Available online at www.sciencedirect.com

ScienceDirect

Procedia Structural Integrity 44 (2023) 1092–1099

Structural Integrity Procedia

www.elsevier.com/locate/procedia

XIX ANIDIS Conference, Seismic Engineering in Italy

Seismic vulnerability analysis and structural rehabilitation of a historical masonry tower

Massimiliano Ferraioli[a]*, Angelo Lavino[b], Donato Abruzzese[c], Alberto Mandara[a], Alberto Maria Avossa[a]

[a]*Department of Engineering, University of Campania "Luigi Vanvitelli", Via Roma 9, 81031 Aversa(CE), Italy*
[b]*Department of Architecture and Industrial Design, University of Campania "Luigi Vanvitelli", Via San Lorenzo 31, 81031, Aversa (Italy)*
[c]*Department of Civil Engineering and Computer Science, University of Rome "Tor Vergata", Via Del Politecnico 1, 00133 Rome, Italy*

Abstract

This paper presents the results of the structural performance assessment of an existing masonry tower and the subsequent repair and strengthening intervention. The study deals with an Italian architectural heritage building in the Cilento National Park (Southern Italy): the bell tower in Torre Orsaia. The tower has been the subject of on-site diagnostic investigations including geometrical surveys, laser scanning surveys, flat jack tests, endoscopic tests, sonic pulse velocity tests, geognostic surveys, and tie-rods tests. The multilevel assessment path proposed by the Italian Guidelines for the assessment and mitigation of the seismic risk of the cultural heritage is followed and the corresponding results are discussed. The global inelastic behavior of the masonry tower is studied through a macro-model approach using the nonlinear static (pushover) analysis. The local collapse mechanisms are studied through a kinematic limit analysis based on rigid block rotation. The repair and strengthening interventions have shown their effectiveness to close the existing cracks, preventing damage to the belfry, and improving the seismic performance of the tower. Both the compatibility, durability, and reversibility of the interventions and their reliability and monitoring are finally highlighted since the bell tower of Torre Orsaia is a historical heritage building.

Peer-review under responsibility of the scientific committee of the XIX ANIDIS Conference, Seismic Engineering in Italy.

Keywords: masonry towers, seismic assessment, strengthening;

* Corresponding author. Tel.: .+39 0815010210; fax: +39 081 5010463.
E-mail address: massimiliano.ferraioli@unicampania.it

Peer-review under responsibility of the scientific committee of the XIX ANIDIS Conference, Seismic Engineering in Italy.
10.1016/j.prostr.2023.01.141

1. Introduction

The Italian experience of recent earthquakes has highlighted the importance of seismic risk mitigation in a country like Italy that presents at the same time medium-high seismic hazards, particularly high exposures (due to its monumental and artistic heritage), and the extreme vulnerability of historical buildings. Many of them exhibit poor geometric characteristics, such as irregularities, interactions with adjacent lower buildings, widespread openings, slender and/or heavy belltowers, excessive slenderness, impressive leaning phenomena, and so on. These important features add to the traditional issues of the masonry constructions, such as the heterogeneity of the material, its low tensile strength and poor ductility, and the inadequate connections of masonry heritage buildings. This is particularly true for slender historical structures, such as towers and minarets, that are also often subjected to high gravity loads that determine working stress very close to the strength of the material. Moreover, widespread cracking often occurs on many of these buildings, thus predisposing them to local or total failure mechanisms even under gravity loads. This situation has stimulated the development of a new generation of international standards and many studies and researches in the literature. Some of them are devoted to models and methods of analysis (Abruzzese et al. 2009, Bartoli et al. 2013, Bernardeschi et al. 2004, Carpinteri et al. 2006, Ferraioli et al. 2017, Milani et al. 2012). Others to ambient vibration surveys (AVS) and modal identification (De Sortis et al. 2005, Bennati et al. 2005, Ivorra et al. 2006, Ferraioli et al. 2018). Still, others are dedicated to the soil-structure interaction and constraint effect of adjacent buildings (Casolo et al. 2017, Bartoli et al. 2019, Castellazzi et al. 2018) or the optimal sensor placement (Civera et al. 2021). Unfortunately, widely generalizable results are still lacking, and, thus, the development of case studies is still of great interest and usefulness.

2. On-site investigations

The bell tower of the church of San Lorenzo Martire in Torre Orsaia (Salerno, Italy) was initially built as a defensive tower in the Lombard and Norman ages. The original structure made of local sandstone had three orders with a square shape. In 1576, two more orders were added, particularly, a belfry with an octagonal plan and a conical trunk roof (Fig. 1). The belfry was made of bricks and local stones covered with grey tuff blocks, and then it was strengthened using four iron tie-rods with rectangular cross-sections arranged horizontally. The tower is connected to the church on the north side and the chapel on the south façade. The tower suffered repeated damage from lightning. At the beginning of the on-site investigations, very wide cracks were observed on the octagonal belfry. Many blocks had moved and/or suffered detachment from the internal structure.

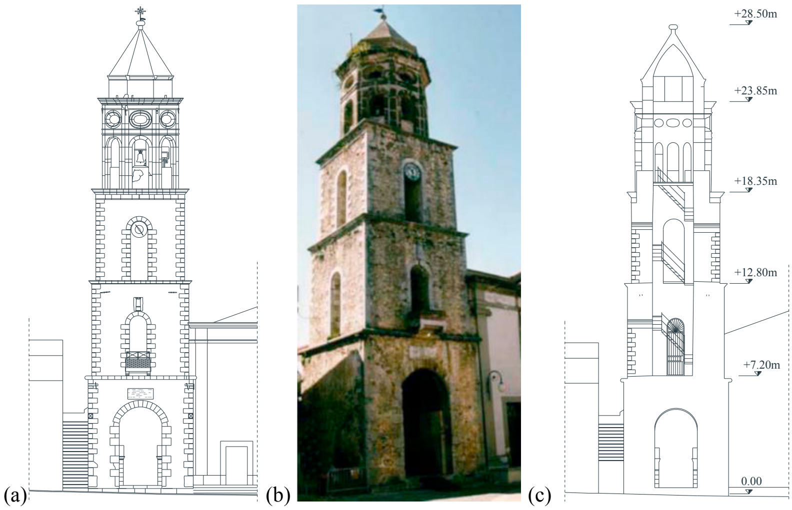

Fig. 1. (a) North side view; b) External photo; (c) Cross-section.

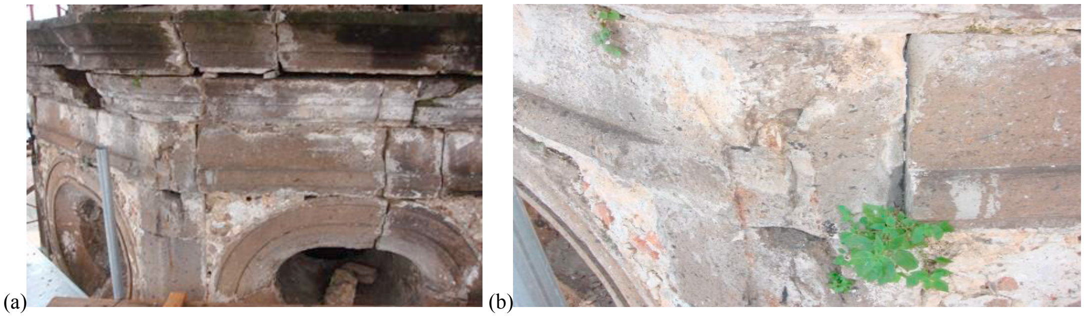

Fig. 2. (a) Cracks on the belfry; (b) No anchor of the tie rod on the west façade.

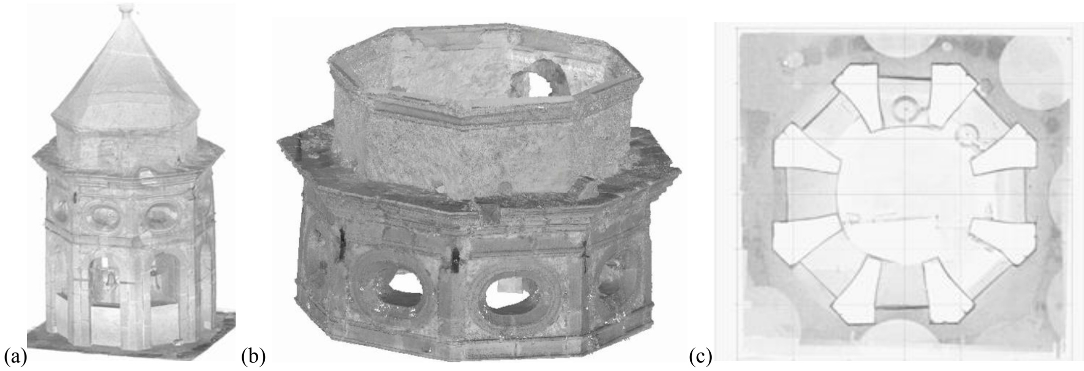

Fig. 3. Laser scanning: (a) 3D view of the belfry; (b) Octagonal plan tambour; (c) Plan view at the height of 20 m.

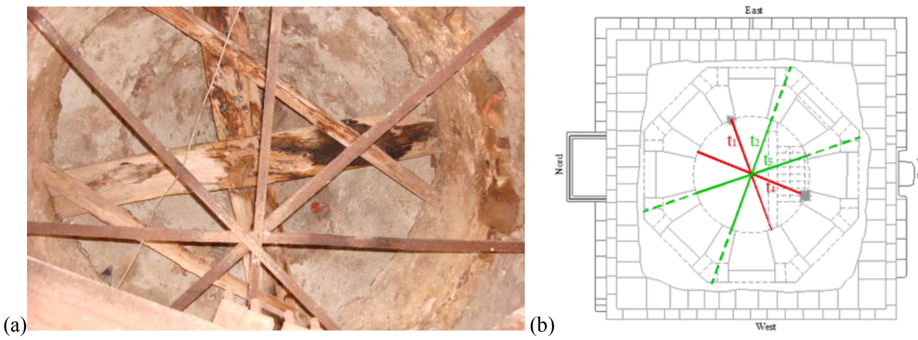

Fig. 4. (a) Ancient tie rods in the belfry; (b) Plan view of the tie rods.

The subsequent diagnostic investigations included: a geometrical survey, laser scanning survey, endoscopic tests, sonic pulse velocity tests, flat jack tests, geognostic surveys, dynamic testing on iron tie-rods, and ambient vibration survey (AVS). The data acquisition system included integrated circuit piezoelectric accelerometers (sensitivity of 1 V/g, frequency range (±5%) of 0.06–450 Hz, resolution of 0.00003 m/s^2, noise of 2.9 × 10-6 m/sec^2/√Hz), a signal conditioning circuitry (noise of 3 × 10-6 m/sec^2/√Hz) and a 16-bit 32-channel A/D converter. The sensors were located at the base of the belfry to estimate the first flexural X and first flexural Y frequencies. More details about these investigations may be found in Ferraioli et al. (2020). The geognostic surveys allowed to exclude leaning phenomena due to a subsidence of the foundations. On the contrary, the dynamic testing and health monitoring on the iron tie-rods, together with the visual inspection of the corresponding anchors, revealed that only tie-rod N.2 was still working, while tie-rod N.1 has a broken anchor and the other ones showed very low axial forces, probably due to high corrosion and deformation of the anchors. The loss of tension of the cables is the main cause of the damage to the belfry.

3. Simplified analysis (Level LV1) and analysis of local collapse mechanisms (Level LV2)

According to the Italian Guidelines (DPCM 2011), a preliminary analysis has been carried out considering a simplified cantilever model (Level of analysis LV1: Evaluation with simplified models) and identifying the collapse mechanisms (Level of analysis LV2: Analysis of local collapse mechanisms). The design material properties of masonry have been evaluated assuming a limited knowledge level (KL1) given the type and extension of the tests performed. The confidence factor FC has been evaluated from the partial confidence factors, which gives FC=1.32. The cohesion and elastic modulus have been calculated according to Table C8.5.I (NTC 2019), using the minimum value for strength and the mean value for the elastic modulus and applying a reduction factor of 0.90 to account for the inner core with poor mechanical properties. The design strength is given by f_d=1·0.90/1.32=0.682N/mm^2. In the simplified mechanical-based approach (Level LV1) the tower is modeled as a cantilever subjected to lateral seismic forces and divided into sectors. The seismic demand is the acting bending moment. The seismic capacity is the resisting bending moment at the base. The lateral forces are calculated from the linear static analysis. The parameters of the design response spectra are shown in Tab. 1. Different constraint conditions (i.e., tower connected or tower unconnected to the adjacent buildings) have been considered in the analysis. The safety checks have been finally carried out using the peak ground acceleration safety index ($f_{a,LS}$) and the return period safety index ($I_{S,LS}$) given by:

$$f_{a,LS} = \frac{a_{LS}}{a_{g,LS}} \; ; \qquad\qquad I_{S,LS} = \frac{T_{LS}}{T_{R,LS}} \tag{1}$$

where T_{LS} and a_{LS} are, respectively, the return period and the corresponding peak ground acceleration for soil type A that leads the tower to the Life Safety (LS) limit state, $T_{R,LS}$ and $a_{g,LS}$ are, respectively, the reference return period and peak ground acceleration for the LS limit state plotted in Tab. 1. The minimum values (as the constraint with adjacent building changes) are given by: $f_{a,LS}$=0.589, and $I_{S,LS}$=0.267. More details can be found in Ferraioli et al. (2020, 2022). In the linear kinematic analysis (Level LV2), the peak ground acceleration is found that mobilizes a local or global collapse mechanism. The multiplier of the lateral loads activating the mechanism is calculated by applying the Principle of Virtual Works. The calculation of the peak ground acceleration and the corresponding safety verification has been carried out using the equivalent SDOF model proposed in C8A.4.2.3 (NTC 2019). Some typical local and global collapse mechanisms have been considered in the analysis using the experience of past seismic events. The minimum value of the safety factor for the LS limit state ($f_{a,LS}$ =0.305, and $I_{S,LS}$ =0.052) is obtained for the local mechanism of the masonry piers shown in Fig. 5a, due to the horizontal thrust of the dome and the poor efficiency of the iron tie-rods. More details about the different collapse mechanisms can be found in Ferraioli et al. (2020, 2022).

4. Global analysis (Level LV3)

The global analysis (Level LV3 according to the Italian Guidelines) has been carried out using a finite element macro-model approach where masonry is considered an equivalent homogeneous isotropic material. To this aim, the model proposed by Avossa et al. (2015) has been used that combines the plasticity criterion, the crushing surface in compression, and the cracking surface in tension. A tridimensional model has been implemented in ANSYS (Kohnke 2001) using 3D Solid65 elements with a prismatic shape for the main body of the tower, and 3D Solid 186 with a hexahedral shape for the belltower (Fig. 5b).

Table 1. Parameters of elastic design response spectra

Parameter	Operation Limit State (OLS)	Damage Limit State (DLS)	Life Safety Limit State	Collapse Prevention Limit State
Probability of exceedance P_{VR}(-)	0.81	0.63	0.10	0.05
Return period T_R(yrs)	30	50	475	975
Peak ground acceleration a_g(g)	0.037	0.047	0.116	0.149
Dynamic amplification factor F_0(-)	2.446	2.443	2.521	2.583
Corner Period T_C(sec)	0.281	0.324	0.447	0.465

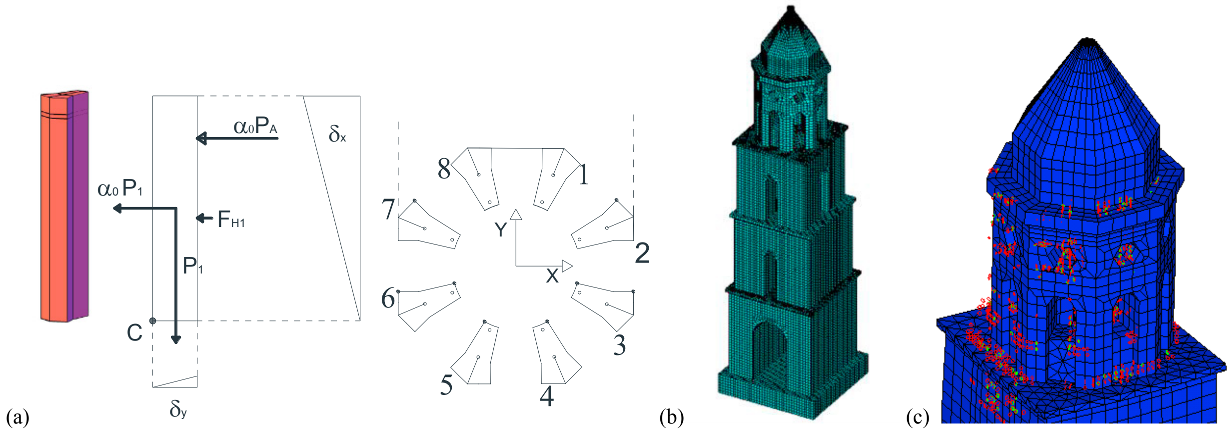

Fig. 5. (a) Local mechanism of the masonry piers (Level LV2); b) Fem model (Level LV3); c) Cracking and crushing on the belfry (LV3).

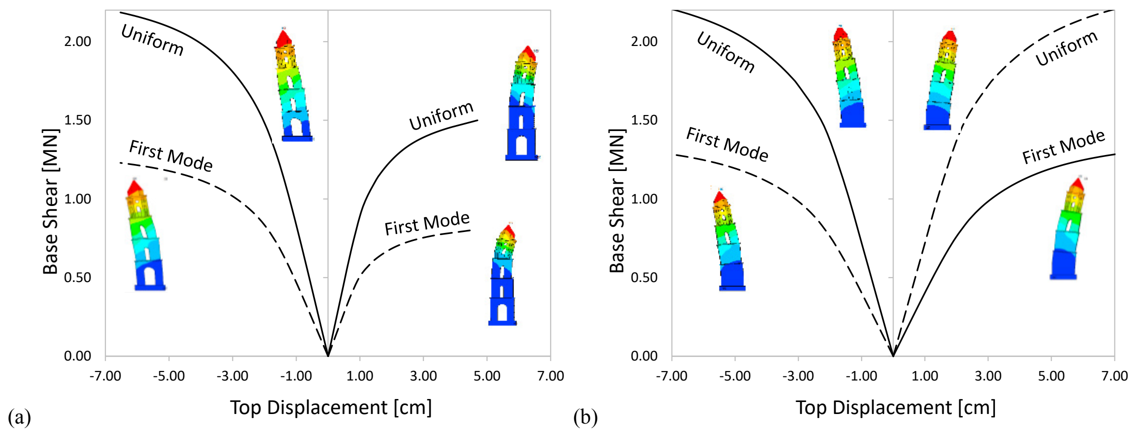

Fig. 6. a) Pushover curves in X-direction; b) Pushover curves In Y-direction.

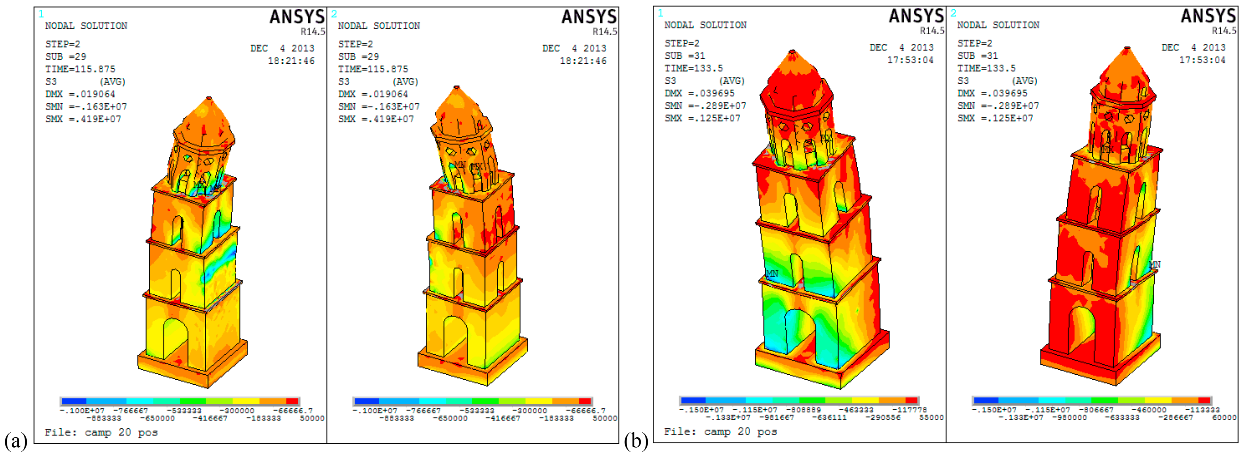

Fig. 7. Maximum principal compressive stress. (a) First-Mode pushover analysis in X-direction; (a) First-Mode pushover analysis in Y-direction.

The parameters of the nonlinear model for masonry are shown in Tab. 2. In particular, the Young modulus (E=1700 MPa) has been calibrated by model tuning using the natural frequencies (3.3Hz and 3.5Hz) identified by the ambient-vibration tests. The compressive strength f_c is given by the mean value from the flat-jack tests. The tensile strength is $f_t = f_c/15$. The friction angle (φ) and the cohesion (c) are evaluated in Table C8.5.I (NTC 2019). The compression cap is f_{cb} =1.2f_c. The shear transfer across the crack faces is considered through β coefficients (equal to β_t and β_c for open and reclosed cracks, respectively). More details on the structural model and material parameters setting can be found in Ferraioli et al. (2020). The seismic safety has been evaluated using the well-known capacity spectrum method (CSM) based on the inelastic demand response spectra (IDRS). The capacity spectrum (CS) has been defined based on the nonlinear static (pushover) analysis using two distributions of the lateral loads over the height of the building: 1) First-mode distribution; 2) Uniform distribution. Fig. 6 shows the pushover curves (i.e., base shear vs top displacement) in the X and Y directions. Figs. 7-8 show the contour map of the maximum principal compressive stress and the corresponding crack and crush patterns. The pushover curve in +X direction is different from that one in -X direction. This situation occurs because of the constraint of the church that acts only if it is in compression.

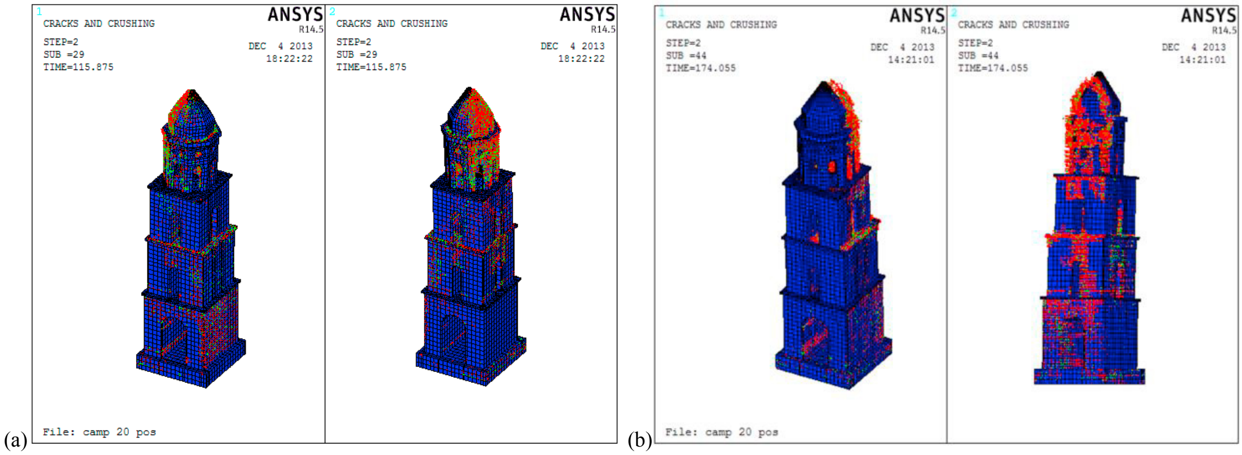

Fig. 8. Crack patterns. (a) First-Mode pushover analysis in X-direction; (a) First-Mode pushover analysis in Y-direction.

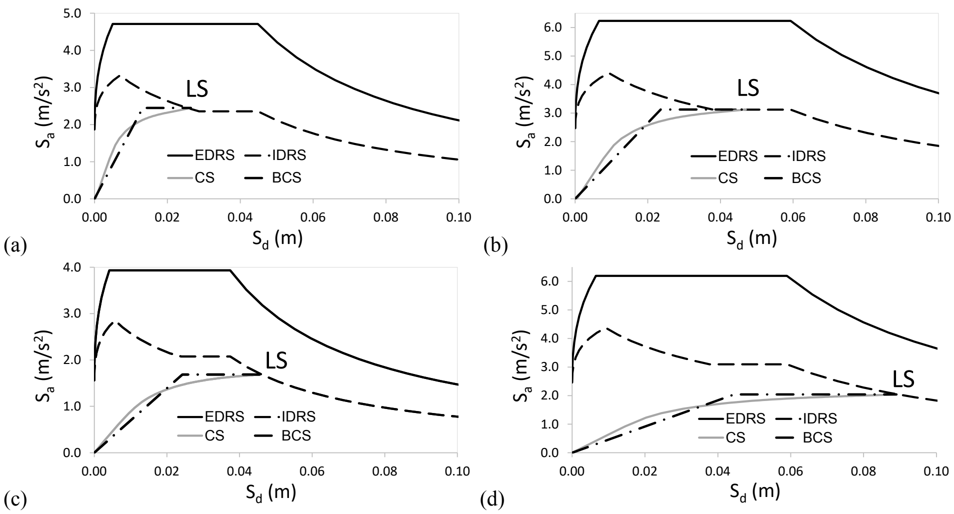

Fig. 9. Capacity Spectrum Method. a) First Mode distribution in X-direction b) Uniform distribution in X-direction; c) First Mode distribution in Y-direction b) Uniform distribution in Y-direction.

On the contrary, the pushover curves in +Y and -Y directions are practically coincident, due to the symmetry of the tower. Likewise, also the contour map and crack patterns of pushover analysis in +X direction are affected by the interaction with the church that causes widespread cracking in the belfry. Fig. 9 shows the calculation of the peak ground acceleration a_{LS} that leads the tower to the (LS) limit state. The values of the safety index (i.e, $f_{a,LS}$ and $I_{S,LS}$) are calculated according to Eq.1 and plotted in Tab. 3. The results show the vulnerability of the tower to seismic action especially in -X direction where the constraint of the church is not effective because tensile stresses occur.

Table 2. Mechanical parameters for masonry.

Elastic Properties			Inelastic Properties (Concrete)				Inelastic Properties (Drucker-Prager)					Inelastic Properties (Compressive Cap)
γ (N/m^3)	E (MPa)	ν	f_c (MPa)	f_t (MPa)	β_c (-)	β_t (-)	c (MPa)	φ (°)	ψ (°)	F_c (MPa)	F_t (MPa)	f_{cb} (MPa)
$1.90 \cdot 10^{-5}$	1700	0.40	0.700	0.047	0.75	0.15	0.150	50	25	0.184	0.055	0.840

Table 3. Acceleration safety index and return period safety index. Level LV3

Direction	First-Mode pushover analysis		Uniform pushover analysis	
	$f_{a,LS}$	$I_{S,LS}$	$f_{a,LS}$	$I_{S,LS}$
+X Direction	1.095	1.247	1.448	2.468
-X Direction	0.793	0.568	1.164	1.448
+Y Direction	0.914	0.803	1.440	2.432
-Y Direction	0.914	0.803	1.440	2.432

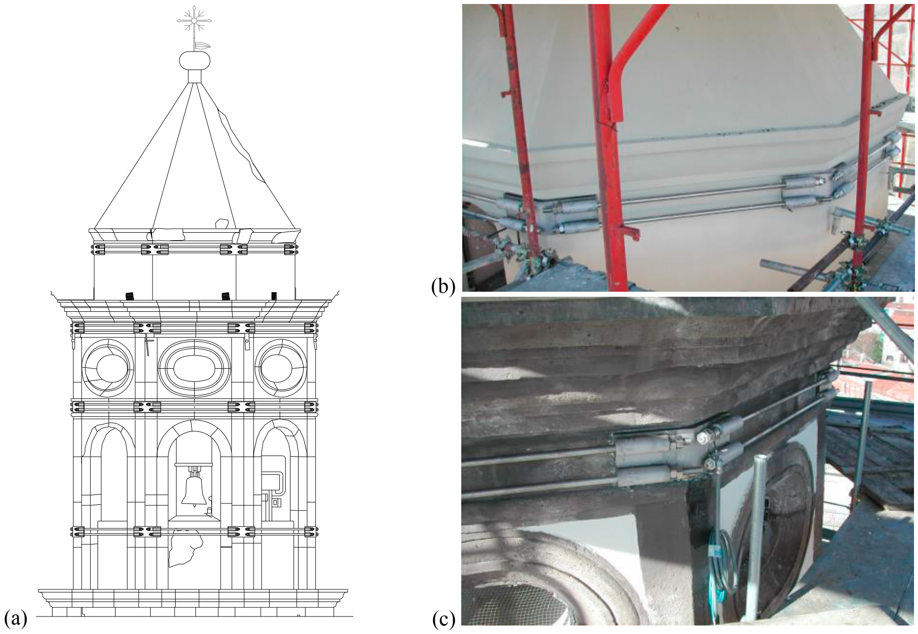

Fig. 10. (a) Front view after retrofit; (b) Stainless tie-rods on the roof; c) Stainless tie-rods on the belfry.

5. Strengthening interventions

The evaluation of the structural vulnerability of the tower is fundamental to establishing both the type and priority of the strengthening interventions. The analyses have shown that the belfry is the most vulnerable part due to its large openings. The critical mechanism is the simple overturning of the masonry piers of the belfry due to cracks in the masonry arches and the ineffective tie-rods. To avoid this collapse mechanism and ensure the static stability of the tower, strengthening interventions were designed and developed. Specifically, four rings, each consisting of a couple of stainless steel bars, were placed in four different horizontal locations of the belfry. Each bar with a diameter of 30 mm was threaded at the edges to allow tensioning through bolting on anchoring corner plates. The pre-tensioning of the bars introduces active confinement in the transverse direction which is very important during the restoration of the belfry. Moreover, the chaining system provides passive confinement in the case of unwanted movements of the masonry piers and resists the outward thrust due to the roof. Finally, it avoids the overturning of the masonry piers under seismic actions, thus ensuring a global behavior of the belfry and increasing the seismic safety of the tower.

6. Conclusions

This paper has presented a comprehensive study on the seismic assessment and rehabilitation of a historical masonry tower. The preliminary investigations (i.e. geometrical survey, laser scanning survey, endoscopic tests, sonic pulse velocity tests, flat jack tests, geognostic surveys, dynamic testing on iron tie-rods, and ambient vibration survey) have been used to identify the main issues of the tower and calibrate the parameters to be used in the subsequent seismic performance assessment. Analysis of different levels (Simplified: LV1; Local: LV2; Global: LV3) have been conducted and the main structural deficiencies have been detected and addressed. The structural rehabilitation using pre-tensioned stainless steel bars has been described, and its effects on static and seismic safety are discussed.

References

Abruzzese, D., Miccoli, L., Yuan, J.L. 2009. Mechanical behavior of leaning masonry Huzhu Pagoda. J Cult Herit 10, 480-486.

Avossa, A.M, Malangone, P., 2015. Seismic performance assessment of masonry structures with a modified “concrete” model. Bull Earthq Eng 13, 2693-2718.

Bartoli, G., Betti, M., Giordano, S., 2013. In situ static and dynamic investigations on the "Torre Grossa" masonry tower. Eng Struct 52, 718-733.

Bartoli, G., Betti, M., Galano, L., Zini, G., 2019. Numerical insights on the seismic risk of confined masonry towers. Eng Struct 180, 713-727.

Bennati, S., Nardini, L., Salvatore, W. 2005. Dynamic behavior of a medieval masonry bell tower. Part II: measurement and modelling of the tower motion. J Struct Eng ASCE 131, 1656-1664.

Bernardeschi, K., Padovani, C., Pasquinelli, G. 2004. Numerical modelling of the structural behaviour of Buti's bell tower. J Cult Herit 5(4), 371-378.

Carpinteri, A., Lacidogna, G., 2006. Structural monitoring and integrity assessment of medieval towers. J Str Eng 132(11), 1681-1690.

Casolo, S., Diana, V., Uva, G., 2017. Influence of soil deformability on the seismic response of a masonry tower. Bull Earth Eng 15, 1991-2014.

Castellazzi, G., D’Altri, A.M., De Miranda, S., Chiozzi, A., Tralli, A., 2018. Numerical insights on the seismic behavior of a nonisolated historical masonry tower. Bull Earth Eng 16(2), 933-961.

Civera, M., Pecorelli, M.L., Ceravolo, R., Surace, C., Zanotti Fragonara, L., 2021. A multi-objective genetic algorithm strategy for robust optimal sensor placement. Comput-Aided Civ Inf 36(9), 1185-1202.

De Sortis, A., Antonacci, E., Vestroni, F., 2005. Dynamic identification of a masonry building using forced vibration tests. Eng Struct 27, 155–165.

DPCM, 2011 Valutazione e riduzione del rischio sismico del patrimonio culturale con riferimento alle norme tecniche per le costruzioni di cui al DM 14 gennaio 2008. Presidenza del Consiglio dei Ministri (Assessment and reduction of seismic risk of cultural heritage in relation to the Building Code included in the Ministerial decree of January 14th 2008. Presidency of the Council of Ministers), Rome (in Italian).

Ferraioli, M., Miccoli, L., Abruzzese, D., Mandara, A. 2017. Dynamic characterisation and seismic assessment of medieval masonry towers. Nat Haz 86, 489-515.

Ferraioli, M., Miccoli, L., Abruzzese, D., Mandara, A., 2018. Dynamic characterisation of a historic bell-tower using a sensitivity-based technique for model tuning. J Civ Struct Health Monit 8(2), 253-269.

Ferraioli, M., Lavino, A., Abruzzese, D., Avossa, A.M., 2020. Seismic Assessment, Repair and Strengthening of a Medieval Masonry Tower in Southern Italy. International Journal of Civil Engineering. 18(9), 967–994.

Ferraioli, M., Lavino, A., Abruzzese, D., Mandara, A., 2022. Safety Analysis and Retrofitting of a Masonry Bell Tower. Lecture Notes in Civil Engineering. 209 LNCE, 938–950.

Italian Code NTC – Instruction, 2019. Circolare N.7 Istruzioni per l’applicazione dell’aggiornamento delle norme tecniche per le costruzioni di cui al decreto ministeriale 17 gennaio 2018 (in Italian).

Ivorra, S., Pallarés, F.J., 2006. Dynamic investigations on a masonry bell tower. Eng Struct 28(5), 660-667.

Kohnke, P., 2001. Ansys, Inc., Theory manual—Twelfth Edition. SAS IP Inc.

Milani, G., Casolo, S., Naliato, A., Tralli, A., 2012. Seismic assessment of a medieval masonry tower in Northern Italy by limit, nonlinear static, and full dynamic analyses. Int J Cult Herit 6(5), 489-524.