

Dynamic characterisation and seismic assessment of medieval masonry towers

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Received: 21 December 2015 / Accepted: 3 August 2016 / Published online: 10 August 2016
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Abstract The paper investigates the dynamic characterisation, the numerical model tuning and the seismic risk assessment of two monumental masonry towers located in Italy: the Capua Cathedral bell tower and the Aversa Cathedral bell tower. Full-scale ambient vibration tests under environmental loads are performed. The modal identification is carried out using techniques of modal extraction in the frequency domain. The refined 3D finite element model (FEM) is calibrated using the in situ investigation survey. The FEM tuning is carried out by varying the mechanical parameters and accounting for the restraint offered by the neighbouring buildings and the role of soil–structure interaction. The assessment of the seismic performance of the bell towers is carried out through a nonlinear static procedure based on the multi-modal pushover analysis and the capacity spectrum method. Through the discussion of the case studies, the paper shows that the modal identification is a reliable technique that can be used in situ for assessing the dynamic behaviour of monumental buildings. By utilising the tuned FEM of the towers, the theoretical fundamental frequencies are determined, which coincide with the previously determined experimental frequencies. The results from seismic performance assessment through a pushover analysis confirm that the masonry towers in this study are particularly vulnerable to strong damage even when subjected to seismic events of moderate intensity.

Keywords Masonry towers · Ambient vibration tests · Dynamic characterisation · Seismic assessment

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List of symbols

a	Parameter of the yield Drucker–Prager surface (–)
d	Soil depth (mm)
E	Young’s modulus (MPa)
E_f	Young’s modulus of foundation (MPa)
E_b	Young’s modulus of superstructure masonry (MPa)
E_s	Secant Young’s modulus (MPa)
E_t	Tangent Young’s modulus (MPa)
f	Frequency (Hz)
f_c	Compressive strength (MPa)
f_t	Tensile strength (MPa)
f_{AVT}	Natural frequencies from ambient vibration test (Hz)
f_{FEM}	Natural frequencies from finite element model (Hz)
$F_{AVT,i}$	i th modal force vectors from ambient vibration test (–)
$F_{FEM,i}$	i th modal force vectors from finite element model (–)
F_o	Amplification factor (–)
h	Bell tower height (m)
K	Parameter of the yield Drucker–Prager surface
k_n	Normal stiffness (MPa)
k_v	Vertical stiffness (MPa)
m	Mass (kg)
M	Mass matrix (kg)
N	Number of the experimental mode shapes (–)
PGA	Peak ground acceleration (g)
PGA _{LS}	Reference peak ground acceleration for the life safe limit state (g)
PSD	Power spectral density (g^2/Hz)
P_{VR}	Probability of exceedance (–)
S_a	Spectral acceleration (m/s^2)
S_d	Spectral displacement (cm)
t	Soil layer thickness (m)
T_R	Return period (years)
V	Base shear (MN)
W	Weight (kN)
α_i	i th modal mass ratio (–)
α_{LS}	Safety index at life safe limit state (–)
Γ_i	i th modal participation factor (–)
δ	Lateral displacement (m)
δ_{top}	Roof lateral displacement (m)
δ_y	Displacement in y -direction (m)
ε_t	Tensile strain (‰)
η_i	Error (–)
η	Weighted arithmetic mean error (–)
μ	Mass density (kg/m^3)
ν	Poisson’s ratio (–)
Φ_i	i th mode shape (–)

1 Introduction

Many ancient masonry structures are located in high-risk seismic areas, and although faultlessly able to withstand self-weight loads, they are inadequate to sustain the effect of horizontal inertia forces induced by earthquakes. Lessons learnt from recent major earthquakes highlighted the intrinsic vulnerability of the majority of historical structures. To address this vulnerability, the development of a new generation of international standards that requires the safety evaluation of historical structures even for seismic loads was proposed. This situation encouraged the development of reliable nonlinear models and methods of analysis suitable for seismic risk assessment of historical constructions. In particular, tall monumental buildings such as towers and mosque minarets are not adequate to withstand the horizontal forces caused by earthquakes. Representative problems of these slender masonry buildings are related to material degradation, intrinsic structural deficiencies, geotechnical problems, buckling behaviour of slender elements and dynamic loading vulnerability. There are many studies in the literature dedicated to seismic performance and damage assessment of historical structures (Lagomarsino and Cattari 2015; Lourenço and Roque 2006, Lourenço et al. 2007; Abruzzese et al. 2008, 2009a, b). Betti and Galano (2012) investigated the seismic vulnerability of a Palazzo del Vicario in Pescia, a small town near Florence. Cakir et al. (2016) discussed the basic principles to be considered in performance-based seismic evaluation of historical structures. In particular, the seismic vulnerability assessment of ancient masonry towers is a topic of great concern in the cultural heritage literature. A great number of studies are dedicated to full-scale ambient vibration tests (AVTs) and dynamic characterisation (De Sortis et al. 2005; Bennati et al. 2005; Ivorra and Pallarés 2006; Júlio et al. 2008; Ferraioli et al. 2010; Ramos et al. 2010; Osmancikli et al. 2012; Ferraioli 2015). Bayraktar et al. (2009) performed the modal identification and finite element analysis of the Hagia Sophia bell tower in Trabzon, Turkey. Russo et al. (2010) presented the experimental analysis of the “Saint Andrea” masonry bell tower in Venice. Tomaszewska and Szymczak (2012) discussed the problem of parametric identification of the Vistula Mounting tower. D’Ambrisi et al. (2012) evaluated the dynamic characteristics and mechanical properties of the medieval civic tower of Soncino (Cremona, Italy) through a dynamic characterisation with AVTs. Gentile et al. (2015) developed a vibration-based methodology for the calibration of a 3D FEM of the historic bell tower of the church Chiesa Collegiata in Arcisate (Varese, Italy). Preciado (2015) proposed a methodology for the seismic vulnerability assessment of all types of towers and slender unreinforced masonry structures. Casolo et al. (2013) presented a comparative seismic vulnerability analysis on ten masonry towers in the coastal Po Valley in Italy. In the same way, many studies are concerned with the need of using an appropriate masonry material model able to represent its nonlinear behaviour (Bernardeschi et al. 2004; Carpinteri et al. 2005; Abruzzese et al. 2009c; Peña et al. 2010; Milani et al. 2012).

As stated, this paper investigates the seismic vulnerability of two ancient masonry towers located in Southern Italy. This type of study is a challenging task because numerous parameters related to the geometrical and the mechanical properties of the structures are uncertain. This uncertainty makes the prediction of structural risk quite critical. Specifically, numerical modelling and simulation of the mechanical behaviour of masonry are of great importance, due to both intrinsic material complexity and large scatter of mechanical properties. Reliable procedures for the characterisation of the structural parameters from in situ investigation should be defined, and the effects of the random characteristics of masonry on the reliability of the results should be estimated. The dynamic characteristics

and the mechanical properties of masonry towers can be effectively assessed through dynamic characterisation. The knowledge of their dynamic properties, together with site seismicity and stratigraphy, represents a key aspect of the seismic assessment of these structures, in particular when an accurate model has to be implemented. Following this approach, this paper describes the full-scale AVTs under environmental loads carried out on two ancient masonry towers in Italy. Based on the modal characteristics of the towers, linear elastic analyses were performed for model validation. The corresponding FEM was calibrated by minimising the difference between theoretical and experimental natural frequencies and mode shapes by means of a trial-and-error procedure. This procedure is based on the in situ investigation survey. In conclusion, a nonlinear material model for the masonry was implemented to perform the seismic assessment through capacity spectrum (CS) method based on inelastic demand spectra.

2 Description of the towers and on-site tests

The present study was carried out considering two representative case studies of historical interest. The first one is the Aversa Cathedral bell tower originally built between 1053 and 1080 at the side of an ancient Lombard church. Figure 1 shows the Cathedral of Saint Paul in the city centre of Aversa. Figure 2 shows the general view, the front elevation and the internal view of the Cathedral bell tower. The present tower was built in 1499 after the collapse of the original tower under a strong earthquake occurred in 1457. The tower is about 45.5 m high from the ground level and has a square cross section of side about 14.0 m. Some anomalies, such as biological colonisation, a general state of deterioration and material degradations, were noticed. No major structural damage was observed following visual inspection apart from a concentration of vertical cracks passing through the



Fig. 1 The Cathedral of Saint Paul in the city centre of Aversa (credits: Ugo Persice Pisanti)

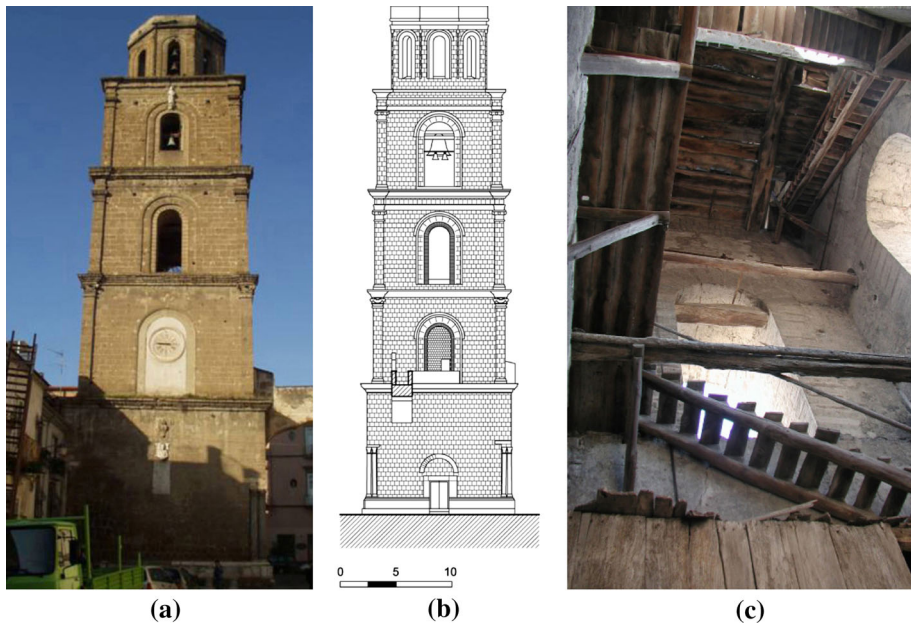


Fig. 2 Aversa Cathedral bell tower. General view (a); north elevation (b); internal view (c)

keystones of the arch openings along the four sides. These typical cracks in the arch lintels are caused by the structural behaviour of the tower under the dead loads. The vertical bearing structure is composed of four masonry piers on the edges, connected by spandrels with masonry arch above the openings. The first level of horizontal structures has masonry vaults, and the upper levels have timber floors. Such structures have poor in-plane stiffness and provide a weak connection to the vertical macro-elements. Thus, the outward-directed horizontal forces of the arch, caused by the overlying wooden beams, facilitate the overturning of the piers. This mechanism is responsible for the cracks above the arched window openings. In order to counteract these horizontal forces, a system composed of horizontal tie bars was installed at the third and fourth levels, which improves the connection between the vertical macro-elements.

No specific investigation was carried out on the foundation structures. Nevertheless, based on a widespread knowledge of the site geology, it is possible to assume that the bell tower lies on a uniform bedrock made of compact Campanian grey tuff. Thus, the high stiffness soil-foundation system allows excluding the possibility of instability conditions by soil settlements. Likewise, the tower does not show typical events of structural deteriorations due to differential settlements of foundation such as loss of connections due to cracking, rotation of the foundation or inclination of the superstructure.

The information provided by historical documentation and past survey results was used. The following activities were carried out: geometrical and material survey, survey of crack patterns, chemical tests, monotonic compressive tests, single flat-jack tests (SBAAAS CE-BN 2008). Table 1 shows the results of monotonic compressive tests (ASTM C1196-91) on three samples of the tower base, where f_c is the compressive strength, E_t is the tangent modulus of elasticity, and E_s is the secant modulus of elasticity. The soil profile down to the investigated depth is composed of a first layer of pyroclastic loose rocks and a second

Table 1 Results from monotonic compressive tests carried out on Aversa Cathedral bell tower

Sample	Material	f_c (MPa)	E_t (MPa)	E_s (MPa)
1	Yellow tuff with grey pumices	4.23	3990	3820
2	Yellow tuff (lithoid facies)	2.85	3660	1920
3	Yellow tuff with grey pumices	3.46	1920	2080

one of Campanian Ignimbrite. The details of the soil stratigraphy are listed in Table 2. The second case study is the Capua Cathedral bell tower. Figure 3 shows the Church of S.S. Annunziata in the city centre of Capua. Figure 4 shows the general view, the front elevation and the internal view of the Cathedral bell tower. Founded in 856 in the Lombard Period by bishop Landolfo I, the Capua Cathedral bell tower was destroyed by a strong earthquake in 990 and then rebuilt during the Norman conquest of southern Italy in twelfth century. Four columns were built on top of the bell tower to support a marble plate. All these columns were destroyed by a strong earthquake ground motion in 1457. The tower is about 41 m high from the ground level and has a square section of about 11.3 m. The first two levels, characterised by corner columns and double lancet windows, are made of limestone blocks obtained from ancient Rome buildings. The upper floors are composed of Campanian tuff blocks and clay bricks. The soil under the Capua Cathedral bell tower includes mostly a group of pyroclastic sedimentary rocks typical of the volcanic Phlegrean area (Table 3).

3 Ambient vibration tests

3.1 Test set-up

The AVTs on the Aversa Cathedral were performed using a data acquisition system (DAQ) and four integrated circuit piezoelectric (ICP) accelerometers (PCB 393C) with a sensitivity of 1 V/g, a frequency range ($\pm 5\%$) of 0.025–800 Hz, a signal conditioning circuitry and a 16-bit analog-to-digital converter. The response of the tower was measured in selected points at different cross sections along the height of the tower. The sensor layout shown in Fig. 5 was selected to account for the dynamic properties of a preliminary FEM of the tower. In the same way, the choice of the experimental measurements range (0–6 Hz) was related to results of the FEM modal analysis. Acceleration time histories induced by ambient excitation were recorded for 2880 s. As far as the AVTs on the Capua Cathedral bell tower are concerned, the data acquisition system (DAQ) included eight

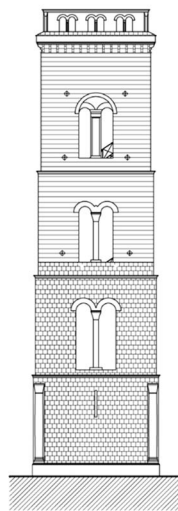
Table 2 Soil profile under the Aversa Cathedral bell tower

Layer	Material	t (m)	d (m)
1	Vegetable soil	3.00	0.00
2	Pyroclastic soil: ashes and pumices	1.70	3.00
3	Paleosoil	0.50	4.70
4	Pyroclastic soil: ashes and pumices	2.00	5.20
5	Campanian grey tuff	Bed rock	7.20

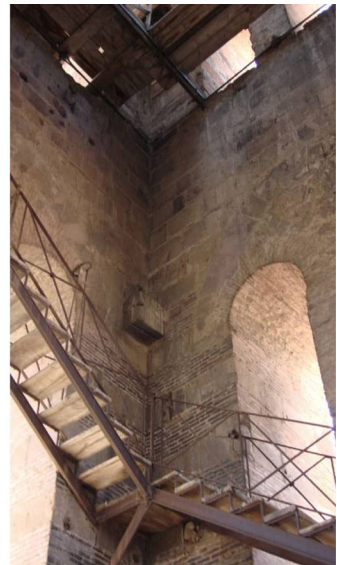
Fig. 3 Church of S.S. Annunziata in the city centre of Capua (credits: Agostino De Maio)



(a)



(b)



(c)

Fig. 4 Capua Cathedral bell tower. General view (a); north elevation (b); internal view (c)

integrated circuit piezoelectric (ICP) accelerometers (PCB 393B04) with a sensitivity of 1 V/g and a frequency range ($\pm 5\%$) of 0.06–450 Hz, a signal conditioning circuitry and a 16-bit analog-to-digital converter. The sensor layout is shown in Fig. 6. The experimental

Table 3 Soil profile under the Capua Cathedral bell tower

Layer	Material	<i>t</i> (m)	<i>d</i> (m)
1	Vegetable soil	3.00	0.00
2	Yellow cinerite	3.00	3.00
3	Brown cinerite	1.00	6.00
4	Brown tuff	2.00	7.00
5	Campanian grey tuff	6.00	9.00

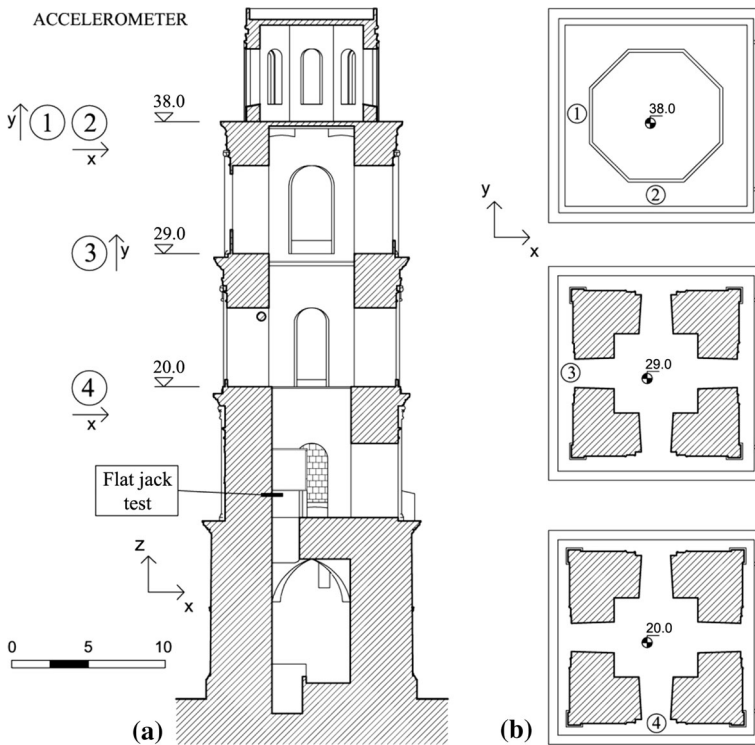


Fig. 5 Ambient vibration sensor layout. Aversa Cathedral bell tower: elevation (a); plans (b)

measurements, acquired with a sampling frequency of 100 Hz and electronically filtered under 30 Hz, were processed, and the results obtained were in a range of 0–7 Hz. The acquisition data were obtained using only the ambient noise due to road traffic and wind. The data acquisition was run by LabVIEW software (2003). This software allowed the signals acquisition, as well as the real-time visualisation of accelerograms and Fourier spectra at 10-s intervals.

3.2 Tests results

The AVTs provided useful information about the modal parameters of the towers. In general, a large number of modal identification methods are available in the literature. These methods range from the simple peak picking (PP) method (Bendat and Piersol 1993)

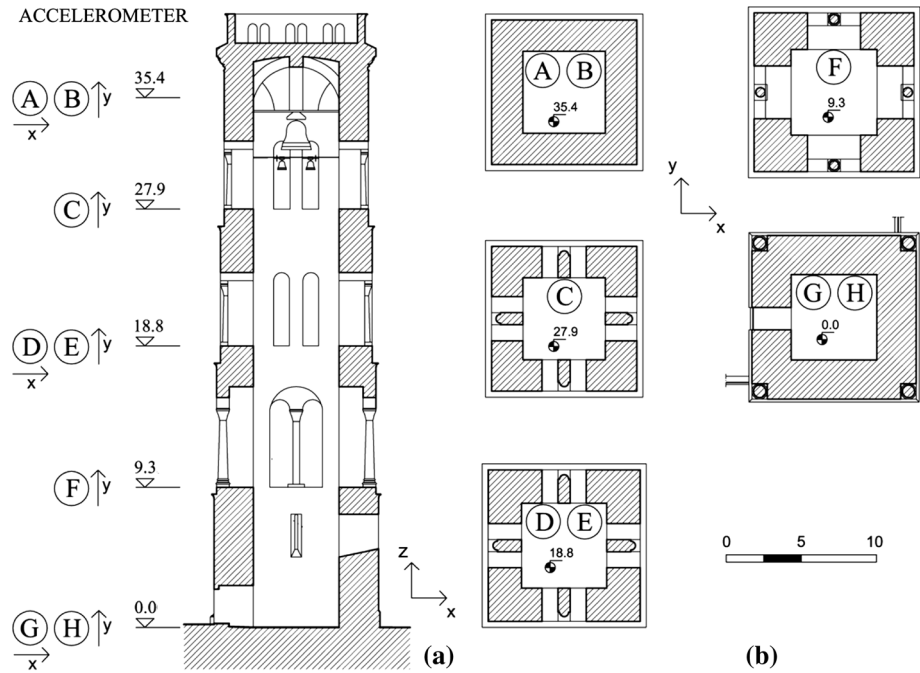


Fig. 6 Ambient vibration sensor layout. Capua Cathedral bell tower: elevation (a); plans (b)

to the more advanced frequency domain decomposition (Brincker et al. 2000) and stochastic subspace identification (Van Overschee and De Moor 2012; Peeters 2000). In this paper, the data obtained from the monitoring were analysed using the PP method. This method, based on the evaluation of the frequency peaks of spectral densities in the frequency domain, is applied to the power spectral density (PSD) of the signals. Generally, the PP method is able to give reliable results only when the basic assumptions of well-separated modes and low damping are satisfied. The main drawback of the PP method is that it is unable to identify closely spaced modes. In the case studies here described, the spectral densities of the signal did not display closely spaced modes. Therefore, there was no need to apply more sophisticated methods of modal extraction. In Fig. 7, the PSD of some signals recorded on the Aversa Cathedral bell tower is plotted. The measured acceleration signals in accelerometer 2 (*X*-direction) and accelerometer 3 (*Y*-direction) were considered (Fig. 5). The peaks corresponding to the resonant frequencies are given at 1.05, 1.37, 4.81, 4.89 and 5.05 Hz. Figure 8 shows the PSD of the signals recorded on the Capua Cathedral bell tower from accelerometer A (*Y*-direction), accelerometer B (*Y*-direction) and accelerometer F (*Y*-direction). Resonant frequencies are given at 1.26, 1.29, 3.10, 6.15 and 6.17 Hz. The natural frequencies identified in the AVTs on both towers are summarised in Table 4. In the case study of the Aversa Cathedral bell tower, both the number and location of the sensors during AVTs were not sufficient to extract the experimental mode shapes. On the other hand, for the Capua Cathedral bell tower the sensor layout allowed also the identification of the flexural mode shapes. In Fig. 9, the *Y*-displacements δ_y corresponding to the first flexural *Y* (mode 2, frequency 1.29 Hz) and second flexural *Y* (mode 5, frequency 6.17 Hz) mode shapes are plotted. The nearly symmetric shape of the tower produced two bending modes with close frequency values

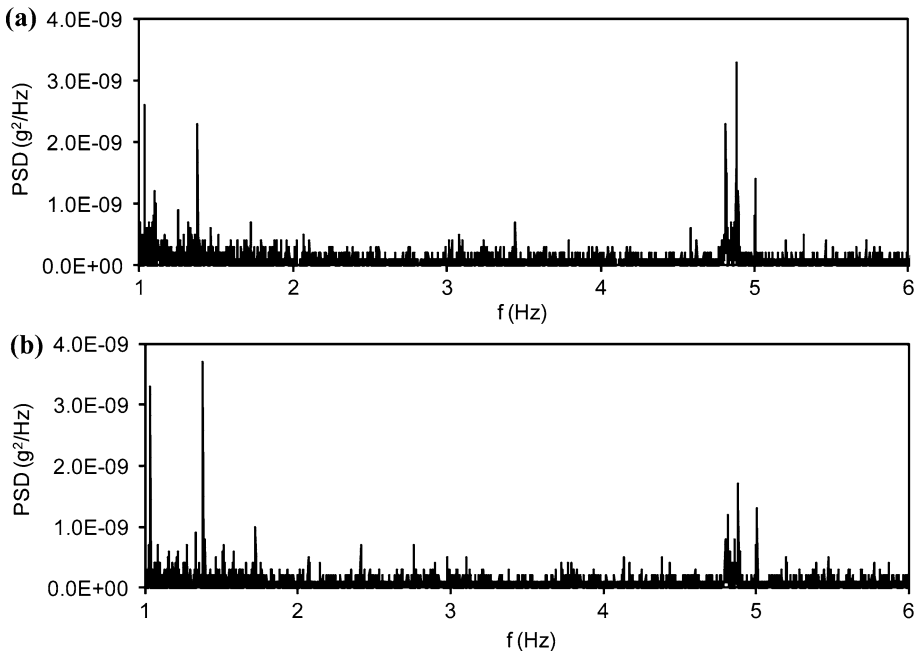


Fig. 7 PSD versus frequency. Signal in accelerometers 2 (a) and 3(b). Aversa Cathedral bell tower

(1.26 Hz for the first flexural *X* mode shape and 1.29 Hz for the first flexural *Y* mode shape). Likewise, the second two bending modes have close frequency values (6.15 Hz for the second flexural *X* mode shape and 6.17 Hz for the second flexural *Y* mode shape). Thus, the first flexural *X* and second flexural *X* modes shapes can be considered closely correspondent to the identified first and second flexural *Y* modes shapes.

4 FEM tuning by structural characterisation

4.1 Finite element models

In this paper, the modal parameters measured via the AVTs (i.e. natural frequencies and mode shapes) were used to identify the uncertain parameters of the numerical model. The linear elastic analysis was applied to update the initial model using the experimental results for adjusting geometry, material properties and interaction with adjacent buildings. The sensors location in AVTs helped to collect information on both bending and torsional modes. To obtain the dynamic properties, a preliminary 3D FEM with 8-node brick solid elements was implemented using the finite elements analysis software LUSAS (2012). A modal analysis was performed using the 3D FEM. The geometry was reproduced thanks to an accurate geometric survey on the towers. The model takes into account the location of openings and masonry floors and the foundation. The actual depth of the foundation was estimated from other towers similar in age and typology. The mesh was refined until the variation of frequency was lower than 1 % for all the considered modes. The model had 50,550 nodes and 41,517 elements for the Aversa Cathedral bell tower. 47,090 elements

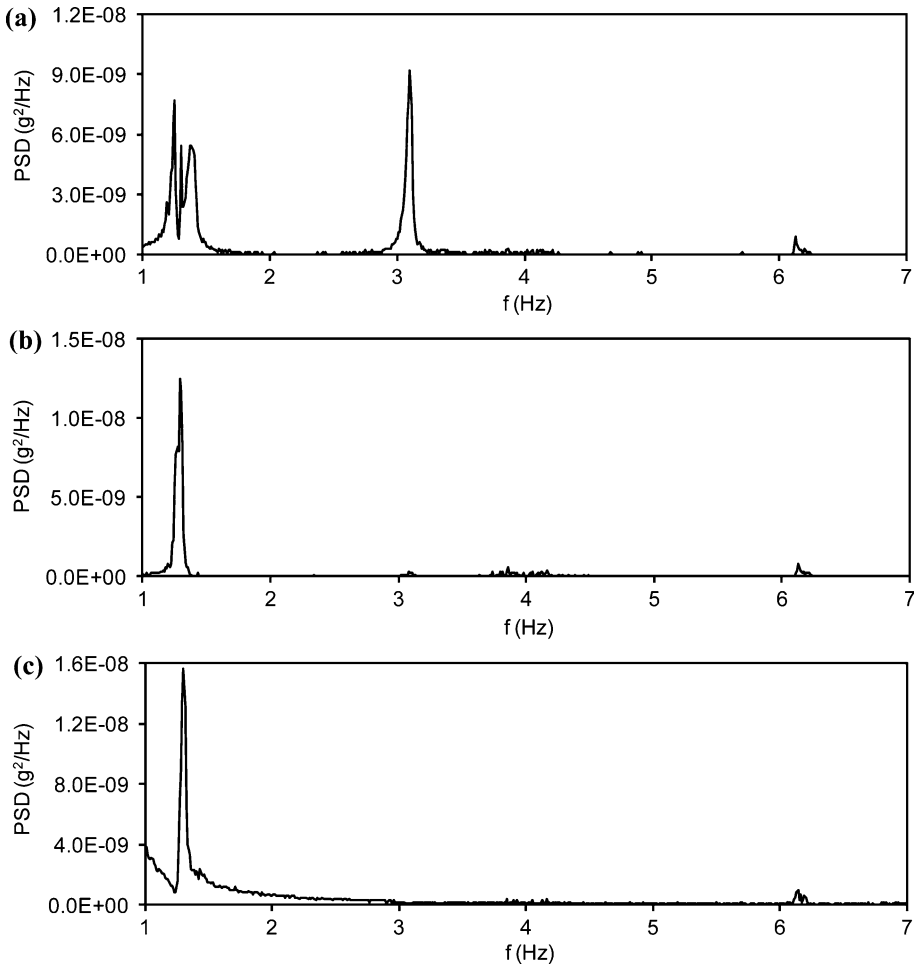


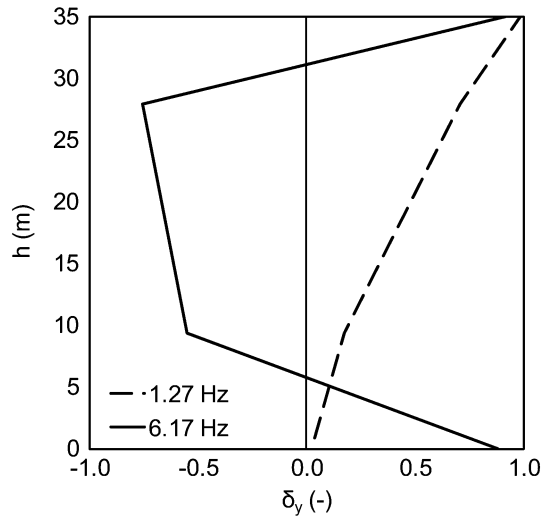
Fig. 8 PSD versus frequency. Signal in accelerometers A (a), B (b) and F (c). Capua Cathedral bell tower

Table 4 Natural frequencies identified from ambient vibration tests

Mode No.	Mode type	Direction	Aversa Cathedral bell tower f_{AVT} (Hz)	Capua Cathedral bell tower f_{AVT} (Hz)
1	1° flexural	X	1.05	1.26
2	1° flexural	Y	1.37	1.29
3	1° torsional	–	4.81	3.10
4	2° flexural	X	4.89	6.15
5	2° flexural	Y	5.05	6.17

and 54,066 nodes were used for the model of Capua Cathedral bell tower. The wall masonry was considered as homogeneous by assuming an equivalent Young’s modulus E and an equivalent shear modulus G . The number of elements and nodes was calibrated to

Fig. 9 Experimental identified mode shapes. Capua Cathedral bell tower



obtain a regular distribution of masses. This distribution allowed to accurately reproduce the spatial characteristics of the building. The soil–structure interaction was considered significant for the analyses here developed. For this reason, this effect was simulated by a uniform distribution of linear elastic springs (both vertical and horizontal) of constant stiffness implemented into the FEM to represent the interaction between the foundation base and the soil. In Figs. 10 and 11, the first and second flexural X mode shapes and the first torsional mode shape, respectively, for the Aversa Cathedral bell tower and for the Capua Cathedral bell tower, are shown.

4.2 Calibration of main parameters

The FEM calibration was carried out considering the uncertainty related to some parameters with major influence on the dynamic behaviour of the tower. These parameters are the restraints offered by the adjacent structures, the soil–structure interaction and the Young’s modulus of masonry. These parameters were considered as updating parameters, and they were iteratively modified so that the main natural frequencies obtained from the model match those measured on the actual structure. First, typical values of Young’s modulus E and Poisson’s ratio ν were used for the constituent parts of the bell tower (foundation, superstructure, top and damaged lintels). The values of the spring constants simulating the soil–structure interaction and the constraining effect of the neighbouring structures were updated following an iterative approach. This iterative process ran until a satisfactory agreement between numerical and experimental frequencies was achieved. Likewise, the assessment of the restraint conditions of the towers by trial-and-error procedure was carried out until an acceptable agreement was achieved between some specific dynamic properties obtained from AVTs and FEM. In particular, the following dynamic properties were used as control parameters: (1) ratio between first flexural X and first torsional frequencies; (2) ratio between first flexural X and first flexural Y frequencies; (3) ratio between first and second flexural X frequencies; (4) ratio between first and second flexural Y frequencies. Following this iterative approach, the constraining effect of the neighbouring arch connecting the bell tower with the Aversa Cathedral was modelled as a restraint in the

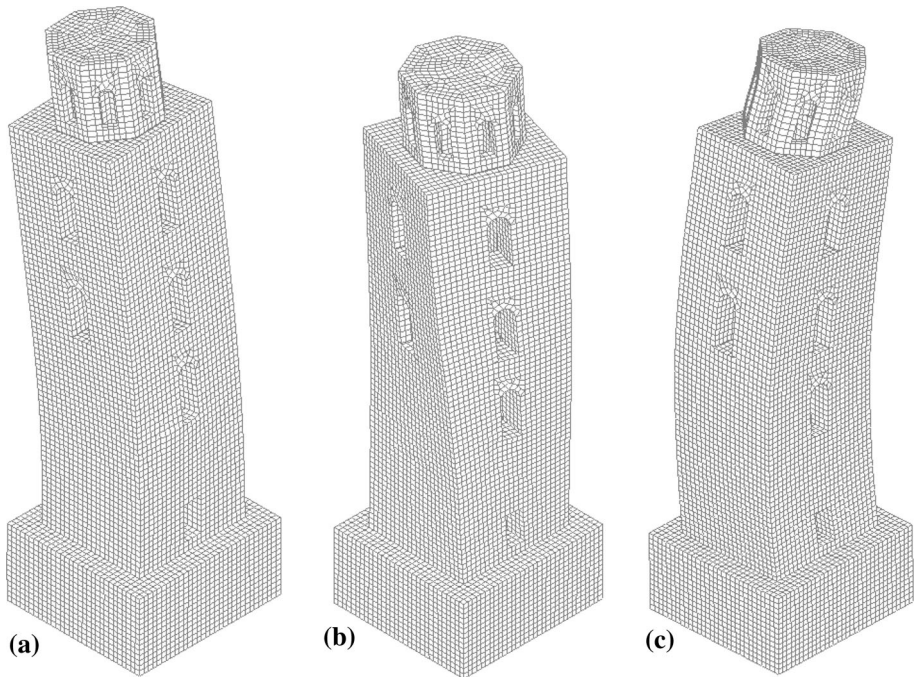


Fig. 10 Numerical mode shapes. Aversa Cathedral bell tower: first flexural (a); first torsional (b); second flexural (c)

direction parallel to the arch (X -direction). Moreover, the horizontal and vertical spring constants accounting for soil–structure interaction were set to: $k_h = 0.5$ MPa and $k_v = 2.0$ MPa, respectively, for the Aversa Cathedral bell tower, and $k_h = 0.9$ MPa and $k_v = 2.0$ MPa for the Capua Cathedral bell tower. After the calibration of the above-mentioned parameters (soil spring constants and the constraining effect of the neighbouring structures), the Young's modulus of masonry was varied so as to get in a closer agreement with the results of AVTs. For the Aversa Cathedral bell tower, the Young's modulus was assumed to have a value in the range of 1600–2000 MPa for foundation masonry (E_f) and of 2000–3000 MPa for superstructure masonry (E_b). For the Capua Cathedral bell tower, the Young's modulus E was found to be in the range of 3400–5800 MPa for the limestone of foundation (E_f) and of 1600–4000 MPa for the clay bricks and Campanian tuff of the superstructure masonry (E_b). As expected, the optimum value of Young's modulus was related to the parameter chosen for calibration, namely the frequency, mode shape and modal force vector. For the Aversa Cathedral bell tower, both the number and location of the sensors during AVTs were not sufficient to give an accurate evaluation of the mode shapes of the tower. For this reason, the optimal values of the Young's modulus were selected by minimising the weighted arithmetic mean of the relative errors between the natural frequencies obtained from AVTs and FEM. This error was computed by making the mean, weighted on the modal mass ratio (α_i), of the errors (η_i) on each frequency, as follows:

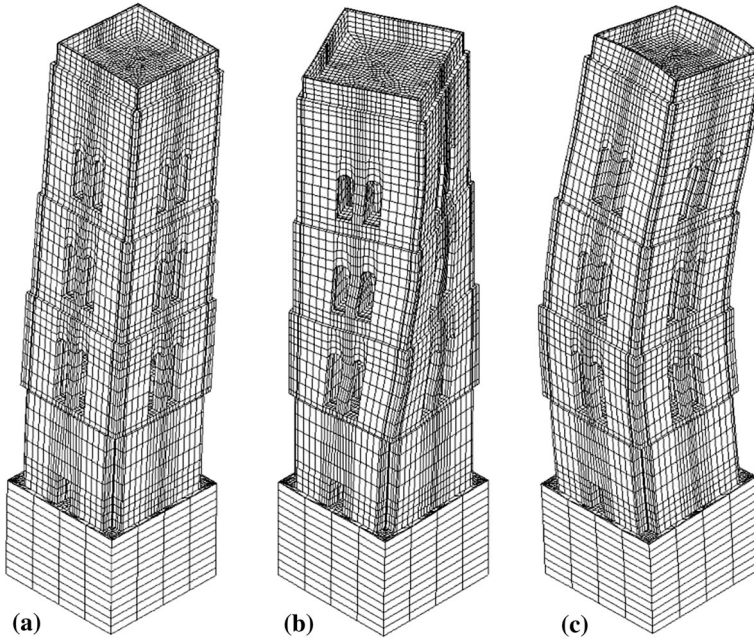
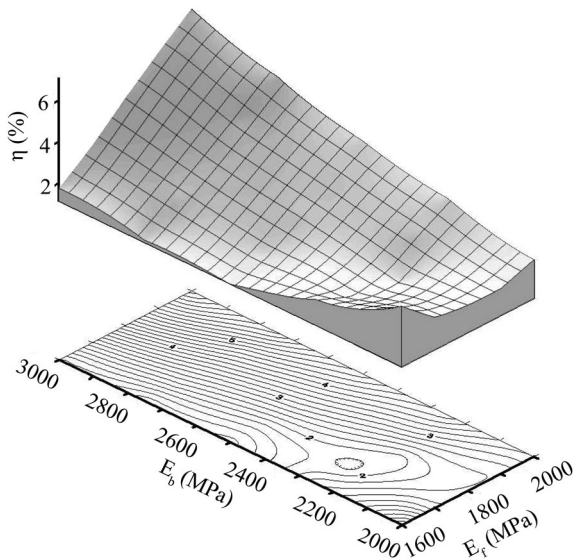


Fig. 11 Numerical mode shapes. Capua Cathedral bell tower: first flexural (a); first torsional (b); second flexural (c)

$$\eta = \frac{\sum_{i=1}^N \eta_i \cdot \alpha_i}{\sum_{i=1}^N \alpha_i} \quad \eta_i = \left| \frac{f_{FEM,i} - f_{AVT,i}}{f_{AVT,i}} \right| \quad (1)$$

By the varying values of the Young’s modulus for masonry in the pre-selected intervals, a number of data sets of the relative errors were found. These irregularly spaced data were

Fig. 12 Error in frequency versus E_b and E_f . Aversa Cathedral bell tower



interpolated using the Kriging interpolation technique (Cressie 2015). This technique allowed plotting contours and surfaces (Fig. 12). The minimum value of the error of 1.12 % was reached for $E_b = 1710$ MPa and $E_f = 2260$ MPa. In Table 5, the values of mechanical parameters after tuning are reported. In Table 6, the comparison between the natural frequencies obtained from AVTs and tuned FEM is shown. The correlation between numerical and experimental frequencies are in a good agreement, especially for the frequencies of first flexural X ($\eta = 0.95$ %) and first flexural Y ($\eta = 0.73$ %) mode shapes. For the Capua Cathedral bell tower, combined frequency and mode shape data were obtained from the AVTs. These data allowed the definition of the error in both the natural frequency and the mode shape data. Assuming that the aim of the dynamic investigations is the seismic vulnerability assessment, the calibration of the FEM was carried out to effectively simulate the modal lateral load pattern. Thus, the optimal values of the Young’s modulus were selected by minimising the weighted arithmetic mean of the errors in terms of modal lateral load patterns in both directions, defined as follows:

$$\eta = \frac{\sum_{i=1}^N \eta_i \cdot \alpha_i}{\sum_{i=1}^N \alpha_i} \quad \eta_i = \frac{\|F_{FEM,i} - F_{AVT,i}\|}{\|F_{AVT,i}\|} \tag{2}$$

In Eq. (2), $F_{AVT,i}$ and $F_{FEM,i}$ are the modal lateral load patterns, defined as follows:

$$F_{AVT,i} = M\Phi_{AVT,i}\Gamma_i(\Phi_{AVT,i})S_a(f_{AVT,i}) \tag{3}$$

$$F_{FEM,i} = M\Phi_{FEM,i}\Gamma_i(\Phi_{FEM,i})S_a(f_{FEM,i}) \tag{4}$$

where $\Phi_{AVT,i}$ and $\Phi_{FEM,i}$ are the i th mode shape obtained from AVT and FEM analysis, respectively, and S_a is the spectral acceleration computed from the 5 % damped elastic demand response spectrum (EDRS) of the Italian Building Code (NTC 2008). Figure 13 shows the contour and surface plots obtained by varying the values of the Young’s modulus for masonry in the pre-selected intervals and using Kriging interpolation technique. The minimum value of the error of 16.92 % was attained for $E_f = 5000$ MPa and $E_b = 1600$ MPa. Table 6 shows the comparison between the natural frequencies obtained from AVTs and tuned FEM. Also in this case, a good agreement between numerical and experimental frequencies was obtained, in particular for the frequencies of first flexural X ($\eta = 1.59$ %) and first flexural Y ($\eta = 2.33$ %) mode shapes.

5 Seismic vulnerability assessment

5.1 Preliminary considerations

The vulnerability analysis is able to provide the potential collapse mechanism in relation to the applied load. As described in the afore-mentioned papers, the seismic performance of masonry towers is greatly influenced by their geometry and, most of all, by their slenderness (defined as the ratio between the height and the base length), thickness of perimeter walls and percentage of openings, especially at belfries. The boundary conditions represent an additional important aspect as the towers are generally part of an urban aggregate. Consequently, the constraining effect of the neighbouring structures greatly influences the behaviour and, hence, the vulnerability of the structure. Other significant aspects are related to the soil conditions that play a major role in seismic vulnerability of towers, especially in the presence of interaction effects with soft soil. Summing up, the most

Table 5 Elastic parameters of masonry

Cathedral bell tower	Foundation		Basement		Body	
	E (MPa)	ν (-)	γ (kg/m ³)	E (MPa)	ν (-)	γ (kg/m ³)
Aversa	1800	0.15	1800	1710	0.15	1900
Capua	2800	0.16	1600	5000	0.20	1800
Cathedral bell tower	Damaged lintels					
	Top					
	E (MPa)	ν (-)	γ (kg/m ³)	E (MPa)	ν (-)	γ (kg/m ³)
Aversa	2000	0.15	1800	2000	0.15	1600
Capua	-	-	-	-	-	-

Table 6 Comparison between experimental and numerical frequencies

Mode No.	Aversa Cathedral bell tower			Capua Cathedral bell tower		
	f_{AVT} (Hz)	f_{FEM} (Hz)	η (%)	f_{AVT} (Hz)	f_{FEM} (Hz)	η (%)
1	1.05	1.06	-0.95	1.26	1.24	-1.59
2	1.37	1.36	-0.73	1.29	1.26	-2.33
3	4.81	3.76	-21.8	3.10	3.57	+15.2
4	4.89	4.64	-5.11	6.15	4.65	-24.4
5	5.05	6.14	+17.7	6.17	4.68	-24.1

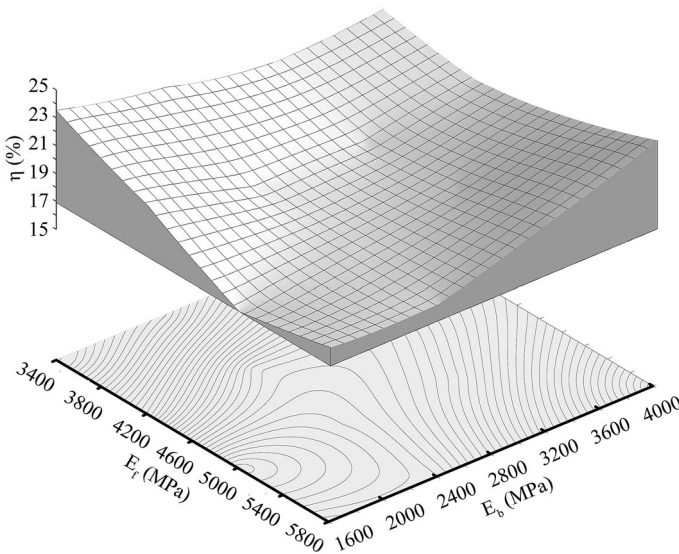


Fig. 13 Error in modal load pattern versus E_b and E_f . Capua Cathedral bell tower

decisive factor affecting both seismic behaviour and failure mechanism of historical towers under seismic loads are related to structural materials and constructional techniques (compressive stress under dead load, connection between structural elements, nonlinear behaviour and progressive deterioration of masonry). The available data obtained from past in situ surveys on the Aversa Cathedral bell tower are much greater than those obtained for the Capua Cathedral bell tower. An extensive experimental investigation with the traditional techniques (monotonic compressive tests and single flat-jack tests) was carried out on the Aversa Cathedral bell tower. At present, similar experimental data are lacking for the Capua Cathedral bell tower. In both cases, the available material data are not sufficient to define the strength characteristics of masonry in an exhaustive way. Consequently, data available in the literature are used, also complying with those reported from other towers of similar age and material. A value of the compressive strength f_c of 3.0 MPa was assumed also based on what suggested by the Italian Building Code (NTC 2008) in Annex C8A.2. A tensile strength f_t of 0.15 MPa was considered in the analysis. This value was assumed equal to 1/20 of the compressive strength f_c .

The single flat-jack tests carried out on the Aversa Cathedral bell tower give the stress state in specific locations, and this is useful information for the model verification and validation process. To achieve this aim, a preliminary static analysis under dead loads was performed. In Fig. 14, the vertical stress maps obtained by the FEM analysis are plotted. These stress maps are in good agreement with the stress measured by the flat-jack tests (SBAAAS CE-BN 2008). The maximum stress at the base of the tower was 1.32 MPa for the Aversa Cathedral bell tower (about 44 % of the adopted f_c) and 1.04 MPa for the Capua Cathedral bell tower (about 35 % of the adopted f_c). These results show that both towers are in elastic conditions, since in all parts of the tower the level of stresses was significantly smaller than the corresponding strength.

5.2 Nonlinear modelling

The nonlinear behaviour of the masonry, the choice of accurate modelling strategies and constitutive equations still represent one of the most challenging areas in the analysis of historical constructions (Lourenço 2002; Roca et al. 2010). The micro-modelling and meso-modelling are powerful tools for the analysis of small-scale structures (Lofti and Shing 1994; Lourenço and Rots 1997; Calderini and Lagomarsino 2006). However, the application of these strategies to larger-scale problems, such as the seismic analysis of historical constructions, does not seem to be suitable because too much input data are required and high computational effort is needed to analyse the detailed structural model. As an alternative, the macro-modelling technique (Gambarotta and Lagomarsino 1997; Lourenço et al. 1998; Schlegel 2004) based on the material homogenisation approach simplifies the implementation of the structural model and reduces the computational effort. Generally, smeared crack isotropic models are adopted for the constitutive law of masonry to assess the nonlinear behaviour (Luciano and Sacco 1998; Marfia and Sacco 2012). Nevertheless, the choice of this model, due to the great computational costs, is not effective when dealing with complex 3D FEMs. Efficient yield criteria such as the Mohr–Coulomb

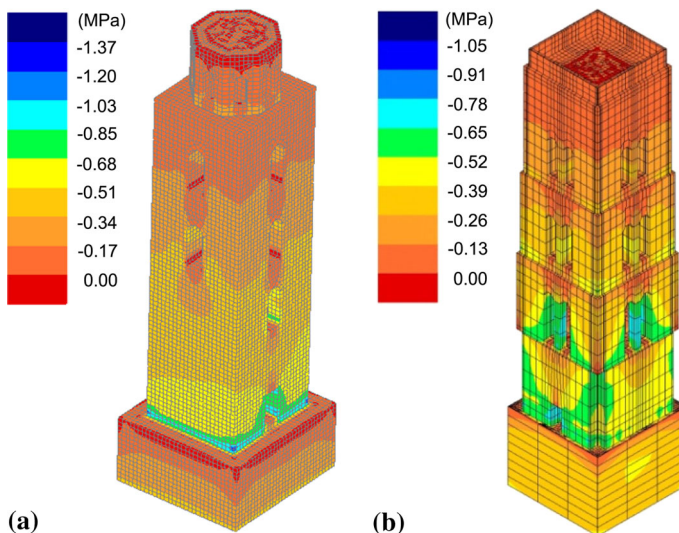


Fig. 14 Vertical stress map due to dead load: **a** Aversa Cathedral bell tower; **b** Capua Cathedral bell tower

and Drucker–Prager (DP) showed the capacity to reproduce the plasticity properties in a simple and reliable way. Their limitations are related to the inefficient modelling of cracking and crushing capabilities of masonry materials as well as their brittle behaviour in the softening range. An alternative to macro-modelling based on a homogenised continuum is the discrete element modelling of masonry structures as proposed in the literature (Casolo 2004; Orduña and Lourenço 2005a, b; Lemos 2007; Foraboschi and Vanin 2013).

In this paper, the perfectly elastic–plastic model based on the DP yield surface was applied in the 3D nonlinear finite element analysis of the masonry towers. This model was widely adopted in the literature to investigate the behaviour of masonry constructions. Zucchini and Lourenço (2007) used the DP criterion to reproduce the plastic deformation in masonry cells. Cerioni et al. (1995) adopted the DP model for the seismic vulnerability assessment of the Parma Cathedral bell tower. Betti and Vignoli (2011) combined the DP and the Willam–Warnke criteria for model crushing and cracking to assess the seismic vulnerability of a masonry church. D’Ambrisi et al. (2012) adopted the DP failure surface for nonlinear analysis of a historical masonry tower. In this paper, the material properties to define the model were introduced in such a way that the circular cone yield surface of the DP model corresponds to the outer vertex of the hexagonal Mohr–Coulomb yield surface. The parameters a and K of the yield DP surface are related to the cohesion (c) and the internal friction angle (ϕ), as follows:

$$\phi = \arcsin\left(\frac{3a\sqrt{3}}{2 + a\sqrt{3}}\right) \quad c = \frac{K\sqrt{3}(3 - \sin\phi)}{6 \cos\phi} \quad (5)$$

The identification of these material parameters for the DP plasticity model was carried out from the intersection between the DP cone and the principal stresses plane, which gives

$$a = \frac{f_t + f_c}{\sqrt{3}(f_c - f_t)} \quad K = \frac{2}{\sqrt{3}} \cdot \frac{f_t \cdot f_c}{(f_c - f_t)} \quad (6)$$

where f_t and f_c are yield stresses in uniaxial tension and compression, respectively. The nonlinear seismic analysis of the masonry towers was performed on the refined 3D model made of 3D solid elements using the computer code LUSAS (2012). In case of the Aversa Cathedral bell tower, the interaction between the tower and the neighbouring arch was neglected as the masonry arch was expected to have relatively low strength under severe seismic loads. In general, a masonry arch is capable of ductile behaviour when subjected to loads in the plane of the arch. On the contrary, under transverse loading due to seismic action, an arch structure may behave in a very brittle way. In this case study, the arch supporting the footbridge between the bell tower and the Cathedral shows limited width, about 2.0 m. Thus, its resistance against the out-of-plane actions is limited. Moreover, this arch is the only structural connection between the bell tower and the cathedral. Considering these causes, even low-intensity earthquakes can induce stresses in the arch, leading to a failure mechanism. Accordingly, the contribution of this masonry arch was neglected in the nonlinear seismic analysis.

5.3 Multi-modal nonlinear static analysis

The application of nonlinear static analysis to low-rise steel or reinforced concrete buildings was extensively studied in the literature and has long been introduced in codified seismic response evaluation procedures (ATC-40 1996; FEMA 440 2005; CEN 2004; NTC 2008; ASCE 2007). This success is due to the advantage of taking into account the

nonlinearities preserving at the same time the simplicity of the static analysis. In addition, this analysis provides an effective graphical representation of the structural capacity. Since 1997, several procedures were proposed to extend the pushover analysis to the evaluation of the inelastic torsional response of asymmetric buildings (Kilar and Fajfar 2001; Chopra and Goel 2004; Fajfar et al. 2005; Ferraioli 2015). First proposed for the application in the earthquake engineering field, this analysis method was recently widely adopted also to investigate the structural performance of buildings against progressive collapse (Ferraioli et al. 2014a). In the last few decades, several papers showed that these nonlinear static procedures provide acceptable predictions of the actual dynamic response of structures. However, some limitations to their application were highlighted. Thus, adaptive and multi-modal pushover procedures were proposed to overcome the most important limitations of traditional methods, especially to estimate seismic demands of high-rise and irregular buildings (Ferraioli et al. 2014b, c). Recently, several guidelines and recommendations extended the application of nonlinear static procedures to existing and monumental masonry structures (NTC 2008; DPCM 2011; CEN 2004). However, the application of conventional nonlinear static procedures to slender masonry towers involves some drawbacks, which need to be overcome. The major limit to the application of these procedures to masonry towers is that they are based on lumped-mass models with rigid floor diaphragms. In these models, the masonry towers show distributed stiffness and mass and poor diaphragm action. Moreover, the conventional nonlinear static procedures assume that the structure vibrates predominantly in a single mode, while in these slender structures, the effect of higher modes may become significant. This effect must be included in the analysis. Finally, the analysis does not consider the progressive changes in the modal properties due to structural yielding. This requires taking into account progressive damage accumulation by the adaptive pushover procedures. In this paper, the pushover analysis was performed considering a distributed mass model without effective rigid floor diaphragms. The body forces were applied throughout the volume of the tower according to the following load pattern:

$$F(x, y, z) = \sqrt{\sum_i [\Gamma_i \cdot \Phi_i(x, y, z) \cdot S_a(T_i) m(x, y, z)]^2} \quad (7)$$

where $m(x, y, z)$ and $\Phi_i(x, y, z)$ are the mass and i th mode shape varying with position (x, y, z) , T_i is the i th modal period, and $S_a(T_i)$ is the spectral acceleration computed from the 5 % damped elastic demand response spectrum (EDRS) of the Italian Building Code (NTC 2008). The square root of sum of squares modal combination includes a sufficient number of modes to obtain a combined modal mass of at least 90 % of the total mass. The so-called target or control node in the pushover analysis was assumed at the centre of mass of the roof. The load pattern given by Eq. 7 includes the effects of higher modes, but does not account for the changes in vibration characteristics due to progressive structural degradation. Consequently, the variations in the mode shapes, modal load distributions and spectral amplifications due to building damage were neglected. This hypothesis can lead to an unconservative overestimation of the seismic capacity of the tower, especially for significant and non-uniform damage. Nevertheless, this multi-modal pushover analysis can give effective information on the damage state without computationally demanding nonlinear dynamic analysis. The pushover analysis was performed loading the FEMs with the gravitational force and then applying the horizontal forces under monotonically increased top displacement control. The horizontal forces were applied in both X - and Y -directions, combining the modal load distributions of all the mode shapes of the structure with significant effective modal mass in the considered direction.

The nonlinear static pushover analysis allowed obtaining the capacity curve (base shear vs. roof displacement in the centre of mass of the roof). The pushover analysis was performed up to the collapse. The maximum principal strain (equivalent to tensile damage and cracks) was used as the controlling parameter to study the crack pattern evolution up to failure. To achieve this aim, the maximum tensile strain ϵ_t of 1 ‰ was considered during the analysis. The two main failure mechanisms of slender bell towers under earthquake loading are tensile cracking and compressive crushing. Horizontal cracking occurs at the tower's body due to bending behaviour, while masonry crushing occurs at the compressed toes. The masonry crushing at the compressed toes is due to the maximum compressive principal stress that reaches the value of the compressive strength. The collapse condition is defined when the structure shows extended cracking and crushing. The corresponding horizontal displacement of the tower top was considered as the ultimate condition for the tower. Figure 15 shows the damage (measured by the maximum principal strains) corresponding to the collapse condition during the pushover analysis of the towers. The horizontal load produces tensile cracking at the superstructure due to bending stress at the piers on the edges. The region most sensitive to cracking is located at the lowest part of the tower. When the principal stress equals the tensile strength, the cracks start to open and the maximum principal strain immediately begins to increase. When the horizontal load increases, the cracking spreads over the vertical direction. First, cracking starts in the foundation, then follows the parts close to the openings at the top of the tower. At the base of the tower, the masonry crushing occurs once the stress in the compressed toes exceeds the value of the masonry strength. When the horizontal load increases, this cracking expands gradually. In the compressed toes, the maximum tensile principal stress increases until it becomes equal to their tensile strength. This condition begins the opening of subvertical cracks oriented in the direction of the most compressed piers. At this step, the

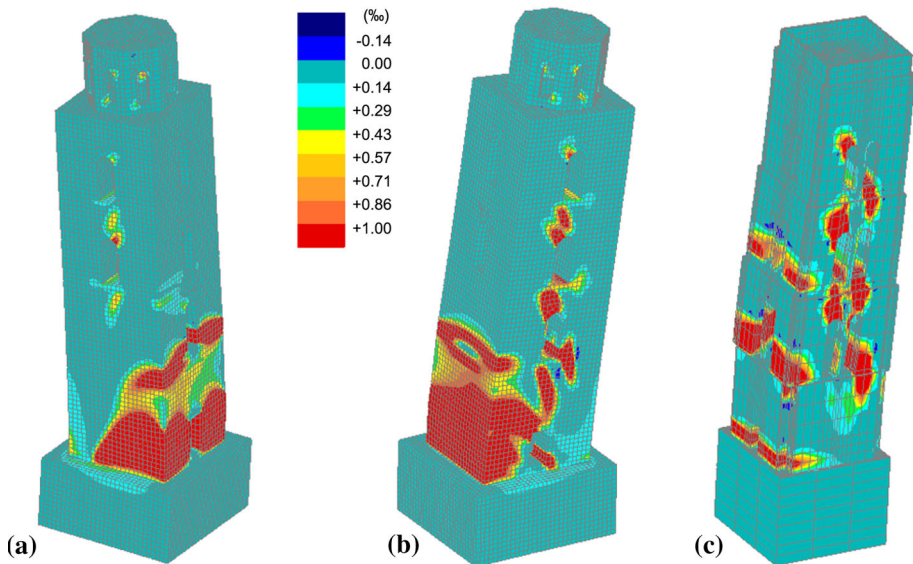


Fig. 15 Maximum principal strain: **a** Aversa Cathedral bell tower, pushover in *X*-direction; **b** Aversa Cathedral bell tower, pushover in *Y*-direction; **c** Capua Cathedral bell tower, pushover in *Y*-direction

collapse capacity of the structure was reached by failure at the compressed piers and the pushover analysis was stopped.

Considering the symmetry of the structure, the principal strain map of the Capua Cathedral bell tower is plotted only in relation to the pushover analysis in Y -direction. Results of nonlinear static analysis of the Aversa Cathedral bell tower show the presence of damaged zones concentrated in the bottom part of the tower and close to the openings. Similar results were obtained from pushover analysis in X -direction, with some differences related to the size and the openings position. Results of the analysis of the Capua Cathedral bell tower show tensile damage and cracks not only in the bottom part of the tower and close to the openings, but also in the bottom part of the upper levels, especially the first one. Figure 16 shows the pushover curves in both X - and Y -directions. Both towers show a brittle damage mechanism, mostly due to tensile failure at the foundation level. For the Aversa Cathedral bell tower, the pushover curves obtained are different for each direction because of the different size and position of openings. On the other hand, the pushover curves of the Capua Cathedral bell tower both in the positive and in the negative X - and Y -directions are identical due to the symmetry of the structure.

5.4 Seismic risk

The seismic performance evaluation was carried out with the CS method based on inelastic demand spectra originally proposed by Fajfar (1999) and then introduced in pre-standards reports and guidelines (NTC 2008; CEN 2004). The procedure is based on an idealised elasto-perfectly plastic force–displacement relationship and on an equivalent linearisation. Also in this case, the major limit to the application of this procedure is that it is based on lumped-mass models, typical of framed structures rather than of towers. To overcome this problem, as an alternative to storey forces applied to rigid floor diaphragms, the load pattern defined in Eq. 7 was applied throughout the volume of the towers. Moreover, it is necessary to extend and reformulate the conventional relations used to transform the global force–displacement capacity curve of the structure (base shear V vs. top displacement δ_{TOP}) in the CS (spectral acceleration S_a vs. spectral displacement S_d). In this study, the CS of the distributed mass model of the tower in acceleration displacement response spectra (ADRS) format was calculated as follows:

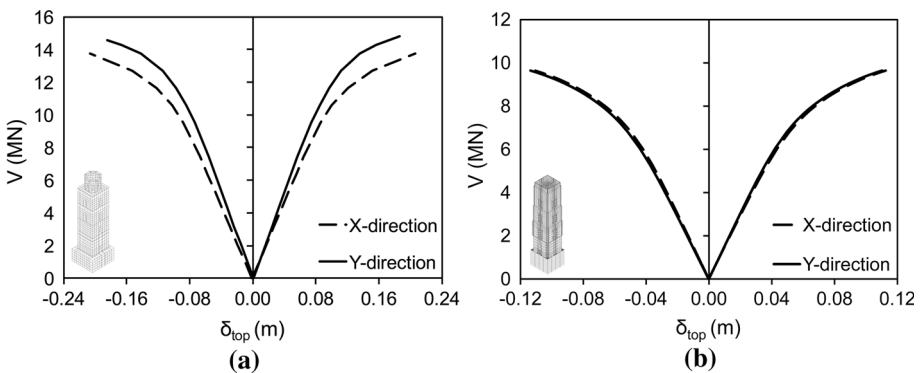


Fig. 16 Pushover curves in both X - and Y -directions: **a** Aversa Cathedral bell tower; **b** Capua Cathedral bell tower

$$S_a = V \frac{\int \mu(x, y, z) \cdot \delta^2(x, y, z) dx dy dz}{(\int \mu(x, y, z) \cdot \delta(x, y, z) dx dy dz)^2} \tag{8}$$

$$S_d = \frac{\delta_{TOP}}{\delta(x, y, z_{TOP})} \cdot \frac{\int \mu(x, y, z) \cdot \delta^2(x, y, z) dx dy dz}{\int \mu(x, y, z) \cdot \delta(x, y, z) dx dy dz} \tag{9}$$

where $\mu(x,y,z)$ is the mass density and $\delta(x,y,z)$ is the lateral displacement pattern. The performance point (PP) is defined by the intersection between the CS and the 5 %-damped inelastic demand response spectrum (IDRS). On the other hand, if the PP was determined by the collapse condition, the value of the peak ground acceleration (PGA) can be increased until the IDRS intersects the CS in the PP. Figures 17 and 18 show the interaction between CS and IDRS in the collapse condition in both X- and Y-directions, the bilinear capacity spectrum (BCS) and the elastic demand response spectrum (EDRS). The lowest value of PGA leading to collapse represents the actual capacity of the tower for the life safe limit state (LSLS). This value is defined as capacity peak ground acceleration PGA_{LS} . The seismic demand is represented by the reference PGA on type A ground for the LSLS provided by the Italian Building Code (NTC 2008). The results for three common limit states are listed in Table 7: serviceability limit state (SLS), damage limit state (DLS) and LSLS. The safety index (α_{LS}) was defined by the capacity/demand quotient in terms of PGA with a value of α_{LS} equal to unity, meaning a complete fulfilment of code requirements. In Table 8, the capacity and safety index of the towers are presented. Despite differences in size and position of openings and, consequently, in pushover curves, the values of safety index of the Aversa Cathedral bell tower in X- and Y-directions are very close. On the other hand, as an effect of the symmetry of the structure, the results from the Capua Cathedral bell tower both in the positive and in the negative X- and Y-directions are identical. The α_{LS} results greater than unity for both towers are evidence of their ability to resist the design seismic action. This result is mainly due to the moderate seismicity of the area. Moreover, it should be noted that the safety index is strongly dependent on the structural compatibility of the obtained displacements in the pushover analysis. Consequently, more effective assessments would require deeper experimental investigations to identify the characteristic parameters of the nonlinear behaviour of masonry that strongly influence the failure displacement defining the collapse condition.

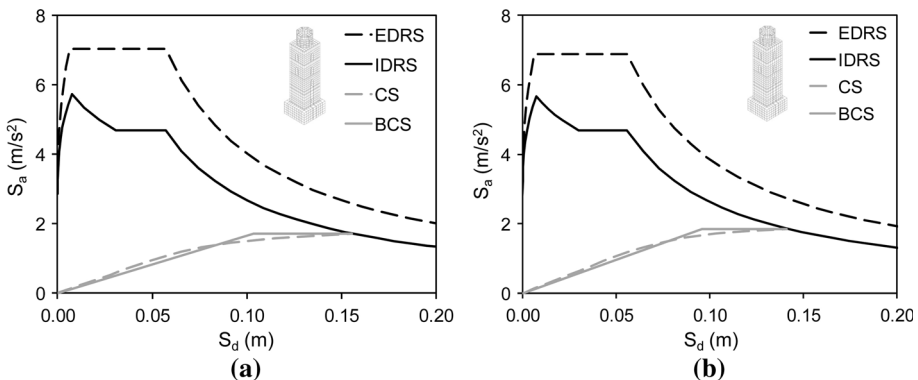


Fig. 17 Capacity spectrum method: **a** X-direction; **b** Y-direction. Aversa Cathedral bell tower

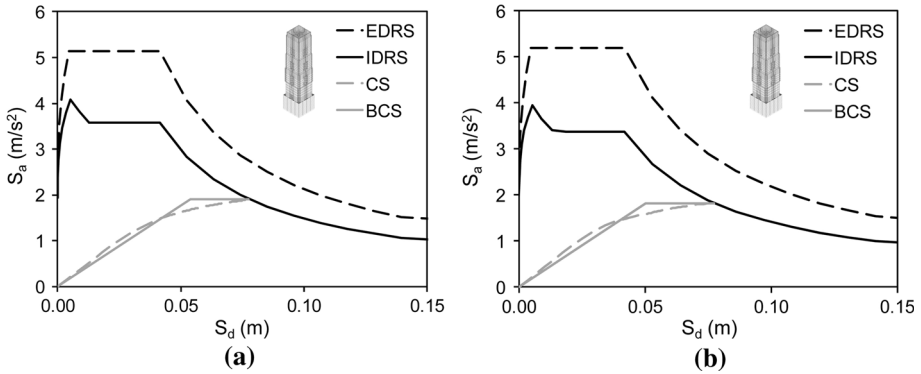


Fig. 18 Capacity spectrum method: **a** X-direction; **b** Y-direction. Capua Cathedral bell tower

Table 7 Parameters of elastic demand response spectra

Parameter	Aversa Cathedral bell tower			Capua Cathedral bell tower		
	SLS	DLS	LSLS	SLS	DLS	LSLS
P_{VR} (-)	0.81	0.63	0.10	0.81	0.63	0.10
T_R (years)	30	50	475	30	50	475
PGA (g)	0.042	0.055	0.136	0.042	0.052	0.113
F_0 (-)	2.379	2.353	2.455	2.418	2.405	2.579
T_C (s)	0.285	0.318	0.368	0.285	0.322	0.434

Table 8 Capacity peak ground acceleration and safety index

Direction	Aversa Cathedral bell tower		Capua Cathedral bell tower	
	PGA_{LS}	α_{LS}	PGA_{LS}	α_{LS}
X+	0.194	1.43	0.165	1.46
X-	0.192	1.41	0.165	1.46
Y+	0.190	1.40	0.167	1.48
Y-	0.189	1.39	0.167	1.48

6 Conclusions

The investigations based on ambient vibrations and numerical analysis performed to assess the seismic safety of two masonry bell towers was presented in this paper. The proposed vibration-based characterisation procedure is based on an iterative approach that simultaneously handles the uncertainties associated with historic buildings (support conditions, mechanical characteristics of materials, soil–structure interaction and constraining effect of the neighbouring structures) until an acceptable correspondence was achieved between

some specific dynamic properties obtained from the AVTs and the FEM. A good agreement between measured and predicted modal parameters was reached by means of the updating procedure, which, through a systematic calibration, provides a reliable tool to investigate the dynamic features of towers without performing destructive tests. Following the tuning procedure, the FEM was used in subsequent investigations to assess the seismic vulnerability of the towers. To achieve this aim, the macro-modelling technique based on the material homogenisation approach and the DP yield criterion was applied to reproduce the plasticity properties of the 3D FEM of the tower. To overcome the limitation due to inefficient modelling of crack, the maximum principal strain (equivalent to tensile damage and cracking) was used as control parameter instead of simulation cracking and crushing capabilities of masonry. The nonlinear static analysis confirmed the susceptibility of masonry towers to extensive damage under seismic loads. However, the towers presented here are located in an area of moderate seismicity and, consequently, the capacity/demand quotient in terms of PGA (i.e. the safety index) resulted, in any case, greater than one. Despite the lack of knowledge and the uncertainties related to parameters influencing the structural behaviour of the towers, the numerical FEM provided useful predictions on the incidence of a collapse mechanism. However, more effective structural assessments of the damage state under seismic loads require additional experimental investigations aimed at characterising the nonlinear mechanical behaviour of masonry buildings.

Acknowledgments The authors wish to express their gratitude to Mr. Salvatore Froncillo, Mr. Luigi Aruta and Mr. Alessandro Vari for their important support in the on-site measurements carried out on both towers.

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