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Seismic Vulnerability Enhancement of Medieval and Masonry Bell Towers Externally Prestressed with Unbonded Smart Tendons

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ABSTRACT

Medieval and masonry bell towers are highly vulnerable to suffer strong earthquake damage due to the mechanical and physical characteristics of masonry and other important factors. An approach for the seismic vulnerability reduction of masonry towers with external prestressing is proposed. The devices are vertically and externally located in order to be removable when needed. The characteristic flexural failure mode of medieval towers and the shear mechanism of bell towers are simulated. Both failure modes are in agreement with earthquake damage in similar towers. Medium prestressing level enhances force capacity of towers failing by bending without reducing ductility. High prestressing level slightly reduces the displacement capability of towers failing ductile. In case of belfry failure, both prestressing levels permit to increase displacement but lower force than towers failing by bending. The proposed medium prestressing level is the optimal for masonry towers and other slender structures failing by bending and shear.

Keywords: *earthquakes; historical masonry; towers; failures; energy dissipation; external prestressing; unbonded tendons; smart materials*

1. INTRODUCTION

Unreinforced masonry (URM) structures are highly vulnerable under earthquake (EQ) conditions and may present a total collapse. This is due to the anisotropy, heterogeneity and poor tensile strength of masonry and other important factors affecting the structural vulnerability such as geometry, structural configuration, EQ source, etc. Seismic risk management of existing buildings located in EQ prone areas is integrated by two huge stages, the vulnerability assessment and its reduction. There is an enormous number of methods to assess the seismic vulnerability of buildings (Carreño et al., 2012), but not completely clear within the scientific community regarding the procedure to follow for assessing the vulnerability and the measures for its reduction. Recent studies in EQ engineering are oriented to the development, validation and application of techniques to assess the seismic vulnerability of existing buildings (Carreño et al., 2007, Barbat et al., 2008, Lantada et al., 2009 and Pujades, 2012). Assessing the seismic vulnerability of a historical building is a complex task if compared to other existing building as explained by Carpinteri and Lacidogna (2007), Barbieri et al. (2013), Foraboschi, 2013, Preciado and Orduña (2014) and Preciado et al. (2014 and 2015a-c). There is a large variety of techniques and materials available for the protection of historical masonry constructions. Among them, two main techniques are distinguished: rehabilitation and retrofitting. On the one hand, in the rehabilitation process is taken into account materials of similar characteristics to the original ones and the same construction technique to locally correct the damaged structural elements to preserve the building in original conditions. On the other hand, retrofitting uses advanced techniques and materials to improve the seismic performance (energy dissipation) of the building in terms of ultimate lateral load and displacement capacity. Compatibility, durability and reversibility are the fundamental aspects recommended in literature to be taken into account when retrofitting is used for the seismic protection of cultural heritage.

The main objective of this research is the achievement of the seismic vulnerability enhancement of historical masonry towers by the implementation of reversible prestressing devices. The approach is integrated by three main stages: initial analyses of the proposed virtual towers; simulation of typical EQ damage and behavior, as well as the seismic enhancement by externally prestressed smart tendons. The devices are vertically and externally located at key locations inside the towers in order to give to the retrofitting the characteristic of reversibility (not invasive), without affecting the architectonic and historic value of the structure. The devices intend to improve the seismic performance by reducing damage with the application of a uniform overall distribution of compressive stresses. This precompression state completely changes the poor response of unreinforced masonry against lateral loading by reducing the tensile stresses at key zones and transforming them into compressive ones. The quasi-brittle behavior of masonry may be changed by means of prestressing in order to obtain a high energy dissipation system by providing more lateral strength and displacement capacity. These improvements at the retrofitted masonry structure in terms of seismic behavior are also represented by more ductile failure mechanisms, which may be interpreted as seismic energy dissipation.

2. SEISMIC VULNERABILITY OF HISTORICAL MASONRY TOWERS

Historical masonry towers were built either isolated or commonly included in different manners into the urban context, such as built as part of churches, castles, municipal buildings and city walls. Bell and clock masonry towers (see Fig. 1), also named civic towers, were built quite tall and with large belfries for informing people about time and extraordinary events such as civil defence, fire alarm and social meetings. Another reason that led to the construction of tall civic towers in the medieval cities of Italy was that they were seen as a symbol of richness and power of the great families. On the other hand, medieval towers were built quite high but with almost no openings mainly for warlike purposes. Strong damage or complete loss suffered by the cultural patrimony due to EQs has occurred through the history of humanity. The occurrence of these unexpected and unavoidable events has demonstrated that towers are one of the most vulnerable structural types to suffer strong damage as shown in Figure 2. Their protection is a topic of great concern among the scientific community. Although recent progress in technology, seismology and EQ engineering, the preservation of these quasi-brittle and massive monuments stills represents a major challenge. Masonry towers in all their uses are highly vulnerable to suffer strong EQ damage, even when subjected to seismic events of low to moderate intensity.

Towers are slender by nature, the slenderness (H/L) is the single most decisive factor affecting their seismic performance, characterized by a ductile behavior where bending and low tensile strength of masonry determine the overall performance. This slenderness may be measured by ambient vibration tests aimed at obtaining the natural frequencies of the tower and the vibration modes may be analysed by commercial software. These linear elastic evaluations of finite element models (FEM) are relatively fast due to the progress of recent decades on computational tools and in combination with the results from the in-situ campaigns permit to define reliable models as explained by D'Ambrisi (2012). Julio et al. (2008) successfully evaluated the structural integrity of a masonry tower by modal identification and concluded that this is a fast and reliable in-situ technique to establish the structural assessment of towers and other buildings. Bachmann et al. (1997), Meli (1998), Casolo (1998) and Preciado (2011) describe in their works that the natural frequencies of slender masonry towers are measured between 0.9 and 2 Hz (periods between 0.5 and 1.11 s). As a reference, the reader may find in Table 1 the natural frequencies of 10 historical masonry towers with variations in height and geometrical characteristics.

Also the position of a tower into the urban context is another important aspect that influences vulnerability (Sepe et al., 2008). These boundary conditions could strongly modify the seismic behavior and failure modes. Non-isolated towers were commonly built as part of churches or next to another building. In addition, the seismic vulnerability of towers is increased by certain important aspects as soil conditions, large openings at belfries, high vertical loading and progressive damage. Towers were built as most of the historical buildings, to mainly withstand their vertically induced self-weight. During construction, wall thicknesses used to be determined by following empirical rules by trial and error, mainly based on the structure´s height and previously observed EQ damage. These empirical rules led to the construction of walls with enormous thicknesses, in some cases higher than 2m. Masonry towers are slender structures under high vertical loading due to the height, wall thickness, tall roof system, high density of masonry and large bells. This loads lead to a concentration of high compressive stresses, mainly at the base. All these issues and moreover taking into account the deterioration of masonry through the centuries make towers extremely vulnerable to suffer a sudden collapse by exceeding the intrinsic compressive strength. These sudden collapses have been occurring since centuries ago in this type of structures as explained in the works of Binda et al. (1992), Macchi (1993), GES (1993) and Binda (2008).

2.1 Earthquake behavior and typical failure modes of masonry towers

Seismic behavior and failure modes identification of masonry towers subjected to lateral and vertical simultaneous loads induced by EQs is a complicated task. This identification strongly depends on many factors such as soil and boundary conditions, geometry and mechanical properties of masonry, vertical loading and EQ characteristics. Compared to other compact structures, towers mainly fail ductile in a predominant bending behavior because of the excessive slenderness (height / length > 4). Due to this, as well as the heavy mass, the lateral vibration at the top of the tower during an EQ is considerably more amplified than the base, inducing important displacements by inertia forces. This behavior could cause different failure modes as illustrated in Figure 3. Meli (1998) describes that during an EQ, masonry bell towers with reduced openings present important horizontal top displacements. Bending mainly generates horizontal cracks but rarely the complete overturning. This is due to the alternation of the movement that causes an opening and closing effect of these cracks, dissipating an important part of the EQ energy with the impact.

Even when bending failure of towers is considered to be ductile, another failure mechanism may be triggered due to the flexural behavior. This mechanism is the crushing of masonry at the compressed in-plane or out-of-plane toes by flexion and heavy vertical loads of towers. This failure mode is considered as brittle, with the sudden formation of an explosive mechanism which may lead to the complete tower failure (see Figure 3a). Foraboschi (2014) gives more information about the differences between masonry towers and general masonry buildings. There are just a few references that may be found in the literature aimed at studying this brittle failure mechanism presented in-plane and out-of-plane (e.g. Preciado, 2011, Vanin and Foraboschi, 2012 and Preciado, 2015). On the other hand, in towers presenting large openings at belfry, the main failure mode may be triggered by shear stresses. Compared to the bending ductile failure, the shear mode is quasi-brittle, which may lead to complete belfry collapse. This brittle failure mode is also represented at the structural capacity curve with a reduced amount of energy dissipated, due to the fact that the formed envelope between ultimate shear force against displacement is reduced. Due to the strong damage and quasi-brittle behavior, the belfry could collapse by instability, endangering the adjacent buildings and mainly people who could be inside or in the surroundings. The last almost happened due to the M7.5 Colima EQ in 2003 as explained in the research work of Preciado (2011).

The key behavior of masonry bell towers during EQs is mainly dominated by in-plane failure in the direction of the façade. The out-of-plane failure is generally less important and is only regarded with the detachment of the façade from the nave (Alcocer et al., 1999). Curti et al. (2008) observed in 31 Italian bell towers damaged by the 1976 Friuli EQs that the belfry is the most vulnerable part of the tower due to the large openings, bending behavior and low tensile strength of masonry. Peña and Meza (2010) developed post-earthquake observations in 172 Colonial masonry churches with bell towers after two major EQs occurred in 1999 in Puebla and Oaxaca, Mexico. The authors identified that the main damage in masonry towers is at belfry, due to the great openings and heavy mass, with no observed masonry crushing at the base. Lagomarsino et al. (2002) propose the damage modes of Figure 4 and may be summarized as follows: in plane and out-of-plane body damage by bending and shear; vertical cracking in both planes by horizontal tension; alternated diagonal in-plane cracking and belfry damage by horizontal and diagonal cracking due to large openings.

In brief, the author of this paper may conclude that the main failure mechanisms presented in masonry bell towers due to EQ loading are the following: (1) horizontal cracking at the tower´s body due to bending behavior; (2) stepped or diagonal cracking at the tower´s body by shear stresses at openings; (3) vertical cracking at the tower´s body due to horizontal tensile stresses generated by the detachment from other vertical elements; (4) partial or total collapse of belfries due to shear or bending stresses and (5) masonry crushing at the compressed toes.

3. SEISMIC ASSESSMENT OF MEDIEVAL AND BELL TOWERS UNDER STUDY

General view and FEM models of the medieval and masonry bell towers under study are illustrated in Figure 5. They were selected taking into account common masonry towers with variations in geometry and openings at belfry (see Fig. 1). In following sections, the models are described in detail and initially analyzed by linear analyses and compared against theoretical background. In a subsequent stage, the towers under study are subjected to nonlinear static analyses to assess their seismic behavior and failure modes. Moreover, the towers are retrofitted with a proposed external prestressing device and posttensioning force. The main goal is the achievement of the EQ performance enhancement by providing a better overall behavior of both main groups of towers, bell and medieval, by converting brittle shear failures into ductile ones with higher energy dissipation capability.

3.1 Description and linear elastic analyses of FEM models

Medieval and masonry bell towers of Figure 5 are isolated (with no neighbor buildings) and have a light timber roof, both with a rectangular section of 5 x 5 m, wall thickness of 1.5 m and a total height of 32 m. The wall thickness is considered as constant along the complete height of both towers and the light timber roof may be neglected in order to simplify the nonlinear assessments. Another technique consists in including the light timber roof with linear elastic behavior for simplifying the nonlinear evaluations. Moreover, the main goal is to assess the damage at masonry elements and not at the timber roof.

The model of Figure 5a is representative of medieval towers and is named MT1 and the MT2 model of Figure 5b is a representation of masonry bell towers with large openings at belfry. Both 3D FEM models are developed by the commercial software ANSYS®. The selected element for walls is Shell43 with six degrees of freedom (DOF) and four nodes. Every node may have different height in order to describe elements with thickness variation. This element is able to represent in-plane and out-of-plane behavior and has plasticity and creep capabilities. The MT1 is built by 640 quadrilateral shell elements with a mesh size of 1m x 1m and the MT2 by 600 elements respectively. In the generation of the numerical models the following main assumptions were taken into account: (1) because the type of foundation and soil properties are not considered, all the base nodes are assumed as fixed; (2) The main mechanical properties of the MT models are proposed by taking into account average values reported in literature: (Meli, 1998, Calderini and Lagomarsino, 2006, Urban, 2007, Sperbeck, 2009, Preciado, 2011 and 2015 and Preciado et al. 2015a-c). The selected masonry is considered as carved stone with lime mortar, average density of 2000 kg/m^3 and a Young´s modulus of 2000 MPa. The Poisson´s ratio is held constant and equal to 0.15. The compressive strength is assumed to be 3.5 MPa and the tensile strength 0.25 MPa.

The natural frequencies and vibration modes of the medieval and masonry bell towers (MT1) with timber roof (MT2) are numerically obtained and presented in Figure 6 and Table 2. The vibration modes and frequencies are similar for both towers (medieval and bell). Analyzing the results of Figure 6 and Table 2, it could be observed that the two fundamental vibration modes of both towers correspond to general bending. This low frequencies (high periods of about 1 s) and vibration modes are representative of the real behavior of slender and tall structures as MT, which are highly vulnerable to EQ loading. The higher modes and frequencies obtained through the modal analyses are related to torsion. Structural modal analysis represents a helpful tool to obtain a first estimation of the dynamic response of FEM in the linear elastic range.

3.2 Seismic behavior and failure mechanisms by nonlinear static analyses

The nonlinear static analyses of the MT are developed by the pushover method including a suitable and validated masonry model representative of this material. As a result, it is possible to obtain the seismic behavior and failure mechanisms of the medieval and masonry bell towers against EQs. In order to have comparative indicators of performance, it is included at the capacity curves the EQ performance limit states established by the European Code (EC-8) (Eurocode 8, 2004); the damage limit state (DLS) at first yielding; significant damage limit state (SDLS) representing large damage and the ultimate limit state (ULS) near collapse. Moreover, these limit states at the capacity curves are correlated to the damage grades (DG) DG2, DG3 and DG4 proposed by the European Macroseismic Scale (EMS-98) reported in Grünthal (1998). For having quantitative indicators of performance at the capacity curves, it is included the seismic coefficient (SC), which is determined by the ratio between the ultimate lateral force at ULS and vertical loading. The SC is typically expressed as a fraction or percentage of the gravity (g). The main drawback of this indicator is that only the lateral strength of the structure is evaluated, disregarding the displacement and ductility capability, which are extremely important in the EQ assessment of structures for energy dissipation capabilities.

In the nonlinear analyses through FEM models, the homogenized masonry material model developed by Gambarotta and Lagomarsino (1997) is implemented. This model is capable to simulate the main failure modes and behavior of masonry structures in static and dynamic conditions and is integrated in ANSYS® by subroutines. The model is based on the macro-modeling approach, which is considered as appropriate for the seismic assessment of large historical constructions. The suitability of the material model in masonry structures has been demonstrated through numerical simulations by Calderini and Lagomarsino (2006), Urban

(2007), Sperbeck (2009) and by Preciado (2011 and 2015) against experimental results reported in literature, e.g. Van der Pluijm and Vermeltfoort (1991), Raijmakers and Vermeltfoort (1992) and Vermeltfoort and Raijmakers (1993). The continuum damage model is based on a micromechanical approach where masonry is assumed as a composite medium made up of an assembly of units connected by bed mortar joints. The contribution of head joints is not considered. The constitutive equations are obtained by homogenizing the composite medium and on the hypothesis of plane stress condition. The model is characterized by three yield surfaces: tensile failure, sliding of mortar joints and compressive failure of units (see Fig. 7). In brief, if tensile stresses act in mortar bed joints $\sigma_y \geq 0$, three damage modes may become active: failure of units, sliding and failure of mortar bed joints. On the other hand, if mortar joints are under compressive stresses $\sigma_y < 0$, then both damage mechanisms of units and mortar are activated. The needed masonry material parameters are described in Table 3.

The FEM models (see Fig. 5) are firstly loaded with the gravitational force, and in a subsequent stage, the horizontal force is applied under monotonically increased top displacement control. From the evaluations, it is possible to obtain the complete capacity curve and failure mechanisms, especially to capture the nonlinear (plastic) range. In the analyses, the displacement-based load pattern is applied through a considerable number of steps and sub-steps, especially in the nonlinear range to attain convergence. The failure modes of the medieval tower (MT1) and bell tower (MT2) are illustrated in Figures 8a, 10a, 12a and 14a. The medieval tower MT1 presents a global bending behavior represented by the initial formation of horizontal cracks due to vertical tensile stresses at the base level at a displacement of 155 mm, which corresponds to a DG3 and SDLS. The tower reaches an ULS and a DG4 at a displacement of 265 mm. The final collapse mechanism (see Fig. 8a and 10a) is formed due to the extension of the horizontal cracks. The failure by masonry crushing is not observed due to the maximum value of stress in the compressed in-plane and out-of-plane toes is in the order of 3.086 MPa, which is lower than the intrinsic strength (3.5 MPa).

On the other hand, the bell tower (MT2) presents a different behavior as illustrated in Figures 12a and 14a. The main mechanism at ULS (U= 130 mm and F= 1839 kN) is identified as quasi-brittle failure by concentration of shear stresses at the large openings of belfry in combination with horizontal cracks at the body by initial bending. This is the most common failure mode presented by masonry bell towers under EQs. The large cracks may lead to the sudden collapse of belfries, placing in a situation of danger the adjacent buildings and people. For towers failing by shear stresses at belfry is quite important to enhance force and displacement capacity by inducing a flexural ductile failure, but mainly confinement. The maximum reached stress at the compressed toes at ULS of this tower is in the order of 2.25 MPa, much lower than in the ductile flexural failure presented by the medieval tower and lower than the intrinsic strength. The capacity curves of the medieval and masonry bell towers (MT1-2) including the damage grades (EMS-98) and limit states (EC-8) are illustrated in Figures 9, 11, 13 and 15. Both towers present similar linear behavior, reaching the yielding (DG 2, DLS) at a displacement of 55 mm in case of the medieval tower and at 65 mm the bell tower. The towers present different nonlinear behavior at a DG3 and SDLS, being more evident the difference at ultimate conditions (DG4, ULS). The medieval tower shows an evident ductile behavior of 265 mm (F= 1600 kN) against the quasi-brittle behavior of 130 mm (F= 1839 kN) of the bell tower. The seismic analyses summary of both towers is presented in Table 4. The two towers have similar vertical loading, with a small variation in the tower with openings MT1 (less mass). The obtained low values of SC represent in quantitative terms the high vulnerability of this type of structures to seismic actions. The SCs are in good agreement with typical values of ancient masonry buildings, between 0.1 and 0.3 (Meli, 1998). In contrast, for seismically designed masonry buildings, the SC is in the range between 0.5 and 0.86. In conclusion, both MT models would reach an ULS or collapse under an EQ ground motion of about 0.1 g. The SC permits to the user to obtain more reliable results (quantitative) than the qualitative damage indicators. On the other hand, it is not possible to obtain information with this coefficient about maximum displacement capability.

4. EARTHQUAKE PERFORMANCE UPGRADING BY EXTERNAL PRESTRESSING

The technique of prestressing has been successfully used to improve the seismic behavior of concrete structures since the beginning of the XX century. The adaptation of this technique to the seismic retrofitting of cultural heritage has gained in recent decades especial interest for many researchers around the world. Post-tensioning (or prestressing) of masonry has shown to successfully improve ductility and strength as explained in the works of Ganz (1990 and 2002), Indirli (2001), Castellano (2001), Sperbeck (2009) and Preciado (2011).

The technical solution that may be adopted to obtain a dissipative structure that adequately reduces the forces due to the elastic spectrum consists of transforming the masonry into high-dissipative reinforced masonry (Foraboschi, 2013). The most effective technique to convert URM into reinforced masonry is to epoxy bond Fiber-Reinforced Polymer (FRP) strips onto the external surface of the masonry (Ascione et al., 2005; D’Ambrisi et al., 2013a and b; Foraboschi and Vanin, 2013; Muciaccia and Biolzi, 2012 and Fedele et al., 2014). FRP composite materials bonded to the masonry surface of towers may be another retrofitting possibility to either prevent the kinematic mechanism from triggering or to provide confinement to prevent crushing as explained by Foraboschi and Vanin (2013). Since historical buildings must be retrofitted with reversible techniques for not affecting its architectonic value (the bare-surface has to be kept unchanged), no plaster and FRP strips may be applied to the masonry. Therefore the need of another technique such as removable prestressed tendons is highly recommended in the relevant literature, Indirli (2001), Castellano (2001), Sperbeck (2009) and Preciado (2011). One solution that may be implemented is the external or internal prestressing by tendons and anchorage system at key identified parts at the structure by the seismic vulnerability assessment. This technique is in compliance with the demand for architectural conservation and may be horizontally and vertically located without bonding (unbonded) in order to be fully removable. Moreover, external prestressing is more economic than internal prestressing because it does not need masonry drilling, which damages the structure and needs specialized and expensive equipment. The unbonded condition permits the future tendon calibration and control of changes in prestressing forces by material relaxation and volumetric changes. The use of FRP stripes bonded to the masonry may be useful in combination with a removable prestressing system, by wrapping the masonry structure at key non visual parts as in the case of belfries (Preciado et al., 2015a-c).

4.1 Seismic retrofitting of ancient masonry towers

Even when external prestressing has been frequently used as seismic retrofitting measure of cultural heritage, very few applications of this technique can be found in ancient masonry towers (Preciado, 2011). Past applied intervention techniques in masonry towers have been used more as local strengthening (to avoid out-of-plane failure) of certain vulnerable structural parts than for a real improvement of the global structural performance against EQs. This is consequence of the limitations in the existing materials in those periods, added to the lack of technology and knowledge about the real behavior of these structural elements. One of the few cases reported in literature are related to the strengthening of the General Post Office clock-tower in Sydney, Australia (Ganz, 2002) and the tower of the church of San Giorgio in Trignano, Italy (Indirli, 2001 and Castellano, 2001). However, in both real applications the retrofitting effectiveness was validated in qualitative terms with no numerical simulations to support the suitability of the retrofitting proposal. Moreover, the way of determining the post-tensioning force is not mentioned and the combination of a high resistance material such as prestressing steel with an extremely poor material as masonry is doubtful in terms of compatibility of deformations and stress concentrations. In the context of this paper, a prestressing device is a structural member axially stressed in tension and is integrated by three main parts, top and bottom anchorages and tendon. The prestressing devices are vertically and externally located at key locations inside the towers to give to the retrofitting the characteristic of reversibility, respecting in all senses the architectonic and historical value (without plaster and invasive elements such as bonded FRPs). Compatibility, durability and reversibility are fundamental aspects recommended in literature to be taken into account for the seismic retrofitting of cultural heritage. Reversibility is definitely the most important aspect, because if the applied technique shows compatibility deficiencies that increase seismic vulnerability or there is a new material that permits to obtain a better structural performance, the initial retrofitting may be substituted.

5. PRESTRESSING DEVICES AND FORCES

The prestressing devices are vertically and externally located at key locations inside the towers (unbonded) in order to give to the retrofitting the characteristic of reversibility. The external prestressing at the MT1 and MT2 (Figure 5a) consists of four vertical devices without drilling, inside of the internal corners of the tower and anchored at the top and foundation. For this case, the tendon material could be made of high resistance steel and smart materials such as FRP and shape memory alloys (SMA). Due to the elevated cost of SMA, this material is regularly used in segments of about 0.30 m. The used FEM elements to simulate tendons correspond to Link10 (only tension) and Solid185 for SMA. Vertical prestressing was selected against horizontal one because it has been demonstrated to be more suitable to increase strength and ductility of masonry structures (Sperbeck, 2009 and Preciado, 2011). The level of improvement strongly depends on the level of prestressing force, so, the higher the initial prestressing force the higher the lateral strength and ductility enhancement. Especial careful may be taken into account in order to use this technique in historical masonry towers. Firstly, an optimal prestressing level may be designed, due to high prestressing levels could lead to local damage at the top anchorage zone, or a sudden collapse even in static conditions by an exceedance of compressive stresses at the bottom (crushing). Moreover, in seismic conditions, the compressed in-plane and out-of-plane toes could fail by crushing as well, and to induce a quasi-brittle failure due to the explosive behavior of this mechanism. From an extensive parametric study on different configurations of old masonry towers, Preciado (2011) proposes an optimal prestressing force and device that may be used in any compact or slender masonry structure ranging from light houses, medieval, civic and bell-towers with large openings at belfries (bells place). The parametric study included different tendon material such as conventional prestressing stainless steel, smart materials as FRPs (Aramid and Carbon) and different SMAs. The last material is also called NiTinol (Nickel-Titanium) and presents a super elastic behavior. This smart material can undergo large deformations in loading and unloading cycles without permanent deformations forming a loop which represents the dissipation of energy. This superelastic material has found very interesting applications as seismic retrofitting of cultural heritage. The main goal of the parametric study developed by Preciado (2011) was the investigation of the impact on the seismic performance of different parameters such as tendon material (steel and FRPs) and combinations with segments of SMAs, prestressing level, changes in tendon forces and SMA superelasticity.

6. SEISMIC ENHANCEMENT OF THE MEDIEVAL AND BELL TOWERS

In the seismic assessment of the medieval and masonry bell towers of Section 3, it was identified a ductile bending behavior of the medieval tower and a quasi-brittle failure of the bell tower due to shear stresses at the large openings of belfry. Taking into account the parametric study developed by Preciado (2011), both towers are retrofitted with four prestressing devices (anchorage plate and tendon) of FRPs. Compared to prestressing steel, smart FRPs are more resistant to corrosion, equal or superior tensile strength, insensitivity to electromagnetic fields, 15 to 20% lighter and the possibility to incorporate optical fiber sensors for monitoring purposes. The disadvantages of FRPs are their vulnerability to fire and brittle failure with no yielding, showing a stress-strain behavior linear at all stress levels up to the point of failure. The recommended prestressing force is of about 40% of the ultimate load capacity for Aramid (AFRP) and 60% for Carbon (CFRP) due to the stress-rupture limitations. It is proposed four tendons of Technora AFRP because its low elasticity modulus of 54000 MPa (allowable stress of 760 MPa) which may be compatible with the poor one of the historical masonry under investigation (E= 2000 MPa). The use of steel tendons is not suitable to interact with degraded historical masonry due to its high elasticity modulus of 210000 MPa.

The devices and anchorage system are made of the same AFRP material and vertically located in the interior part of the tower with no-bonding. The anchorage system is connected at the foundation and top of the tower (see Fig. 5a). The selected FE for the post-tensioned tendon is a uniaxial tension-only 3D spar element (Link10) with linear-elastic behavior. The device is simulated as connected to the supports (foundation) and fixed at the upper level of belfry (32 m) to a perimetral load-distribution beam (Beam4) to avoid force eccentricities. This 3D uniaxial element has linear-elastic behavior with tension, compression, torsion, and bending capabilities.

The prestressing force is applied at the tendons by means of strains. This technique is more realistic to account for restoring forces at the tendon than only applying external normal forces. Restoring forces have a high impact in the realistic simulation of prestressed masonry. This trend was investigated in detail by comparing externally prestressed walls in laboratory and numerically by Sperbeck (2009).

The proposed prestressing force is calculated considering percentages of the total vertical loading. The towers are retrofitted with four Technora devices and two prestressing levels due to the different observed failure modes, 15% of vertical loading 0.15Fv (At= 1000 mm^2, 15 bars of 8 mm per tendon) and 30% of vertical loading 0.30Fv (At= 2000 mm^2, 30 bars of 8 mm per tendon). To develop a comparison between original state and retrofitted, the level of seismic vulnerability reduction is assessed in terms of lateral strength and ductility enhancement based on the EC-8 and EMS-98.

6.1 Ductile flexural failure mechanism of the medieval tower

The ductile flexural failure mode of the medieval tower was successfully simulated and was in complete agreement with the typical observed failure mode of this type of towers, which is governed by global bending and horizontal cracks at the bottom part. In the case of bending failure, becomes extremely important to enhance the seismic performance of the tower by increasing bending resistance without reducing ductility. As aforementioned in previous paragraphs, the medieval tower (MT1) is retrofitted by means of four Technora devices and two prestressing levels, 15% of the vertical loading and 30% respectively. By retrofitting the tower with both post-tensioning levels, it presents at ULS the same failure mode as in original conditions, but with a clear increasing of its lateral force in the order of 13.8% with 0.15Fv and 25.5% with 0.30Fv (see Table 4 and Figs. 8-11). On the one hand, medium prestressing permitted to obtain a displacement enhancement of 7.6% at ULS. On the other hand, the high prestressing level does not enhance the displacement capability of masonry towers failing by bending. Contrary to this, the high prestressing level reduces the bending behavior of towers failing ductile (displacement reduction of approximately 1.9%).

In original conditions, the medieval tower presents a maximum value of stress in the compressed in-plane and out-of-plane toes of 3.086 MPa, which is lower than the intrinsic strength of 3.5 MPa. Retrofitted with the medium prestressing level, the tower reaches 20 mm more displacement than in original state (see Fig. 9), which leads to a more ductile behavior and with this, to a higher concentration of stresses at the compressed toes in the order of 3.342 MPa. The changes of prestressing forces in left tendons are of about +2.38% (increasing) and -1.86 % (decreasing) in the right ones. The direction of the displacement load pattern in the pushover analyses is from left to right (see Figs. 8 and 10), therefore the changes of prestressing forces are higher in the left tendons due to top rotation (cracks opening). In the high prestressing level, the medieval tower presents 3.344 MPa, very close of failing by compressive stresses. The changes of prestressing forces at ULS are reduced, and are of about 2.19% in the left tendons and 1.07% in the right ones. This low change in prestressing forces is due to the reduction of top rotation induced by the high normal forces. In brief, the seismic vulnerability enhancement is obtained by the medium prestressing level, which permits to enhance force and displacement, being fundamental for energy dissipation (see Figs. 9 and 11). This enhancement is also reflected at the reduction of plastic activity at ULS in the comparison of failure modes between original conditions and retrofitted of Figures 8 and 10.

6.2 Quasi-brittle shear failure mechanism of the masonry bell tower

The quasi-brittle failure mode is commonly represented by shear stresses at belfry due to the large openings and is the most common failure mode presented by masonry bell towers under EQs. For towers failing quasi-brittle by shear at belfry and reduced bending behavior, it is quite important to enhance force and displacement capacity by inducing a flexural ductile failure, but mainly confinement. The bell tower (MT2) was retrofitted with medium (0.15Fv) and high prestressing levels (0.30Fv), enhancing both cases the seismic performance in terms of force, displacement and confinement (see Figs. 12-15). In this case, retrofitting permits lower force enhancement than in the flexural mode presented by the medieval tower, but more displacement, which is fundamental for energy dissipation. In original conditions, the tower fails by initial bending at the lower body and a loss of belfry by a combination of in-plane shear and out-of-plane bending. It is worth noting in the comparison of Figures 12 and 14 the way that both retrofitting measures substantially reduce damage by decreasing the plastic activity and extension of cracks especially at belfry.

At ULS, the retrofitted bell tower presents similar failure mode with both retrofitting levels if compared to the one in original state, but presents a more ductile post-peak behavior, which could be interpreted as an increase of confinement that avoids belfry collapse (Figs. 13 and 15). The maximum reached stress at the compressed toes at ULS in original state is in the order of 2.25 MPa, much lower than in the ductile flexural failure presented by the medieval tower. Retrofitted with the medium prestressing level, the bell tower reaches stresses of about 2.535 MPa and changes of prestressing forces in the left tendons of +2.38% and -1.86% in the right ones. In the high post-tensioning level, the bell tower presents compressive stresses in the order of 2.634 MPa and lower changes of prestressing forces than in the medium level due to the high normal forces (left: +1.06%, right: -0.95%). The seismic analysis summary of the medieval and masonry bell towers in original state and retrofitted with the medium and high post-tensioning levels is presented in Table 4. Both towers are compared in terms of seismic vulnerability reduction by taking into account force, displacement and seismic coefficient including the EC-8 and EMS-98. Analyzing the results of Table 4 and Figures 12-15, it is worth noting that the displacement capability at ULS of the tower is increased about 15.4% (11.8% in force capability) due to the medium prestressing level (0.15Fv). In contrast, in the high prestressing level (0.30Fv), the displacement is just enhanced about 7.7% and 4.5% in terms of force.

7. CONCLUSIONS

Masonry towers are highly vulnerable to suffer strong EQ damage due to the anisotropy, heterogeneity and poor tensile strength of masonry and other important factors. An approach for the seismic vulnerability reduction of masonry towers with external prestressing devices was proposed in this paper. The unbonded devices were vertically and externally located at the four internal corners of the towers without drilling. Vertical prestressing was selected against horizontal one because it has been demonstrated to be more suitable to increase strength and ductility of masonry structures. The ductile flexural failure mode of the medieval tower was simulated and was in complete agreement with typical EQ damage. In the case of flexural failure, the main objective is the seismic performance enhancement by increasing bending resistance without reducing ductility. The quasi-brittle failure of the bell tower was simulated and was also in good agreement with typical EQ damage, loss of belfry by shear stresses at large openings. In this failure mode is extremely important to increase ductility.

The Technora FRP device showed good performance in force and displacement enhancement with low changes in tendon forces because of its low elasticity modulus. The suitability of both prestressing levels and device was demonstrated through the simulation of two different typical failure modes of masonry towers. The vulnerability reduction was evaluated by combining the capacity curves and the seismic hazard. Especial attention is suggested when using high prestressing level, because it could generate brittle failure by masonry crushing. On the one hand, medium prestressing level enhanced force capacity of towers failing by pure bending without a reduction of the ductility. On the other hand, the high prestressing level slightly reduced the displacement capability of towers failing ductile. In the case of towers with belfry failure, both prestressing levels increased displacement but allowed a lower force than towers failing by bending due to the brittleness of this mode. The post-peak behavior showed a more ductile behavior, which may be interpreted as an increase in confinement, being favorable to avoid belfry brittle failure. The improvement level in the two failure modes was also reflected in the seismic vulnerability reduction. The proposed medium prestressing level is the optimal for towers failing ductile and quasi-brittle because of the energy dissipation achievement. In conclusion, the proposed approach may be used for any slender masonry structure such as towers, minarets, light houses and so on.

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(a)

(b)

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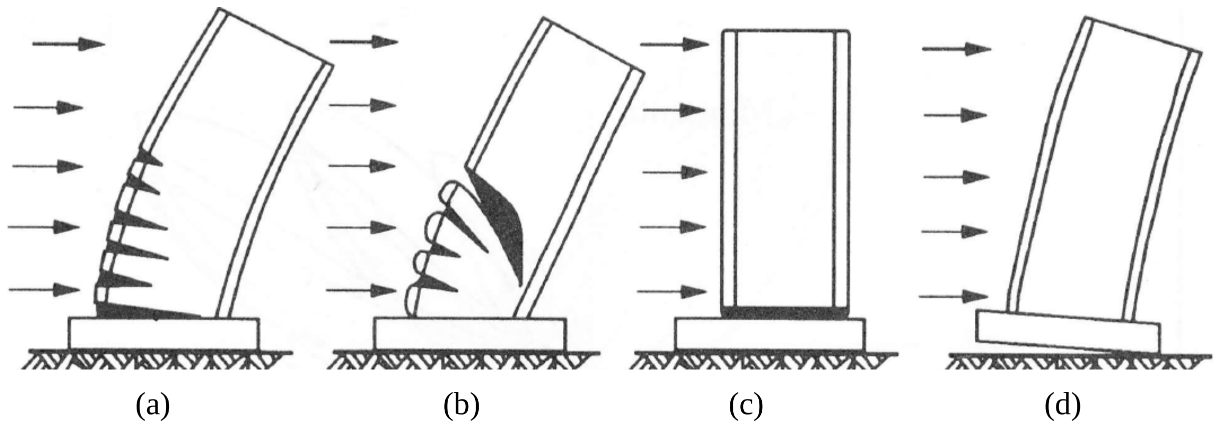
(a) (b)

Figure 2: Typical collapses of historical masonry towers under EQ loading; (a) medieval tower (used as clock tower) before and after the EQ and (b) bell tower with large openings at belfry



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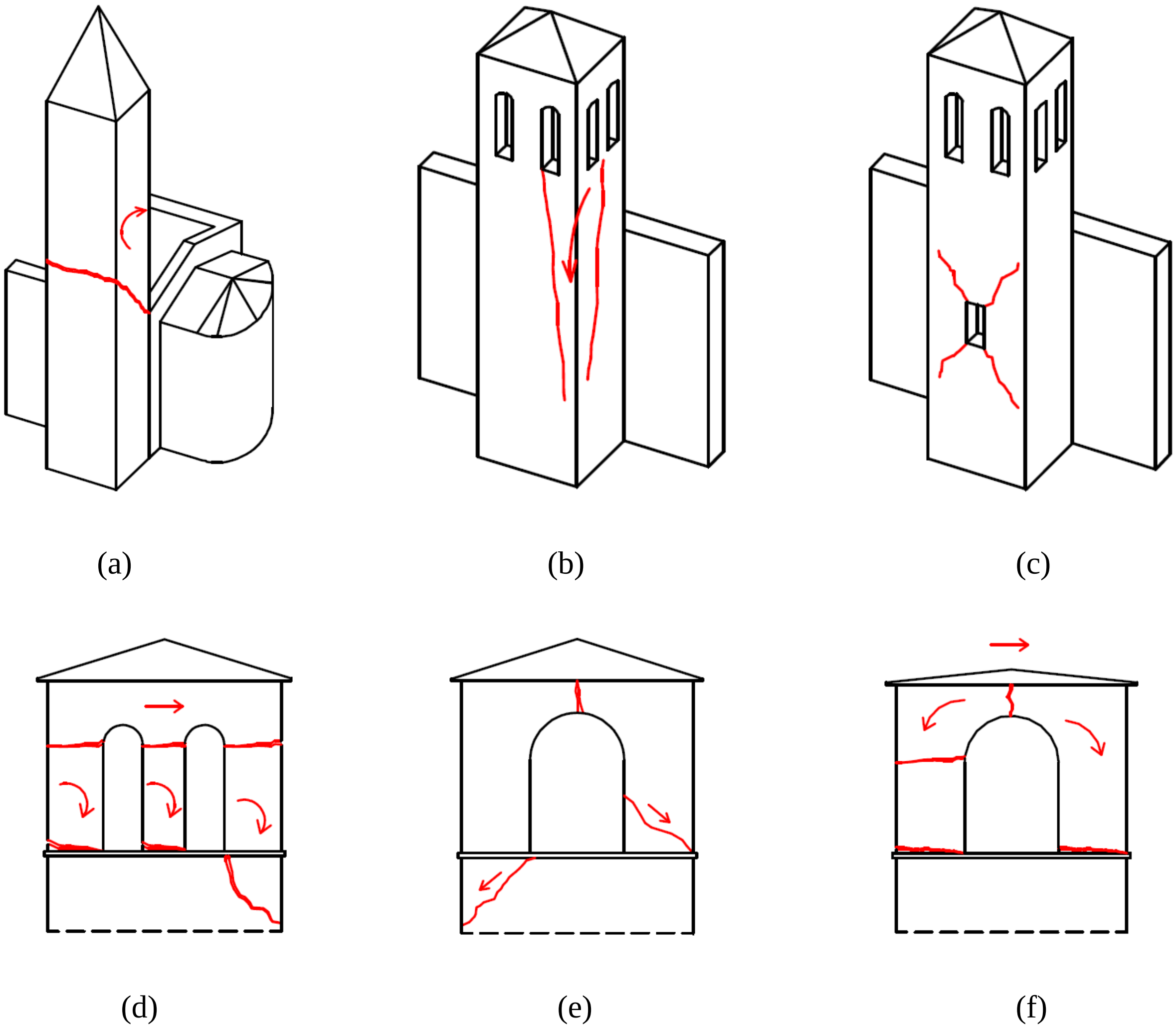
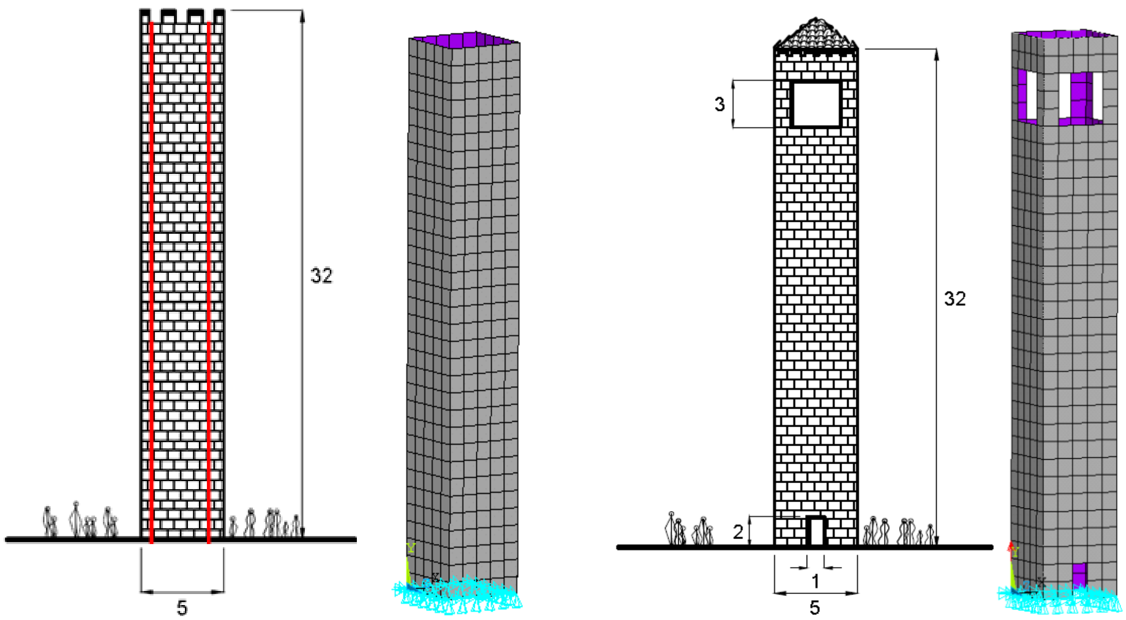


Figure 4: Observed EQ damage mechanisms of masonry towers; (a-c) at the body of the tower and (d-f) at the level of belfry (Lagomarsino et al., 2002)



(a) (b)

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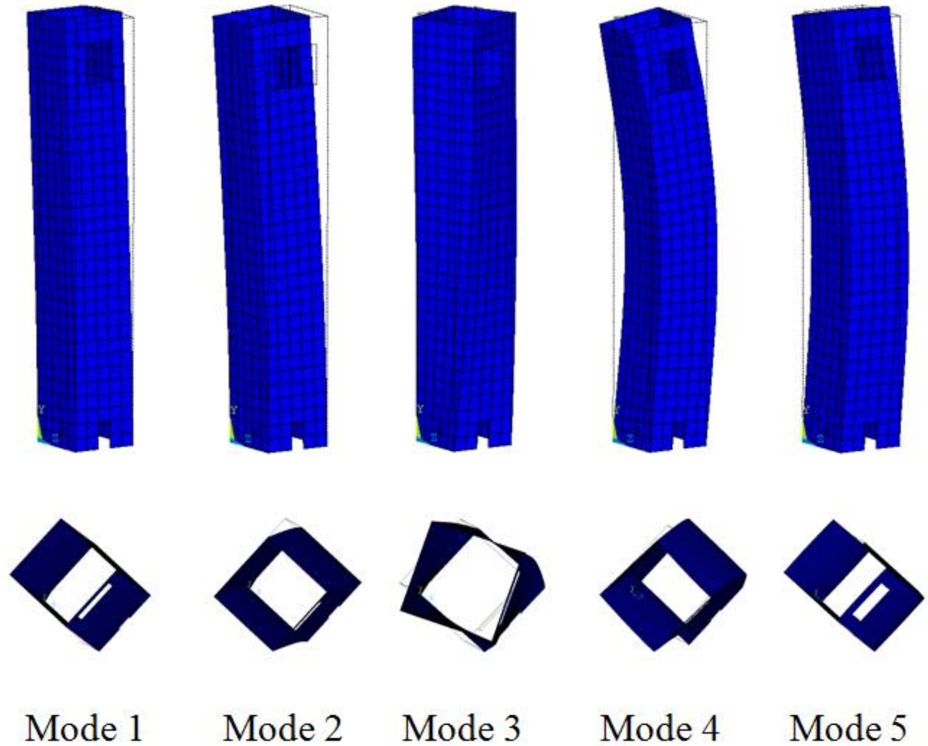

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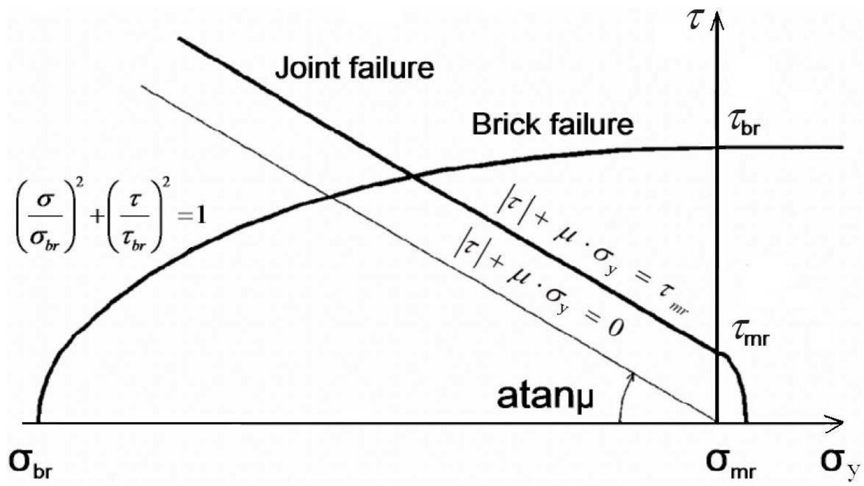

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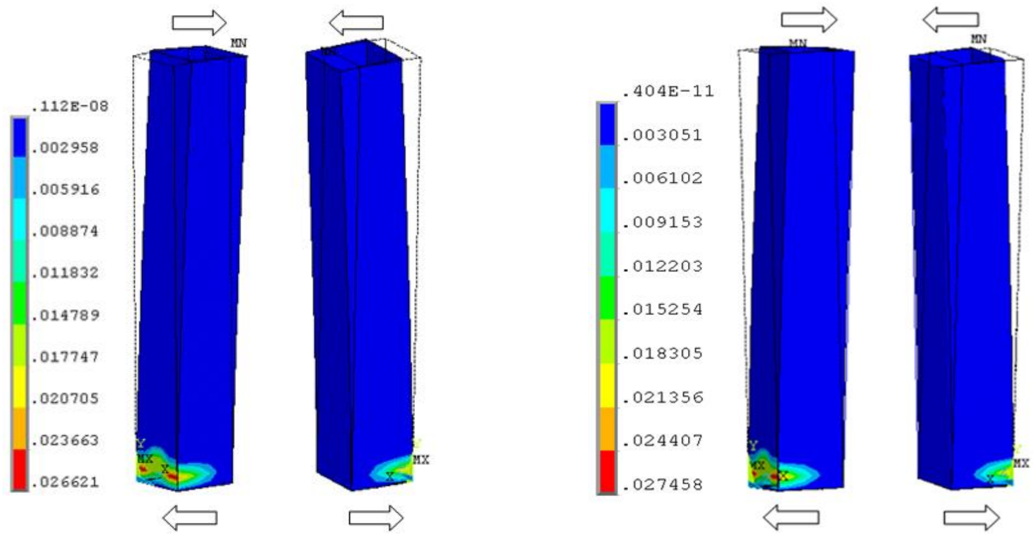
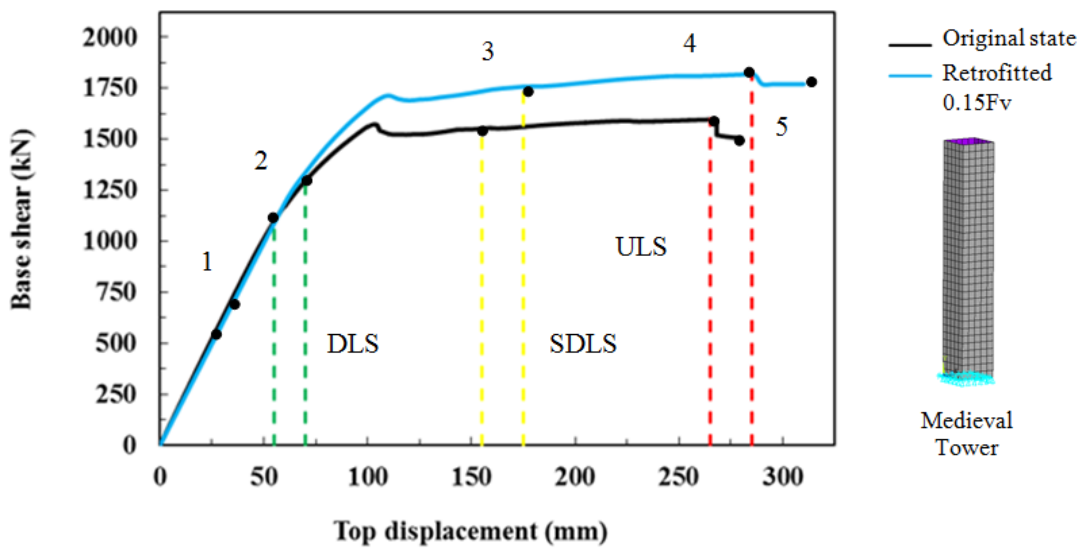


(a) (b)

Figure 8: Medieval tower. Comparison of principal plastic strain contours (front and back) at ULS: (a) original state at 265 mm and (b) retrofitted 0.15Fv Technora at 265 mm



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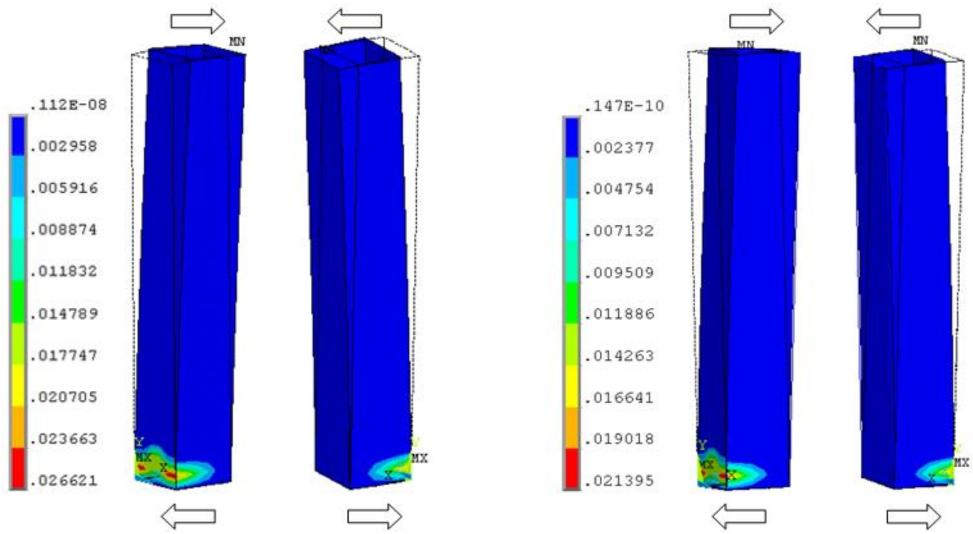

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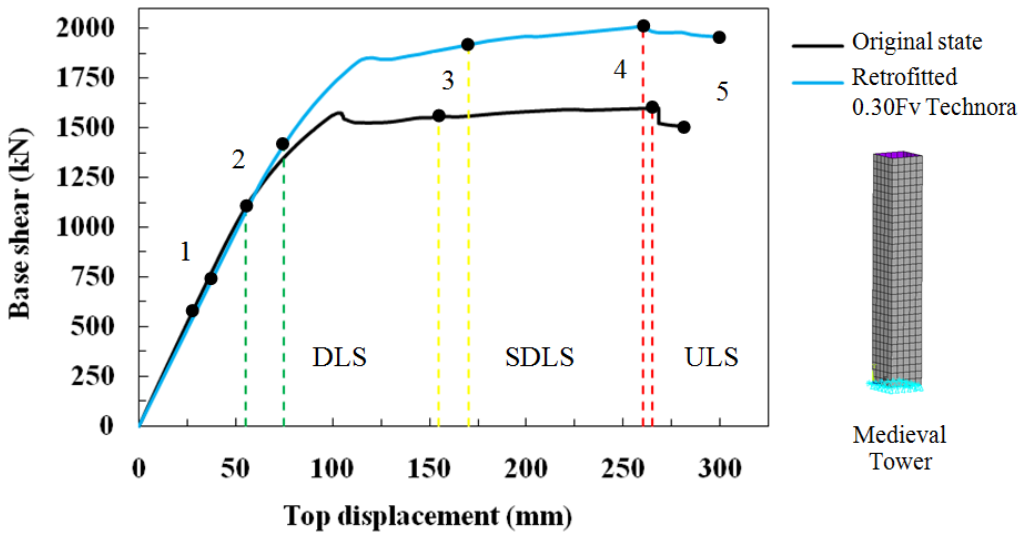

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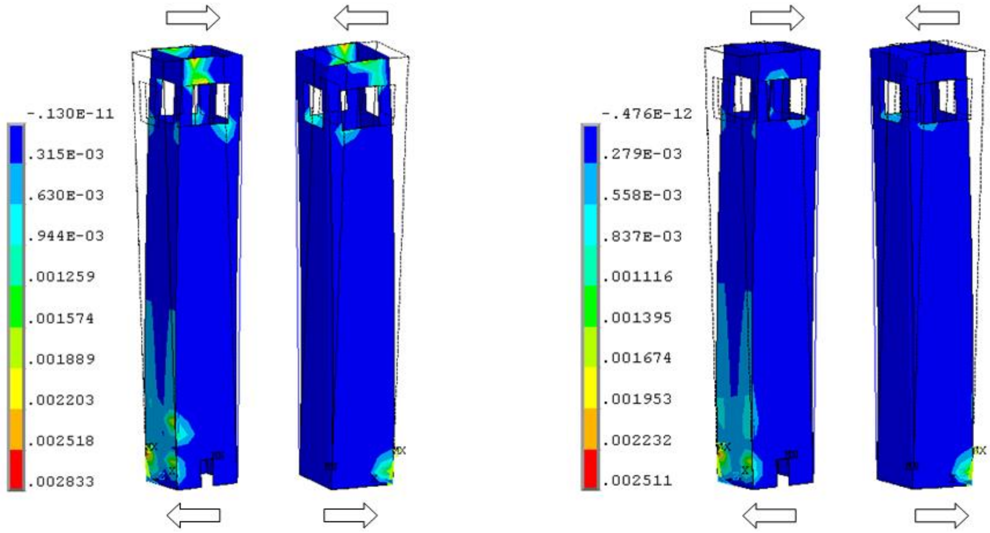
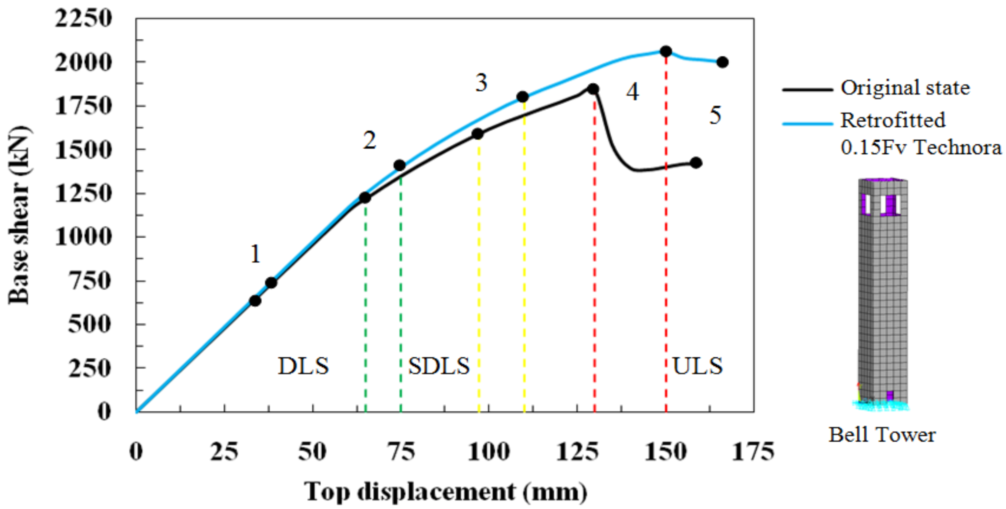


(a) (b)

Figure 12: Bell tower. Comparison of principal plastic strain contours (front and back) at a displacement of 130 mm: (a) original state (ULS) and (b) retrofitted 0.15Fv Technora



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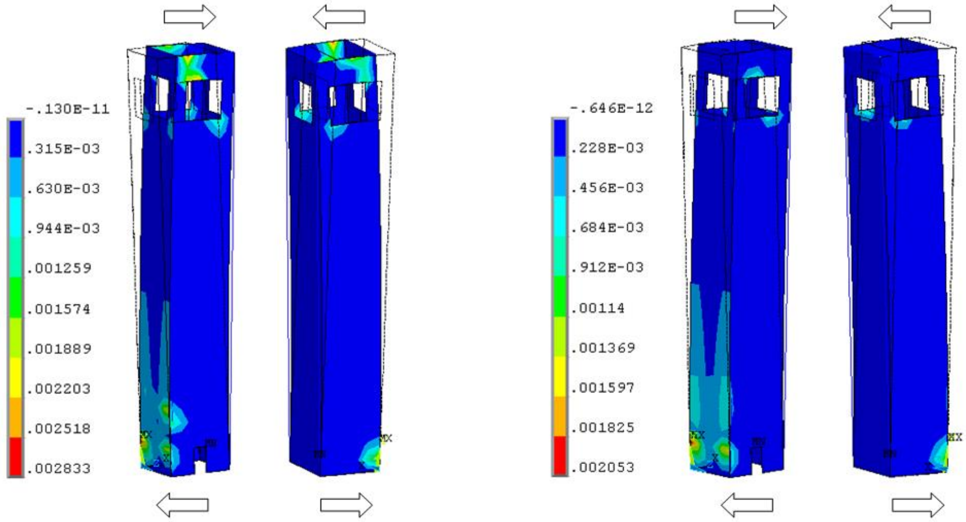

(a) (b)

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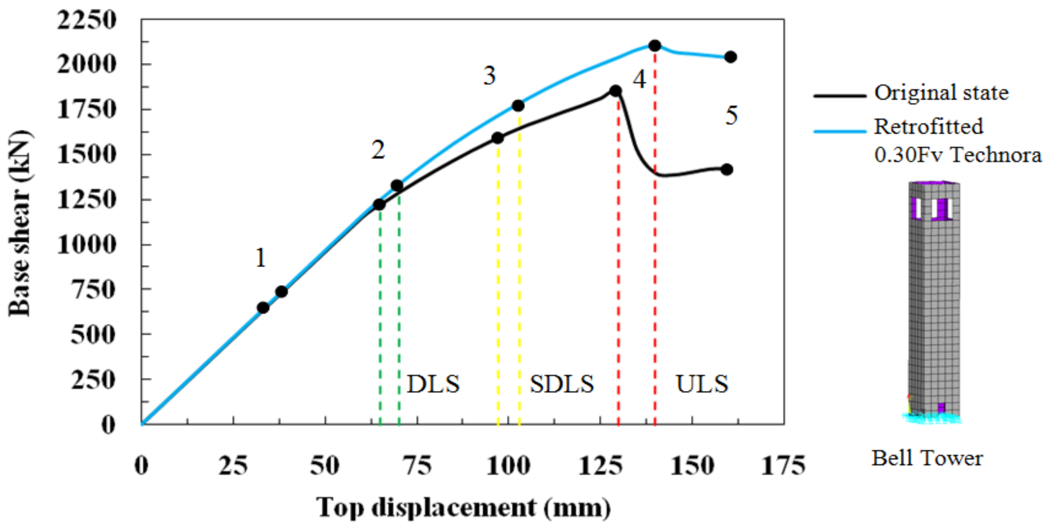

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Table 1: Reference natural frequencies of 10 historical masonry towers

Reference	**Tower height**	**Frequency (Hz)**	
		1st	**2nd**
Ramos et al. (2010)	20.40	2.15	2.58
Bayraktar et al. (2009)	22.00	2.56	2.66
Ivorra et al. (2008)	33.90	2.15	2.24
Slavik (2002)	35.00	1.10	1.30
Ivorra and Pallares (2006)	41.00	1.29	1.49
Abruzzese et al. (2009)	41.00	1.26	1.29
Lund et al. (1995)	43.50	1.38	1.82
Abruzzese et al. (2009)	45.50	1.05	1.37
Russo et al. (2010)	58.00	0.61	0.73
Gentile and Saisi (2007)	74.10	0.59	0.71

Table 2: Natural frequencies of the medieval and bell towers

Mode no.	Vibration mode	Frequency (Hz)	
		MT1	MT2
1^{st}	Bending N-S	1.064	1.076
2^{nd}	Bending E-W	1.064	1.083
3^{rd}	Torsion	4.732	4.723
4^{th}	Bending E-W	5.255	5.162
5^{th}	Bending N-S	5.255	5.272

Table 3: Summary of masonry inelastic parameters for the material model

Parameter	**Value**	**Unit**
σ_m: tensile strength for mortar	0.25	MPa
τ_m: shear strength for mortar	0.35	MPa
c_m: shear inelastic compliance for mortar	1	-
β_m: softening coefficient for mortar	0.7	-
μ : friction coefficient for mortar	0.6	-
σ_M : compressive strength of masonry	2.5	MPa
τ_b : shear strength of units	1.5	MPa
c_M : inelastic compliance of masonry in compression	1	-
β_M : softening coefficient of masonry	0.4	-

Table 4: Seismic analysis summary of the towers in original state and retrofitted

FEM model	Limit states EC-8 and Damage grades EMS-98														
	DLS DG 2				SDLS DG 3				ULS DG 4				Enhancement at ULS		
	F_{OS}	***U_{OS}***	***F_R***	***U_R***	***F_{OS}***	***U_{OS}***	***F_R***	***U_R***	***F_{OS}***	***U_{OS}***	***F_R***	***U_R***	***F %***	***U %***	***S.C. %***
MT1 15% Fv	1100	55	1336	70	1553	155	1758	175	1600	265	1820	285	13.8	7.5	12.9
MT1 30% Fv	1100	55	1430	75	1553	155	1916	170	1600	265	2008	260	25.5	0.0	24.7
MT2 15% Fv	1245	65	1397	75	1600	97	1771	110	1839	130	2056	150	11.8	15.4	12.5
MT2 30% Fv	1245	65	1340	70	1600	97	1755	103	1839	130	2106	140	14.5	7.7	14.4

MT1: medieval tower; MT2: bell tower; Fv: vertical loading; $_{OS}$: original state; $_{R}$: retrofitted; F: force (kN), U: displacement (mm) and S.C.: seismic coefficient