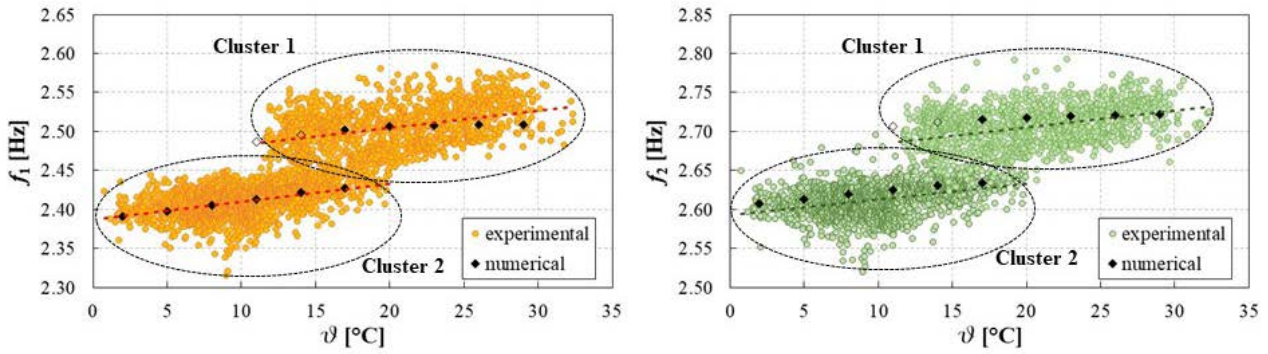
Figure 14 - Experimental (circular dots) versus numerical (rhomboid black dots) trend (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Figure 14 shows, for cluster 1, the experimental trend of the first (left side in orange) and of the second (right side in green) frequency along with their numerical counterparts (rhomboid black dots). The rhomboid black dots without filling (in cluster 1) represent the extrapolation of the numerical simulation with the cluster 2 neglected. Figure 15 and Figure 16 report the fracture strain patterns of Mogadouro clock tower corresponding, respectively, to the reference temperature ϑ_0 and the maximum reached temperature ϑ_{max}.

It is clear from the plots that the numerical algorithms implemented are able to capture the variations of the dynamic properties of the masonry structure as a function of the temperature until 20°C: indeed, up to this value, the numerical frequencies slightly increase, but then they remain constant till 29 °C. This behaviour does not fully reflect the experimental evidence where a direct frequency-temperature correlation is found for the entire period. This phenomenon is directly related to the crack pattern exhibited by the model at the different temperatures ϑ_0 and ϑ_{max} which is practically unchanged; the discretized structure expands freely and the existing micro fractures remain the same (Figure 15 and Figure 16). Although the interior of the tower is not thermally isolated from exogenous factors, a linear temperature distribution through the tower walls' thickness was also considered. The results obtained coincide with those of uniform temperature distribution, without making any improvements to the modelling, thus for the sake of brevity they are not reported in the paper.
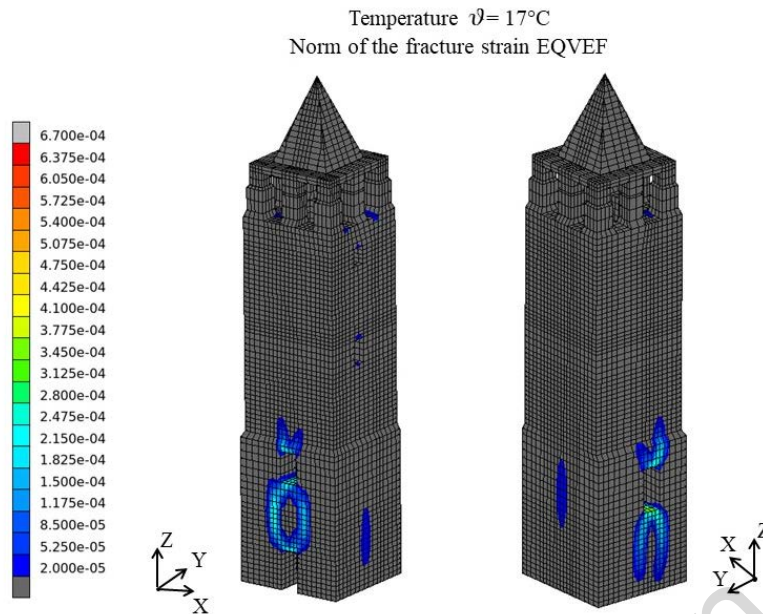
Figure 15- Pattern of fracture strain norm at the reference temperature ϑ_0. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)
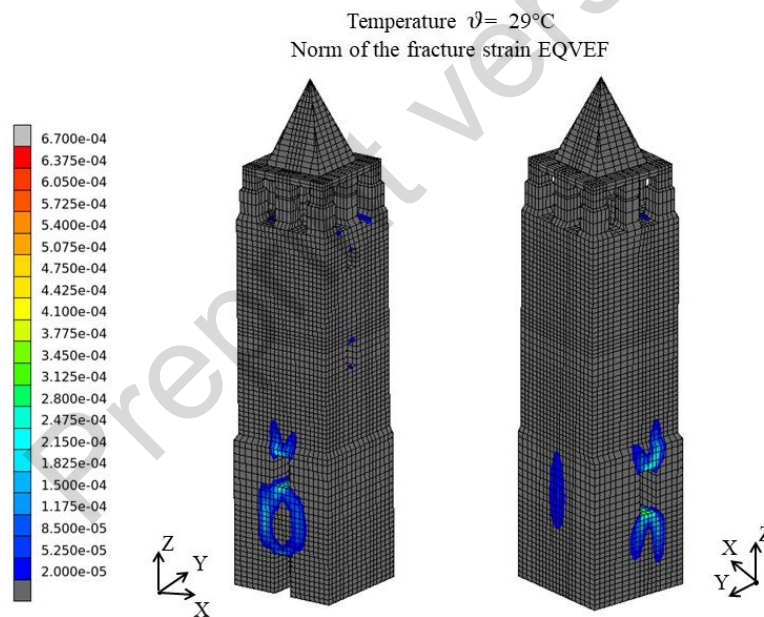

Figure 16- Pattern of fracture strain norm of the tower at temperature ϑ_{max}. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Aiming at following the dynamic behaviour of Mogadouro clock tower across the entire temperature interval, the frequency trends in the second cluster must be considered as well. To this end, it is necessary to catch the frequency gap observed in the experimental data between the two identified clusters that, as already mentioned, is likely due to the humidity and water absorption during the rainy season. As well-known in the literature, the influence of moisture inside masonry walls can result either into a mass increase [50] or a stiffness decrease ([51], [52]). Since NOSA-ITACA does not allow to directly consider the humidity variation within the masonry, its effect is artificially

introduced in the numerical simulations by exploiting the ability of the code to manage mechanical characteristics which depend on temperature. Hence, for temperatures below 17° C (higher humidity content), a reduction of the elastic modulus of about 6-7% is applied for masonry-like materials in order to fit the experimental frequencies at 17°C in cluster 2. The numerical simulations are repeated by adopting the new Young's moduli $E_0{}^2$ reported in Table 8 (in the same table, the Young's moduli $E_0{}^1$ used for cluster 1 are also recalled for the sake of clarity).

Table 8 - Young's moduli used in the numerical simulations for both cluster 1 and cluster 2.

Material	Tower's portion	$E_0{}^1$ [N/mm^2]	$E_0{}^2$ [N/mm^2]	$E_0{}^2/E_0{}^1$
1	South and North Façades	$1.760 \cdot 10^9$	$1.640 \cdot 10^9$	0.932
2	East and West Façades	$0.810 \cdot 10^9$	$0.764 \cdot 10^9$	0.943
3	Walls' Corners	$4.960 \cdot 10^9$	$4.670 \cdot 10^9$	0.941
4	Top Pillars	$1.210 \cdot 10^9$	$1.210 \cdot 10^9$	1.000
5	Roof	$0.195 \cdot 10^9$	$0.195 \cdot 10^9$	1.000

Table 9- Cluster 2: Comparison between experimental ($f_{0,\exp}{}^2$) and numerical frequencies ($f_{0,N}{}^2$); relative frequency error $\Delta f_0{}^2 = (f_{0,\exp}{}^2 - f_{0,N}{}^2)/f_{0,\exp}{}^2$ and MAC^2 values

Mode shape	$f_{0,\exp}{}^2$ [Hz]	$f_{0,N}{}^2$ [Hz]	$\Delta f_0{}^2$ [%]	MAC^2
1	2.432	2.427	0.21	0.98
2	2.634	2.634	0.00	0.98
3	6.993	6.422	8.17	0.97
4	8.476	8.121	4.19	0.74

Table 9 summarizes the results obtained for the FE model of the tower at the reference temperature scenario ϑ_0 for cluster 2 in terms of experimental ($f_{0,\exp}{}^2$) and numerical ($f_{0,N}{}^2$) frequencies, relative frequency error, and MAC values between experimental and numerical mode shapes. Once again, the first two frequencies and the mode shapes of the structure are very well approximated, while higher frequency errors are found for modes 3 and 4, reading a maximum relative percentage error of 8.2% for the frequency of the third mode.

After concluding this second calibration process, the dynamic properties of the tower are calculated as a function of the temperature variations recorded experimentally in cluster 2. Uniform decreases in temperature are applied up to a minimum temperature of 2°C. As done for cluster 1, after each temperature decrease, the first two natural frequencies and mode shapes of the tower are calculated via the linear perturbation analysis implemented in NOSA-ITACA and the MAC values between the current mode shapes and those corresponding to the reference temperature are estimated. Figure 14 shows, for cluster 2, the experimental trend of the first (left side in orange) and of the second (right side in green) frequency versus the numerical counterpart. For comparative purposes, the trend obtained for cluster 1 data is overlapped as well. Figure 17 depicts the fracture strain pattern of Mogadouro clock tower corresponding to a temperature value of 2°C.
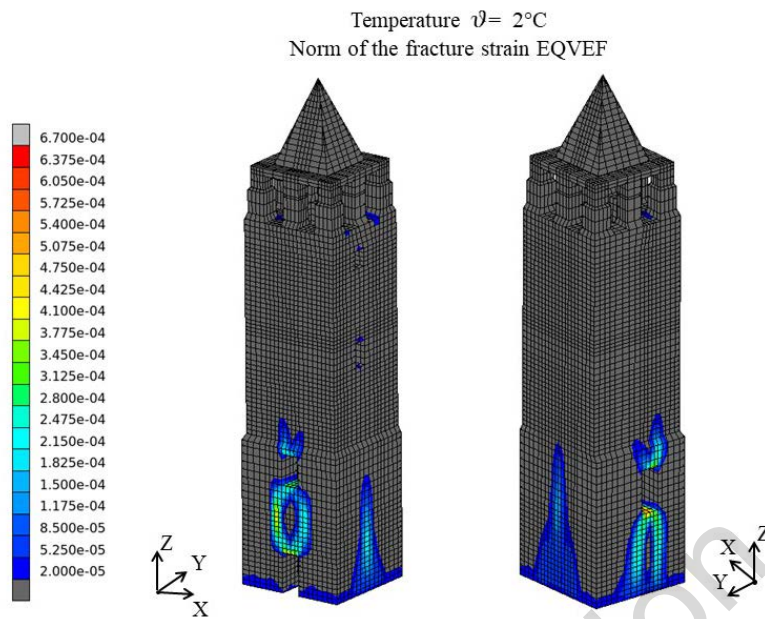
Figure 17- Pattern of fracture strain norm of the tower at temperature ϑ_{min}. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

In this case, the numerical model is able to satisfactorily reproduce the actual dynamic behaviour of the structure across the entire temperature interval, exhibiting the same frequency-temperature trend observed experimentally for cluster 2, i.e. decreasing frequencies as the temperature decreases and vice versa. Indeed, as observed in Figure 17, new cracks arise at the base of the tower at 2°C and the size of the cracks already existing at the reference temperature (see Figure 15) tends to increase, leading to frequency downshifts. It is worth noting that the same numerical results have been obtained considering an increase of mass instead of a decrease of stiffness to simulate the humidity effect in the tower. For the sake of brevity, these results are not reported.

5. Conclusions

The paper presents a numerical investigation of the effects of temperature variation on masonry structures' static and dynamic behaviour. The influence of temperature is assessed by resorting to a finite element code (NOSA-ITACA, www.nosaitaca.it/software/) able to model the peculiar constitutive behaviour of masonry under static, dynamic and thermal loads. The effects of thermal variation over the modal properties of a masonry structure are evaluated via a linear perturbation procedure. The investigation is carried out on two trial examples (a masonry portal with and without tie-rods reinforcement). The code is then applied to a real case study, the Mogadouro tower, which has been the subject of a long-term dynamic monitoring campaign, whose results highlighted the dependence of the tower's natural frequencies on temperature.

The primary outcomes of the paper can be summarized as follows.

- The code can model the effects of the crack pattern in the static and dynamic behaviour of a masonry structure.
- Temperature variations affect a masonry structure's static and dynamic behaviour because they induce changes in the stress field and in the crack distribution over the structure.

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- The Mogadouro tower shows a positive correlation between frequencies and temperature, as evidenced in different long-term dynamic monitoring campaigns.
- The experimental results of the Mogadouro tower monitoring campaign show a peculiar behaviour likely attributed to yearly changes in the moisture content of the masonry walls.
- The code is able to reproduce the results of the Mogadouro experiment, with regard to the effects of both temperature and moisture variations on the modal properties of the tower.

Future works that are recommended include: i) the development of experimental tests in climate chamber to evaluate the variation of the Young´s modulus as a function of the temperature and relative humidity variations for different types of masonry; ii) the construction of a masonry mock-up which will be damaged and whose dynamic properties will be evaluated as the climatic conditions vary; iii) the execution of numerical simulations via different FE codes to replicate the experimental results and compare different constitutive equations.

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