ANALISI SISMICHE DI UNA STRUTTURA IN ACCIAIO CON DISPOSITIVI VISCOSI PER LA PROTEZIONE DI CAMPANILI IN MURATURA

SEISMIC ANALYSIS OF A STEEL STRUCTURE WITH VISCOUS DAMPERS FOR THE PROTECTION OF MASONRY TOWERS

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ABSTRACT

This paper presents a solution for seismic retrofitting of existing historical masonry towers, consisting of an internal steel structure equipped with dissipative devices that does modify the vertical bearing mechanisms of the masonry tower and does not alter its external architectural appearance. A historic masonry bell tower in the town of Fermo (Italy) is adopted as testbed structure to evaluate the potentialities of the proposed retrofit strategy. A finite element model is developed of both the masonry tower and the dissipation system and numerical nonlinear dynamic analyses are performed to investigate and compare the seismic response before and after the intervention. The outcomes of the study highlight the suitability of the proposed retrofit strategy in mitigating the seismic response of the upgraded structure.

SOMMARIO

Questo articolo presenta una soluzione per l'adeguamento sismico di torri storiche in muratura, consistente in una struttura in acciaio, dotata di dispositivi dissipativi viscosi, interna alla torre in muratura, che non ne modifica la statica verticale e l’aspetto architettonico esterno. Per valutare le potenzialità della soluzione proposta, è stato preso come caso studio un campanile storico in muratura situato nel comune di Fermo. Sono stati sviluppati dei modelli agli elementi finiti al fine di condurre delle analisi numeriche, dinamiche non lineari, utili ad investigare e confrontare la risposta dinamica strutturale prima e dopo l'intervento. I risultati evidenziano l'idoneità della soluzione proposta nel migliorare significativamente la risposta sismica del campanile.

1 INTRODUCTION

Masonry towers are vulnerable structures that are prone to earthquake damage, e.g., [1]-[4]. Many studies on the structural modeling and analysis of masonry towers can be found in the technical literature, e.g., [5]-[11], which testifies to the interest of the structural engineering community in understanding and predicting their seismic behavior. Vulnerability assessments are crucial to design proper maintenance and retrofit interventions. The review of the seismic interventions in historic masonry towers documented in the technical literature shows a variety of possible solutions such as: the use of strengthening techniques that include local injections, rebuilding and repointing of mortar joints, local or diffused metallic or composite reinforcements, tie rods and confining rings, e.g., [12]-[16]; the application of vertical external prestressing tendons made of steel, composite or smart materials, e.g.,[17]; the adoption of vertical prestressing tendons with added hysteretic dissipative devices at the base [18]; the use of base isolation [19]; and the realization of internal steel structures to bear horizontal loads [20].
The objective of this study is to explore the possibility of a solution that starts from the idea of a steel structure that is internal to the masonry tower as proposed and realized by Jurina [20] and adds to such steel structure dissipative devices. In this way there is no alteration of the external architectural appearance and no modifications of the path followed by the vertical loads. In addition, this meets the principle of architectural restoration (intervention reversibility). The idea of exploiting dissipative towers for the seismic upgrading of existing structures with deficient seismic performance is not new in structural engineering, e.g., [21]-[23]. The retrofit solution proposed in this article exploits fluid viscous dampers (FVDs) [24]. A case study is considered, and numerical seismic simulations are presented to evaluate the potentialities of the proposed approach.

2 CASE STUDY

2.1 The Church of San Zenone

The case study considered in this article is the bell tower of the Church of San Zenone (Fig. 1, Fig. 2) in Fermo, a town in the Marche region (Central Italy) that is characterized by historical stratification dating back to the Roman age, including medieval transformations and reconstructions. The reader interested in the whole historical analyses and the sequence of construction phases can refer to [25].
The bell tower (see tower plans in Fig. 3 and vertical sections in Fig. 4, both derived from geometric survey), located on the right side of the main elevation, measures 6.26 x 6.33 m^2 in plan and 29.13 m in height. From the access atrium to the main altar the altitude jump is 2.22 m. Two different masonry typologies that characterize the building: bricks and mortar at the upper part of the North fronts, including the bell tower, and sandstone blocks and mortar at the bottom of the main elevation, which belong to the early construction phases of the church.
A shallow foundation system is present and based on preliminary studies conducted on the site, the soil condition is characterized by values of the shear-wave velocity (V_{S30}) within the interval of 180 m/s and 360 m/s (soil category C according to the Eurocode 8 classification [26]).

Fig. 1. Façade and bell tower view of the San Zenone Church (Fermo, Italy).

Fig. 2. Northern façade of the church from architectural survey.

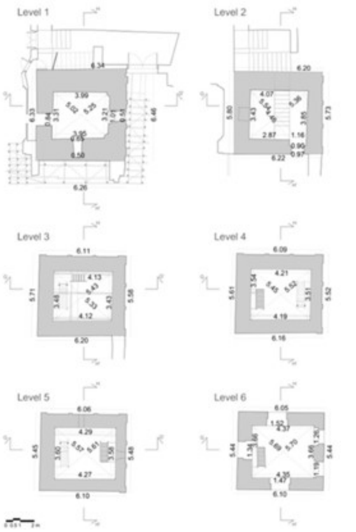

Fig. 3. Tower plans at different heights.

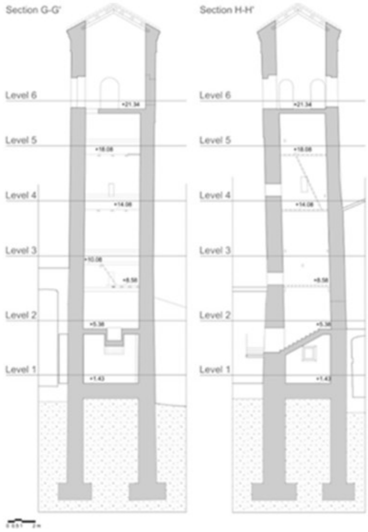

Fig. 4. vertical sections.

2.2 Seismic Hazard

The location of the tower is characterized by medium/high seismicity according to the Italian seismic map, and shows peak ground acceleration (on rigid soil, with $V_{S30} > 800$ m/s) of about 0.2 g with an exceedance probability of 10% over 50 years. In this study, a stochastic ground motion model [25][27] was exploited to characterize the underlying seismicity and provide a set of simulated earthquake samples to perform structural analyses. The values of the parameters governing the seismic scenario were set according to [25] to provide a hazard curve representative of the seismicity expected at the site of the tower. The maximum component $S_a(T^*)$ was used as the intensity measure (IM) in this study, where T^* is equal to 0.75 s, i.e., the average period from the two principal modes of vibration of the tower. The IM hazard curve is shown in Fig. 5, while Fig. 6 depicts the response spectra of a set of 20 pairs of earthquake samples selected at intensity level T_R = 475 years.

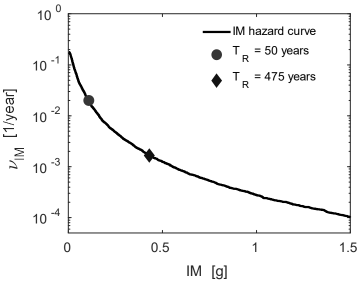

Fig. 5. Intensity measure (IM) hazard curve for $S_a(T^*)$ at T^* = 0.75 s and reference IM levels.

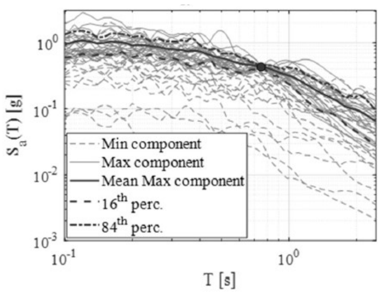

Fig. 6. Response spectra of horizontal seismic components at the intensity level T_R = 475 years.

3 DISSIPATIVE TOWER

3.1 Concept and global configuration

The intervention strategy has three main goals, namely: (1) to provide the tower with a source of supplemental damping that is able to reduce the seismic demand in terms of displacement, acceleration and stress on the masonry elements; (2) to preserve the modal dynamic properties of the original tower without adding stiffness to the system in order not to force the structure to behave differently from its natural tendency, i.e., modal shapes and periods are preserved; and (3) to leave unaltered the vertical static behavior of the tower that does not reveal deficiencies. Accordingly, the solution that was chosen was a dissipative tower inside the masonry bell tower and extended from the base up to the bell chamber level, located 21.50 m from the ground level. For the sake of brevity, the dissipative tower will be referred to as the “inner tower” and the original masonry tower as the “outer tower”.

The inner tower consists of a steel structure with in-plane dimensions of 2.00 × 2.00 m^2 and a total height of 20.00 m, with 8 stories having inter-storey height of 2.50 m each. Every level of the inner tower is equipped with four fluid viscous dampers (two along each direction). Dampers are not located according to standard diagonal configuration, rather they are placed exploiting a scissor-jack configuration [28] (Fig. 7), whose geometry was defined in order to significantly amplify the damper’s axial deformation, and thus, the effectiveness of the damping system, without requiring high displacements of the main structural system. The steel structure has the role of hosting the viscous devices and transferring the motion of the outer tower. For this reason, the structure does not need to be rigid, on the contrary, it is designed to be as flexible as possible in order not to alter the dynamic behavior of the masonry bell tower. The inner tower is made of four HEA 300 columns continuous from the ground to the top with the strong axis oriented along the Y direction, which is the most deformable direction. Beams are made of IPE 270 profiles, with webs linked to the columns through bolted connections working as hinge restraints. In this way, the inner tower stiffness is basically governed by the four columns, working like cantilevers. As can be observed from Fig. 7, the frame structure of the inner tower is arrested one level below the bell chamber floor. Indeed, the latter set of scissor-jack dampers is directly connected from the top of the inner tower to the bottom of the reinforced concrete slab. The connection between the inner and the outer tower is realized through a strut-and-tie steel system (Fig. 7) that can rigidly connect the towers in the horizontal plan by uniformly distributing the stresses (generated from viscous devices) on the masonry structure. At the same time, the connection is sufficiently flexible to accommodate the relative rotations arising from the dynamic motion without undergoing bending.

3.2 Viscous Dampers Design

Fluid viscous dampers are designed to add supplemental damping for a total amount equal to ξ_d = 20%, which is high enough to improve the seismic performance of the outer tower without negatively affecting the response in absolute acceleration. The target design strategy of the retrofit intervention is the mitigation of the seismic response at an intensity level characterized by T_R = 475 years. Linear viscous dampers (α = 1.0) are considered in this application. Once assigned the target damping ratio (ξ_d = 20%), the size of the dampers (viscous coefficients c) was set according to a storey-shear proportional distribution approach (based on the first mode of vibration), for its simplicity and given its acknowledged suitability [29]. Dampers' coefficients c_i, expressed in $(kN\cdot s/m)^\alpha$ and obtained by the design procedure in [25][30][31], are listed as follows from level 1 to level 8: 1000, 993, 968, 927, 868, 788, 676, 543.

4 NUMERICAL MODEL AND SEISMIC ANALYSIS

4.1 Finite Element Model

A three-dimensional finite element model of the tower (Fig. 8) was developed in SAP2000 [32]. The masonry walls were modelled through four-node thick shell elements, thus, accounting for transverse shear deformation according to the Mindlin-Reissner formulation. The in-plan geometry of the tower was accurately reproduced as well as the variation in the wall thickness and the openings along the height of the structure. The tilted configuration (almost 2 degrees with respect to the vertical) of the tower was also modelled. Concerning the boundary conditions, the base of the tower was assumed to be fixed. The presence of the adjacent building was considered in the model by following a simplified approach, according to which the portions of the structure at which the interference is expected are restrained along specific degree of freedom by means of proper supports. It was assumed that the walls of the adjacent building (the main façade of the church and the inner parallel wall) only affect the tower motion along the X direction, hence a set of simply supports effective in X were distributed at a height of 10.00 m along the two corners of the tower. The equivalent height of 10.00 m was set based on the consideration that the part of the façade up to the rose window inferior level (up to 10.00 m from the base) is sufficiently rigid to affect the dynamic behavior of the tower, whereas the upper part is more deformable and thus it is neglected in terms of boundary conditions

The masonry tower model is integrated with the model of the inner steel tower equipped with fluid viscous devices. Steel elements are modelled as frame elements, while the dampers are introduced as a link element with an "exponential damper" property.

4.2 Dynamic Time-History Analyses

The structural seismic performance of the retrofitted tower was evaluated and compared to the seismic response of the structure before the intervention. A set of 20 seismic time-history analyses were performed at the intensity level with T_R = 475 years, and a pair of X-Y ground motion time-series was used in each case of analysis.

A comparison between the seismic performance before and after retrofitting is provided in the following figures: a direct comparison of the maximum displacement profiles along the height of the tower, before and after retrofitting, is given in Fig. 9; a chart of the maximum stress components on the masonry structure (a single shell element at the base of the tower was monitored) is provided in Fig. 10. From both figures it can be observed that the response of the retrofitted tower is notably reduced.

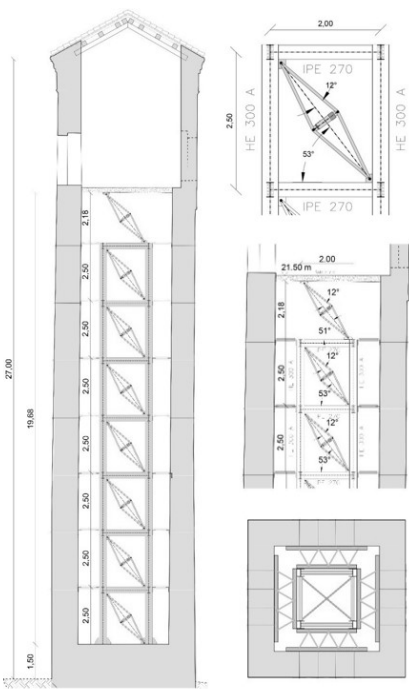

Fig. 7. Scheme of the inner tower, connections, scissor-jack-damper system.

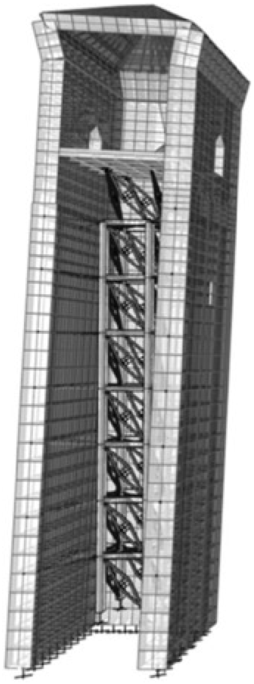

Fig. 8. Inner and outer tower finite element model.

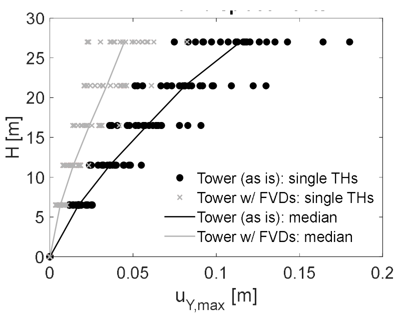

Fig. 9. Maximum displacements along the height of the tower, before and after retrofitting along the Y direction.

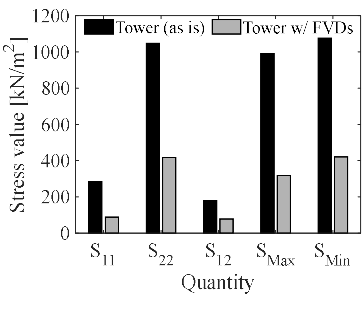

Fig. 10. Masonry median stress values (single shell element at the base of the tower) before and after retrofitting.

5 CONCLUSIONS

A preliminary study for seismic upgrading of a bell masonry tower through an internal dissipative steel tower with added dissipative fluid viscous dampers was conducted in this paper. A numerical model of the masonry tower and the dissipation system was first developed, and then numerical seismic simulations were performed to assess the feasibility of the proposed retrofit strategy. The numerical results indicates that the seismic response of the upgraded bell tower is significantly improved. Seismic displacements were mitigated, and effective seismic energy dissipation was observed with consequent beneficial effects on the level of stress of masonry elements. This preliminary investigation needs further developments before real-world application.

REFERENCES

[1] Carbonari, S., Dall'Asta, A., Dezi, L., Gara, F., Leoni, G., Morici, M., Prota, A., Zona, A. First analysis of data concerning damage occurred to churches of the Marche region following the 2016 Central Italy earthquakes. Boll. Geofis. Teor. Appl. 2019, 60, 183–196.

[2] Canuti, C., Carbonari, S., Dall'Asta, A., Dezi, L., Gara, F., Leoni, G., Morici, M., Petrucci, E., Prota, A., Zona, A. Post-earthquake damage and vulnerability assessment of churches in the Marche region struck by the 2016 Central Italy seismic sequence. Int. J. Archit. Heritage 2019, 1–22.

[3] Morici, M., Canuti, C., Dall'Asta, A., Leoni, G. Empirical predictive model for seismic damage of historical churches. Bull. Earthq. Eng. 2020, 18, 6015–6037.

[4] Cescatti, E., Salzano, P., Casapulla, C., Ceroni, F., Da Porto, F., Prota, A. Damages to masonry churches after 2016–2017 Central Italy seismic sequence and definition of fragility curves. Bull. Earthq. Eng. 2020, 18, 297–329.

[5] Ruggieri, S., Tosto, C., Rosati, G., Uva, G., Ferro, G.A. Seismic vulnerability analysis of masonry churches in Piemonte after 2003 Valle Scrivia earthquake: Post-event screening and situation 17 years later. Int. J. Archit. Heritage 2020, 1–29.

[6] Casolo, S. A three-dimensional model for vulnerability analysis of slender medieval masonry towers. J. Earthq. Eng. 1998, 2, 487–512.

[7] Riva, P., Perotti, F., Guidoboni, E., Boschi, E. Seismic analysis of the Asinelli Tower and earthquakes in Bologna. Soil Dynam. Earthq. Eng. 1998, 17, 525–550.

[8] Carpinteri, A., Invernizzi, S., Lacidogna, G. In situ damage assessment and nonlinear modelling of a historical masonry tower. Eng. Struct. 2005, 27, 387–395.

[9] Ceroni, F., Pecce, M., Voto, S., Manfredi, G. Historical, architectural, and structural assessment of the bell tower of Santa Maria del Carmine. Int. J. Archit. Herit. 2009, 3, 169–194.

[10] Dall'Asta, A., Leoni, G., Meschini, A., Petrucci, E., Zona, A. Integrated approach for seismic vulnerability analysis of historic massive defensive structures. J. Cult. Herit. 2019, 35, 86–98.

[11] Milani, G. Fast vulnerability evaluation of masonry towers by means of an interactive and adaptive 3D kinematic limit analysis with pre-assigned failure mechanisms. Int. J. Archit. Herit. 2019, 13, 941–962.

[12] Modena, C., Valluzzi, M.R., Tongini Folli, R., Binda, L. Design choices and intervention techniques for repairing and strengthening of the Monza cathedral bell-tower. Constr. Build. Mater. 2002, 16, 385–395. [

[13] Valluzzi, M.R., Binda, L., Modena, C. Mechanical behaviour of historic masonry structures strengthened by bed joints structural repointing. Constr. Build. Mater. 2005, 19, 63–73.

[14] Cosenza, E., Iervolino, I. Case study: Seismic retrofitting of a medieval bell tower with FRP. J. Compos. Constr. 2007, 11, 319–327.

[15] Milani, G., Shehu, R., Valente, M. A kinematic limit analysis approach for seismic retrofitting of masonry towers through steel tie-rods. Eng. Struct. 2018, 160, 212–228.
[16] Formisano, A., Milani, G. Seismic vulnerability analysis and retrofitting of the SS. Rosario Church Bell Tower in Finale Emilia (Modena, Italy). Front. Built Environ. 2019, 5, 70.
[17] Preciado, A., Sperbeck, S.T., Ramírez-Gaytán, A. Seismic vulnerability enhancement of medieval and masonry bell towers externally prestressed with unbonded smart tendons. Eng. Struct. 2016, 122, 50–61.
[18] De Stefani, L., Scotta, R., Lazzari, M., Saetta, A. Seismic improvement of slender masonry tower by using hysteretic devices and partial prestressing technique. Bull. Earthq. Eng. 2014, 12, 829–853.
[19] Habieb, A.H., Valente, M., Milani, G. Effectiveness of different base isolation systems for seismic protection: Numerical insights into an existing masonry bell tower. Soil Dyn. Earthq. Eng. 2019, 125, 105752.
[20] Jurina, L. La "Torre nella Torre: Due casi di consolidamento a Pavia. In Proceedings of the REUSO 2016, IV Convegno Internazionale Sulla Documentazione, Conservazione e Recupero del Patrimonio Architettonico e Sulla Tutela Paesaggistica, Pavia, Italy, 6–8 October 2016; Parrinello, S., Besana, D., Eds.; Edifir Edizioni: Firenze, Italy, 2016.
[21] Gioiella, L., Tubaldi, E., Gara, F., Dezi, L., Dall'Asta, A. Modal properties and seismic behaviour of buildings equipped with external dissipative pinned rocking braced frames. Eng. Struct. 2018, 172, 807–818.
[22] Gioiella, L., Tubaldi, E., Gara, F., Dezi, L., Dall'Asta, A. Stochastic seismic analysis and comparison of alternative external dissipative systems. Shock Vib. 2018, 5403737.
[23] Tubaldi, E., Gioiella, L., Scozzese, F., Ragni, L., Dall'Asta, A. A design method for viscous dampers connecting adjacent structures. Front. Built Environ. 2020, 6, 25.
[24] Scozzese, F., Dall'Asta, A., Tubaldi, E. Seismic risk sensitivity of structures equipped with anti-seismic devices with uncertain properties. Struct. Saf. 2019, 77, 30–47.
[25] Pavia, A., Scozzese, F., Petrucci, E., Zona, A., 2021. Seismic Upgrading of a historical masonry bell tower through an internal dissipative steel structure. Buildings, 11(1), 24.
[26] European Committee for Standardization. Eurocode 8-Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings; European Committee for Standardization: Brussels, Belgium, 2004.
[27] Micozzi F., Scozzese F., Ragni L., Dall'Asta A. 2022. Seismic reliability of base isolated systems: sensitivity to design choices. Engineering Structures 256, 114056.
[28] Sigaher, A.N., Constantinou, M.C. Scissor-jack-damper energy dissipation system. Earthq. Spectra 2003, 19, 133–158.
[29] Pekcan, G., Mander, J., Chen, S. Design and Retrofit Methodology for Building Structures with Supplemental Energy Dissipating Systems; Technical Report MCEER-99-0021; University at Buffalo, State University of New York: Buffalo, NY, USA, 1999; pp. 1–177.
[30] Dall'Asta, A., Scozzese, F., Ragni, L., Tubaldi, E. Effect of the damper property variability on the seismic reliability of linear systems equipped with viscous dampers. Bull. Earthq. Eng. 2017, 15, 5025–5053.
[31] Scozzese, F., Gioiella, L., Dall'Asta, A., Ragni, L., Tubaldi, E., 2021. Influence of viscous dampers ultimate capacity on the seismic reliability of building structures. Structural Safety.
[32] SAP2000, Computers and Structures Inc. v20 Integrated Finite Element Analysis and Design of Structures; Computers and Structures Inc.: Berkely, CA, USA, 2018.

KEYWORDS

Anti-seismic devices, seismic retrofit, steel structures, fluid viscous dampers.