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Seismic Assessment of Four Historical Masonry Towers in Southern Italy

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Keywords
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On-site surveys,
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ABSTRACT

The paper synthesizes the main results of investigations devoted to the evaluation of the seismic performance of four historical masonry towers. On-site diagnostic investigations were carried out using non-destructive or slightly destructive tests, including geometric surveys, laser scanning, endoscopic tests, sonic pulse velocity tests, geognostic surveys, flat-jack tests, environmental vibration tests, dynamic tests on tie-rods. The results of the on-site surveys were employed to calibrate refined 3D finite element models of the towers accounting for the materials' mechanical parameters, restraint of the neighboring constructions, and effect of soil-structure interaction. The FEM model was usefully employed to assess the seismic risk of the towers based on the Italian Guidelines. To this aim, the nonlinear FEM global analysis was developed using the pushover technique for the estimation of the seismic safety of the towers.

1. INTRODUCTION

Many ancient masonry structures are vulnerable to lateral inertial forces due to seismic events. The poor ductility of the masonry, combined with other elements of weakness such as irregular geometry or slenderness, may increase the risk of severe damages or collapse even under moderate earthquake ground motions. The lessons learned from earthquakes in recent decades indicate the vulnerability of the historical constructions and the need to ensure the seismic safety of cultural heritage. This has given rise to a new generation of international guidelines and codes [Eurocode 8, 2004; FEMA 356, 2000; ASCE/SEI 31-03, 2003; FEMA 547, 2006; ASCE/SEI 41-06, 2007; DPCM, 2011; ASCE/SEI 41-13, 2014] that include specific procedures to evaluate the seismic safety of historical constructions. Moreover, the present circumstance stimulated the advancement in models and analysis methods for this type of structure. The tall monumental buildings, for example, towers and mosque minarets, don't have sufficient structural capacity to resist to seismic actions. Some of the major deficiencies relate to geotechnical problems, intrinsic structural defects, degradation of materials, buckling of slender members, and sensitivity to dynamic loading. Many studies in the literature focused on the damage evaluation and seismic assessment of historical constructions, with particular concern to ancient masonry towers (Lagomarsino &. Cattari, 2015; Valente & Milani, 2016; Fragonara *et al.*, 2017; Bartoli *et al.*, 2017). A large number of studies developed ambient vibration tests (AVTs) on old buildings and suitable experimental techniques for their dynamic identification (De Sortis *et al.*, 2005; Gentile *et al.*, 2015; Ferraioli *et al.*, 2018). Bayraktar *et al.* (2009) presented a study on the dynamic identification and model updating of the Hagia Sophia bell tower in Turkey. Russo *et al.* (2010) performed an experimental campaign aimed at the evaluation of the performances of the bell tower of Saint Andrea in Venice. Tomaszewska *et al.* (2012) presented the dynamic identification of the Vistula Mounting tower. D'Ambrisi *et al.* (2012) used the ambient vibration tests to identify first the dynamic modal parameters and then the material properties of the civic tower of Soncino in Cremona (Italy). Gentile *et al.* (2015) proposed a vibration-based procedure to calibrate the finite element model of the bell tower of the Church Collegiata in Arcisate (Varese, Italy). Preciado (2015) focused on towers and other slender masonry structures developing a seismic vulnerability assessment method. Casolo *et al.*

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(2013) carried out a comprehensive study where ten masonry towers in the coastal Po Valley (Italy) were analyzed, and their seismic vulnerability was evaluated and discussed. On the other side, many studies focused on the mechanical model of masonry that should be able to characterize all the aspects of its complex non-linear behavior (Bernardeschi *et al.*, 2004; Carpinteri *et al.*, 2005; Peña *et al.*, 2010, Milani *et al.*, 2012). This paper synthetically reports the experience of the extended experimental and theoretical campaign dedicated to four historical masonry towers in Southern Italy. The experimental campaign included geometric surveys, laser scanning, endoscopic tests, flat-jack tests, sonic pulse velocity tests, environmental vibration tests, geognostic surveys, and dynamic tests on the tie-rods. The ultimate goal of these activities was to address several uncertainties related to the geometry and mechanical properties of materials that, along with soil stratigraphy and site seismicity, address a critical part of the seismic evaluation of these structures. The on-site investigation survey was the basis to calibrate refined nonlinear finite element models that were employed to evaluate the seismic safety of the towers using the capacity spectrum method (Fajfar, 1999) based on the inelastic demand spectra.

2. INVESTIGATIONS, ON-SITE SURVEYS, and TESTS

This paper presents the results of a comprehensive study carried out on four historical masonry towers in Campania, Italy (Fig. 1): 1) the bell tower of the Cathedral of Aversa; 2) the bell tower of the Cathedral of Capua; 3) the bell tower of the Church of Assumption in Santa Maria a Vico; 4) the bell tower of Torre Orsaia (Cilento National Park in Southern Italy).

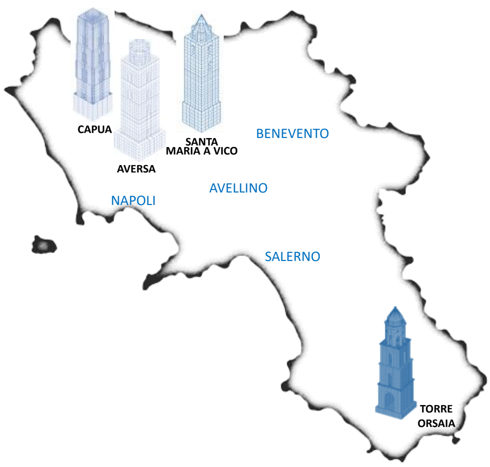


Figure 1. The geographical location of the towers

2.1. The Bell Tower of Aversa

The bell tower of the Cathedral of Aversa (Fig. 2), originally built between 1053 and 1080 next to the Lombard church, was rebuilt in 1499 after the earthquake of 1457. The tower is 45.5 m tall and its cross-section is a square of side 14 m. Four massive masonry piers on the edge of the square cross-section are coupled by spandrels with masonry arch above the openings. The horizontal structures are composed of masonry vaults for the first floor, and wooden structures for the upper floors. However, they have low in-plane stiffness and, therefore, they are not able to guarantee an adequate strong constraint linking the four masonry piers. The tower has been the subject of investigations, on-site surveys, and tests including geometrical material and survey of the crack pattern, chemical tests, single flat-jack tests, and monotonic compressive tests (Fig. 3). Finally, full-scale ambient vibration tests (Fig. 4) were carried out allowing the identification of the modal parameters of the tower (Ferraioli *et al.*, 2017). The foundations have not been the subject of any investigation. The underlying soil is bedrock made of compact Campanian grey tuff.

2.2. The Bell Tower of Capua

The bell tower of the Cathedral of Capua (Fig. 5), originally built in 856, was rebuilt in the 12th century after the earthquake of 990. The tower is 41 m tall and its cross-section is a square of side 11.3 m. The first two levels are composed of limestone blocks removed from antique constructions, while the upper part of the structure is composed of clay bricks and Campanian tuff blocks. The soil under the construction is composed mainly of pyroclastic sedimentary rocks that are typical of the volcanic Phlegrean area. The ambient vibration tests (Fig. 6) were carried out to measure the modal parameters (i.e. mode shapes and natural frequencies). This information was usefully applied to identify all the unknown parameters of the finite element model.

2.3. The Bell Tower of Santa Maria a Vico;

The bell tower of the Church of Assumption in Santa Maria a Vico (Fig. 7), originally built during the 15th century, was retrofitted in 1749 after the earthquake of 1732. The tower is 40 m tall and its cross-section is a square of side 8.3 m. The vertical load-bearing structures are made of tuff masonry with a variable thickness along with the height. The horizontal structures are masonry vaults for the first three levels and a wooden floor strengthened with a concrete slab for the fourth level. Horizontal tie bars are inserted in both horizontal directions at 27, 30 m, and 33 m from the ground level. The soil stratigraphy is composed of two layers. The first layer is made of pyroclastic loose rocks. The second layer is made of Campanian ignimbrite. The modal parameters were identified using ambient vibration tests under traffic and wind loading (Fig. 8) (Ferraioli *et al.*, 2018).

2.4. The Bell Tower of Torre Orsaia

The bell tower of the church of San Lorenzo Martire in Torre Orsaia is located in the Cilento National Park (Southern Italy). The building was originally a three-story fortress in the Lombard age. In 1576 the overall height was increased to about 30 m by adding two more orders with an octagonal plan and a conical roof (Fig. 9). The vertical load-bearing structures are made of local sandstone. The structure of the octagonal belfry has an internal structure made of local bricks and stones covered with a cement-based plaster.

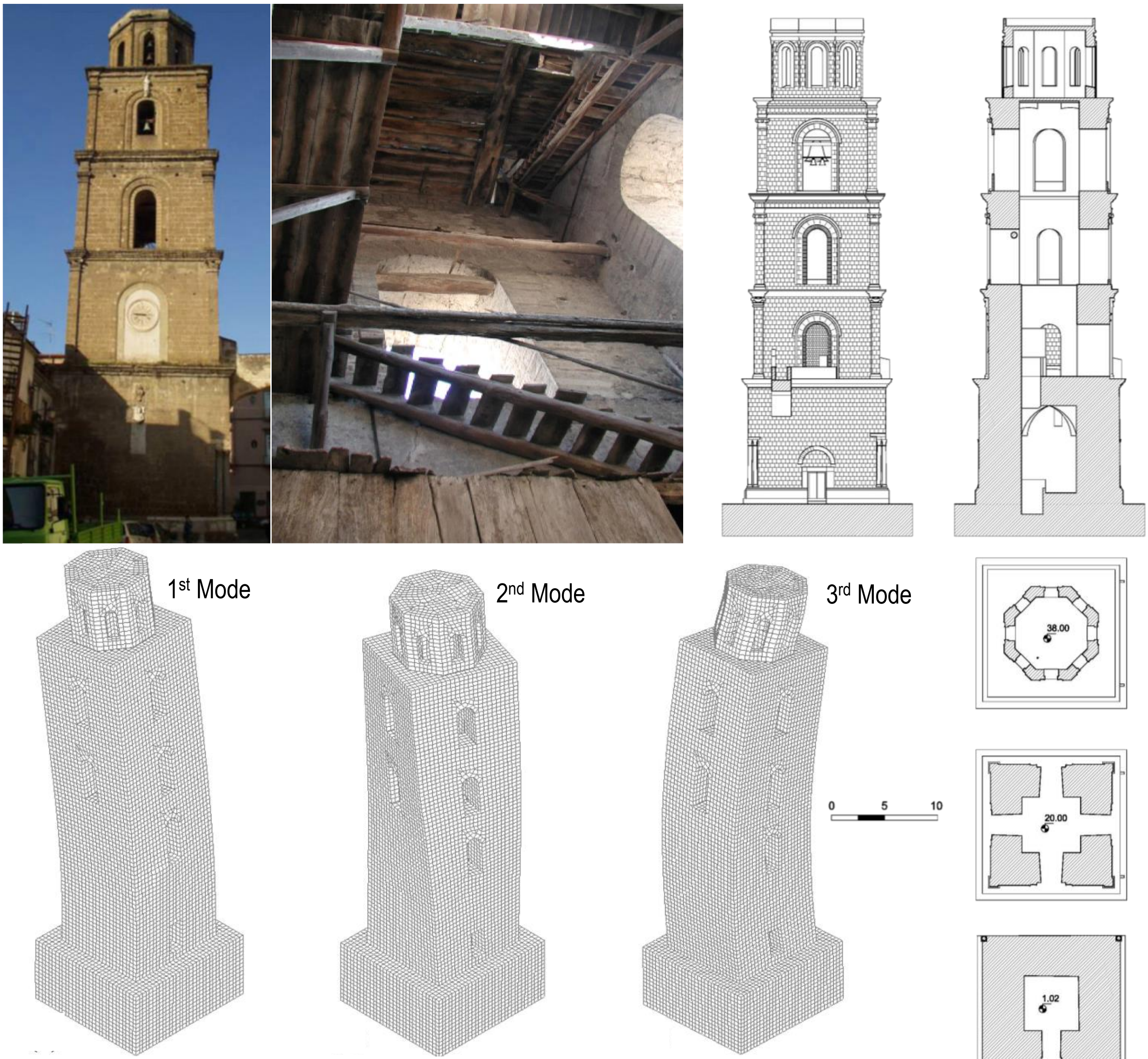
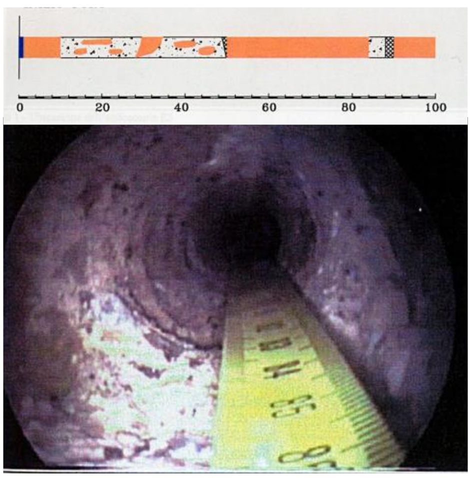
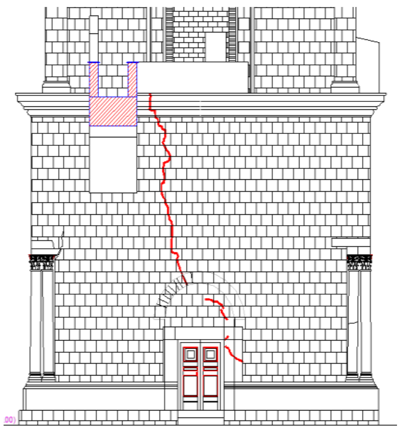


Figure 2. The bell tower of the Cathedral of Aversa: general view, internal view, façade, sections, mode shapes



Endoscopic test



Survey of the crack patterns

Specimen	1	2	3
Description	yellow tuff with grey pumices	yellow tuff (lithoid facies)	yellow tuff with grey pumices
Compressive Strength [MPa]	4.23	2.85	3.46
Tangent Modulus of Elasticity [GPa]	3.99	3.66	1.92
Secant Modulus of Elasticity [GPa]	3.82	3.33	2.08

Flat-jack tests

Figure 3. The bell tower of the Cathedral of Aversa: endoscopic tests, surveys of the crack pattern, flat-jack tests

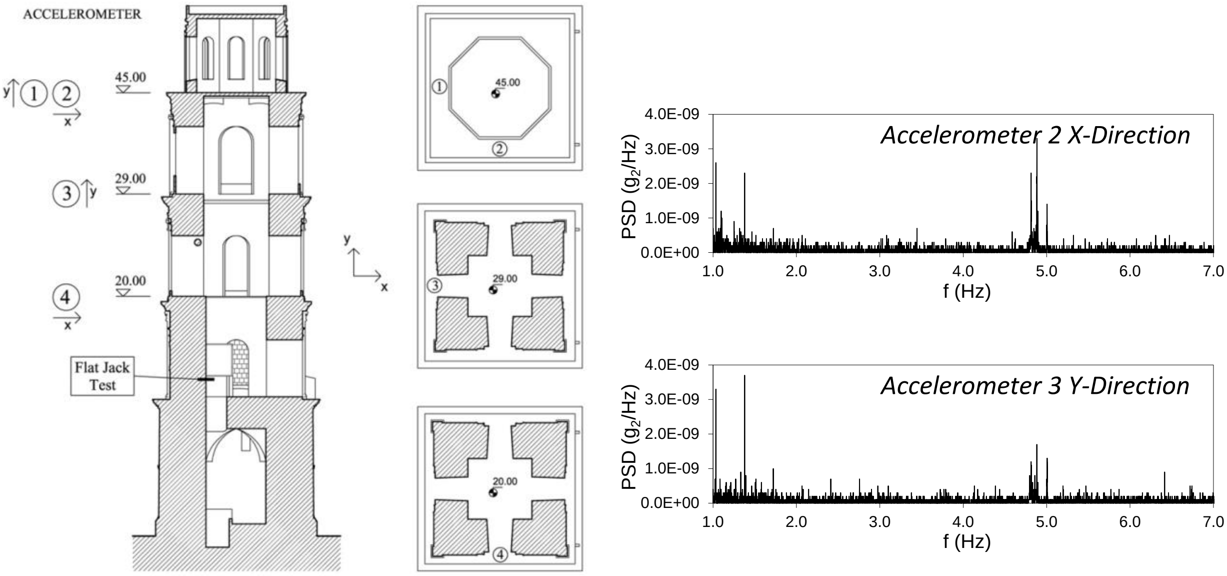


Figure 4. The bell tower of the Cathedral of Aversa: ambient vibration tests

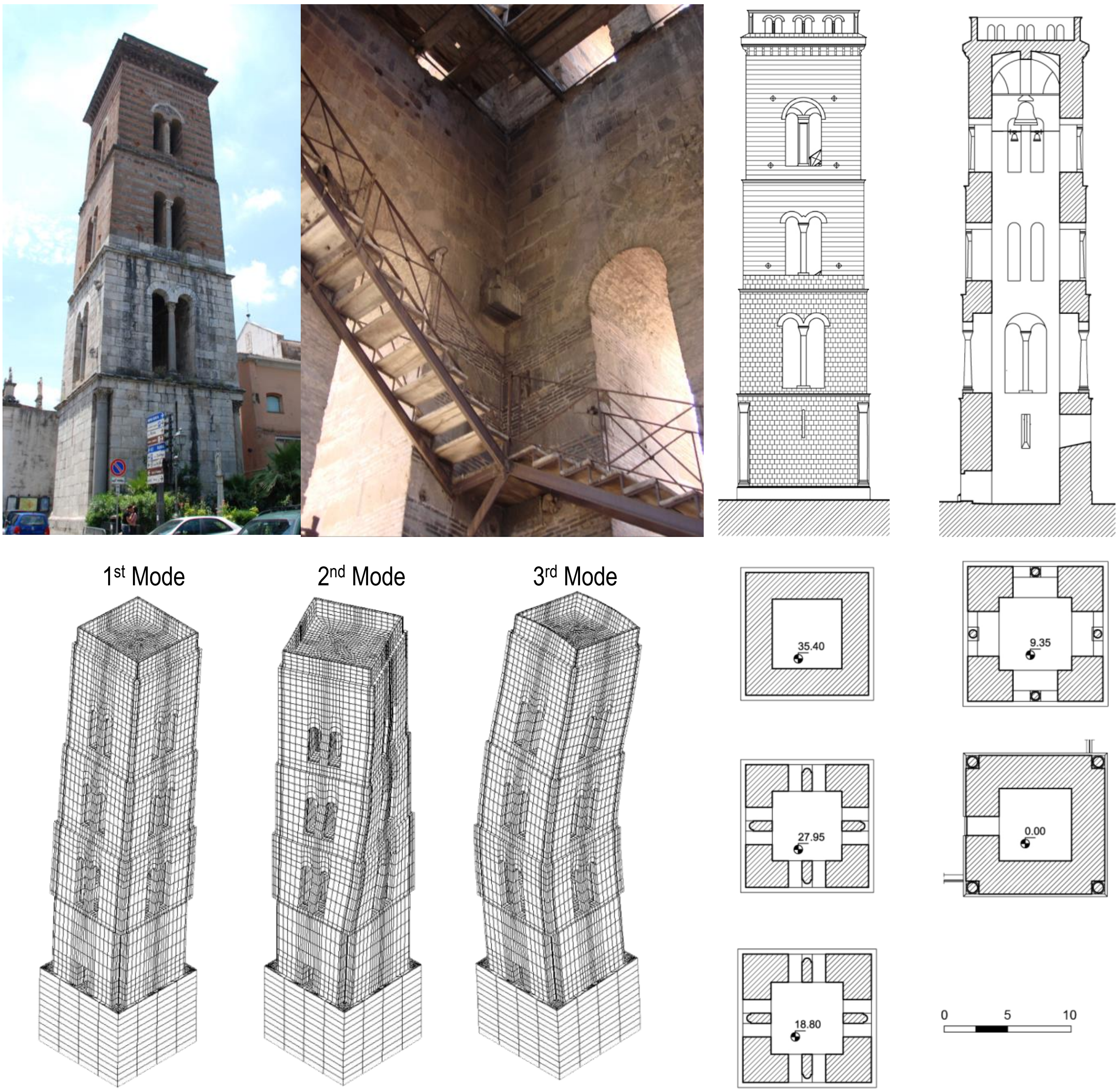


Figure 5. The bell tower of the Cathedral of Capua: general view, internal view, façade, sections, mode shapes

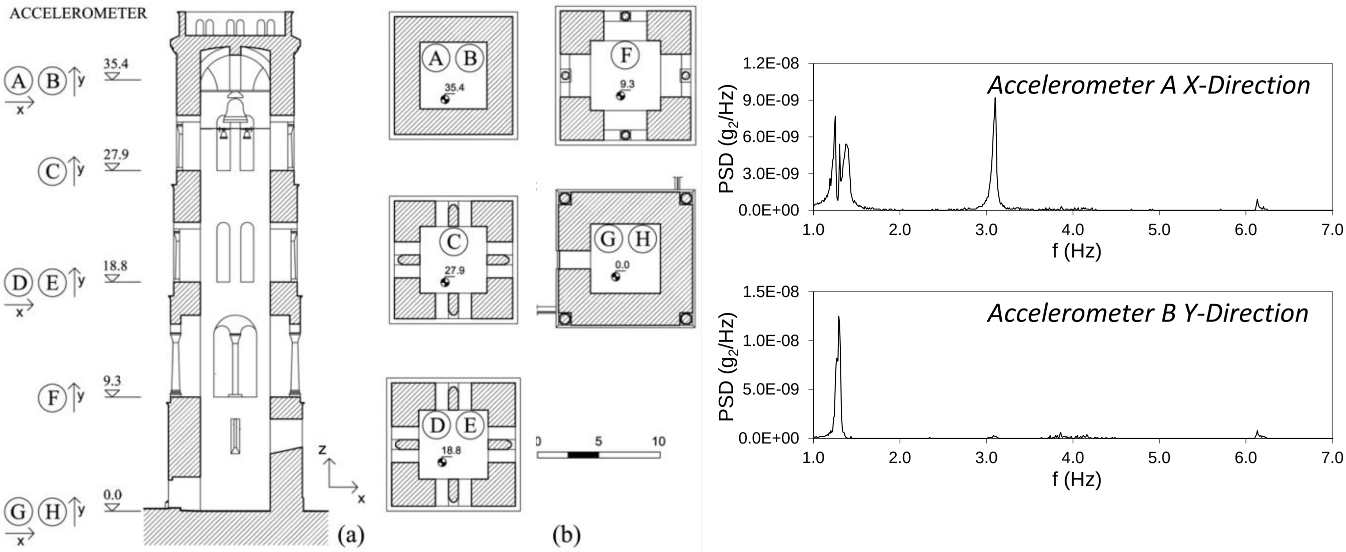


Figure 6. The bell tower of the Cathedral of Capua: ambient vibration tests

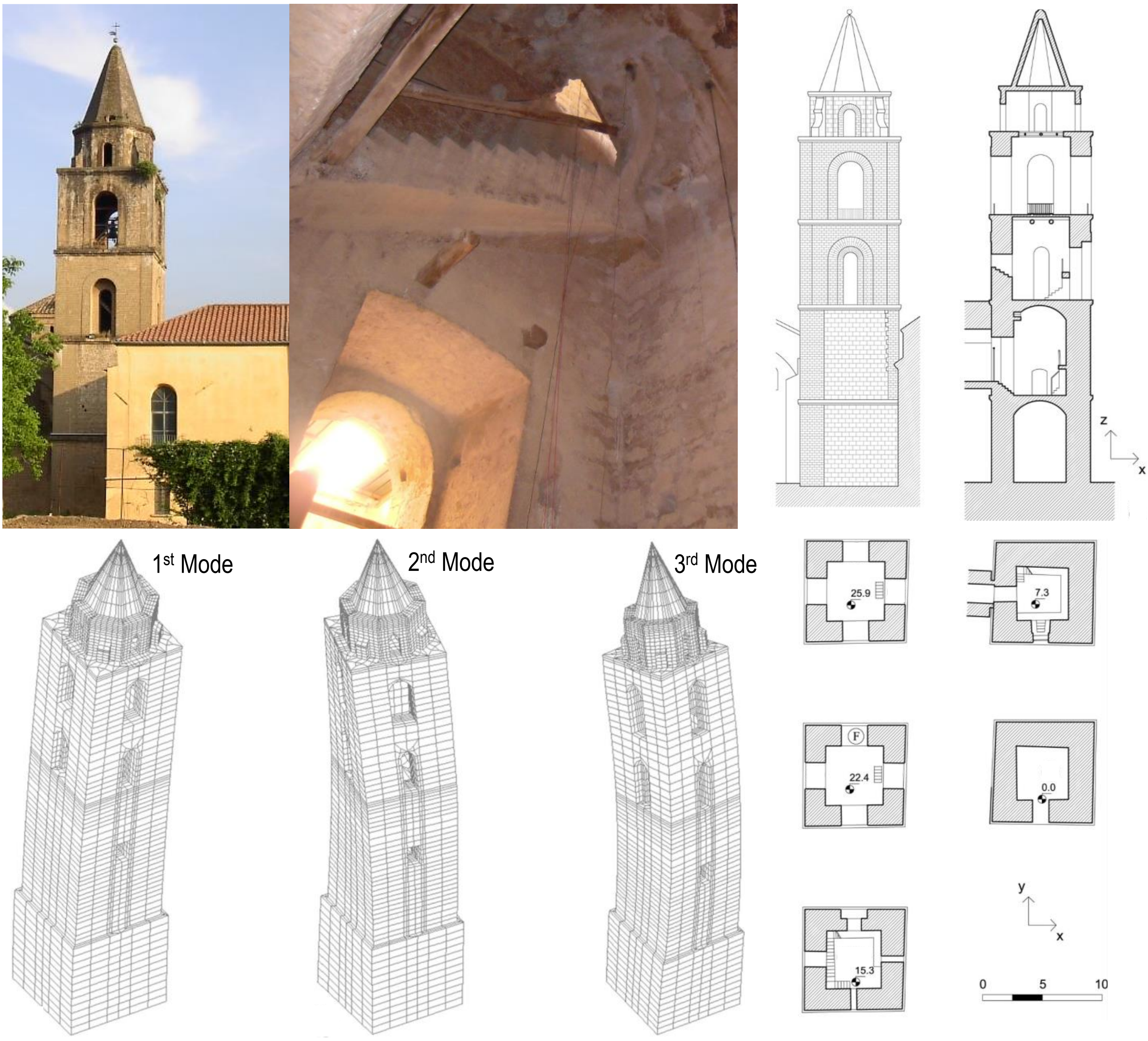


Figure 7. The bell tower of Santa Maria a Vico: general view, internal view, façade, sections, mode shapes

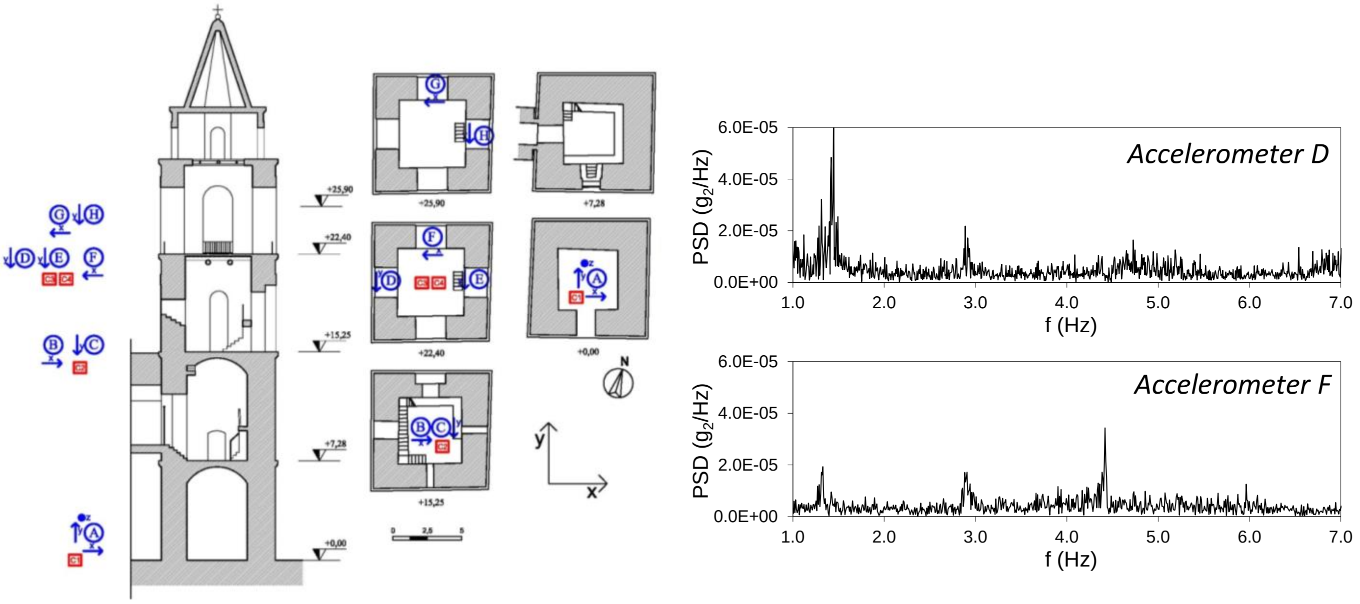


Figure 8. The bell tower of Santa Maria a Vico: ambient vibration tests

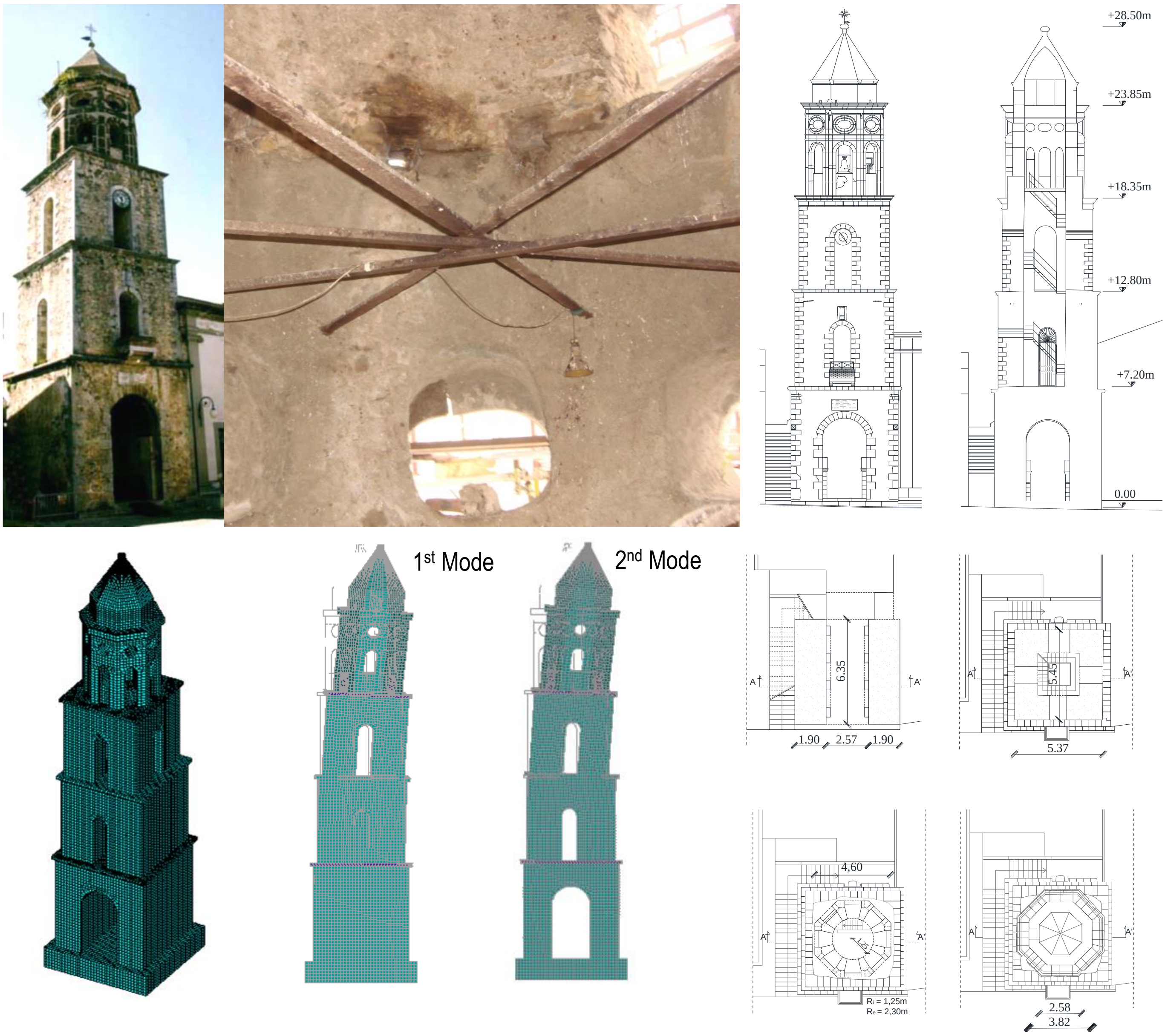


Figure 9. The bell tower of Torre Orsaia: general view, internal view, façade, sections, mode shapes

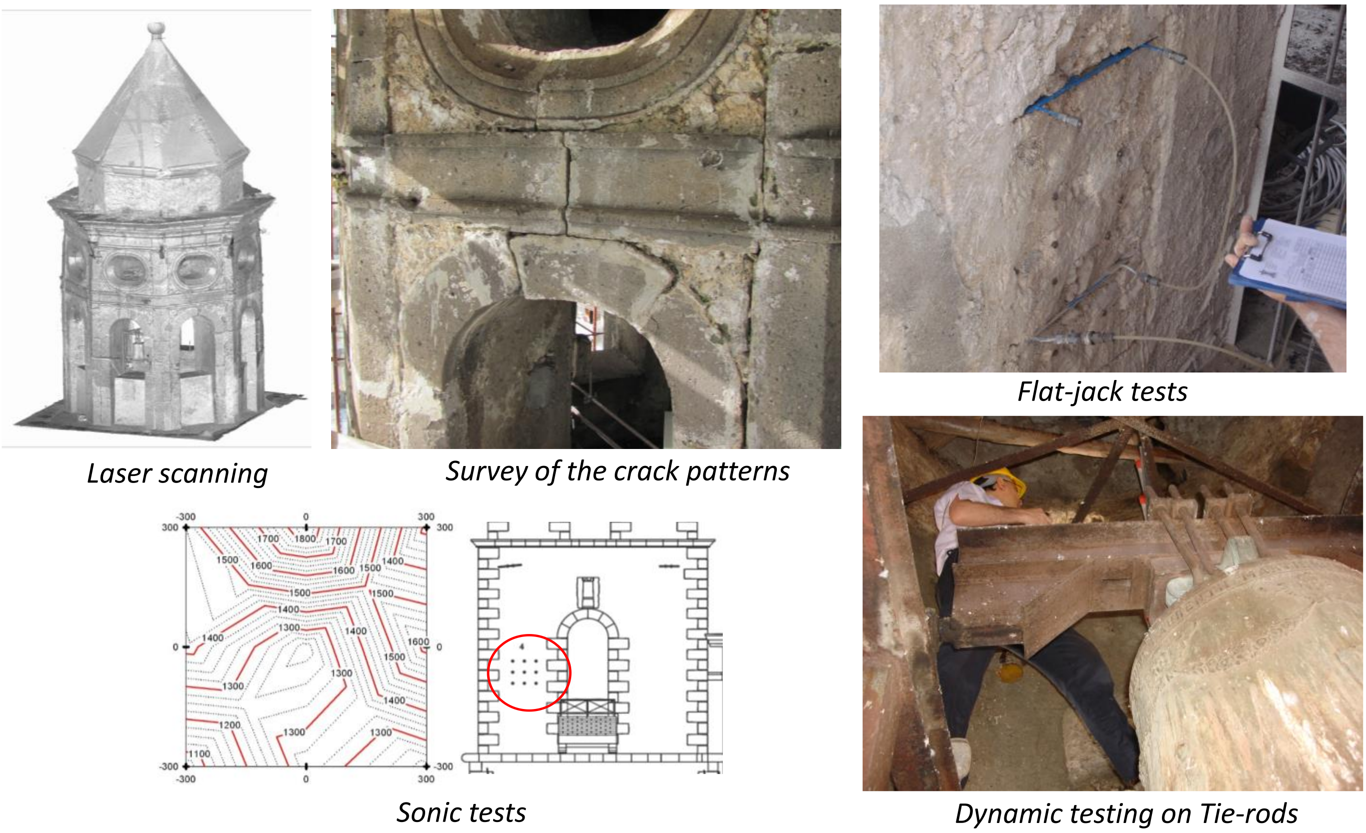


Figure 10. The bell tower of Torre Orsaia: laser scanning, crack patterns, flat-jack tests, sonic tests, dynamic testing on tie-rods

The windows, openings, and octagonal belfry are covered with squared grey tuff blocks. Horizontal iron tie-rods are placed to keep the walls from flexing outward. Investigations, on-site surveys, and tests were carried out on the tower including, geometrical survey, laser scanning survey, endoscopic tests, surveys of the crack patterns, sonic pulse velocity tests, flat jack tests, geognostic surveys, environmental vibration tests (Fig.10) (Ferraioli *et al.*, 2020)

3. MODELING and SEISMIC ASSESSMENT

3.1. FEM Tuning

The seismic behavior of masonry towers largely depends on their geometric properties and, particularly, their slenderness (i.e., height to base length ratio), the percentage of openings particularly in slender belfries, and the width of the perimeter walls. Moreover, it should be observed that the historical masonry towers are part of the urban area. Therefore, the constraints of the neighboring structures considerably affect the seismic behavior of the towers as well as their vulnerability. Another important aspect is the soil conditions that may play a key role, especially in the case of soil-structure interaction effects in soft soils. The modal parameters (i.e. mode shapes and natural frequencies) assessed via the environmental vibration tests were used to identify the unknown parameters of the numerical model. To update the initial model, the modal analysis was carried out and its results were matched to the experimental results to adjust the material properties, the geometry, and interaction with the adjacent buildings. The soil-structure interaction was modeled using linear elastic springs uniformly distributed in both vertical and horizontal directions. An iterative approach was followed and as a result, the values of all the unknown parameters (i.e., mechanical properties of materials, constants of the springs modeling the soil-structure interaction, constants of the springs modeling the constraints of the adjacent buildings). This iterative process was continued until a good match was observed between theoretical and experimental results.

3.2. Nonlinear Modeling

The seismic performance and collapse mechanism of the masonry towers when subjected to earthquake loading are considerably affected by the materials and construction techniques used. Moreover, the compressive stress of masonry due to the dead load, as well as the connection between the structural members may play an important role. Finally, the complex nonlinear behavior of masonry and its deterioration under cyclic loading should be accounted for. Often the compressive strength of masonry cannot be determined exhaustively through on-site destructive tests. Therefore, the data reported in the literature may be used along with the data that also complies with those available from other masonry towers similar for their main geometrical and structural characteristics. In this paper, the 3D nonlinear finite element analysis was carried out using the macro-modeling approach based on material homogenization. A model extensively applied in the literature was used, i.e., the perfectly elastic-plastic model based on the Drucker–Prager (DP) yield surface. The material properties of the model were determined by making sure that the circular cone yield surface of the Drucker–Prager model corresponds to the outer vertex of the hexagonal Mohr–Coulomb yield surface (Ferraioli *et al.*, 2017, 2018, 2020). The compressive strength of masonry was calculated as the mean value from the flat-jack tests if available. The tensile strength was considered as 1/15 of the compressive strength. The friction angle and cohesion were calculated based on the Italian Code (NTC-Guidelines, 2018; NTC-Instructions, 2019). The computer code LUSAS (2012) was used to perform the nonlinear analyses of the towers of Aversa, Capua, and Santa Maria a Vico, while ANSYS (2001) was applied for the tower of Torre Orsaia. The lateral restraint offered by the adjacent building was treated conservatively. Practically, it was considered in cases where the connection of the tower to the adjacent building is compressed, while it is neglected otherwise.

3.3. Seismic assessment

The seismic safety of the towers was evaluated based on the Italian Guidelines (2011) according to the level of analysis LV3 (global finite element analysis). According to the pushover approach provided by the Italian Code (NTC-Guidelines, 2018) two lateral load patterns (i.e., uniform distribution and first mode distribution) were considered in the nonlinear static analysis. This gives the capacity curve that plots the base shear as a function of top displacement. The last point of the curve corresponds to the structural collapse of the structure that is when occurring extensive cracking and crushing. In this case, a visible softening of the pushover curve should be observed. Even if this softening does not occur, the Italian Guidelines (2011) allow defining the ultimate limit state displacement in the range of 3–6 times the elastic displacement of the equivalent SDOF system. Moreover, the ductility ratio (μ) equals the behavior factor (q) based on the hypothesis of equality of the maximum displacements. Thus, the conservative choice of μ=q=2 was made centered in the range of 1.5–2.5 provided by Eurocode 8 (2004) for the behavior factor of unreinforced masonry buildings. The target displacement at the performance point was calculated using the capacity spectrum method (CSM) based on the inelastic demand response spectra originally proposed by Fajfar (1999) and then introduced in Eurocode 8 (2004) and Italian Code (NTC-Instructions, 2019). Fig. 11 compares capacity and demand in the ADRS (Acceleration-Displacement Response Spectrum) format. With increasing the peak ground acceleration also the target displacement increases. Thus, the peak ground acceleration may be amplified until the target displacement equals the life safety (LS) displacement. In this situation, the Inelastic Demand Response Spectrum (IDRS) intersects the Bilinear Capacity Spectrum (BCS) in the Life Safety performance point. The minimum value obtained from all the nonlinear static analyses is a measure of the actual capacity of the tower for the life safety (LS) limit state (i.e., capacity peak acceleration a_{LS}). The seismic demand is given by the reference peak ground acceleration (a_g) of the Italian Code (NTC-Guidelines, 2018) for the life safety (LS) limit state. Tables 1-4 show the seismic parameters of the elastic design response spectra of the site of each tower. The capacity to demand ratio gives the safety index (α_{LS}) in terms of peak ground acceleration (Table 5). The results show that only for the bell tower of Torre Orsaia the safety factor is lower than 1 indicating that seismic retrofit is required in this case study.

Table 1. Parameters of elastic design response spectra Bell Tower of Aversa

Parameter	S	DL	LS
P_{VR} (-)	0.81	0.63	0.10
T_R (yrs)	30	50	475
a_g (g)	0.042	0.055	0.116
F_o (-)	2.379	2.353	2.455
T_C (sec)	0.285	0.318	0.368

Table 2. Parameters of elastic design response spectra Bell Tower of Capua

Parameter	S	DL	LS
P_{VR} (-)	0.81	0.63	0.10
T_R (yrs)	30	50	475
a_g (g)	0.042	0.052	0.113
F_o (-)	2.418	2.405	2.579
T_C (sec)	0.285	0.322	0.434

Table 3. Parameters of elastic design response spectra Bell Tower of Santa Maria a Vico

Parameter	S	DL	LS
P_{VR} (-)	0.81	0.63	0.10
T_R (yrs)	30	50	475
a_g (g)	0.050	0.064	0.166
F_o (-)	2.340	2.355	2.424
T_C (sec)	0.286	0.313	0.372

Table 4. Parameters of elastic design response spectra Bell Tower of Torre Orsaia

Parameter	S	DL	LS
P_{VR} (-)	0.81	0.63	0.10
T_R (yrs)	30	50	475
a_g (g)	0.037	0.047	0.116
F_o (-)	2.446	2.443	2.521
T_C (sec)	0.281	0.324	0.447

Table 5. Capacity peak ground acceleration - safety index

Tower	X-Direction		Y-Direction	
	a_{LS}(g)	α_{LS}	a_{LS}(g)	α_{LS}
Aversa	0.192	1.41	0.189	1.39
Capua	0.165	1.46	0.167	1.48
S. Maria a Vico	0.187	1.13	0.187	1.13
Torre Orsaia	0.127	0.79	0.146	0.91

4. CONCLUSIONS

Many uncertainties lie ahead about the historic constructions including the mechanical properties of materials, foundation structures, soil-structure interaction, restraint of adjacent buildings, and so on.

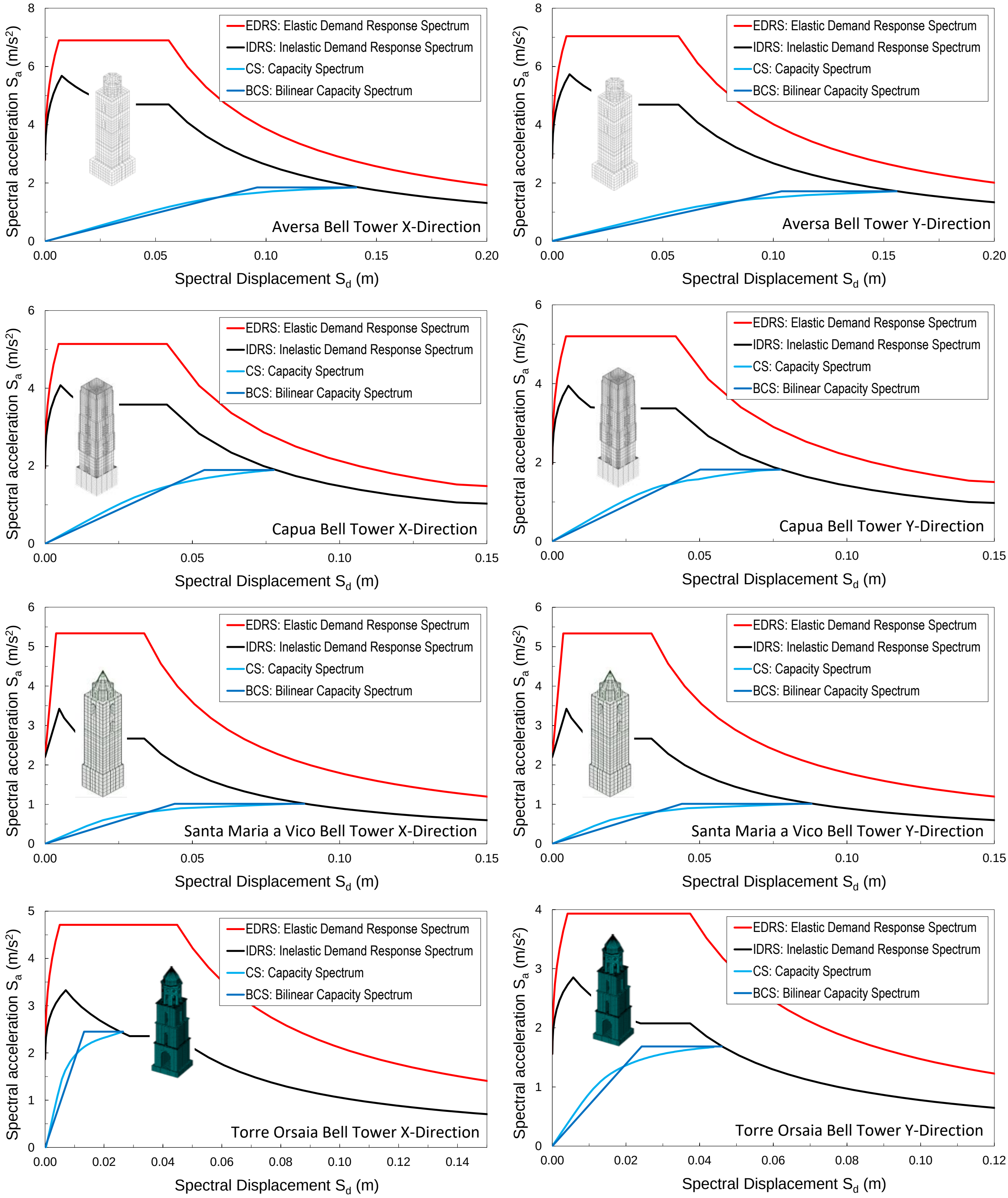


Figure 11. Comparison between capacity and demand in the acceleration–displacement response spectrum plane

This paper summarizes the experience gained in the development and implementation of investigations, on-site surveys, and tests on four historical masonry towers in Southern Italy. The experimental campaign included geometric surveys, laser scanning, endoscopic tests, sonic pulse velocity tests, geognostic surveys, flat-jack tests, ambient vibration tests, and dynamic tests on tie-rods. This experimental campaign made it possible to implement an accurate nonlinear model that was then used for global finite element analysis. The subsequent theoretical investigations were conducted to investigate the seismic vulnerability of the towers. The results showed that only the bell tower of Torre Orsaia exhibited a safety factor lower than one (i.e., structure to be considered unsafe). This is due to the choice of the most accurate level of analysis according to Italian Guidelines (2011) (i.e., level of analysis LV3) that allowed to obtain more effective and less conservative results.

Author contributions

Massimiliano Ferraioli: Study conception and design, Methodology, Analysis and interpretation of results, Writing-Reviewing and Editing.
Donato Abruzzese: Study conception and design, Methodology, Data curation, Investigation, Software, Validation

Conflicts of interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper

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