Seismic Risk Mitigation of Historical Masonry Towers by Means of Prestressing Devices

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Abstract This work presents the investigation of the efficiency of different prestressing devices as a rehabilitation measure for the seismic risk mitigation of historical masonry towers. As a first phase, the seismic vulnerability of theoretical masonry towers was assessed by means of numerical models validated with information from the literature, observed damage and behavior of these structures due to passed earthquakes (crack pattern and failure mechanisms), and mainly taking into account the engineering experience. Afterwards, the validated models were rehabilitated with different prestressing devices; analyzing the results and concluding which device or the combination of them improved in a better way the seismic performance of the masonry towers. Finally, the methodology will be applied in two historical masonry towers located in seismic areas; the medieval tower "Torre Grossa" of San Gimignano, Italy, and one of the bell towers of the Cathedral of Colima, Mexico.

Keywords: historical masonry towers, seismic risk management (risk assessment and mitigation), linear and nonlinear analyses, and prestressing devices as structural rehabilitation.

Introduction

As it has been demonstrated for passed earthquakes and the damage they could generate in the cultural heritage; historical masonry towers are considered one of the most vulnerable structural types; due to its height, slenderness, significant compressive stress levels (that could lead to a suddenly collapse as it has been presented previously), and the nonlinear behavior of the masonry which allows low load levels due to its poor tensile strength. All these issues and behavior represent a huge task when the dynamic characteristics of the seismic action are taken into account. Inside the framework of the *seismic risk management* there are two main stages that have to be follow as a measure to ensure the protection of the cultural heritage (historical buildings). Those stages correspond with the *seismic risk assessment* and *seismic risk mitigation*. Nowadays there is an enormous variety of methodologies to assess the seismic risk of buildings (the first stage of the seismic risk management), and exists a big confusion within the scientific community regarding which is the best procedure to follow for achieving finally the seismic risk mitigation of buildings, and mainly, to ensure the protection of the persons. For the seismic risk assessment of buildings the most important methodologies reported in the literature are classified in empirical or qualitative (e.g. vulnerability and damage classes of the EMS-98 (Grünthal, 1998) and the vulnerability index method GNDT (1990)); the quantitative ones, integrated by the analytical methodologies (e.g. more refined methods such as FEM and limit analysis) and the hybrid. This last classification is a combination of the analytical methods with real data of the building obtained experimentally (mechanical and dynamic information). This experimental data is a valuable tool for the calibration (or updating) of the model, obtaining more reliable results towards the real seismic performance of the structure. All the above mentioned methodologies for assessing the seismic risk of a building are based in the close relation between the *hazard of the research zone* (seismic action) *and the vulnerability of the structure*. The selection of every methodology to assess the seismic vulnerability depends on different factors such as number of buildings (an isolated building or a complete city), importance of the structure, available information, and the destination of the study results. It means that for a building or a large group of buildings the empirical methodologies allow determining the vulnerability (low, medium or high) in a fast and qualitative way. These methodologies are helpful as well to determine briefly the seismic scenario before or after the occurrence of an earthquake. For assessing the seismic vulnerability of an essential building (e.g. historical building, hospital, school, jail and so on) the procedure is different and more in detail (quantitative) than in the qualitative evaluations of a large number of buildings. It is more complicated, requires more computer resources and especial equipment, and represents more time consuming. A combination of the empirical, analytical and hybrid methodologies have to be applied to obtain reliable results of the vulnerability assessment, in order to measure the amount of damage produced by the seismic action over the structure. These results of the evaluation are helpful to implement as a final stage of the seismic risk management the *rehabilitation proposals*, in order to improve the behavior of the building against seismic actions, and finally, to obtain with this, the ending goal which corresponds with the seismic risk mitigation.

Methodology

The main objective of the present research work consists on the investigation of the efficiency of different prestressing devices as a rehabilitation measure for the seismic risk mitigation of historical masonry towers. The methodology to achieve the main objective has been divided into two main phases: The first one pretends to assess the seismic vulnerability of two theoretical masonry towers by means of numerical simulations, and to validate them with information from the literature, observed damages and behavior of these structures due to passed earthquakes (crack pattern and failure mechanisms), and mainly taking into account the engineering experience. Afterwards, the validated models will be rehabilitated with different prestressing devices such as post-tensioned tendons of different materials (steel, fiber reinforced plastic (FRP) and shape memory alloys (SMAs)), analyzing the results and concluding which device or the combination of them improves in a better way the seismic performance and decreases the damage of the masonry towers (seismic risk mitigation). As a second and final phase, the methodology will be applied in two historical masonry towers located in seismic zones. One of them corresponds with a medieval masonry tower named “Torre Grossa” located in the historical town of San Gimignano, Italy. The second study case is one of the masonry bell towers of the Cathedral of Colima, Mexico; which is located in the highest seismic area of the country (with earthquakes of M7.6; like the occurred last 21.01.2003 and others of more than magnitude 6 in recent years. The results will be analyzed as in the case of the theoretical towers of the first phase, as well as the generation of the concluding remarks regarding the best prestressing device or the combination of them that allows improving the seismic performance of the towers (by means of an increment of the overall strength and ductility), achieving with this the seismic risk mitigation.

Seismic risk assessment of the theoretical ancient masonry towers

Definition of the finite element models (FEM) 3D FEM were developed for the seismic vulnerability evaluation of the theoretical historical masonry towers using the commercial finite element program ANSYS® (see Fig. 1). Two models of towers with the same geometrical and mechanical characteristics were considered for the numerical simulations. The only difference between the models corresponds with the boundary conditions generated by neighbor buildings. For the first model the tower was considered isolated, and for the second one the interaction between neighbor buildings was taken into account (non-isolated); connected in the East façade at the height of 10 m and in the North façade at the height of 15 m as it is depicted in Fig. 1c. As a reference point at the models, the small window and the door at the base level are located in the South façade of the towers (right side corresponds with the East façade). The geometry of both models was determined considering similar characteristics of real historical masonry tall towers (very vulnerable to seismic actions), including the presence of large wall openings and a tall and heavy roof. The models have a rectangular plant with dimensions of 10 m x 10 m, the height of the load-bearing walls is 45 m (constant thickness of 1.5 m) and the triangular roof 10 m (thickness of 0.15 m), reaching the total height from the bottom to the top of 55 m. The selected finite elements for the walls and roof were shell 43; each FEM consists in a total of 2125 nodes and 2050 shell elements. 3D views of the FEM are shown in Fig. 1. In the generation of the FEM the following main assumptions were taken into account: since the type of foundation and soil characteristics of the towers were not considered, all the base nodes were assumed as fixed as shown in Fig. 1c. The main mechanical properties of the masonry were determined considering average values of ancient masonry reported in the literature. The selected assembly was considered as carved stone with lime mortar, and the properties are resumed as follows: an average density of 2000 kg/m^3 and a Young's modulus of 2000 N/mm^2. The Poisson's ratio was held constant and equal to 0.1. Regarding the strengths, it was considered for compression 3.5 N/mm^2 and tension 0.25 N/mm^2. For the non-isolated FEM (see Fig. 1c) the interaction between the tower and neighbor buildings in the North and East façades was simulated by a uniform distribution of linear elastic springs of constant stiffness.

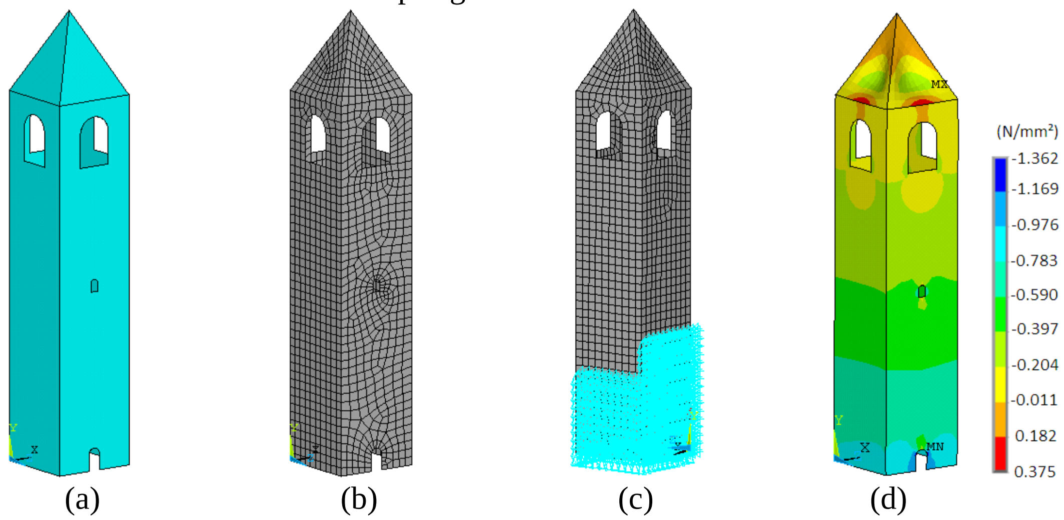
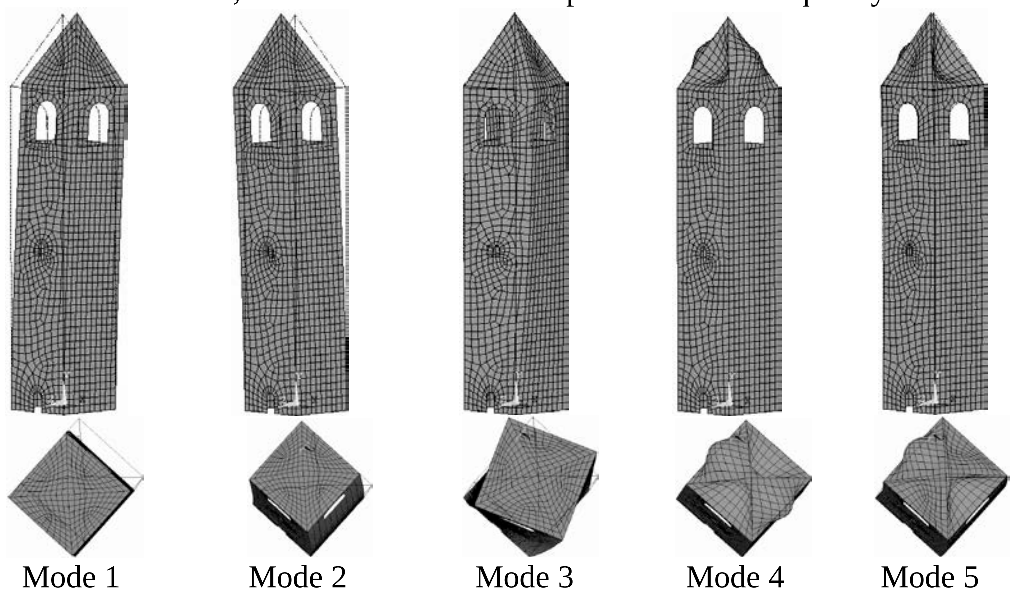


Figure 1: FEM of the theoretical towers; (a) 3D view; (b) isolated; (c) non-isolated with all the applied boundary conditions; and (d) vertical distribution of stresses

Linear analyses of the FEM As a first approach and with the aiming of obtaining significant progress towards the seismic risk assessment of the FEM without the convergence problems related with nonlinear analyses, linear static and dynamic analyses were developed. This first approach based in linear principles enables to determine the presence and magnitude of tensile and compressive stresses in the masonry structure generated by vertical loading, as well as the dynamic properties such as natural frequencies and vibration modes in the modal analysis.

Vertical loading analysis Both FEM were analyzed considering only the presence of dead load generated by the self weight of the structure, the results show that the obtained distribution of vertical stresses is similar. It means that in the non-isolated model the interaction with the neighbor buildings do not have any effect in the results. The distribution of vertical stresses of the complete tower due to self weight shows that the highest compressive stresses are present at the lower part of the structure, with values of 0.98 N/mm^2 in the four façades, and the most overloaded elements correspond with the ones located around the main door at the bottom of the tower, with stresses of about 1.17 and 1.36 N/mm^2 (see Fig. 1d). Regarding the tensile stresses, the models present in some elements at the bottom of the roof (in the connection between walls and roof) in the four façades values of about 0.18 and 0.38 N/mm^2. These results obtained in the roof are in complete agreement with the real behavior of this kind of heavy structural elements as in the case of domes and cupolas of ancient masonry, that tend to open at the lower part due to the tensile stresses generated for the elevated self weight and the effects of temperature as well. Anyway, even when tensile stresses were detected in some elements at the bottom part of the triangular roof, the analysis revealed that the complete tower is basically in linear conditions, since the level of compressive stresses are lower than the masonry strength, and tensile stresses are not present in a large zone of the structure. These results allowed a first validation of the FEM, concluding that the towers are stable to resist at least their own self weight satisfactorily as it is expected in most of the historical constructions.

Modal analysis The linear investigation was extended to a modal analysis; it was applied in both FEM in order to obtain a first estimation of the dynamic response of the structures. As a first stage, the modal parameters of the isolated tower were obtained; Fig. 2 shows the resulting vibration modes. For the non-isolated tower the vibration modes are similar than those obtained for the isolated one; the only difference between both models is in the natural frequencies which are a bit higher (lower periods) for the non-isolated tower due to the increasing of stiffness generated by the assumed contact with the neighbor buildings (see Table 1). Analyzing the results it is observed that the two main vibration modes of both FEM correspond with a general flexion. The third mode represents a torsional vibration and the fourth one a particular problem due to the vertical vibration of the tall and heavy triangular roof. The lower frequencies of these two first modes compared with the last three lead to higher periods, being representative of slender and tall structures (very vulnerable to seismic actions), as in the case of historical masonry towers. The rest three modes have higher frequencies (lower periods), being considered with this less important than the two first ones, due that they could be less excited if an earthquake occurs. This case of higher frequencies and low periods becomes more important for compact and rigid structures as in the case of most of the historical masonry buildings. For the validation of both FEM regarding modal analysis, it was considered the mentioned about the lower frequencies (the two main vibration modes), dynamic property characteristic of slender and tall masonry towers, as well as the increment in the frequency of the first vibration mode of the non-isolated tower due to the boundary conditions, becoming stiffer than the isolated one. As a complement in the validation it was used the reported in Bachmann et al. (1997) and Casolo (1998), where the authors comment that the main two frequencies for slender and tall towers are measured between 0.9 and 2 Hz. Moreover it was considered the equation proposed by NCSE (2002) (see Eq. 1), to assess approximately the first frequency of bell towers. Ivorra et al. (2008) and Bayraktar et al. (2009) have demonstrated that Eq. 1 allows successfully the estimation of the modal parameters (first frequency) of real bell towers, and then it could be compared with the frequency of the FEM.



Mode 1 Mode 2 Mode 3 Mode 4 Mode 5

Figure 2: Vibration modes of the isolated tower

In Eq. 1, L corresponds with the plant dimension along the vibration direction, and H is the height of the tower. As a result it is expected to acquire in the isolated tower an approximated first frequency of

1.119 Hz. The result is in complete agreement with the one obtained in the modal analysis, 1.046 Hz. For the case of the non-isolated tower it is estimated to obtain a greater first frequency since its stiffness is higher as a consequence of the contact with the neighbor buildings.

Table 1: Modal parameters of the FEM

Mode No.	*Vibration Mode*	*Frequency [Hz]*		*Period [sec]*	
		Isolated Tower	*Non-Isolated Tower*	*Isolated Tower*	*Non-Isolated Tower*
1st	Bending N-S	1.046	1.293	0.956	0.773
2nd	Bending E-W	1.051	1.133	0.951	0.883
3rd	Torsion	3.313	3.702	0.302	0.270
4th	Vertical	3.464	3.464	0.289	0.289
5th	Bending N-S	3.935	4.138	0.254	0.242

$$\omega_1 = \frac{\sqrt{L}}{0.06 * H * \sqrt{\frac{H}{2 * L + H}}} \qquad (1)$$

Nonlinear analyses of the FEM Afterwards the validated FEM were analyzed by means of the displacement-based (all the nodes at the connection between cover and walls) pushover method to obtain the capacity curves. The applied material model was developed by Gambarotta and Lagomarsino (1997), and Calderini and Lagomarsino (2004) and used in combination with ANSYS®. To obtain a link between the capacity curves of the FEM and the seismic vulnerability, it was used the method developed by Lang (2002) where the author related the capacity curves of existing buildings with the damage grades of the EMS-98 (Fig. 3a). For including the seismic demand (the hazard) in the seismic risk evaluation of the FEM and in order to measure the amount of damage produced by the seismic action over the structure, it was possible to take into account as well the seismic intensity scale of the EMS-98 for every one of the damage grades. DG1: Negligible to slight *damage* (*Intensity* V), DG2: Moderate (VI), DG3: Substantial to heavy (VII), DG4: Very heavy and DG5: Destruction (VIII to XII). The capacity curves of both FEM are presented in Fig. 3b. It is observed that definitely the non-isolated tower is stiffer than the isolated one as it was expected. The isolated tower is considered then as more ductile, reaching the yielding at the displacement of 60 mm and a lateral force of 3000 KN for an intensity of VI (DG2), and ultimate load capacity at 106 mm (4380 KN) for an intensity of VIII (DG4). Meanwhile for the same intensities and damage grades the non-isolated tower reached the yielding at 55 mm (3028 KN) and ultimate load capacity at 97 mm (4328 KN).

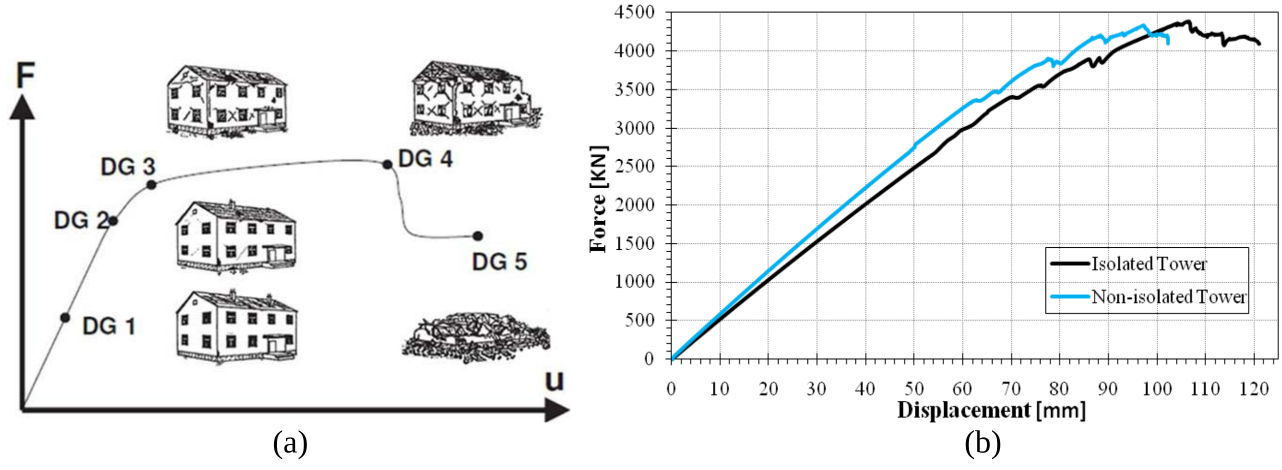


Figure 3: Results of the nonlinear analyses; (a) connection of the EMS-98 with a capacity curve (Lang, 2002); and (b) capacity curves of the theoretical towers in their original state

Seismic risk mitigation of the theoretical ancient masonry towers

Rehabilitation proposals Results of Fig. 3b were quite helpful to compare the different prestressing devices aiming to increase the overall strength (shear and flexion) and ductility of the towers against seismic actions (seismic risk mitigation). Nonlinear static analyses were developed in ANSYS® using the same pushover method and material model in the FEM, in order to evaluate now the impact in the seismic behavior of different prestressing devices such as post-tensioned tendons located vertically and externally inside the four corners of the towers in order to give to the rehabilitation the characteristic of reversibility. In this first approximation the investigated post-tensioned tendons were prestressing steel, Aramid FRP (AFRP) (Arapree), NiTi SMAs and the combination of them. The devices were anchored at the top and bottom of the towers and post-tensioned to produce a uniform overall distribution of compressive prestressing to the masonry. Several numerical simulations were developed varying the material of the tendon and combination of them, as well as two levels (low and high) of the applied compressive prestressing, 0.1 and 0.4 N/mm^2. It was observed that the strength and ductility was increased by the rehabilitation measure depending on the prestressing level and the device. In the low prestressing level the stiffest device was steel with an increasing of the ultimate load capacity of 11.25%, meanwhile the least stiff device corresponded to the combination of SMAs with AFRP showing an increment of 5.63%. For the high prestressing level, steel presents an increment of 32.81% and SMAs with AFRP 15.90%. The ductility is increased around 7% in both prestressing levels for all the devices.

Conclusions

The linear analyses allowed obtaining a first approach towards the seismic risk assessment of the towers, giving information such as distribution of stresses and dynamic characteristics. This information in addition to the reported in the literature, observed damages and engineering experience was quite helpful to validate the FEM. The nonlinear static analyses by the pushover method and the material model allowed *assessing the seismic risk* successfully combining the capacity curves of the towers with the damage grades of the EMS-98. Results showed that the ultimate load capacity and ductility could be improved satisfactory depending on the prestressing level and the device, allowing with this the vulnerability reduction of the towers (*seismic risk mitigation*). Even when SMAs combined with AFRP showed less stiff behavior compared to prestressing steel, results interesting to conclude that this combined device allowed less change (increment + and reduction -) of the prestressing force in the order of ± 5%, compared to prestressing steel with a change of ± 20%. This is a quite important property of the SMAs, to limit the stresses applied to the masonry by means of keeping the applied prestressing force almost constant (force controlled). In the final phase of this research, the same methodology will be applied in two real ancient masonry towers located in seismic areas (Italy and Mexico). The nonlinear investigations (behavior of the prestressing devices) will be extended to dynamic by time-histories for earthquakes of different intensities considering the seismic hazard of the research zones and the response spectrums reported in the codes.

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