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"Seismic Vulnerability and Failure Modes Simulation of Ancient Masonry Towers by Validated Virtual Finite Element Models"

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ABSTRACT

Seismic protection of ancient masonry towers is a topic of great concern among the scientific community. A methodology for the seismic vulnerability assessment of all types of towers and slender unreinforced masonry structures (e.g. light houses and minarets) is presented. The approach is developed by four validated 3D FEM models representative of European towers. The models are subjected to linear elastic investigations to establish load carrying capacity and dynamic properties for validation against similar towers. Seismic simulations are developed through intensive nonlinear static pushover analyses. From the assessments, the failure modes and overall seismic response of the towers are obtained. Low tensile strength of masonry and large openings at belfries have significant influence on the seismic behavior, resulting in a quasi-brittle failure. All the towers presented an imminent high vulnerability to seismic actions. The few investigations reported in literature on the seismic behavior of towers are focused on in-plane behavior, disregarding out-of-plane behavior and toe crushing, both aspects are investigated in this paper. The more flexible towers are close to present toe crushing in both planes. The failure mechanisms are validated with reported post-earthquake observations on real damaged towers.

Keywords: *Strong earthquakes; historical towers; old masonry; failure mechanisms; damage assessment; seismic vulnerability; validated virtual models; nonlinear Finite Element Method*

1. CULTURAL HERITAGE IN EARTHQUAKE PRONE COUNTRIES

Most of cultural heritage (masonry monuments) of the world is located in earthquake (EQ) prone zones with different levels of seismic hazard and source characteristics (e.g. Mexico, Chili, Italy, Portugal, Turkey, China and New Zealand). These monuments were built following empirical rules to mainly withstand the vertical loading induced by their self weight, disregarding the effect of horizontal inertia forces induced by EQs. This was due to the limitations in materials technology and knowledge about EQs and structural behavior in that time. The construction of historical buildings was carried out by empirical rules transmitted from generation to generation mainly by means of geometrical approaches and by trial and error about structural stability. The lack of knowledge, quasi-brittle and heavy materials such as masonry and other factors make historical buildings extremely vulnerable to suffer partial or total collapse even by EQs of low intensity. This trend has been observed through centuries and nowadays (Fig. 1) almost after every EQ of considerable intensity (e.g. 2003 M7.5 in Colima, Mexico; 2009 M6.3 in L'Aquila, Italy and 2011 M6.3 in Christchurch, New Zealand). There is a high interest among the nations and scientific community in preserving the cultural heritage of humanity.

EQ assessment of cultural heritage located in seismic areas is an issue of very intensive research in recent years. The main difficulties on the seismic analysis of these buildings arise from the complex geometry, high heterogeneity, anisotropy and heavy mass of masonry. Moreover, the poor behavior of masonry due to its low tensile strength if compared to the compressive one, induces cracking since very low lateral loads. All these factors in combination with the EQ loading tends to separate the structure into macro-blocks that behave independently with different failure mechanisms. Degradation of masonry through time (long-term heavy loads) is another important factor affecting the seismic behavior of old buildings, reducing the strength of masonry and the possible structural failure even in static conditions.

2. SEISMIC VULNERABILITY ASSESSMENT OF CULTURAL HERITAGE

The seismic vulnerability assessment of a historical building is a complex task if compared to another existing building as explained in the works of Preciado (2011), Barbieri et al. (2013), Foraboschi (2013), Preciado et al. (2014) and Preciado and Orduña (2014). This section is aimed at describing the most important and current methodologies reported in literature for assessing in a satisfactory way the seismic vulnerability of a historical masonry building. It is explained the need of using a masonry material model able to represent its nonlinear behavior, and the use of liner elastic analyses just as model verification and validation. Moreover, the analytical approaches are compared against Finite Element Method (FEM), highlighting advantages and drawbacks.

2.1 Historical masonry

Masonry is known as the combination of units (natural and carved stones, bricks, adobe and combinations) with a mixture named mortar that aims to bind the construction units together and to fill the gaps between them. Mortars in ancient structures are mainly integrated of clay or lime in combination with water. In some cases other materials or compounds used to be added to the mortar (e.g. ashes, fibers, blood and cactus extract) for increasing its capacity of adherence, resistance, durability and malleability during the construction. This additives aimed at reducing the contraction of adobe units and mortar generated by drying, and to enhance its resistance to climate change effects. Unreinforced masonry (URM) is one of the most durable and ancient materials commonly found worldwide in historical constructions. This is due to the fact that the use of this especial material as structure goes back to the first civilizations that populated the earth. From the ancient time until now, masonry has been widely appreciated around the world by different important factors such as availability, durability, bioclimatic characteristics, and its low cost if compared to other materials (e.g. steel or reinforced concrete). In the construction of historical structures multiple typologies of masonry were used depending on many factors such as availability of materials, structural element (arch, wall, buttress, dome, or vault), construction technique and appearance.

2.2 Seismic vulnerability assessment methods

Inside the framework of the seismic risk management there are two main stages recommended to be follow as a measure to achieve the protection of cultural heritage. These stages correspond to the seismic risk assessment and its reduction. Nowadays there is an enormous variety of methodologies to assess the seismic risk (or seismic vulnerability) of buildings ranging from simple (e.g. empirical or qualitative) to more complex quantitative approaches (e.g. analytical-experimental). The selection of the most suitable method depends on factors such as number of buildings, importance, available data, and aim of the study. The empirical methods satisfactorily allow the evaluation of a single building or a complete city in a fast and qualitative way before or after the occurrence of a seismic event (EQ scenarios). For assessing the vulnerability of an historical building the procedure is different and more in detail than in the qualitative and rough evaluations by empirical methods. It is more complex, requires more computer resources and especial equipment, and represents more time consuming. The literature recommends applying a hybrid approach by combining empirical, analytical and experimental methods to obtain more reliable and quantitative results about the amount of damage caused by the EQ over the structure.

Seismic vulnerability assessment of buildings is an issue of most importance at present time and is a concept widely used in works related to the protection of buildings. Nevertheless, there is not a rigorous and widely accepted definition of it. In general terms, vulnerability measures the amount of damage caused by an EQ of given intensity over a structure. However, “amount of damage” and “seismic intensity” are concepts without a clear and rigorous numerical definition (Orduña et al., 2008). There is no general approach for assessing the seismic vulnerability of a complex historical building. One approximation may consists of obtaining at a first instance all the relevant information such as identification of structural elements, damages, plans, historical analysis and restorations, as well as experimental vibration tests. Furthermore, with the obtained information is possible to construct a 3D geometrical model with computational tools. After building the initial 3D model (e.g. FEM, Limit Analysis, etc.), the mechanical properties of materials constituting the structure and boundary conditions (BCs) are assigned. Together with a suitable constitutive material model able to satisfactory represent the nonlinear behavior of URM, the model is statically or dynamically assessed. These evaluations are linear or nonlinear depending on the aim of the study and the action under analysis (e.g. self weight, seismic loading, wind, etc.) to define the levels of damage at the structure (vulnerability). Once the seismic assessment of the building is developed and identified its behavior, failure modes and key vulnerable parts, the most suitable retrofitting measure is proposed to improve the overall seismic capacity.

2.3 Linear vs nonlinear approaches

In the case of assessing the seismic vulnerability of masonry buildings, linear analyses suffer from the absence of correlation between linear behavior and ultimate limit state. More specifically, the stress results of a linear analysis are not significant, since a masonry structure does not fail due to excessive stresses but due to a mechanism (either rotating or translating) (Blasi and Foraboschi, 1994). Nonlinear static analyses by means of the pushover approach relate the resistance and energy-dissipation capacity to be assigned to the structure to the extent to which its non-linear response is to be exploited. Therefore, non-linear static analyses account for both the actual force-resisting system of the building, in particular the overstrength, and the actual energy-dissipation system of the building, in particular not only the plastic dissipation (Foraboschi and Vanin, 2013). In brief, linear elastic analyses are only used to verify the load carrying capacity of a certain structure in terms of distribution of stresses, as well as to compare the numerical frequencies with the experimental ones for model calibration/updating.

2.4 Analytical approaches vs Finite Element Method

In the framework of the FEM analysis, three main modeling strategies for masonry are identified to be the most used in the relevant literature. The micro-modeling of single elements (unit, mortar and interface) and meso-modeling (unit and interface), are suitable for the analysis of small structures, e.g. Lofti and Shing (1994) and Lourenço and Rots (1997). The large amount of time for the generation of the detailed structural model and high calculation effort prevent their use in the seismic analysis of sophisticated and large-scale structures as in the case of historical constructions. On the other hand, the macro-modeling (smeared, continuum or homogenized), considers masonry as an anisotropic composite material, e.g. Gambarotta and Lagomarsino (1997), Lourenço et al. (1998) and Schlegel (2004). This simplifies the generation of the structural model, and due to the significantly reduction of the degrees of freedom, less calculation effort is needed, being considered as suitable for the seismic analysis of large historical constructions. Macro-modeling of masonry through analytical models is also gaining the attention of the scientific community for static nonlinear analysis purposes. Among them are the 3D limit analysis approach by rigid macro-blocks (Orduña and Lourenço, 2005a and b) (Orduña et al. 2008) and the strut-and-tie model (Foraboschi and Vanin, 2013). The first approach is based on a rigid-perfectly plastic material that does not need parameters of stiffness and softening, only strength parameters. On the other hand, it is not possible to evaluate the displacements and deformations of the structure, which are fundamental for seismic energy dissipation assessments.

The strut-and-tie modeling approach was developed for reinforced concrete members and can include externally reinforced concrete members (Biolzi et al., 2013). The strut-and-tie modeling approach is supported by the lower bound theorem of the limit analysis, as well as by the maximum stiffness or minimum deformation energy criteria (Blasi and Foraboschi, 1994). Actually, the original form of the lower bound theorem refers to an elasto–plastic constitutive law of the material, which does not include masonry. However, the lower bound theorem can be extended to masonry structures, under the assumption that masonry has an elasto-plastic compression behavior (or perfectly elastic) and a no-tension behavior, which is an assumption that suits masonry adequately (Foraboschi and Vanin, 2013). However, FEM modeling is still the most powerful tool and recommended to assess the vulnerability of large historical constructions against EQs. This is due to its capability to calibrate the model with real experimental data and possibility to simulate a nonlinear dynamic analysis, taking into account the EQ characteristics, damping and dissipation of energy.

3. SEISMIC VULNERABILITY OF OLD MASONRY TOWERS

Existing ancient masonry towers (AMT) with different characteristics and functions are distributed all over the world and constitute a relevant part of the architectural and cultural heritage of humanity. These vertical structures 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 towers (see Fig. 2), also named civic towers, were built quite tall for informing people visually and with sounds about time and extraordinary events such as civil defence or fire alarm, and to call the community to 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. Strong damage or complete loss suffered by the cultural patrimony due to EQs has been occurring through the history of humanity.

3.1 Fundamental aspects determining the seismic vulnerability of towers

The occurrence of unexpected and unavoidable events such as EQs has demonstrated that AMT are one of the most vulnerable structural types to suffer strong damage or collapse as depicted in Figure 1. Their protection is a topic of great concern among the scientific community. This concern mainly arises from the observed damages after every considerable EQ and the need and interest to preserve this cultural heritage. Although the 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 (bell, clock and medieval towers) are highly vulnerable to suffer strong damage or collapse in EQ conditions, even when subjected to seismic events of low to moderate intensity.

These vertical structures are slender by nature, the slenderness (H/L) of towers is the single most decisive factor affecting their seismic performance, characterized by a ductile behavior where bending and low tensile strength of masonry determinate the overall performance. The position of a tower in the urban context is an important aspect that influences the vulnerability of the structure (Sepe et al., 2008). These boundary conditions could strongly modify its seismic behavior and have big impact in the generation of different failure modes. Non-isolated towers were commonly built as part of churches or next to another building. Adjacent walls or façades with different height than the tower and the lack of connection between elements by the poor tensile strength of masonry could generate during an EQ a detachment of the different bodies. In addition, the seismic vulnerability of towers is increased by certain important aspects such as soil conditions, large openings at belfries, nonlinear behavior of masonry, lack of good connection between structural elements, high vertical loading and progressive damage. These fundamental aspects determine the seismic vulnerability of towers in terms of behavior and failure mechanisms that differentiate them from most of compact historical constructions.

AMT were built as most of the historical buildings to mainly withstand the vertical loading generated by their self-weight. The thickness of walls used to be determined by following empirical rules transmitted from generation to generation by trial and error mainly based on the height and observed EQ damage. These empirical rules led to the construction of walls with enormous thicknesses higher than 2 m. The roof system of towers was usually made of the same material of the walls, even when reduced thicknesses were considered, the elevated mass of masonry generated problems of instability that could lead to collapse even during the construction works. For avoiding heavy roofs, it is quite frequent to especially find in Italy masonry towers with a plane roof system integrated by wooden beams and fired-clay bricks. AMT are slender structures under high vertical loading due to the height, wall thickness, presence of a 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 AMT 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. The most famous cases are reported in Binda et al. (1992), Macchi (1993), GES (1993) and Binda (2008). They relate to the collapses of the bell tower of “Piazza San Marco”, Venice in, the civic tower of Pavia in 1989 and the bell tower of the church of “St. Maria Magdalena” in Goch, Germany in 1992.

3.2 Post-earthquake observations and typical failure modes of towers

The identification of seismic behavior and failure mechanisms of AMT subjected to in-plane and out-of-plane loading is a complicated task. This identification strongly depends on many factors such as soil and boundary conditions, geometrical characteristics and mechanical properties of masonry (mortar and units), level of vertical loading and the EQ characteristics. All these factors play an important role in the determination of the seismic behavior and failure mechanisms of AMT. Compared to other compact structures, masonry towers mainly fail ductile in a predominant bending behavior due to the excessive slenderness (height / length > 4). Due to this, and 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 and inertia forces. This behavior could cause different failure mechanisms as illustrated in Figure 1. Meli (1998) describes that during an EQ, masonry towers present important horizontal top displacements. Bending generates horizontal cracks but rarely the overturning of the structure. This is due to the alternation of the movement that causes an opening and closing effect of these cracks, dissipating with the impact an important part of the EQ energy.

On the other hand, in bell towers, the presence of large openings at belfry could increase the vulnerability of the structure, being more frequent the failure by shear. Due to the strong damage, 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, where one belfry collapsed by overturning on the basketball court of a neighbor building (see Fig. 1b). The remaining damaged belfry was removed during the rehabilitation and retrofitting works, and in the end it was decided to leave the church without belfries for security reasons (Preciado, 2011).

Alcocer et al. (1999) describe that the key behavior of bell towers during EQs is dominated by in-plane failure in the direction of the façade. The out-of-plane failure of towers is generally less important and is only regarded with the detachment of the façade from the nave. 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 presence of large openings, natural bending behavior and low tensile strength of masonry. This amplifies the seismic motion causing critical effects at the top part of the tower. Peña and Meza (2010) developed post-earthquake observations in 172 Colonial 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 of these structures, with no masonry crushing at the base of the tower. Based on observed damage on AMT after considerable EQs occurred in Italy, Lagomarsino et al. (2002) propose the damage mechanisms of Figure 3. The body damage of Figure 3a corresponds to horizontal cracking out-of-plane due to bending behavior and diagonal cracking by shear stresses in-plane, leading to overturning over the nave. The type of damage of Figure 3b consists of vertical cracking in both planes due to horizontal tension, resulting in the detachment of walls and collapse by instability. On the other hand, the damage mode of Figure 3c is represented by alternated diagonal cracking in-plane due to shear which could be repaired. The damages at belfries are mainly characterized by horizontal and diagonal cracking due to the presence of large openings, leading to the collapse by overturning (Figs. 3d-f). In brief, the author of this paper may conclude that the main failure mechanisms presented in 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, (3) vertical cracking at the tower´s body due to horizontal tensile stresses induced by the detachment from other vertical elements (e.g. the façade or nave of a church) (4) partial or total collapse of belfries due to shear stresses and bending behavior, and (5) masonry crushing at the compressed toes.

4. SEISMIC FAILURE AND BEHAVIOR SIMULATION OF OLD MASONRY TOWERS

The main objective of this paper is to develop a methodology for the seismic vulnerability assessment of all types of towers and slender URM structures (e.g. light houses and minarets), through the correct simulation of failure modes and behavior. The simulation of seismic response and failure modes is developed through validated FEM models of four virtual historical masonry towers commonly found in Europe with variations in geometry, roof system and boundary conditions (see Fig. 4). As a first approximation, the generated 3D FEM models of the towers are evaluated by linear elastic procedures to obtain in relatively simple way information about the load carrying capacity and dynamic characteristics (natural frequencies and vibration modes). In order to obtain representative models of real AMT, the numerical results are validated with theoretical back ground and experimental results on similar towers reported in literature. Before starting with the static and dynamic nonlinear analyses of the towers, the capability of the applied masonry model to simulate the nonlinear behavior of masonry is validated with selected experimental examples reported in literature. Since the towers are theoretical, the seismic hazard is determined at a first instance in qualitative terms by the damage grades of the European Macroseismic Scale (EMS-98) proposed by Grünthal (1998) and the limit states of the performance-based design (PBD) philosophy for different EQ intensities. The seismic action is evaluated in quantitative terms by the seismic coefficient obtained in the analyses. As a final approximation, intensive numerical simulations through a series of nonlinear static analyses are carried out for the EQ evaluation of the AMT. The results are validated with reported key-behavioral characteristics and observed EQ damage.

4.1 Characteristics of the virtual AMT and FEM models

The general view and dimensions of the virtual AMT under study are illustrated in Figure 4. The towers were selected taking into account common AMT (see Fig. 2) with variations in roof system, height, boundary conditions and openings at belfry. The main objective is to obtain different failure mechanisms and behavior, in order to compare them with the observed damage after moderate to strong EQs. The first two towers (AMT 1-2) of Figure 5, correspond to bell towers with large openings at the four sides of belfry and tall and heavy masonry roof. The tower AMT 1 (Fig. 4a) is isolated and the AMT 2 (Fig. 4b) has neighbor buildings (non-isolated). The last two towers (AMT 3-4) of Figure 4 are isolated and have light timber roof. The AMT 3 model is representative of bell towers with only one opening at belfry (Fig. 4c) and AMT 4 of medieval towers (Fig. 4d) with no belfry (see Table 1). Table 1 presents the 3D FEM models of the proposed virtual AMT, which are developed by means of the commercial software ANSYS®. The first two models (AMT 1-2) have the same geometry and roof system but different BCs. The interaction between neighbor buildings at the AMT 2 model is taken into account at the East façade (at 10 m height) and at the North one (at 15 m height). The simulated interaction with neighbor buildings is illustrated in Figure 4b and Table 1b. The third and fourth models (AMT 3-4) have a light timber roof common of this type of structures that could be neglected in the analyses (see Figs. 4c-d and Table 1c-d).

The selected element for walls and roofs is Shell43, which has four nodes and four thicknesses with six degrees of freedom (DOF) at each node. This element can represent in-plane and out-of-plane behavior and has plasticity and creep capabilities. In the generation of the four 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 were assumed as fixed. (2)

The main mechanical properties of the AMT were proposed by taking into account average values reported in literature. The selected masonry was considered as carved stone with lime mortar, with an average density of 2000 kg/m^3 and a Young´s modulus of 2000 MPa. The Poisson´s ratio was held constant and equal to 0.15. The compressive strength was assumed to be 3.5 MPa and the tensile strength 0.25 MPa. (3) At the non-isolated model AMT 2 (Table 1b), the interaction with neighbor buildings in the North and East façades was simulated by a uniform distribution of linear elastic springs of constant stiffness (275 Combin14 elements). To simulate the interaction induced by neighbor masonry buildings it is proposed Ec. 1, based on the works of Pandey and Meguro (2004), Crisafulli and Carr (2007) and Mondal and Jain (2008), where the authors assess the lateral stiffness contribution on masonry infill panels. The axial spring stiffness K_{sp} is assumed to be equal to a fraction γ of the total stiffness of a masonry block.

$$K_{sp} = \gamma \; \frac{E_m \, A_m}{T_m} \tag{1}$$

Where E_m is the elastic modulus of masonry, A_m is the area of a composite masonry block of 1x1 m (4 springs) and T_m is the wall thickness. The factor γ is recommended in literature to be estimated between 0.50 and 0.75 depending on the researcher when calibrating the model. During the calibration process it is decided to use a factor of 0.30, resulting in a spring stiffness of 100 kN/mm. This value is in good agreement with the proposed by Ivorra and Pallares (2006), where the authors experimentally evaluated the lateral stiffness contribution of masonry façades in old bell-towers.

4.2 Validation of the virtual AMT by linear static and dynamic analyses

Static and dynamic linear evaluations such as vertical loading and modal were firstly developed to obtain an important progress on the seismic vulnerability assessment without the convergence problems related to nonlinear analyses. These linear elastic approximations permit to determine the presence and magnitude of tensile and compressive stresses at the masonry structure by vertical loading, as well as the frequencies and vibration modes in the modal analysis. In the generation of structural models of complex historical constructions there are many assumptions and uncertainties regarding the determination of geometry, material properties, and boundary conditions. In this case, the linear analyses could be used to calibrate (or up-date) the initial model with the experimental data by adjusting geometry, material properties and interaction with adjacent buildings. This permits to obtain models more representative of the structure under study, and with this, a reliable seismic vulnerability assessment. In case of masonry towers the vertical load represents an important factor in the seismic behavior, because these structures were constructed by empirical rules only to withstand their self-weight. In historical towers, usually the zone most over stressed is the bottom part. High compressive stresses could generate local failure of masonry and may be the trigger of sudden collapse as explained in Section 3.1. The models with triangular roof (AMT 1-2) present the same vertical distribution of stresses because they have the same mass (interaction with neighbor buildings has no influence), therefore only one tower is presented (Fig. 5a). In case of the other two towers (AMT 3-4) with timber roof (Figs. 5b and c), there is a small variation of mass by the presence of openings. However, the maximum values are present at the doors due to the reduction of the resistant area. The two towers with triangular roof present tensile stresses at the base of the cover (Fig. 5a). This trend is in agreement with real behavior observed in similar masonry towers. The roof bends due to the heavy weight and height, generating vertical cracking similar to domes. Therefore is more common to observe in this type of towers tall triangular roofs made of timber. The vertical analyses have revealed that the towers (AMT 1-4) are in linear conditions, because the levels of compressive stresses are lower than the intrinsic strength and tensile stresses are not present in large zones. These results allowed the validation of the FEM models regarding static conditions, concluding that the towers are stable to satisfactorily resist their own self weight as most of historical constructions.

The linear investigations were extended to dynamic analyses in order to obtain a first estimation of the dynamic response of the four virtual towers. As in the case of the vertical loading analyses, the modal evaluations of FEM models are relatively fast due to the progress of recent decades on computational tools. As a first stage, the dynamic parameters of the isolated and non-isolated towers with masonry roof are numerically obtained. The resulting vibration modes of both towers are similar, therefore only the modes of the isolated tower (AMT 1) are depicted in Figure 6a. The natural frequencies of the non-isolated model (AMT 2) are higher (lower periods) as expected, due to the increment of stiffness (about 24 % in the N-S direction and 8 % in the E-W) generated by the assumed contact with neighbor buildings (Table 2). Analyzing the results of Figure 6a and Table 2, it could be observed that the two fundamental vibration modes of both towers correspond to a general bending. This low frequencies (high periods of about 1 s) and vibration modes, are representative of real behavior of slender and tall structures as AMT, which are highly vulnerable to EQ motions. The higher modes represent torsion and a particular problem of vertical vibration due to the tall and heavy roof. Afterwards, the natural frequencies and vibration modes of the isolated towers with timber roof (AMT 3-4) are numerically obtained as presented in Figure 6b and Table 2. In this case the vibration modes and frequencies are similar as in the case of the towers with heavy roof.

To validate the numerical natural frequencies of the virtual towers obtained in the modal analyses, an extensive literature review was developed. Bachmann et al. (1997) and Casolo (1998) 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). The Spanish Standard NCSE (2002) considers a masonry structure as slender when its first natural period is comprised between 0.75 s < T < 1.25 s (0.8 Hz < f < 1.33 Hz). The same Standard proposes an analytical formula to approximately assess the first frequency ω of masonry bell towers (see Eq. 2). Where L corresponds to the plan dimension in the vibration direction and H is the height of the tower. The suitability and efficiency of this equation as a first and quick estimation (or validation of numerical and experimental results) of the first natural frequency of real masonry bell towers have been proved by many researchers, e.g. Ivorra and Pallares (2006), Ivorra et al. (2008), Bayraktar et al. (2009), Preciado (2011).

$$\omega_1 = \frac{\sqrt{L}}{0.06\, H \sqrt{\frac{H}{2\, L + H}}} \quad \text{(Hz)} \qquad (2)$$

As a result of applying Ec. 2 on the four FEM models of the virtual towers in the vibration direction E-W, the isolated tower with masonry roof (AMT 1) is supposed to have a first natural frequency of 1.119 Hz. The result is in good agreement with the obtained in the numerical simulation for the same direction (1.051 Hz). For the case of the non-isolated tower with masonry roof (AMT 2) is expected a greater first natural frequency as a consequence of the contact with neighbor buildings. The increment in stiffness induced by neighbor buildings is obtained in the numerical simulations (see Table 2). For the case of the two towers with timber roof (AMT 3-4), the equation does not consider the influence of openings in the total mass. Therefore both first natural frequencies in the E-W direction are the same and correspond to 1.334 Hz (modal analysis: 1.064 Hz and 1.083 Hz respectively). As a final validation, the obtained natural frequencies by modal analyses and Eq. 2 are compared to experimental results in similar masonry towers reported in literature (see Table 3). It is worth noting that the frequency reduces with the increment in height, being the structure more slender, and as a consequence more flexible. The masonry tower of 35 m assessed by Slavik (2002) has a first natural frequency of 1.10 Hz, which is in very good agreement to the presented by the AMT 3-4 models of 32 m (1.076 and 1.064 Hz respectively). The same trend is observed between the first frequencies of the 45.5 m isolated tower (1.05 Hz) evaluated by Abruzzese et al. (2009) and AMT 1-2 of 45 m (1.046 Hz).

4.3 Seismic failure mechanisms of the AMT by nonlinear static analyses

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 mechanisms 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 proved through numerical simulations by Calderini and Lagomarsino (2006), Urban (2007), Sperbeck (2009) and Preciado (2011) against experimental results reported in literature, e.g. Van der Pluijm and Vermeltfoort (1991), Raijmakers and Vermeltfoort (1992) and Vermeltfoort and Raijmakers (1993). The 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. 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 4. In order to assess the seismic response of an historical building is recommended to obtain the material parameters through detailed experimental campaigns. This is always a complex and expensive task, mainly due to the heterogeneity of masonry, lack of representative samples and the need of non-destructive tests. In case that it is not possible to obtain all the material parameters, the ones proposed and calibrated through numerical simulations by Preciado (2011) are recommended.

The towers of Figure 2 are subjected to the pushover method with the integrated material model. The FEM models 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 analysis it is possible to obtain the complete capacity curve and failure mechanisms during the analysis, 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 in order to attain convergence. The time of computational calculation for every analysis is in the order of 8 hours by means of a standard desktop work station. 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 significant damage and the ultimate limit state (ULS) near collapse. Moreover, these limit states at the capacity curves are correlated to the damage grades (DG) DG 2, DG 3 and DG 4 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) determined by the ratio between the ultimate lateral force and the 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 which is extremely important in the EQ assessment of structures for energy dissipation capabilities.

In both towers with masonry roof AMT 1 and AMT 2 (Figs. 7 and 8), the analyses illustrate a failure mode governed by diagonal cracking due to in-plane shear stresses at the large openings (front and back) at belfries. This is due to the reduction of the resistant area at this weakened part. The final failure mode in both towers is suddenly formed by the extension of the in-plane diagonal cracks at openings of belfries (Figs. 7b and 8b). These large cracks lead to the collapse of belfries, placing in a situation of danger the adjacent buildings and people inside or in the surroundings. Masonry crushing at the in-plane and out-of-plane compressed toes is not observed, due to the fact that these towers present quasi-brittle behavior by belfry failure. The maximum compressive stresses of about 1.4 MPa, being lower than the intrinsic strength (3.5 MPa). Figure 9 illustrates the capacity curves of the two towers with triangular roof (AMT 1 and AMT 2), including the damage grades of the EMS-98 and the limit states of EC-8. It is worth noting that the linear behavior of both towers is different. The non-isolated tower (AMT 2) is stiffer than the isolated one (AMT 1) due to the interaction with adjacent buildings as it was observed in the modal analysis, reaching the yielding (DG 2) at a displacement of 40 mm and a lateral force of 2220 kN. By the other hand, the isolated tower approximately presents 22 % more lateral force and about 33 % more displacement capacity (F= 2700 kN and U= 53 mm) at the same yielding stage. In the nonlinear range, both towers present similar lateral load capacity but different displacement. This behavior continues until both towers reach ultimate conditions, showing the isolated one about 10 % more displacement (F= 4350 kN and U= 115 mm).

The failure mechanisms of the virtual masonry towers with timber roof AMT 3 and AMT 4 are illustrated in Figures 10 and 11. The medieval tower AMT 4 presents a global bending behavior represented by the initial formation of horizontal cracks (see Fig. 11a) due to vertical tensile stresses at the base level at a displacement of 155 mm, which corresponds to a DG 3 and a limit state of significant damage (SDLS). The tower reaches an ULS and a damage grade 4 at a displacement of 265 mm. The final collapse mechanism (Fig. 11b) 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 isolated bell tower with timber roof and openings (AMT 3) presents a different behavior as illustrated in Figure 10. The tower shows at a displacement of 185 mm the initial formation of horizontal cracks due to vertical tensile stresses as in the case of the medieval tower (AMT 4) but at a different height in both planes of the posterior part (Fig. 10a). The presence of diagonal cracks is evident by shear stresses in the plane of the main door. The tower reaches an ULS at a displacement of 325 mm, represented by a final failure mode due to the extension of horizontal and diagonal cracks (Fig. 10b). This tower is close of failing by masonry crushing at the compressed toes in both planes, with a maximum compressive stress of 3.305 MPa. The obtained seismic failure mechanisms through validated virtual models of AMT are characteristic of this type of structures and are in complete agreement with the described in post-earthquake observations (Section 3.2).

4.4 Capacity curves and behavior of the AMT by nonlinear static analyses

The capacity curves of the bell and medieval towers (AMT 3-4) with timber roof including the damage grades (EMS-98) and limit states (EC-8) are illustrated in Figure 12. It could be observed that both towers present similar linear behavior, reaching the yielding (DG 2, DLS) at the same load of 1100 kN and a displacement of 55 mm. The towers present different nonlinear behavior at a DG 3 and SDLS, being more evident the difference in the ultimate limit state (DG 4, ULS). The tower with openings shows about 9% more lateral force and 23% more displacement (F= 1750 kN and U= 325 mm) than the tower with no openings. This trend is similar to the numerically results on ancient masonry structures with different configuration reported in Preciado (2011) and the experimental tests of Raijmakers and Vermeltfoort (1992) and Vermeltfoort and Raijmakers (1993). Comparing the capacity curves of the four FEM models of the virtual towers illustrated in Figures 9 and 12, it is worth noting that the towers with masonry roof are more resistant to lateral loading, but in contrast present less ductile behavior. Table 5 summarizes the results of the seismic evaluation of the four virtual historical masonry towers by the pushover method. The SCs are calculated at ultimate lateral conditions and are presented in Table 6.

In the seismic analysis summary of Table 5, it could be observed that in the DLS and DG 2, the four towers present similar displacement, being stiffer the tower with the assumed adjacent buildings. The difference is evident in the lateral carrying capacity, withstanding the stiffer tower (AMT 2) about 100 % more lateral load, and the isolated with masonry roof (AMT 1) about 145 %. In the SDLS and DG 3 the towers with masonry roof (AMT 1-2) present more lateral strength capacity but in contrast less ductility. The towers with timber roof (AMT 3-4) show different seismic behavior between them at ultimate conditions (ULS and DG 4), presenting the tower with openings (AMT 3) about 9 % more force and 23 % more displacement.

In the summary of SCs of Table 6, it could be observed that the two towers with timber roof (AMT 3-4) have similar vertical loading, with a small variation in the tower with openings AMT 3 (less mass). This tower shows more lateral force capacity of about 150 kN due to the different seismic behavior induced by the main door opening. The towers with masonry roof (AMT 1-2) present the same vertical loading because they have the same mass. Regarding lateral force, both towers show similar capacity, with 50 kN more the isolated one (AMT 1). Compared to the towers with masonry roof, the ones with timber show about 2.5 times less force and vertical loading. Because of this relationship, the four towers have similar SC. The obtained low values of SC represent in quantitative terms, the high vulnerability of this type of structures to seismic actions. These SCs are in complete agreement with the typical values of ancient masonry buildings, in the range between 0.1 and 0.3 as mentioned by Meli (1998). In contrast, for seismically designed masonry buildings, the SC is in the range between 0.5 and 0.86. In conclusion, the four virtual historical masonry towers would reach an ULS or collapse under an EQ ground motion of 0.1 g. The SC allows obtaining 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.

5. CONCLUSIONS

A proposed methodology for the validation of virtual AMT and seismic vulnerability assessment through failure mechanisms and behavior was described. The research was developed through four validated 3D FEM models representative of towers usually found in Europe. As a first approximation on the seismic assessment, the FE models were subjected to linear elastic investigations on their load carrying capacity and dynamic characteristics. These initial analyses permitted to validate the models with theoretical background and experimental data on similar towers reported in literature. This validation plays an important role to obtain models representative of real towers, and with this, more reliable results in the seismic vulnerability evaluation. This validation could be useful when there is no experimental data available to calibrate the model, and when available, as a practical pre-calibration. The described strategy to simulate the interaction with neighbor buildings is envisaged to simplify the model construction and the nonlinear analyses, because normally the modeling of non-isolated towers is done by including the complete façade or nave of the church. Intensive numerical simulations by nonlinear static analyses were carried out. The seismic analyses by the pushover approach successfully permitted to obtain the overall seismic response of the towers, represented by the capacity curves and the in-plane and out-of-plane failure modes. The huge impact of the low tensile strength of masonry and large openings at belfries on the seismic behavior was observed, failing the AMT 1 and AMT 2 models in a quasi-brittle mode by shear stresses. The medieval tower AMT 4 presented the characteristic bending behavior with horizontal cracks in-plane and out-of-plane. The similar tower with openings AMT 3 presented a mixed failure mode of bending and shear stresses at the bottom (attracted by the main door opening), being more resistant and ductile. The same trend was observed in the validation of the material model stage and was corroborated with reported experimental observations.

The few investigations reported in literature on the seismic behavior of AMT are mainly focused on the in-plane behavior and disregard horizontal cracking out-of-plane and masonry crushing at the tower's bottom. The more flexible towers (AMT 3-4) were close to present crushing in both planes. The behavior and damage types were validated with the seismic vulnerability aspects described in Section 3.1 and the reported post-earthquake observations on masonry towers of Section 3.2. The capability of the applied model to simulate the nonlinear behavior of masonry and collapse mechanisms at masonry towers in post-earthquake observations showed a very good agreement. The seismic hazard was included in qualitative terms at the capacity curves for different DGs and limit states, and quantitatively by the SC. A drawback of the SC is that ductility is not considered, which is quite important to evaluate energy dissipation. The three approaches permitted to satisfactorily assess the seismic vulnerability of the four AMT. All of them presented an imminent high vulnerability to seismic actions.

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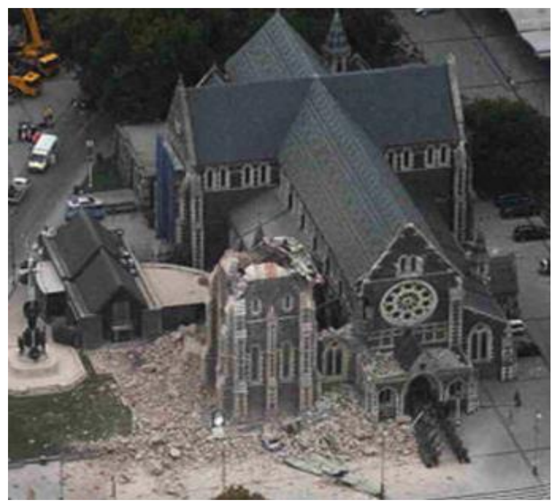
LIST OF FIGURES

(a)



(b) (c)

Figure 1: Observed EQ damage on cultural heritage; (a) L'Aquila, Italy in 2009 M6.3; (b) Colima, Mexico in 2003 M7.5 and (c) Christchurch, New Zealand in 2011 M6.3

(a) (b)

Figure 2: Ancient masonry civic towers; (a) bell-towers and (b) clock tower

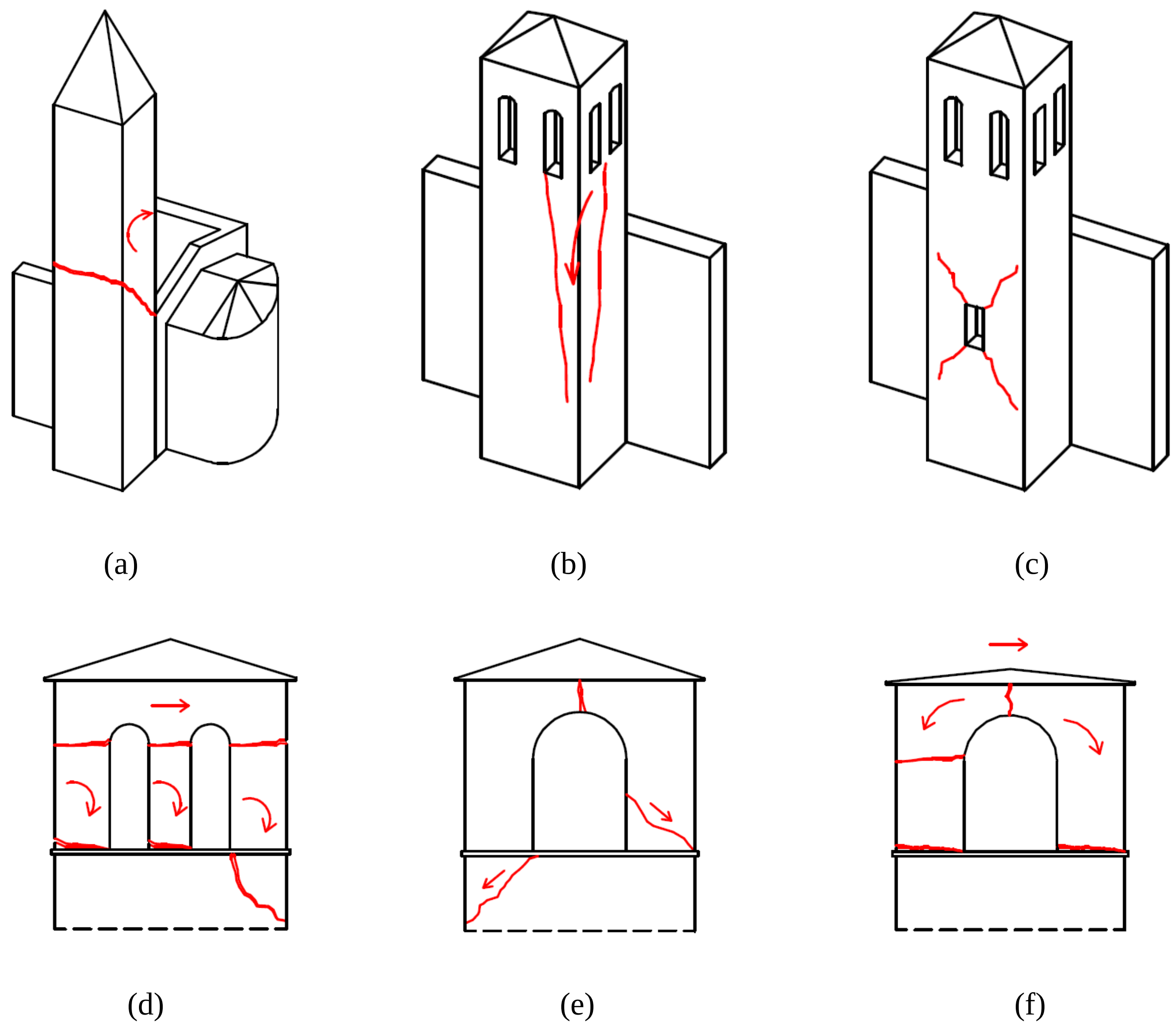


Figure 3: Observed EQ damage mechanisms at masonry towers; (a-c) at the body of the tower and (d-f) at the level of belfry (Lagomarsino et al., 2002)

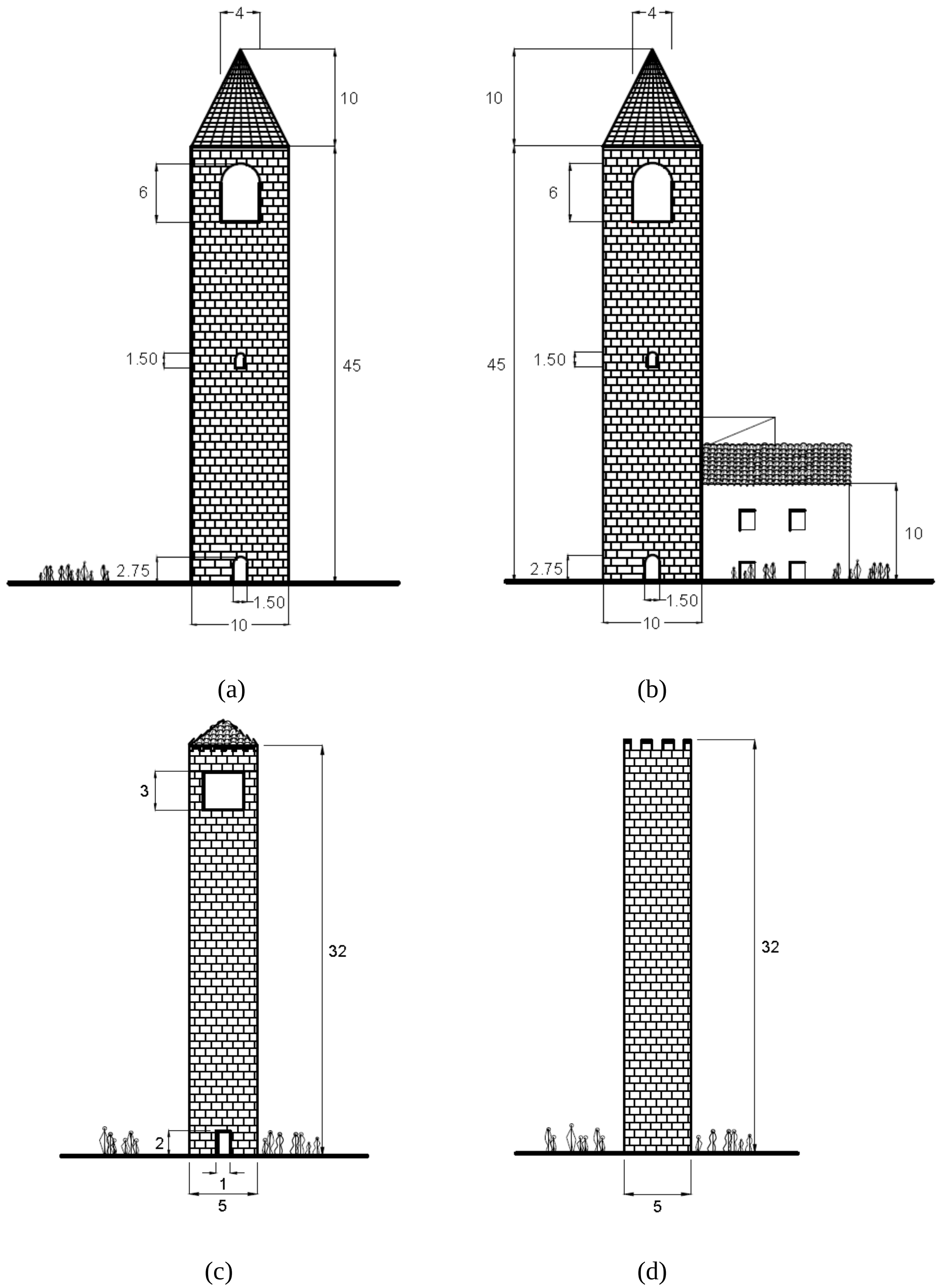
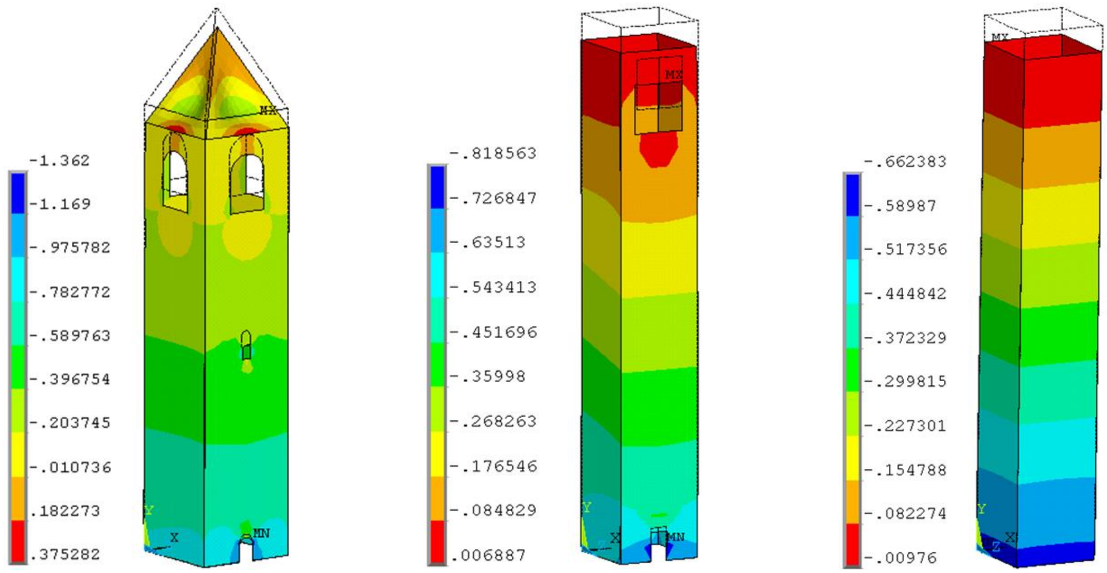


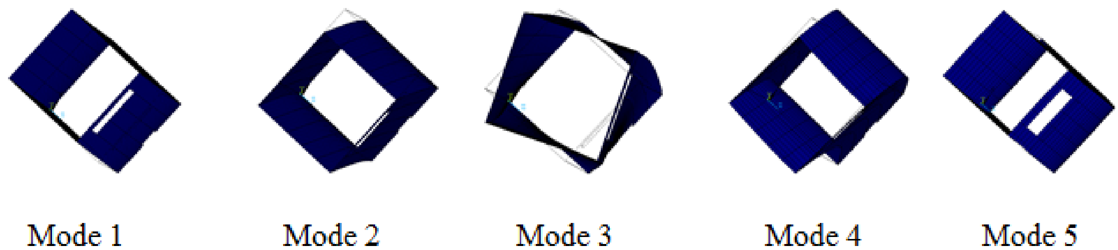
Figure 4: General view and dimensions (in m) of the four virtual old masonry towers; (a) AMT 1 heavy roof; (b) AMT 2 heavy roof; (c) AMT 3 light roof and (d) AMT 4 light roof



(a) (b) (c)

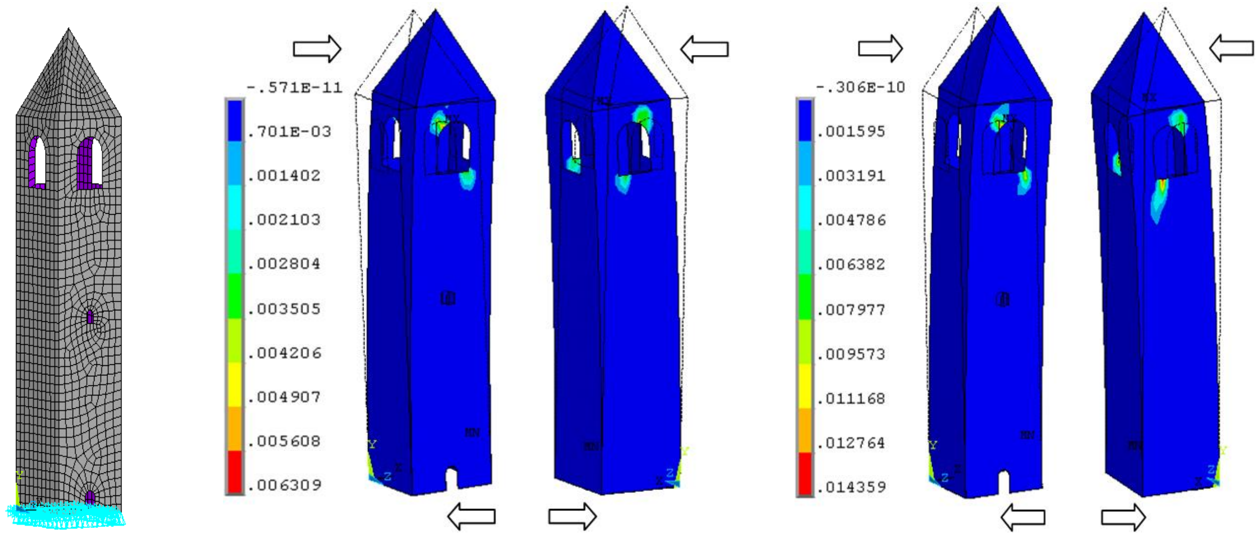
Figure 5: Vertical distribution of stresses at the FEM models (units in MPa); (a) AMT 1-2 masonry roof; (b) AMT 3 timber roof and (c) AMT 4 timber roof

(a)



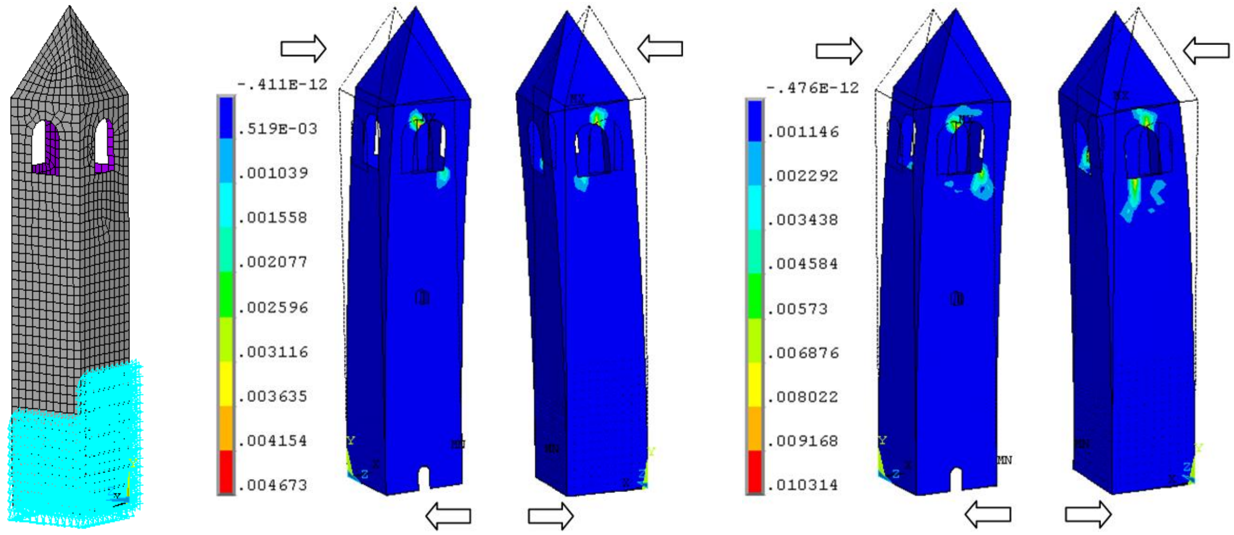
(b)

Figure 6: Top view of vibration modes of the four AMT; (a) AMT 1-2 and (b) AMT 3



(a) (b)

Figure 7: Isolated tower masonry roof (AMT 1). Principal plastic strain contours (front and back) at a displacement of: (a) 80 mm (DG 3, SDLS) and (b) 115 mm (DG 4, ULS)



(a) (b)

Figure 8: Non-isolated tower masonry roof (AMT 2). Principal plastic strain contours (front and back) at: (a) 70 mm (DG 3, SDLS) and (b) 105 mm (DG 4, ULS)

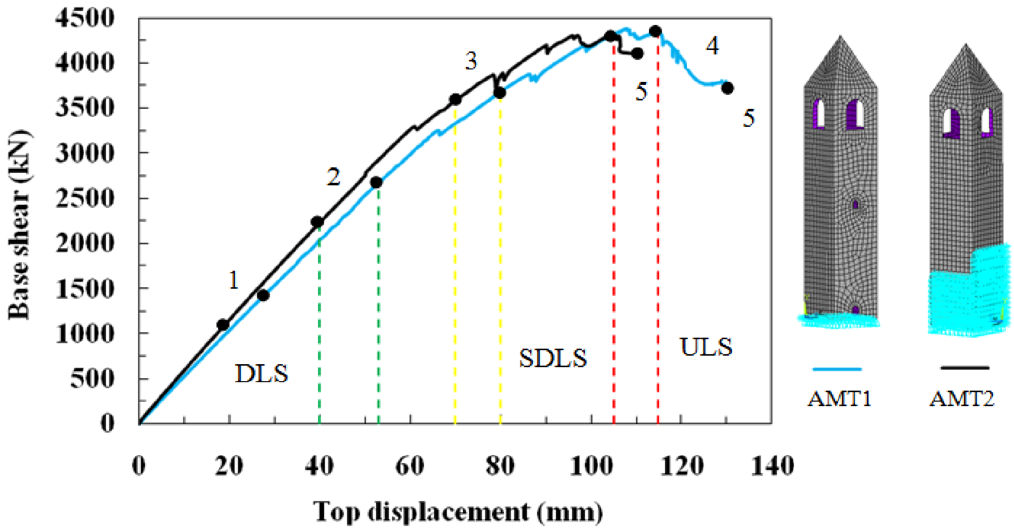
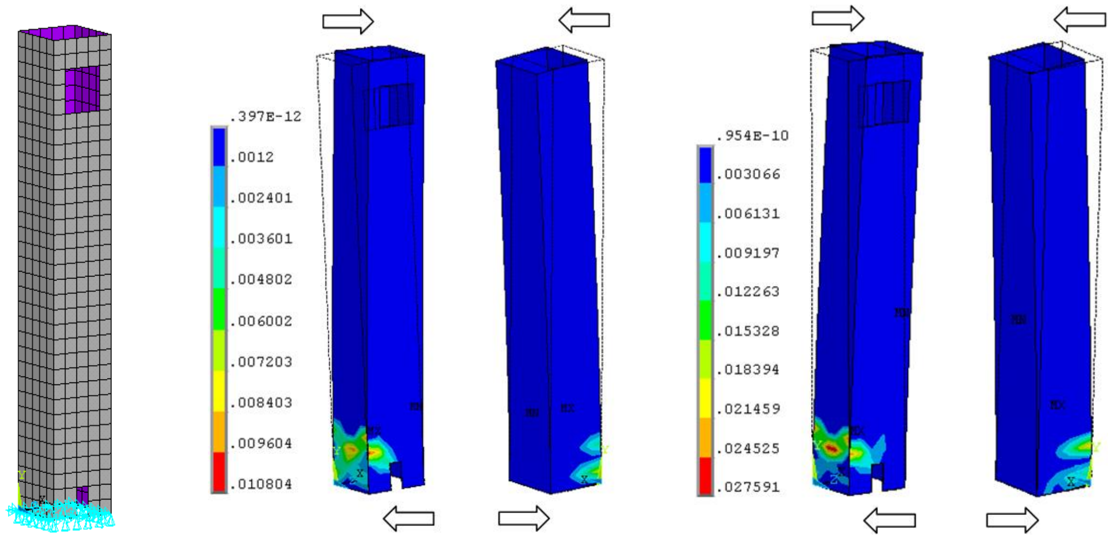
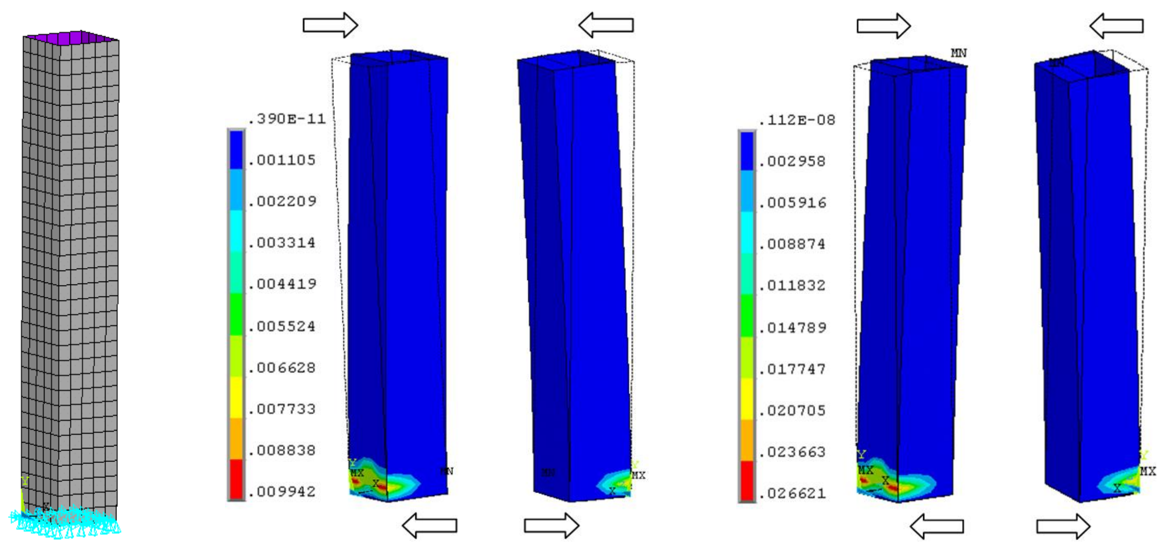


Figure 9: Isolated and non-isolated bell towers with masonry roof (AMT 1 and AMT 2). Capacity curves with the damage grades of the EMS-98 and limit states EC-8



(a) (b)

Figure 10: Isolated bell tower with light roof and openings (AMT 3). Principal plastic strain contours (front and back) at: (a) 185 mm (DG 3, SDLS) and (b) 325 mm (DG 4, ULS)



(a) (b)

Figure 11: Medieval tower light roof (AMT 4). Principal plastic strain contours (front and back) at a displacement of: (a) 155 mm (DG 3, SDLS) and (b) 265 mm (DG 4, ULS)

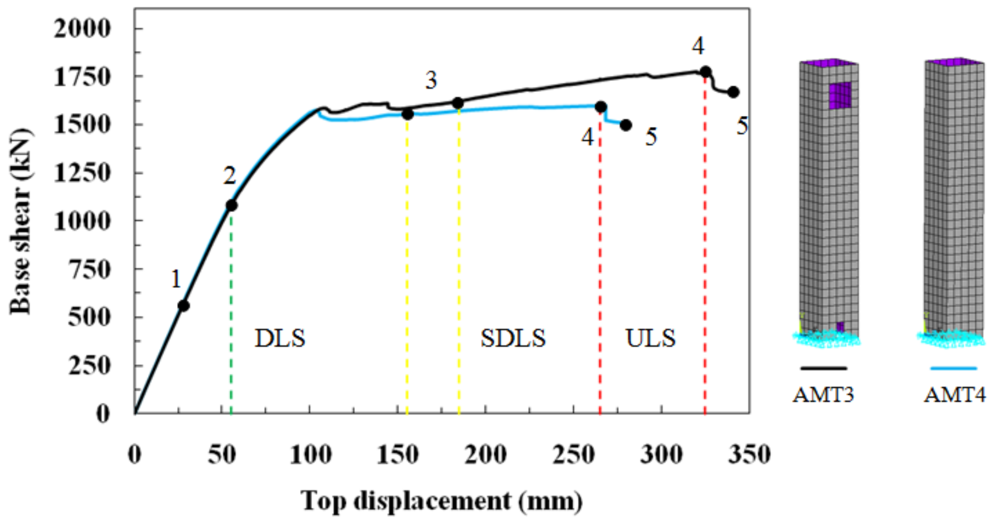


Figure 12: Bell and medieval towers with timber roof (AMT 3 and AMT 4). Capacity curves with the five damage grades of the EMS-98 and three limit states of the EC-8

LIST OF TABLES

Table 1: Summary of dimensions and FEM models of the four AMT

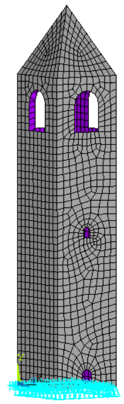
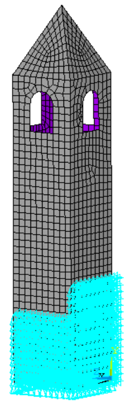
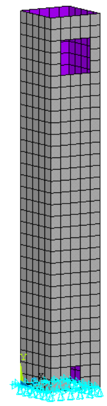
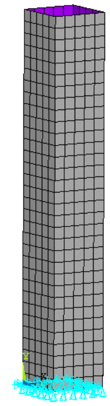
Dimensions in (m) **No Scale**	 **(a)**	 **(b)**	 **(c)**	 **(d)**
	AMT 1	**AMT 2**	**AMT 3**	**AMT 4**
Plan	10 x 10	10 x 10	5 x 5	5 x 5
Walls height (thk.)	45 (1.5)	45 (1.5)	32 (1.5)	32 (1.5)
Cover height (thk.)	10 (0.15)	10 (0.15)	-----	-----
Elements (nodes)	2050 (2125)	2050 (2125)	629 (656)	640 (660)
DOF	12627	12627	3876	3900

Table 2: Natural frequencies of the of the four AMT

Mode no.	Vibration mode	Frequency (Hz)			
		AMT 1	AMT 2	AMT 3	AMT 4
1st	Bending N-S	1.046	1.293	1.076	1.064
2nd	Bending E-W	1.051	1.133	1.083	1.064
3rd	Torsion	3.313	3.702	4.723	4.732
4th	Bending E-W	3.464	3.464	5.162	5.255
5th	Bending N-S	3.935	4.138	5.272	5.255

Table 3: Reference natural frequencies and periods of 10 historical masonry towers

Reference	**Tower height**	**Frequency (Hz)**		**Period (sec)**	
		1st	**2nd**	**1st**	**2nd**
Ramos et al. (2010)	20.40	2.15	2.58	0.47	0.39
Bayraktar et al. (2009)	22.00	2.56	2.66	0.39	0.38
Ivorra et al. (2008)	33.90	2.15	2.24	0.47	0.45
Slavik (2002)	35.00	1.10	1.30	0.91	0.77
Ivorra and Pallares (2006)	41.00	1.29	1.49	0.78	0.67
Abruzzese et al. (2009)	41.00	1.26	1.29	0.79	0.78
Lund et al. (1995)	43.50	1.38	1.82	0.72	0.55
Abruzzese et al. (2009)	45.50	1.05	1.37	0.95	0.73
Russo et al. (2010)	58.00	0.61	0.73	1.64	1.37
Gentile and Saisi (2007)	74.10	0.59	0.71	1.69	1.41

Table 4: 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 5: Summary of seismic analyses by the pushover method on the four AMT

FEM model reference	Limit states EC-8 and Damage grades EMS-98					
	DLS and DG 2		SDLS and DG 3		ULS and DG 4	
	F **(kN)**	***U*** **(mm)**	***F*** **(kN)**	***U*** **(mm)**	***F*** **(kN)**	***U*** **(mm)**
AMT 1	2700	53	3670	80	4350	115
AMT 2	2220	40	3600	70	4300	105
AMT 3	1100	55	1623	185	1750	325
AMT 4	1100	55	1553	155	1600	265

Table 6: Summary of SCs of the four AMT

FEM model reference	**Lateral force (kN)**	**Vertical loading (kN)**	**SC**
AMT 1	4350	50876	0.086
AMT 2	4300	50876	0.085
AMT 3	1750	18511	0.095
AMT 4	1600	18900	0.085