Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2025) 2835-2846

ECCOMAS
Proceedia

COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

MULTILEVEL SEISMIC VULNERABILITY ASSESSMENT OF AN ITALIAN MEDIEVAL BELL TOWER

Maddalena Della Pietra[1], Mattia Zizi[1], Corrado Chisari[1] and Gianfranco De Matteis[1]

[1] Department of Architecture and Industrial Design, University of Campania “Luigi Vanvitelli”
Via San Lorenzo, 81031, Aversa (CE), Italy
{maddalena.dellapietra, mattia.zizi, corrado.chisari, gianfranco.dematteis}@unicampania.it

Abstract

Masonry bell towers represent a widely diffused architectural typology and, consequently, a valuable part of our historical and cultural heritage. These structures are highly vulnerable to seismic events due to their slender form, structural irregularities, and properties of masonry. Ensuring their seismic safety is therefore essential, as earthquakes can lead to significant damage or even collapse, posing risks to both the towers themselves and their surrounding areas. This study aims to assess the seismic vulnerability of a significant case study: the Medieval bell tower of San Michele Arcangelo in Casertavecchia (Caserta, Italy). The seismic assessment was conducted using a multi-level approach encompassing three levels of evaluation, in compliance with Italian Guidelines for the assessment and mitigation of seismic risks to cultural heritage. A simplified mechanic-based approach (Level 1) was first followed, based on a cantilever schematization of the tower and considering bending failures at each relevant level. Then, limit analyses with a static kinematic approach were developed by considering recurrent and possible collapse mechanisms (Level 2). Finally, a refined Finite Element Model (FEM) was employed to perform nonlinear static analyses and estimate the tower capacity under seismic loads (Level 3). Results from each level were compared to the local seismic demand to determine the tower's safety level and identify possible retrofitting measures.

Keywords: Masonry bell tower, Multilevel seismic vulnerability assessment, Simplified mechanics-based approach. Limit analysis, Non-linear static analysis, Seismic risk assessment.

ISSN:2623-3347 © 2025 The Authors. Published by Eccomas Proceedia.
Peer-review under responsibility of the organizing committee of COMPDYN 2025.
doi: 10.7712/120125.12613.25276

1 INTRODUCTION

Masonry bell towers are among the most widespread architectural features in Italy, keepers of local identity. However, their widespread presence hides an intrinsic fragility due to their peculiar structural characteristics. Their slenderness, the rigidity of their masonry structures and the discontinuities between construction elements make them particularly vulnerable to earthquakes.

Recent earthquakes in Italy, such as those in L'Aquila (2009), Emilia-Romagna (2012), and Central Italy (2016–2017), have exposed the vulnerability of historic churches and their bell towers. Post-seismic analyses reveal recurring collapse mechanisms, mainly due to weakened structural connections and material fragility [1-3].

In this context, the bell tower of the Church of San Michele Arcangelo in Casertavecchia represents an ideal case study for further investigation into the seismic vulnerability of historic structures. This study stems from a desire to deepen the knowledge of the tower, building upon previous research such as [4, 5], and further unveiling the complexity of its structural and historical evolution. The tower is distinguished by its architectural configuration and the transformations it has undergone over the centuries, which have modified both its appearance and structural behaviour. The assessment of its seismic vulnerability was conducted using a multi-level approach, in accordance with the methodologies suggested by the Italian Guidelines for the assessment and mitigation of seismic risk of cultural heritage [6]. Starting from a historical and geometrical analysis of the monument, the work develops through different levels of seismic assessment, incorporating simplified methods, kinematic analyses and finite element modeling.

2 SAN MICHELE ARCANGELO BELL TOWER

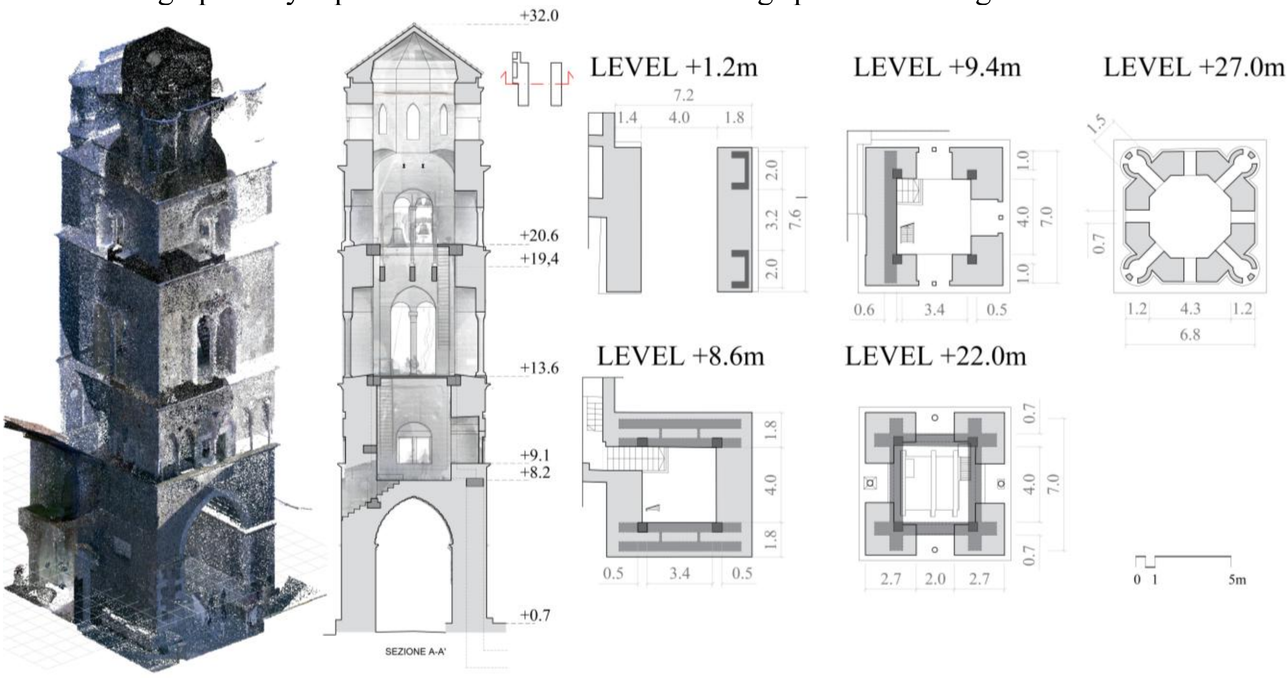
2.1 Historical construction notes

The bell tower of the Church of San Michele Arcangelo, located in the medieval village of Casertavecchia, is an extraordinary example of Italian Romanesque architecture. The tower was built in the 12th century, approximately 80 years after the church. The integrative intervention is evident at the connection with the right nave, where structural discontinuities are visible in the access passage carved into the original masonry, approximately at the 8.6 m height, as shown in Figure 1. Originally, the tower was covered by a 7-meter-high pyramidal roof but, following lightning damage in the 18th century, it was demolished and later replaced with a simpler tiled roof [7]. Further modifications were made in the 19th century, including the addition of rectangular windows on the second level and the installation of iron ties, likely introduced to overcome pre-existing structural instabilities [8]. In the 20th century, the bell tower underwent the most significant and invasive interventions in its history. Notably, the 1955 restoration had a decisive impact on the structure, as it involved the installation of concealed reinforced concrete curbs within the masonry and the reconstruction of deteriorated masonry sections. Due to the 1980 earthquake, the tower suffered severe damage, the masonry showed mortar loss and a general detachment of ashlars. In response, the disconnected blocks were removed and repositioned, and steel bars were installed through the masonry thickness to reconnect detached blocks. This complex historical evolution suggests that simplified assessment methods, especially those neglecting the presence of reinforced concrete elements, may lead to inaccurate evaluations of the tower's actual behaviour.

2.2 Geometrical features and material characterization

The bell tower in question is located on the right side of the Cathedral and rises to a height of approximately 30 meters with a square base measuring 7.6 meters on each side. The structure is divided into five levels and features a slight tapering in the upper two levels, where the wall thickness decreases from 1.8 meters at the base to 1.3 meters. The base of the tower is traversed by a large pointed arch, allowing the *via dell' Annunziata* to pass beneath it. The bell tower entrance is located inside the church at a height of 5.36 metres, above an existing small chapel; inside, movement between levels is only possible via iron ladder rungs.

Rectangular openings and an intertwined arch motif, characteristic of Campanian Romanesque architecture, appear on the second level and reoccur on the top level of the belfry, reinforcing the structure’s stylistic identity. The third and fourth level are characterized by bifora windows on all sides, while above it, the structure transitions into an octagonal plan (2.2 meters wide), enriched by small cylindrical turrets at the corners and culminating in a pyramidal roof. These geometric characteristics are graphically represented in the technical drawings provided in Figure 1.



(a) (b) (c)

Figure 1. Geometrical survey of the tower: (a) point cloud representation, (b) vertical section, (c) plan views.

The bell tower of the Church of San Michele Arcangelo was built using a regional variety of grey tuff. The entire wall structure exhibits a regular texture, consisting of square-cut tuff blocks with thin mortar joints. In the first four meters of height, at the corners, tuff blocks are combined with limestone ashlars, likely used to reinforce the vertical supports. Additionally, marble elements are present to a lesser extent, mainly employed in decorative features such as cornices, brackets, and portals [9, 10]. The mechanical properties of the bell tower's materials were previously investigated and documented in [5]. The investigations provided compressive strength values for the tuff units ranging between 2.66 and 2.73 MPa. Also, penetrometer tests on mortar returned low penetration resistance, further corroborating the scarce mechanical characteristics of the masonry. Thus, for the subsequent analyses, the minimum value of the compressive strength proposed by Italian code [11] for tuff masonry was adopted (i.e. 2 MPa). As for masonry density, ρ=1100 kg/m³ was assumed, as resulted from the tests.

Given the extensive investigation performed and the good level of knowledge achieved, a confidence factor FC=1 was assumed.

3 SEISMIC VULNERABILITY ASSESSMENT

3.1 Methodology

The seismic vulnerability analysis was performed using the data acquired during the knowledge phase of the bell tower. An initial qualitative evaluation of the seismic risk was carried out through the application of the Evaluation Level 1 (EL1) methodology, aimed at identifying preliminary global capacity in terms of overall strength and seismic safety index.

Following in-situ surveys and the historical analysis of the structure, it was possible to reconstruct the cracking pattern, which highlighted the need for a detailed investigation of local collapse mechanisms and out-of-plane instabilities. A Level 2 (EL2) approach was subsequently employed, enabling the identification of these local failure mechanisms and the assessment of their associated vulnerabilities.

The complex stratification of interventions carried out over time on the structure suggested the adoption of a more advanced approach, corresponding to Level 3 (EL3). This phase required the use of finite element modeling (FEM), made possible by detailed knowledge of the mechanical properties of materials and construction phases. The FEM analysis allowed for an accurate simulation of the structural behaviour under seismic actions, providing a realistic representation of internal stress and damage distribution and potential failure modes.

Finally, the integration of the three levels of analysis facilitated a global seismic vulnerability assessment, identifying structural weaknesses and offering insights for potential future intervention strategies.

3.2 EL1 Assessment

The EL1 analysis evaluates the seismic response of the tower based on several simplified hypotheses: conservation of plane sections, masonry with no tensile strength and a compressive stress distribution modelled as a stress block. Although preliminary, this methodology is important for establishing intervention priorities.

The analysis was carried out using the software program *PREVENTSoft*, originally developed to support the research project "*PREVENT – Integrated Procedure for Assessing and Improving the Resilience of Existing Masonry Bell Towers on a Territorial Scale*"[12, 13].

Specifically, through the development of a parametric model of the bell tower and the input of data related to seismic hazard and construction material properties, the software enables the automated calculation of the seismic safety index, in compliance with the current regulatory framework.

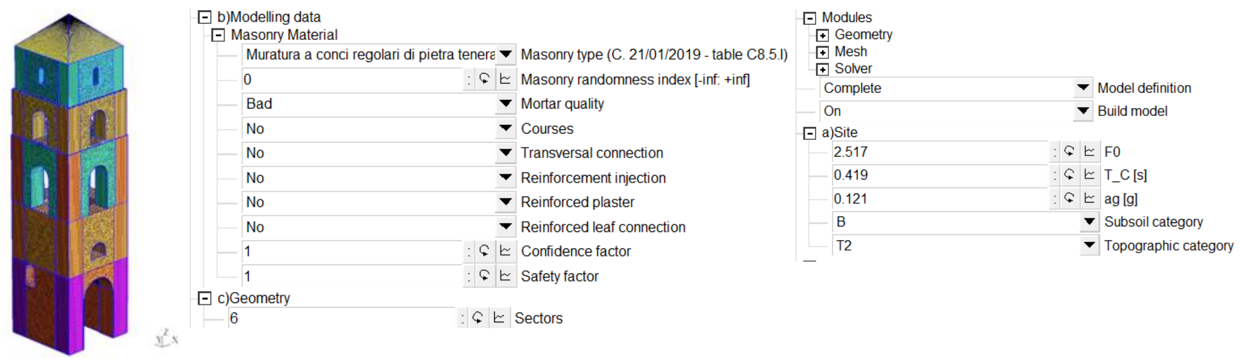


Figure 2. PreventSoft interface and site input parameters.

The geographical coordinates of the site were considered, a nominal reference life of 50 years was assumed, and a use coefficient of 1.5 (Use Class III) was adopted, as defined by the Italian NTC2018 [14]. Based on these inputs, the parameters shown in Table 1 were obtained: the return period at Life Safety Limit State ($T_{R,SLV}$), the maximum horizontal peak ground acceleration (a_{SLV}), the maximum spectral amplification factor ($F_{0,SLV}$), and the characteristic period ($T_{C,SLV}$*). The subsoil and topographic categories were determined from geotechnical and morphological data.

$T_{R,SLV}$ [years]	$a_{g,SLV}$ [g]	$F_{0,SLV}$ [-]	$T_{C,SLV}$* [s]	Subsoil category	Topographic category
712	0.137	2.549	0.429	B	T2

Table 1. Seismic parameters based on the geographical coordinates, nominal life values and coefficient of use.

Although the software can estimate the fundamental period of the structure, the values obtained from previous modal analyses (1.95 Hz and 2.03 Hz) [5] were used instead to calculate a_{SLV}, as they provide a more accurate representation of the tower's dynamic behaviour.

A compression-bending verification was then performed, i.e. the structure is treated as a cantilever subjected to a system of horizontal forces. Failure may occur at a generic section due to crushing in the compressed zone. The tower was divided into six sectors, corresponding to the different levels and the verifications were performed at each section transition and in both principal directions, considering the roof as an additional load.

The seismic vulnerability of the tower within the EL1 analysis was assessed by comparing the structural capacity, expressed in terms of the collapse peak ground acceleration (a_{SLV}), evaluated from spectral acceleration $S_{e.SLV}$, with the design seismic demand ($a_{g,SLV}$). To account for the limited dissipative capacity of masonry structures, the seismic demand was appropriately reduced using a behavior factor q, assumed as 1.5 in accordance with Chapter C8.7.1.2.1.1 of Italian Standards [11].

The results of the EL1 analysis are presented in Table 2 for six vertical sectors of the tower and along both principal directions (N–S and E–W). The parameters are defined as follows:

- z is the elevation of the section;
- M_u is the overturning moment capacity of the section;
- a_{SLV} is the collapse acceleration, i.e., the horizontal acceleration that would trigger failure at the considered section;
- $a_{g,SLV}$ is the peak ground acceleration corresponding to the SLV;
- $f_{a,SLV}$ is the ratio between a_{SLV} and $a_{g,SLV}$.

As shown in Table 2, all $f_{a,SLV}$ values exceed 1.0 across the six evaluated sections and in both principal directions (N–S and E–W), ranging from approximately 1.27 at the base to over 16.4 at the top. The lowest values are found in the first sector, where the combination of higher mass and bending effects results in the most critical conditions. Nevertheless, even in the lower section, the safety index remains above unity, suggesting that the structure satisfies the seismic demand.

Sector	Direction	z [m]	M_u [kNm]	$S_{e.SLV}$ [m/s^2]	a_{SLV} [m/s^2]	$f_{a,SLV}$ [-]
1	N - S (x)	0.00	36883.95	0.26	0.17	1.27
	E - W (y)		40595.20	0.28	0.19	1.39
2	N - S (x)	7.67	31340.09	0.36	0.25	1.79
	E - W (y)		34021.57	0.39	0.27	1.94
3	N - S (x)	13.27	20442.56	0.41	0.28	2.01
	E - W (y)		20442.56	0.41	0.28	2.01
4	N - S (x)	19.87	10369.10	0.58	0.39	2.85
	E - W (y)		10369.10	0.58	0.39	2.85
5	N - S (x)	24.37	4283.81	0.93	0.63	4.63
	E - W (y)		4283.81	0.93	0.63	4.63
6	N - S (x)	28.57	298.51	2.25	1.52	16.39
	E - W (y)		298.51	2.25	1.52	16.39

Table 2. EL1 results in terms of acceleration factors f_a.

3.3 EL2 Assessment

In order to identify potential collapse mechanisms in a structure, it is necessary to conduct a deep analysis that considers both pre-existing damage conditions and the behaviour observed in similar buildings subjected to seismic actions. Data from past earthquakes provide essential insights into structural vulnerabilities and help formulate reliable hypotheses about possible failure models [6]. For bell towers, seismic behaviour is influenced by multiple factors, including the structural slenderness, the type of connection between walls, the presence of adjacent buildings, and the arrangement of slender elements at the top. In the specific case of the San Michele Arcangelo bell tower, direct inspection revealed a low level of cracking.

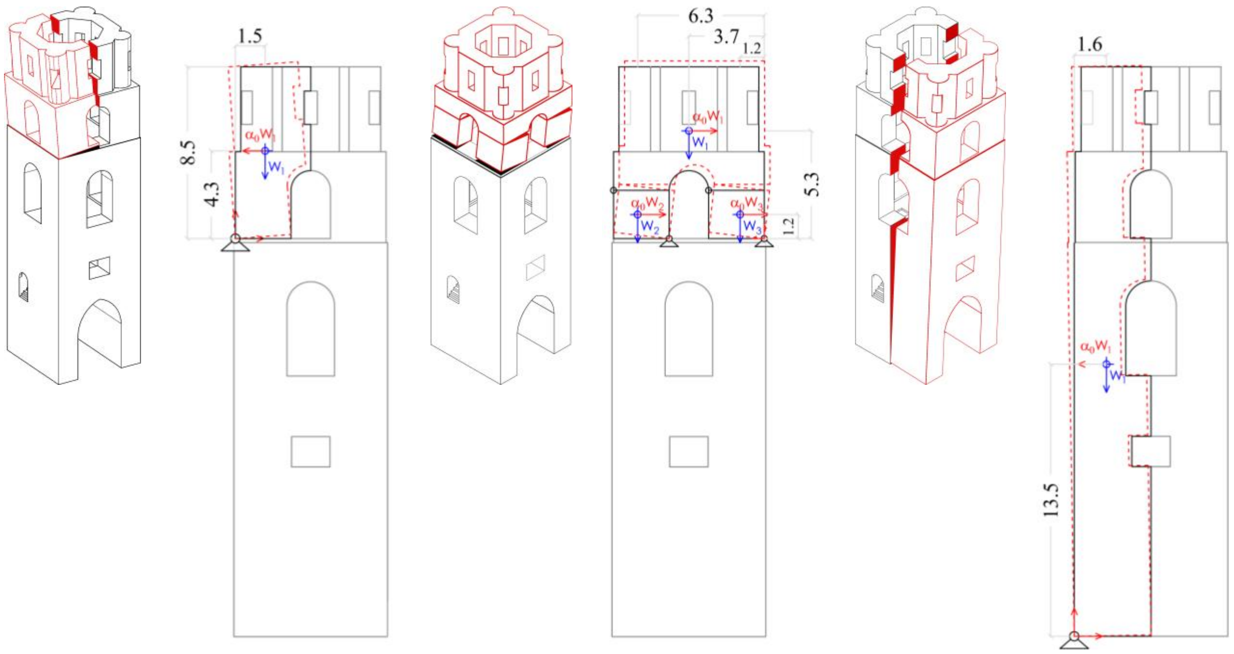
A significant crack, oriented perpendicular to the axis of the basement vault, was observed during the on-site survey. This damage, along with a significant stiffness variation at a height of approximately 20 meters, due to the transition from the lower section supported by a reinforced concrete frame to the upper masonry portion, suggests the presence of potential structural discontinuities.

Based on a combined analysis of the crack pattern and the common failure modes observed, three main local collapse mechanisms were examined:

a) Overturning of the upper portion of the structure, with the formation of a hinge line at the stiffness transition occurring at the end of the reinforced concrete frame (Figure 3a);
b) Simultaneous rotation of the belfry piers, with the development of plastic hinges at the ends (Figure 3b);
c) Overturning of an entire portion of the west side of the structure, along the existing cracks and the preferential paths provided by the openings (Figure 3c).

The linear kinematic analysis is based on the evaluation of the horizontal action capable of triggering the mechanism. The verification consists of comparing the acceleration required to activate the mechanism with the maximum ground acceleration corresponding to the SLV, appropriately reduced by the behaviour factor (q) assumed as 1.5 [6].

For each kinematic mechanism, it is necessary to define the collapse multiplier (α_0), which represents the seismic input required to trigger overturning. In this specific case, the verification of the three mechanisms was carried out considering only the self-weights (W_k) of the blocks, applied at their centroid, and a system of horizontal seismic forces (αW_k) proportional to these weights.



(a) (b) (c)

Figure 3. The analysed collapse mechanisms for EL2: (a) overturning of the upper portion; (b) rotation of the belfry piers and (c) overturning of the west side portion.

The results of the EL2 assessment are summarized in the Table 3. For each mechanism, the table presents:

- $\frac{S_e(T_1)\psi(Z)\gamma}{q}$ seismic demand calibrated considering amplification effects due to the height Z of the collapse mechanism, through the first mode shape $\psi(Z)$ and the modal participation factor γ;
- a_0* the spectral acceleration;

- $f_{a,SLV}$ the acceleration factor.

Mechanism	$\frac{S_e(T_1)\psi(Z)\gamma}{q}$	a_0*	$f_{a,SLV}$
a	2.00	3.32	1.66
b	2.87	10.04	3.50
c	1.08	1.16	1.08

Table 3. Structural check of collapse mechanisms relevant to EL2 procedure.

EL2 returned positive values of acceleration factors, confirming the safety of the tower under the design-level earthquake. The most critical collapse mechanism was the third one ($f_{a,SLV}$ = 1.08), characterized by the formation of a rotational hinge at the base of the tower. It is worth noting that the presence of the embedded reinforced concrete frame could potentially inhibit the activation of this failure mode, a condition that was not considered in the assessment. Therefore, the analysis was conducted under highly conservative assumptions.

As for the other collapse mechanisms, the overturning of the upper portion of the structure (first mechanism) resulted in lower acceleration factors compared to the typical belfry collapse (second mechanism). However, in both cases, the seismic demand remained below the threshold required to activate the mechanisms.

3.4 EL3 Assessment

For the level of assessment EL3, nonlinear static analyses were performed by exploiting a FE Model realized in Abaqus software [15]. The geometrical features of the model were defined according to the extensive information retrieved from both historical documents and investigation activity. Thus, both masonry and reinforced concrete portions were included in the model (Figure 4), by explicitly modelling the steel rebars in the concrete frame, as well as solid elements simulating the interaction with the adjacent church.

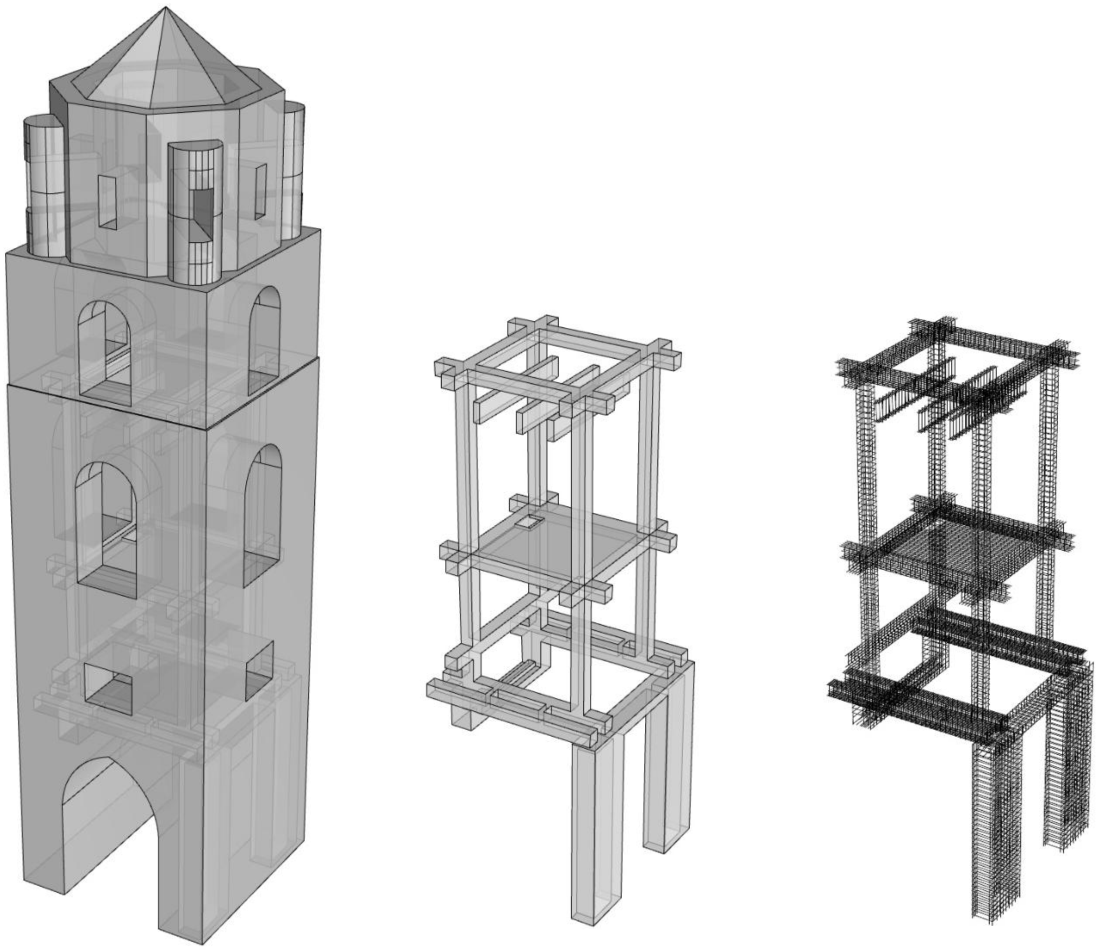
10-node quadratic tetrahedron (C3D10) elements were used for masonry and internal concrete elements (average mesh size of 0.5 m). The roof was modelled with 4-node shell elements (thickness of 0.20 m) with reduced integration (S4R) (average mesh size of 0.5 m) and for steel rebars, truss elements were adopted (T3D2) with a mesh size of 0.2 m. The rebars are assumed fully embedded in the concrete matrix.

A fully fixed condition was assumed at the base of the tower, while the influence of the adjacent church was simulated using no-mass solid linear elements (8-node linear brick with reduced integration – C3D8R – with a mesh size of 0.3 m) with predefined stiffness (further detailed below). These elements interact with the tower following a *Hard contact* behaviour, preventing penetration while allowing separation under tensile forces. Perfect connections were defined among different materials, by guaranteeing node congruence between masonry and concrete portions.

Material elastic parameters were defined according to an optimization procedure based on the ambient vibration tests described in [5] that allowed for defining the dynamical characteristics of the tower (both vibrational frequencies and modal shapes).

In particular, it was assumed:

- Elastic modulus of masonry: 1000 MPa;
- Density of masonry: 1100 kg/m^3;
- Elastic modulus of concrete: 45000 MPa;
- Density of concrete: 2500 kg/m^3;
- Density of concrete beam supporting bells: 3000 kg/m^3, to account for the bell mass;
- Elastic modulus of steel: 210000 MPa;
- Density of steel: 7850 kg/m^3.



(a) (b) (c)

Figure 4. The adopted geometrical model: (a) masonry portion, (b) concrete frame and (c) rebars in concrete frame.

The elastic modulus of the elements used to simulate the interaction with the church was assumed to ensure a perfect connection with the façade wall and a negligible normal interaction with the longitudinal wall, as determined through the calibration process described in [5].

As per nonlinear behaviour of the materials, for both masonry and concrete the *Concrete Damage Plasticity* material model was adopted [16,17]. Parabolic laws were defined for uniaxial compressive behaviour, while a fracture energy approach was adopted for the tensile constitutive law. Also, damage linear laws were introduced for both compression and tension, starting from the peak strength. The values of the parameters listed in Table 4. Parameters adopted for masonry and concrete uniaxial laws.Table 4 were adopted for the materials.

Material	Compressive strength [MPa]	Tensile strength [MPa]	Tensile fracture energy [N/m]
Masonry	2.00	0.1	20
Concrete	28	2.77	133

Table 4. Parameters adopted for masonry and concrete uniaxial laws.

The remaining parameters were defining for both materials as follows: uniaxial-to-biaxial compressive strength ratio f_{c0}/f_{b0}=1.16, dilation angle Ψ = 10°, K_c= 0.667, eccentricity ε = 0.1.

Steel was simulated with a typical *Plastic* material behaviour, with a yield strength of 215 MPa and ultimate strength of 355 MPa (ultimate strain of 10%).

Analyses were phased in two steps. During the first steps (*Static/General*) self-weights were applied. The second steps were of the *Dynamic/Implicit* type, with mass proportional horizontal loads applied by means of *Body Force* in the four principal direction of the tower (for convenience of the reader, in the +x direction, the bell tower is pushed perpendicular to the generatrix of the vault of the base sector, moving away from the church, while the +y direction is parallel to the generatrix of the vault and acts towards the church body).

The results are shown in terms of load multiplier-displacement in Figure 5 (the control nodes were selected at the roof base in the masonry portion), while in Figure 6 the damage pattern (tensile damage parameter for masonry and concrete and plastic strains for steel elements) are shown at the last step of the analyses.

The tower generally demonstrated good performance against lateral loads, as indicated by the load multipliers. Under seismic action along the x-direction, load multipliers ranged from 0.35g (in the -x direction) to 0.55g (in the +x direction). The capacity curves in the x-direction show notable differences in initial stiffness, primarily due to the interaction between the tower and the adjacent church. Under seismic loads in the y-direction, the structural response was coincident, reflecting the tower's symmetry and the fact that the interaction with the church in this direction was neglected.

Conversely, the tower exhibited limited deformability, with ultimate drift values remaining below 0.35% in all analysed directions. This behaviour is primarily attributed to the high stiffness of the tower, provided by the reinforced concrete elements and their interaction with the surrounding masonry. It must also be mentioned that, due to the force-controlled character of the simulation, the numerical curve is limited to the ascending branch and possible ductility reserves are then disregarded. However, at the assumed failure points extensive damage were observed in both masonry and concrete elements.

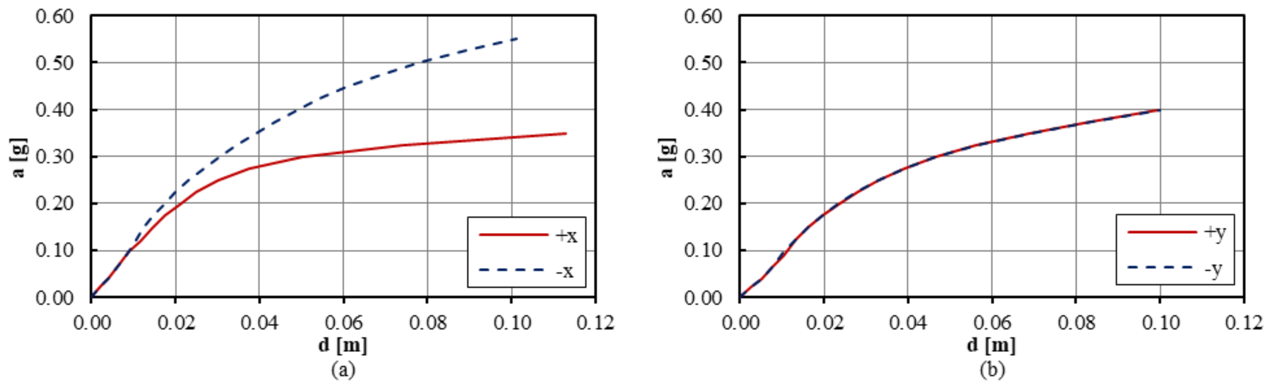


Figure 5. Results of pushover analyses in terms of load multiplier-displacement curves: (a) x directions and (b) y directions.

Based on the observed damage patterns, both masonry and concrete components sustained significant damage in the final step of the analyses. Typically, the masonry elements exhibited flexural failure at the base and shear mechanisms in the upper sections of the tower.

In the case of the -x direction, damage at the base was relatively limited, with no evident failure in the masonry piers. This confirms the significant structural contribution of the reinforced concrete elements, as well as the beneficial influence of the interaction with the adjacent church structure. In this configuration, the most severely affected area was the second sector, where both masonry elements and concrete columns experienced notable damage. The concrete slab was also highly stressed by seismic loads, as demonstrated by the extensive damage distribution.

On the other hand, the steel reinforcement remained largely within the elastic range throughout all analyses, with only localized plastic strains observed in the most highly stressed tensile regions.

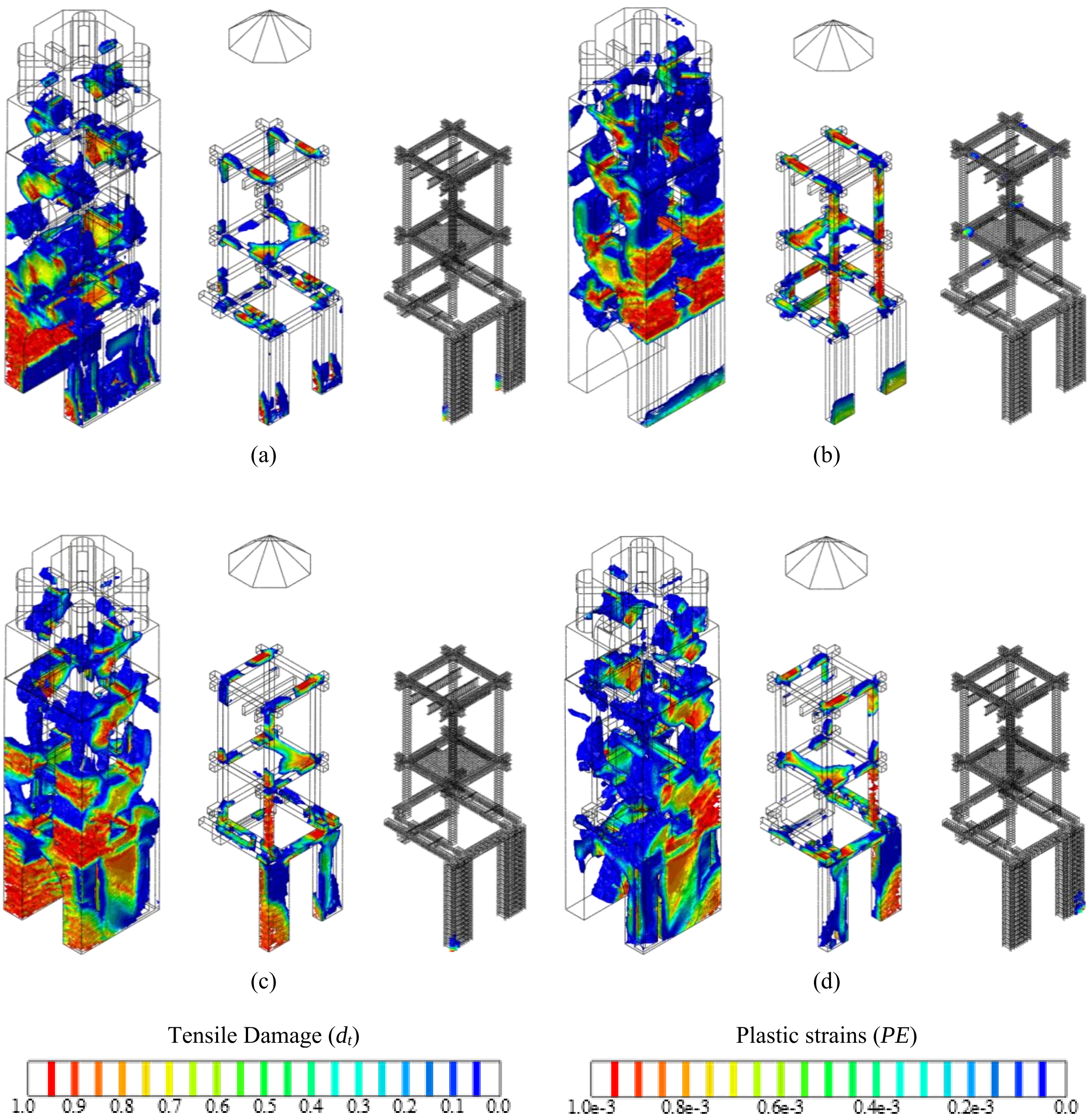


Figure 6. Results of pushover analyses in terms of damage patterns (tensile damage d_t parameter for masonry and concrete and plastic strains *PE* for steel elements): (a) +x direction and (b) -x direction, (c) +y direction, (d) -y direction, (e) legend for tensile damage (masonry and concrete) and (f) legend for plastic strains (steel).

4 CONCLUSIONS

In the present study, a comprehensive seismic vulnerability assessment of a historic masonry bell tower located in Caserta (Italy) has been carried out. The tower, which exhibits typical Medieval architectural features, includes a reinforced concrete frame embedded within the masonry structure, result of a retrofitting intervention implemented during the 20th century.

The assessment was conducted following the three Evaluation Levels (EL1, EL2, and EL3) defined by the Italian Guidelines for the Seismic Risk Assessment of Cultural Heritage. Evaluation Levels 1 and 2, which rely on simplified mechanical assumptions and neglect the contribution of the concrete frame and the structural interaction with the adjacent church, nonetheless revealed a satisfactory seismic capacity of the tower when subjected to design-level earthquake actions. In both cases,

the most critical conditions were identified in the lower portion of the tower, where stress concentrations were more significant (EL1) and most penalizing collapse mechanisms provided the formation of rotational hinge (EL2).
The advanced assessment at Evaluation Level 3, which incorporates a detailed numerical model accounting for the presence of the reinforced concrete frame as well as the interaction with the church structure, confirmed the tower's adequate seismic capacity. The model highlights the structural benefits provided by the retrofitting interventions, particularly in terms of strength and stiffness, despite a limited ultimate drift attained at the maximum force capacity.

ACKNOWLEDGMENTS

The activities described in this paper were carried out within the research project "GENESIS – seismic risk manaGEmeNt for tourist valorization thE hiStorIcal centers of Southern Italy", funded by the Italian Ministry of University and Research under the PNR 2015-2020 programme (call: D.D. n. 1735, 13/07/2017 – Industrial Research and Experimental Development Projects in the 12 Areas of Specialization identified by the PNR 2015-2020 – Specialization Area: Cultural Heritage – Project proposal: ARS01_00883). M.D.P. is funded by MUR (Ministry of University and Research), European Union - Next Generation EU, CUP B53D23027300001 through the project PRIN 2022 PNRR "SAFE_MOTION - Structural Assessment and development of innovative saFEguard Measures fOr hisTOrIcal masONry towers".

REFERENCES

[1] F. Testa, A. Barontini, N. Chieffo, P.B. Lourenço, Observed damage and simplified risk assessment of Italian masonry bell towers struck by past seismic events. *Bulletin of Earthquake Engineering*, **22**(6), 3353–3385, 2024. https://doi.org/10.1007/s10518-024-01878-0

[2] G. De Matteis, M. Zizi, Seismic damage prediction of masonry churches by a PGA-based approach. *International Journal of Architectural Heritage*, **13**(7), 1165–1179, 2019. https://doi.org/10.1080/15583058.2019.1597215

[3] S. Lagomarsino, S. Cattari, PERPETUATE guidelines for seismic performance-based assessment of cultural heritage masonry structures. *Bulletin of Earthquake Engineering*, **13**(1), 13–47, 2015. https://doi.org/10.1007/s10518-014-9674-1

[4] C. Chisari, M. Zizi, J. Rouhi, A. Lavino, G. De Matteis, Ambient vibration testing and model updating of the bell tower of St. Michele Arcangelo Cathedral in Casertavecchia, Italy. *Procedia Structural Integrity*, 44, 1100–1107, 2023. https://doi.org/10.1016/j.prostr.2023.01.142

[5] C. Chisari, M. Zizi, G. De Matteis, Dynamic model identification of the medieval bell tower of Casertavecchia (Italy). *Engineering Failure Analysis*, **167**, 109055, 2025. https://doi.org/10.1016/j.engfailanal.2024.109055

[6] Ministero dei Beni e le Attività Culturali, *Linee Guida per la valutazione e riduzione del rischio sismico del patrimonio culturale allineate alle nuove Norme tecniche per le costruzioni*. 2010, 164 pp.

[7] M. D'Onofrio, *La cattedrale di Caserta Vecchia*. 1993.

[8] G. Daniele, *Il duomo di Casertavecchia*. Caserta, 1873.

[9] J. Rouhi, M. Zizi, C. Chisari, G. De Matteis, A. Gioia, Structural features of St. Michele Arcangelo Cathedral in Casertavecchia, Italy: A preliminary investigation. In *The 3rd International Conference 'Coordinating Engineering for Sustainability and Resilience' CESARE'22*, Irbid, Jordan, 2022.

[10] D. Calcaterra, P. Cappelletti, A. Langella, A. Colella, M. de Gennaro, The ornamental stones of Caserta province: the Campanian Ignimbrite in the medieval architecture of Casertavecchia. *Journal of Cultural Heritage*, **5**(2), 137–148, 2004. https://doi.org/10.1016/j.culher.2003.08.003

[11] Ministero delle Infrastrutture e dei Trasporti, *Istruzioni per l'applicazione dell'«Aggiornamento delle "Norme tecniche per le costruzioni"» di cui al decreto ministeriale 17 gennaio 2018*. Roma, 2019.

[12] C. Chisari, M. Zizi, D. Cacace, G. De Matteis, Seismic vulnerability assessment of ten bell towers in Naples. In *2nd Conference on Sustainability in Architecture, Planning and Design (Beyond All Limits 2022)*, Caserta, Italy, 2022, 235–240.

[13] C. Chisari, D. Cacace, G. De Matteis, A mechanics-based model for simplified seismic vulnerability assessment of masonry bell towers. *Engineering Structures*, **270**, 114876, 2022. https://doi.org/10.1016/j.engstruct.2022.114876

[14] Ministero delle Infrastrutture e dei Trasporti, *NTC 2018 - Norme Tecniche per le costruzioni*. *Gazzetta Ufficiale* n.42 del 20 febbraio 2018.

[15] Dassault Systèmes Simulia Corp., *ABAQUS*. 2023.

[16] J. Lee, G.L. Fenves, Plastic-damage model for cyclic loading of concrete structures. *Journal of Engineering Mechanics*, **124**(8), 892–900, 1998.

[17] J. Lubliner, J. Oliver, S. Oller, E. Oñate, A plastic-damage model for concrete. *International Journal of Solids and Structures*, **25**(3), 299–326, 1989.