Seismic Risk Assessment of Historic Masonry Towers: Comparison of Four Case Studies

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Gianni Bartoli[1]; Michele Betti[2]; and Silvia Monchetti[3]

Abstract: This paper focuses on the seismic risk assessment of historic masonry towers according to the Italian "Guidelines for the Assessment and Mitigation of the Seismic Risk of the Cultural Heritage." The latter identifies a methodology of analysis based on three different levels of evaluation, according to increasing requirements on the structural knowledge: LV1 (analysis at territorial level), LV2 (local analysis), and LV3 (global analysis). Regardless of the methodology of analysis, the more advanced the achieved level of knowledge, the higher the reliability of these approaches becomes. In this field, a fundamental task is the estimation of the uncertain parameters (both material properties and boundary conditions) affecting the structural behavior. The effect of these uncertainties on the global structural response is herein approached through the discussion of an illustrative case study of some of the historic masonry towers in the city center of San Gimignano (Siena, Italy). The seismic risk of these towers was analyzed in the framework of Seismic Risk of Monumental Buildings (RiSEM is the Italian acronym), a research project granted by the Tuscany Regional Administration, and this paper summarizes the results obtained for two of the preceding three levels, which highlights a few issues concerning the seismic risk of historic masonry towers. Useful conclusions are drawn in order to quantify, when performing an LV3 approach through nonlinear models, the effects of the uncertainties on the seismic risk evaluation of such structural typology. The paper, in particular, confirms once more how strongly the effect of confinement reflected on tower seismic performances and stresses that specific attention should be paid to the definition of the effective portion of the structure to be considered as confined (with respect to adjacent buildings). **DOI:** 10.1061/(ASCE)CF.1943-5509.0001039.

Author keywords: Historic masonry tower; Italian guidelines; Seismic assessment; Finite element modeling; Nonlinear static analysis.

Introduction

Preservation of cultural heritage and passing it on to future generations is considered by modern societies a major issue as its conservation is a historical and cultural process, as well as an economic source of wealth (Fioravanti and Mecca 2011). From a social point of view the preservation of cultural heritage, ranging from a local to a European level, contributes to consolidating a collective memory and a European identity. From an economic point of view, especially in contexts where tourism is becoming a major industry, accessibility to cultural heritage significantly contributes to the community's development (Bowitz and Ibenholt 2009).

Safety and functionality of buildings and infrastructures that constitute the urban environment strongly affect the quality of the life of a community (D'Ayala and Paganoni 2011; Brandonisio et al. 2013) and, in the case of Italy, where the territory is characterized by a massive presence of historic and monumental buildings, life quality is intensely connected with the functionality and the safety of these historic structures. These buildings, as demonstrated by the recent earthquakes in L'Aquila (April 2009) (D'Ayala and Paganoni 2011) and Reggio Emilia (May 2012) (Fragonara et al. 2016), are extremely vulnerable to seismic loads. In addition, although the earthquake that affected L'Aquila was a seismic event of exceptional power within the Italian scenario [the main shock had a moment magnitude equal to 6.3 M_w, with a peak ground acceleration (PGA) equal to 0.68 g], not infrequently damages, and in some cases, collapses, of monumental buildings or parts of them were also recorded as a result of not extremely violent earthquakes (e.g., the damage to the Basilica of San Francesco d'Assisi after the seismic shock in November 2007). It is then clear the need of reliable test methods and analysis methodologies that may allow a fairly expeditious seismic risk quantification to be carried out and to be used with a certain repetitiveness and on a territorial scale, to provide general guidelines to establish priority of intervention to protect historic monuments.

Among the different typologies of historic monumental buildings, masonry towers, representing a hallmark of many Italian town centers and widely diffused in the European territory, embody an important heritage to preserve. These structures, built to withstand only vertical loads, show unique typological and morphological features, which have severe effects on their behavior under horizontal loads: due to the high slenderness and mass their seismic risk assessment is a significant concern. As also demonstrated by recent Italian earthquakes and discussed by other authors (Casolo et al. 2013; Valente and Milani 2016), the slenderness and base shear area are among the most important parameters ruling the structural response of towers under seismic loading. In the case of isolated masonry towers, damages are usually most severe at the base (as a result of a combination of bending and shear loads) although cracks along the whole height of the building have been observed. In some cases (Acito et al. 2014), vertical shear cracks are observed during strong earthquakes thus showing that the damage evolution during a dynamic excitation plays a crucial role in reducing the

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Note. This manuscript was submitted on July 29, 2016; approved on December 22, 2016; published online on March 14, 2017. Discussion period open until August 14, 2017; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Performance of Constructed Facilities***, © ASCE, ISSN 0887-3828.

resisting geometry of the structure, activating higher vibration modes, and reducing the cross section stiffness.

These aspects have been taken into account by the Italian Standard (NTC 2008), which involves the territory sensitivity to the seismic hazard and also introduces the use of sophisticated nonlinear analysis methods. Furthermore, an additional document, the Italian "Guidelines for the Assessment and Mitigation of the Seismic Risk of the Cultural Heritage" (DPCM 2011), with reference to three classes of monumental buildings, proposes a methodology of analysis based on three different levels of assessment, according to an increasing knowledge of the structure. The first level of analysis (LV1, analysis at territorial scale), by means of simplified mechanical models based on a limited number of geometrical and material parameters (and qualitative tools such as visual inspections), allows the evaluation of the seismic collapse acceleration of the structure. The second level of evaluation (LV2, local analysis) is based on a kinematic approach and analyzes the local collapse mechanisms that can develop on several macroelements. The knowledge of the structural details of the building (cracking pattern, construction technique, connections between the architectonic elements, etc.) is required for the proper identification of the macroelements. The last level of assessment (LV3, global analysis) asks for a global analysis of the entire construction under seismic loading to be performed by employing suitable numerical codes. If compared with the previous two levels, the LV3 should be the most accurate but, depending on the employed numerical approach, it requires a large amount of experimental input data, together with a high computational effort.

The present research has been developed in this field and illustrates a representative case study, useful to investigate the effects of material parameter and boundary condition uncertainties on the results, expressed in terms of seismic behavior of the towers. The seismic risk of the historic towers in San Gimignano (Siena, Italy) was recently analyzed, as an illustrative case study within the 2-year research project Seismic Risk of Monumental Buildings (RiSEM). The project aimed at developing and testing expeditious and innovative methodologies (i.e., without direct contact with the masonry construction) to assess a minimal set of structural data needed for the subsequent evaluation of the seismic risk at a territorial scale. The whole project, funded by the Tuscany Region (Italy), was developed by a consortium that included the Italian universities of Florence and Siena through four departments from different scientific areas. The city of San Gimignano (Fig. 1) was identified as a prototypical case study due to the typological

Fig. 1. Historic towers in city center of San Gimignano (CH = Chigi tower; BE = Becci tower; CU = Cugnanesi tower) (reprinted from Comune di San Gimignano 2016, with permission from Gianni Bartoli)

structural homogeneity of its historic tall masonry towers. In fact, the availability of several structures with a similar dynamic behavior makes the case study particularly significant for testing new techniques of investigation and analysis. The paper, as a first step toward the synthesis of the results obtained within the project, summarizes the analyses executed and the results obtained for four of the analyzed towers: the Becci tower, the Coppi-Campatelli tower, the Cugnanesi tower, and the Chigi tower (Fig. 2); all the towers were named after the last owner's family name.

In the following, after a brief discussion of the state of the art, the geometry of the considered towers is summarized. Subsequently, the methodology employed to cover the unknowns deriving from the knowledge process and the performed parametric analyses are critically discussed. Finally, some considerations are reported and some conclusions are drawn.

State of the Art

Recent decades have seen the rapid growth of both experimental researches and analytical studies aimed at performing structural

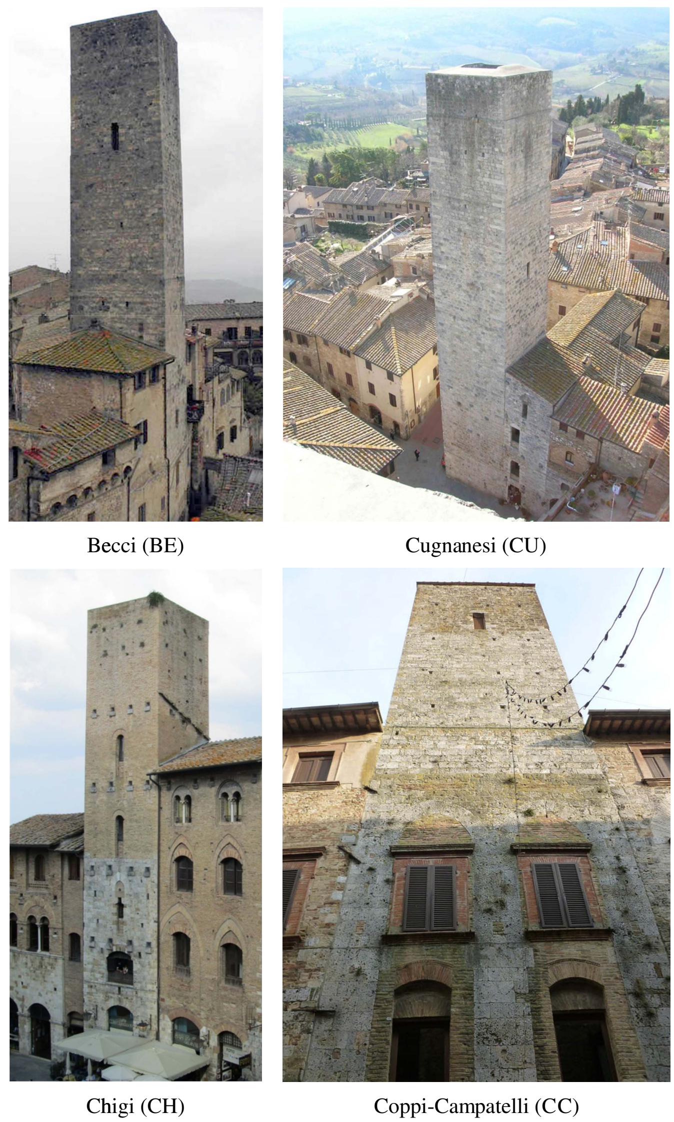


Fig. 2. Four investigated historic towers (images by authors): Becci (BE), Cugnanesi (CU), Chigi (CH), and Coppi-Campatelli (CC)

identification and seismic risk assessment of masonry towers. This growth offers today a wide panorama of researches (use of nonlinear finite element (FE) codes versus use of ad hoc specific numerical codes, experimental identification studies, etc.) and practical applications (aimed at conservation and rehabilitation proposals). Recent examples of these studies in Europe are: the eighth-century masonry tower Torre Sineo (Alba, Italy) (Carpinteri et al. 2006), the bell tower of the Church of Nuestra Sra. de la Misericordia (Valencia, Spain) (Ivorra and Pallarés 2006), the bell tower of the Monza Cathedral (Monza, Italy) (Gentile and Saisi 2007), the Saint Andrea masonry bell tower (Venice, Italy) (Russo et al. 2010), and the bell tower of the Church of Santas Justa and Rufina in Orihuela (Alicante, Spain) (Ivorra et al. 2010).

The researches range from investigation surveys and experimental works (Binda et al. 2005; Ivorra et al. 2009; Anzani et al. 2010; Russo et al. 2010; Bartoli et al. 2013) to dynamic identification studies (Ivorra and Pallarés 2006; Gentile and Saisi 2007; Ramos et al. 2010; Pieraccini et al. 2014). With respect to the experimental activities, in some cases, starting from a field survey of the actual configuration including (if present) the cracking pattern, nondestructive tests (e.g., dynamic tests, sonic pulse velocity tests, thermography, etc.) and/or slightly destructive tests (e.g., flat-jack tests, coring hole, etc.) as well as laboratory tests on cored samples are executed. The aim of the experimental tests is to assess quantities to be subsequently used in the tuning of numerical models employed to evaluate the vulnerability of the tower (usually performed through nonlinear analyses).

The nonlinear analysis approaches differ according to (1) the type of employed method of analysis: nonlinear static (pushover) and/or time-history (Casolo 1998; Bernardeschi et al. 2004; Girardi et al. 2010; Peña et al. 2010; Milani et al. 2012; D'Ambrisi et al. 2012; Casolo et al. 2013; Salvatori et al. 2015). The pushover methods comprise both standard (Milani et al. 2012; Casolo et al. 2013) and multimodal approaches. Recently, Peña et al. (2010), comparing nonlinear static and time-history analyses, showed that differences in the results between the two analysis methods arise due to the changes in the dynamic properties of the tower during the damage process; and (2) the employed modeling technique. Commonly, to have a realistic insight into tower weakness and seismic vulnerability, the numerical model is built by employing the FE technique. In this case, the FE modeling differs according to the level of complexity and of geometric discretization, varying from one-dimensional to three-dimensional (3D) models (Bernardeschi et al. 2004; Carpinteri et al. 2006; Girardi et al. 2010). Simplified monodimensional approaches have been additionally proposed through no tension material approaches (Lucchesi and Pintucchi 2007) or through equivalent Bouc and Wen hysteretic models employed to account for spatial material randomness (Facchini and Betti 2014, 2015).

Recently, comparative studies were performed by Casolo et al. (2013) on 10 masonry towers in the costal Po Valley (Italy); in this case full nonlinear dynamic analyses were performed on two-dimensional discretization aimed at understanding the effect of the geometry on the seismic behavior. Some failure mechanisms are shown for all the investigated towers, which can also be used productively in other studies. Furthermore, the authors show that while simplified analyses can provide reasonable predictions of the towers' seismic risk, sophisticated analyses (i.e., able to predict the failure mechanisms and to assess the areas that undergo severe damages) are required in order to design proper retrofitting. The interest for the topic has been increased after the May 2012 Emilia Romagna (Italy) earthquake sequence; as an example, the behavior of the tower of Finale Emilia was studied by Acito et al. (2014) to understand the causes of the collapse.

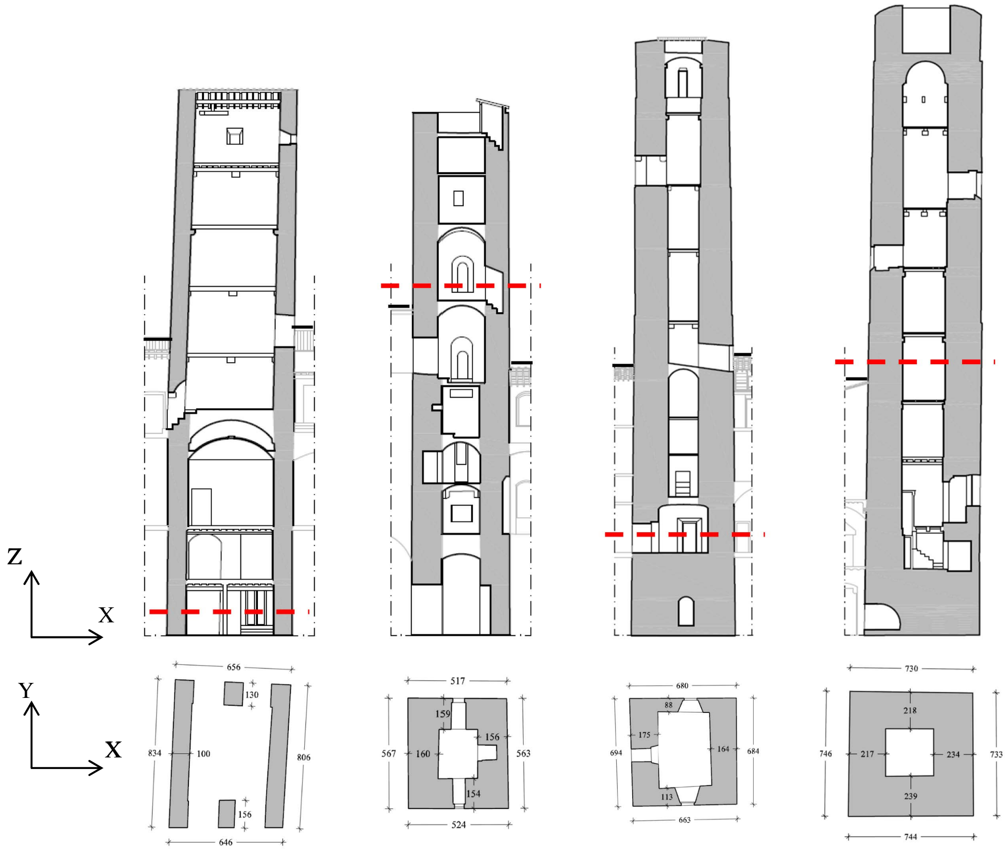
All the preceding researches agree that the slenderness is one of the key parameters that rules the tower seismic vulnerability. Such an easily evaluated parameter is effective from a qualitative point of view or when used for comparative purposes only; nevertheless, its knowledge only is scarcely helpful when a quantitative assessment of the structural behavior is needed. For this reason, different procedures and methodologies have been proposed to obtain a deeper insight into the problem from both a theoretical and a technical point of view.

Historic Towers of San Gimignano

The RiSEM project was aimed at developing and testing expeditious and innovative methodologies (i.e., without direct contact with the masonry construction) to assess the basic structural data needed for the subsequent evaluation of the seismic risk. The methodology adopted in the research was based on the following elements: (1) assessment of seismic hazard and soil-structure interactions; (2) acquisition of the geometric characteristics and reconstruction of the historical evolution of masonry buildings; (3) evaluation of the static and dynamic behavior of structures through nonconventional and innovative investigation techniques; (4) evaluation of seismic vulnerability (through the definition of proper limit states aimed at identifying the safety levels for cultural heritage, considering both the problem of preservation and safety); and finally (5) evaluation of the seismic risk. As an illustrative case study within the research project, the seismic risk of the historic towers of San Gimignano (a small town between Florence and Siena in Tuscany, Italy; Fig. 1), included in the list of UNESCO World Heritage Sites, was analyzed. In its period of maximum splendor San Gimignano is supposed to have had over 70 tower houses (some as high as 50 m). Today, only 13 of these towers have survived.

The survived towers date back to the 12th and 13th centuries. The sustaining walls of the towers are multileaf stone masonry walls with the internal and external faces usually made by stone masonry; the thick internal core is composed of heterogeneous stone blocks tied by a good mortar. Among the analyzed towers, the paper compares the results obtained with respect to four of these. Brief descriptions of the considered towers follow:

- The Coppi-Campatelli Tower (CC; Fig. 2). CC tower represents a typological example of the constructive prototype called *casa torre* (literally *tower house*) that began appearing in San Gimignano during the the 12th and 13th centuries. The tower today is incorporated into an architectural complex that has developed along the medieval entrance in San Gimignano (Via San Giovanni), near the Becci and the Cugnanesi towers. The CC complex is composed of three main buildings: (1) the tower house in Pisan style with two fornices (i.e., vaulted openings first introduced in the Italian town of Pisa), subsequently raised to obtain the present tower; (2) the building on its right (Fig. 2), still in Pisan style, with a single vaulted arch; and (3) a main building on its left, characterized by a portico with four openings in stone blocks on the ground floor and another lower level with vaulted ceilings in the basement. Two more buildings were added on the back façade. With respect to the geometric profile, the main dimensions of the tower are as follows: At street level, the tower has an almost trapezoidal plan, with two equal sides spanning about 6.6 m, while the other two measure about 7.8 m (South side) and about 8.3 m (North side), respectively. The walls' thickness ranges between 1.5 m at the base and 1.0 m at the top. With respect to Via San Giovanni, the tower height is about 27.6 m; on the opposite side, due to the slope of the hill,



		Coppi-Campatelli	Chigi	Becci	Cugnanesi
H	m	27.6	26.9	39.4	42.8
S	m	~ 6.5×8.1	~ 5.2×5.6	~ 6.6×6.8	~7.6×7.5
λ_x	-	4.2	5.2	5.9	5.6
λ_y	-	3.4	4.8	5.8	5.7

Fig. 3. Towers sections and cross sections (H = height; S = base section dimension; $\lambda_{x,y}$ = slenderness)

the height is about 33.0 m (this can be considered the height of the tower with respect to the foundation level). A vertical section of the tower, together with the cross section, is shown in Fig. 3. Internally, the tower is divided into six levels, plus an underground level. The ground floor, the first floor, and the second floor are made up of wooden elements. The third floor is a barrel vault (probably the first intervention during the raising of the tower). The fourth, fifth, and sixth levels are wooden walkways that, through a series of wooden stairs, allow the access to the last level. This level, the seventh, is a wooden floor. From a structural point of view, some specific features of the tower worth highlighting are as follows: (1) a tilt of its axis of about 0.7–0.8 m in the southern direction; (2) the presence in the basement section of the two fornices (West and East sides, Fig. 3); and (3) the incorporation of the lower part of the tower, for about half of its height, into the aggregated buildings along Via San Giovanni (Bartoli et al. 2016);

- The Chigi Tower (CH; Fig. 2). A specific feature that differentiates the CH tower from the others is the use of two different materials: the lower part was built with stones, while the upper part was built with masonry bricks. As in the Coppi-Campatelli Tower, this one is currently incorporated (along the North, South, and East sides) into an architectural complex facing the main square of San Gimignano, a few meters away from the Rognosa, the Salvucci, and the Torre Grossa towers. The entire complex is the result of changes that occurred over the centuries; first of all, the construction of the confining buildings, and later the reshaping of the walls to extend the internal space. The tower was probably built in the second half of the 13th century, as is possibly confirmed by the presence of a sophisticated façade with openings of different shapes, such as single-lancet windows and segmental arches. Originally, it was separate from the other constructions and after about 50 years, the palaces on the South and East side and later the one on the North side were built. CH tower presents a square cross section with an external side of about 5.5 m and height of about 27 m. The sustaining walls have a variable thickness: from 1.6 m at the base up to 1.3 m at the highest level. The construction technique is again that of multilayer masonry walls: two facing walls with internal filling material and, possibly, several pinning. The lower part is formed by multilayer walls with both external and internal faces composed of 0.3-m-thick stone masonry (mainly travertine); the internal filling, of unknown mechanical properties, is composed of heterogeneous material (brick and stone tied by a poor mortar) and it appears, where it was possible to investigate, coherent. From a height of 13 m on, the multilayer walls are composed of internal and external facings of brick with a thickness of 0.25 cm, and an internal filling with the same properties of those forming the lower part of the tower. The dimensions of the prismatic blocks, used for the masonry walls, are about $50 \times 30 \times 30$ cm for the stone and $12.5 \times 30 \times 5.5$ cm for the bricks. For all the types of masonry walls, the mortar joints' thickness is minimal, thus producing excellent mechanical properties. The tower is surrounded by masonry buildings of different levels (about 17 m for the South side and about 14 m on the North and East sides). Only the main façade on the West side is unconstrained, due to the absence of other buildings. The tower's internal floors consist of masonry vaults, except for the fourth, the seventh, and the eighth floors, which are made of wooden slabs;

- The Cugnanesi Tower (CU; Fig. 2). CU tower, located in the heart of the city, is connected to two adjacent masonry buildings at the lower levels along its North side. The total height of the tower is about 42.8 m (Fig. 3). The base is a compact square parallelepiped made of stones with a side length of about 7.6 m and a height of about 5.2 m. On the lateral surface of the base, along the North side, a little hole has been dug in correspondence to an adjacent commercial activity. The inspection of such hole allowed verifying that the inner part of the parallelepiped is composed of a thoroughly mixed conglomerate (medium-size stones with good quality mortar). Above this block, the four lateral sustaining walls gradually decrease their thickness from about 2.4 m at the base to 1.9 m at the top of the structure. A visual analysis of the tiny cavities of the walls along the height reveals their multileaf structure: an internal core of heterogeneous stone blocks tied by a good mortar is confined by two external cavernous limestone masonry layers. Along the tower, seven light timber floors exist, whose influence on the global structural behavior can be neglected. The last level of the tower is composed of a concrete slab sustained by a barrel vault. It is noteworthy to underline that all the tower structural elements appear to be in a good state of conservation and no crack pattern has been visually detected on the structure; and
- The Becci Tower (BE; Fig. 2). BE tower dates back to the 13th century: It overlooks the central Piazza della Cisterna (literally *tank square*) and, at the lower levels with the exception of the South side, it is incorporated in adjacent buildings built in a later period. From a structural point of view the tower has a sufficiently regular geometry. Most parts of the few openings have limited dimensions, the only exceptions can be observed in relation to some zones connecting the tower with the adjacent buildings where larger size openings were created. The external dimensions of the tower, together with the thickness of the wall, were obtained by a geometric survey: the overall height is about 38 m and the cross section has a slight taper to the upper levels. At the level of Via San Giovanni, the section sizes are as follows: 6.7 m on the North side, 6.8 m on the East side, 6.6 m on the South side, and 6.9 m on the Western side. At the last level, these dimensions become 6.2, 6.3, 6.2, and 6.4 m, respectively. The wall section does not present discontinuities along the vertical development and masonry walls are multileaf (two stone external faces and an inner core). It is possible to assume that the external stone layer has a thickness of about 0.4 m, while the internal layer is about 0.25 m; the inner core has an average thickness of about 1.6 m.

A summarizing section of the four analyzed towers, together with the base cross section, is shown in Fig. 3. Slenderness of the towers ranges between 3.4 (CC tower) and 5.9 (BE tower); the thickness of the walls is almost uniform for three towers, while a reduced thickness is observed for the CC tower. At the lower levels, the towers are largely incorporated into the neighboring buildings, and hence the lower sections present several openings (in most cases subsequent to the tower construction) to allow communication with the confining buildings.

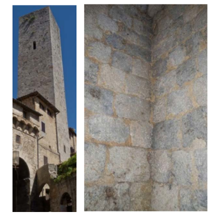
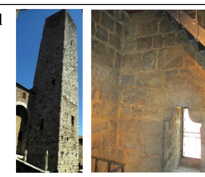
Tower	Masonry typology	Tower walls	View
BE	**D**: Soft stone masonry (SSM).	internal and external faces similar / thick inner core / good quality mortar.	
CC	**B**: Uncut stone masonry with facing walls of limited thickness and infill core (USM). **E**: Dressed rectangular stone masonry (DRS).	internal and external faces similar / thick inner core / good quality mortar.	
CH (lower part)	**D**: Soft stone masonry (SSM).	internal and external faces similar / thick inner core / good quality mortar.	
CH (upper part)	**F**: Full brick masonry with lime mortar (FBM).	internal and external faces similar / thick inner core / good quality mortar.	
CU	**E**: Dressed rectangular stone masonry (DRS).	internal and external faces similar / thick inner core / good quality mortar.	

Fig. 4. Visual characterization of masonry typologies (images by authors)

The internal and external faces of the towers are made by a local cavernous limestone except for the upper part of the CH tower, which was built with masonry bricks (Fig. 2). Since only visual inspections were allowable, no tests (such as core drillings, mineralogical surveys, etc.) have been performed; the mechanical properties of the walls have been characterized by taking into account the provisions of the Italian recommendations (MIT 2009). In particular, four typologies of masonry textures were considered (refer also to Fig. 4): (1) type B: uncut stone masonry (USM) with facing walls of limited thickness and infill core; (2) type D: soft stone masonry (SSM) (tuff, limestone, etc.); (3) type E: dressed rectangular stone masonry (DRS); and (4) type F: full brick masonry (FBM) with lime mortar (used for characterizing the upper part of the CH tower only). The reference intervals for the value of the mechanical properties adopted by MIT (2009) were selected according to the mechanical characteristics of the masonry typologies existing in the Italian territory. These values refer to masonry with mortar of poor mechanical characteristics; some correction factors can be introduced accounting for increasing mechanical characteristics (due to possible good quality mortar and thin joints) and decreasing ones (thick or poor internal core). When only a limited level of knowledge [denoted as KL1 according to the NTC (2008) classification] is reached, minimum values for resistance parameters (uniaxial compressive strength f_m and characteristic shear strength τ_0) and average values for elastic moduli (Young's modulus E and shear modulus G) were used. According to the Italian Recommendation (MIT 2009) and in the absence of more accurate investigations, the

Table 1. Mechanical Properties according to MIT (2009) (Data from MIT 2009)

	Mechanical characteristics			Correction factors		
Type of masonry	f_m (N/mm^2)	τ_0 (N/mm^2)	E (N/mm^2)	Thick or poor internal core	Good quality of the mortar	Thin joints
B	2.00	0.035	1,020	0.8	1.4	1.2
	3.00	0.051	1,440			
D	1.40	0.028	900	0.9	1.5	1.5
	2.40	0.042	1,260			
E	6.00	0.090	2,400	0.7	1.2	1.2
	8.00	0.012	3,200			
F	2.40	0.060	1,200	0.7	1.5	1.5
	4.00	0.092	1,800			

Note: E = modulus of elasticity, correction factor corresponding to (1) thick or poor internal core, (2) good quality of the mortar, and (3) thin joint; f_m = compressive strength; τ_0 = shear strength.

correction factors shown in Table 1 have been applied as multiplicative terms to the mechanical parameters.

The performed estimation of the masonry mechanical parameters through literature results was in accordance with the project goals, aiming at testing expeditious techniques to assess the seismic risk of monumental buildings without direct contact with the masonry construction. Then properties for the selected types of masonry, with similar morphology to those visually detected in situ, were assumed as the lower and upper bound for the actual masonry parameters to be employed in the subsequent analysis models.

Risk Assessment

The seismic assessment of the towers was developed according to the provisions of the Italian Guidelines for the assessment and mitigation of the seismic risk of the Cultural Heritage (DPCM 2011). The Guidelines, which represent an innovative tool in the European context, propose an assessment methodology organized on three levels:

- Level 1 (LV1) is a territorial risk level, in which the input is represented by the macroseismic intensity parameters and the vulnerability is evaluated according to a qualitative knowledge of the relevant structural parameters. The safety indexes are based on typological studies, related to the kind of the building (palace, church, tower), at a territorial scale.
- Level 2 (LV2) is a local mechanical risk level, in which the spectral coordinates of the earthquake represent the input and the vulnerability is evaluated by analyzing the activation of partial collapse mechanisms in single parts of the structure (macroelements). The evaluation of the safety indexes still requires a few geometrical and mechanical parameters.
- Level 3 (LV3) is a global mechanical risk level, in which the spectral coordinates of the earthquake represent the input and the vulnerability is evaluated performing nonlinear analyses (through a capacity curve if a pushover approach is employed; by means of earthquake records if a time-history analysis is used). The model asks for a detailed analysis of the single building, considered as a whole (or as an assembly of macroelements).

Among the three levels of analysis, the paper reports, for 4 out of the 13 analyzed towers of San Gimignano, the results obtained with models related to LV1 and LV3 as both approaches are aimed to assess a global structural behavior of the structure. The two levels are compared through the examination of two safety indexes, evaluated with reference to the life safety limit state (SLV). The first index is the seismic safety index ($I_{S,\mathrm{SLV}}$), the ratio between the return period of the seismic action that brings the tower to the life safety limit state (T_{SLV}) and the expected return time of the earthquake of the site, corresponding to the life safety limit state (usually, as a reference, it can be assumed $T_{R,\mathrm{SLV}} = 475$ years), defined as follows:

$$I_{S,\mathrm{SLV}} = \frac{T_{\mathrm{SLV}}}{T_{R,\mathrm{SLV}}} \tag{1}$$

A seismic safety index greater than 1 corresponds to a safe state for the tower; a safety index lower than 1 highlights possible critical issues thus requiring in-depth investigations.

The second examined index is the acceleration factor ($f_{a,\mathrm{SLV}}$), the ratio between the acceleration that brings the tower to the life safety limit state (a_{SLV}) and the reference acceleration for the life safety limit state ($a_{g,\mathrm{SLV}}$), both referred to a rigid soil condition:

$$f_{a,\mathrm{SLV}} = \frac{a_{\mathrm{SLV}}}{a_{g,\mathrm{SLV}}} \tag{2}$$

The acceleration factor, while considering only one of the parameters that defines the seismic action spectrum, has the advantage of providing a quantitative indication of any deficiency in terms of mechanical strength of the structural system, as the acceleration factor $f_{a,\mathrm{SLV}}$ is a purely mechanical parameter. The seismic safety index $I_{S,\mathrm{SLV}}$, as based on the return periods of the seismic demand and of the capacity of the structure, provides a direct evaluation of the possible vulnerability of the tower over time.

The indexes were evaluated according to the two principal directions of each section of the towers, not (usually) being possible to identify in advance the most critical section.

LV1 Analyses

The simplified LV1 approach proposed by DPCM (2011) is aimed at evaluating the seismic risk of monumental buildings at a territorial level. It aims at evaluating the collapse acceleration of the structures based on a limited number of geometrical and mechanical parameters (or qualitative tools such as visual tests, construction features, and stratigraphic surveys). It is hence mainly aimed at evaluating a comparative ranking risk between similar structures in order to highlight the need for subsequent in-depth investigations with LV2 and LV3 approaches. Results of seismic vulnerability at the territorial level are intended as useful tools for the public administration for highlighting the most critical situations in the territory and for establishing priorities for future interventions. It is implicitly assumed that lower LV1 safety indexes actually correspond to lower safety indexes in case of refined LV3 analyses.

In case of masonry towers, the LV1 approach foresees analyzing the tower as a cantilever beam, subject to a system of horizontal forces, assuming that the collapse can occur according to a combined compressive and bending stress mode. From the operative point of view, the structure is subdivided into n sectors having uniform geometrical and mechanical characteristics. This subdivision is performed by taking into account several aspects, such as (1) the beginning and ending of the openings; (2) the level of detachment of the tower from the neighboring buildings (if the tower is not isolated); (3) the levels at which there is a reduction in the thickness of the masonry walls; and (4) the levels where there are changes of materials and/or changes in the construction techniques. Sections where safety checks are performed correspond to each sector into which the tower is subdivided. Afterward, the safety checks are carried out by comparing, for each sector and for each load direction, the seismic capacity (the correspondent ultimate resistant moment) with the seismic demand (the acting bending moment).

The ultimate bending moment at the base of the ith sector, under the hypothesis that the compressive stress does not exceed $0.85 \cdot f_d$, (f_d, design compressive strength) is evaluated through the following expression:

$$M_{u,i} = \frac{\sigma_{0i} \cdot A_i}{2} \left(b_i - \frac{\sigma_{0i} \cdot A_i}{0.85 \cdot a_i \cdot f_d} \right) \qquad (3)$$

where a_i and b_i = transversal and longitudinal length of the ith sector with respect to the considered seismic load direction, respectively; A_i = section area of the ith sector (cleared of the openings); and σ_{0i} = vertical compressive stress in the analyzed section ($\sigma_{0i} = W_i/A_i$, W_i being the weight of the portion of the tower beyond the analyzed section). The design compressive strength, f_d, has been assumed according to values reported in Table 1.

The evaluation of the acting bending moment requires the estimation of the ordinate of the elastic response spectrum $S_e(T_1)$, which is a function of the main period T_1 of the tower. The main period is thus a fundamental parameter that needs to be assessed and that drives the results of the seismic risk analyses. The Italian Building Code (NTC 2008) provides the following empirical correlation to estimate such a period:

$$T_1 = 0.050 \cdot H^{0.75} \qquad (4)$$

Rainieri and Fabbrocino (2011) showed that Eq. (4) tends to overestimate the natural period for values less than 1 s, while tends to underestimate the actual period for values greater than 1 s when used for slender masonry towers. On the basis of experimental results concerning the main periods of historic masonry towers, they proposed the following empirical correlation, used here for the estimation of the fundamental period:

$$T_1 = 0.013 \cdot H^{1.10} \qquad (5)$$

Eq. (5), as the empirical correlation provided by the Italian Code, provides the main period of the structure as a function of the height H of the tower only. For comparative purposes, the main period of the towers was also estimated by employing the classical formula of the linear elasticity for a cantilever beam (Clough and Penzien 2003):

$$T_1 = 1.787 \cdot H^2 \cdot \sqrt{\frac{\gamma \cdot A}{E \cdot J \cdot g}} \qquad (6)$$

where A = base cross-sectional area; γ = specific weight; E = modulus of elasticity and J denotes the base area moment of inertia along the analyzed load direction.

The Italian Guidelines require, in order to account for the behavior of the structure at the ultimate limit state (i.e., to consider the nonlinear phenomena that occur as a result of the increasing levels of damage induced by the seismic loads) to amplify the linear elastic period T_1 by a coefficient that can vary between 1.40 and 1.75. The main periods T_1 evaluated by Eqs. (4)–(6) were then amplified with a factor equal to 1.40 to obtain a period T_1^* representative of the damage phenomena induced by seismic loads at the ultimate limit state.

Since the towers (Fig. 1) are largely incorporated into the neighboring buildings at the lower levels, in order to apply the LV1 model, the following schemes were considered:

- Model A: The towers are analyzed as isolated constructions; i.e., without considering the presence of the neighboring buildings (it is implicitly assumed that the action offered by the neighboring structures is ineffective or that it can be lost during severe earthquakes).
- Models B and C: The towers are still assumed as isolated constructions, but the tower height has been made equal to the portion of the structure emerging from surrounding buildings (depending on the different height of the neighboring buildings, different models B and C were considered accounting for the different emerging tower heights).

The previous models aimed at introducing lower and upper bounds. The presence of adjacent lower constructions can significantly alter the towers' structural behavior and the actual behavior of the towers can be supposed to lie within the obtained range. In fact, on the one hand, the confining buildings reduce the effective slenderness (thus reducing the period); on the other hand, these constructions constitute stiffeners that might produce localized areas of possible stress concentration (and pounding). The estimated main periods of the four towers for the three models A, B, and C are summarized in Table 2.

Results of the LV1 analyses are summarized in Table 3 (Chigi tower, CH), Table 4 (Coppi-Campatelli tower, CC), Table 5 (Cugnanesi tower, CU) and Table 6 (Becci tower, BE), in terms of acceleration factor $f_{a,\mathrm{SLV}}$, seismic safety index $I_{S,\mathrm{SLV}}$, and return period of the action causing the tower collapse T_R.

It should be noted that due to the low variation of the geometrical characteristics along the height, in all the analyzed cases the minimum spectral capacity acceleration (and hence the lowest values for the safety indexes) has been obviously obtained in correspondence of the base section of each considered model (A, B, or C).

The LV1 results show that no critical situations are detected [although, in the case of the Chigi tower, the A model with the main period evaluated according to Eq. (5) without the amplification factor 1.40 provides an acceleration factor slightly less than one]

Table 2. Main Periods of the Towers (Empirical Correlations and Analytical Expression)

		CH			CC			BE			CU		
Model	Equations	H (m)	T_1 (s)	T_1^* (s)	H (m)	T_1 (s)	T_1^* (s)	H (m)	T_1 (s)	T_1^* (s)	H (m)	T_1 (s)	T_1^* (s)
A	(4)	26.9	0.59	0.83	33.1	0.69	0.97	39.4	0.79	1.10	42.8	0.84	1.17
	(5)		0.49	0.68		0.61	0.85		0.74	1.04		0.81	1.13
	(6)		0.63	0.88		1.09	1.52		1.33	1.86		1.23	1.72
B	(4)	13.4	0.35	0.48	27.6	0.60	0.84	17.4	0.43	0.60	27.8	0.61	0.85
	(5)		0.22	0.31		0.49	0.69		0.29	0.41		0.50	0.70
	(6)		0.16	0.22		0.89	1.05		0.28	0.39		0.66	0.93
C	(4)	10.5	0.28	0.40	13.4	0.35	0.49	15.5	0.39	0.55	26.1	0.58	0.81
	(5)		0.17	0.23		0.23	0.32		0.26	0.36		0.47	0.65
	(6)		0.08	0.11		0.75	0.20		0.22	0.31		0.58	0.82

Note: $T_1^* = 1.4 \cdot T_1$.

Table 3. CH LV1 Safety Indexes

CH	Direction	T_1^* (s)	$f_{a,\mathrm{SLV}}$	$I_{S,\mathrm{SLV}}$	T_R (years)
Model A	N-S (X)	0.68	1.01	1.03	487
		0.88	1.29	2.29	1089
	E-W (Y)	0.68	1.08	1.30	618
		0.85	1.38	2.95	1399
Model B	N-S (X)	0.22	>1.60	>5.21	>2,475
		0.48	>1.60	>5.21	>2,475
	E-W (Y)	0.21	>1.60	>5.21	>2,475
		0.48	>1.60	>5.21	>2,475
Model C	N-S (X)	0.11	>1.60	>5.21	>2,475
		0.40	>1.60	>5.21	>2,475
	E-W (Y)	0.11	>1.60	>5.21	>2,475
		0.40	>1.60	>5.21	>2,475

Table 4. CC LV1 Safety Indexes

CC	Direction	T_1^* (s)	$f_{a,\mathrm{SLV}}$	$I_{S,\mathrm{SLV}}$	T_R (years)
Model A	N-S (X)	0.85	1.38	1.58	749
		1.52	>1.60	>5.21	>2,475
	E-W (Y)	0.85	1.23	1.09	517
		1.25	>1.60	>5.21	>2,475
Model B	E-W (Y)	0.69	1.31	1.34	637
		0.87	1.58	2.51	1,195
Model C	N-S (X)	0.25	1.85	4.54	2,158
		0.32	1.85	4.54	2,159
	E-W(Y)	0.20	>1.60	>5.21	>2475
		0.32	>1.60	>5.21	>2475

Table 5. CU LV1 Safety Indexes

CU	Direction	T_1^* (s)	$f_{a,\mathrm{SLV}}$	$I_{S,\mathrm{SLV}}$	T_R (years)
Model A	N-S (X)	1.13	1.34	2.66	1,260
		1.95	>1.60	>5.21	>2,475
	E-W (Y)	1.13	1.34	2.66	1,260
		1.99	>1.60	>5.21	>2,475
Model B	N-S (X)	0.70	1.57	4.81	2,283
		0.85	>1.60	>5.21	>2,475
	E-W (Y)	0.70	1.57	4.81	2,283
		0.85	>1.60	>5.21	>2,475
Model C	N-S (X)	0.65	1.56	4.74	2,251
		0.81	>1.60	>5.21	>2,475
	E-W (Y)	0.65	1.56	4.74	2,251
		0.81	>1.60	>5.21	>2,475

Table 6. BE LV1 Safety Indexes

BE	Direction	T_1 (s)	$f_{a,\mathrm{SLV}}$	$I_{S,\mathrm{SLV}}$	T_R (years)
Model A	N-S (X)	1.04	1.07	1.26	600
		1.86	>1.60	>5.21	>2,475
	E-W (Y)	1.04	1.03	1.10	521
		1.94	>1.60	>5.21	>2,475
Model B	N-S (X)	0.39	0.77	0.91	368
		0.60	1.36	2.85	1,355
	E-W (Y)	0.40	0.96	0.89	425
		0.60	1.40	3.17	1,507
Model C	N-S (X)	0.31	0.96	0.89	425
		0.55	1.36	2.86	1,359
	E-W (Y)	0.32	0.98	0.96	454
		0.55	1.39	3.10	1,473

and a general consistency of the results in terms of safety indexes is observed. In addition, it is also possible to observe, with respect to both $f_{a,\mathrm{SLV}}$ and $I_{S,\mathrm{SLV}}$, that the smaller values are obtained with the A models (isolated towers). It is noteworthy that the approach adopted by the Italian Guidelines (DPCM 2011) is conceived employing equivalence with a cantilever masonry beam, where the assumed failure modes are due only to the formation of a flexural hinge at the base. Failure modes as a result of shear failures, local collapse near the top due to lower tower slenderness, presence of irregularities, and high perforations and bell towers, cannot be properly taken into account by the adopted procedure.

LV3 Analyses

The third level of analysis is based on the use of numerical models able to simulate the global structural behavior in order to evaluate the accelerations leading the structure to each analyzed limit state. This level, compared with the previous ones, is more demanding since it requires a deeper knowledge of both the constructive techniques and the structural details, together with the material properties (at least tensile and compressive strength of the materials), to perform a consistent evaluation of the seismic capacity of the building. The reliability of the model, and consequently the obtained results, are closely connected to the level of investigation and the available experimental data. In addition, when the construction is inserted into a context of aggregated buildings, as in the case of the towers of San Gimignano, the identification of the construction and transformation phases (edification of new buildings, raising, internal changes with partial demolition, and/or reconstructions) is a fundamental element of knowledge required to assess the structural continuity of the construction with the surrounding area. These, and other aspects not expressly called up (such as the one connected with the proper reproduction of the masonry postelastic behavior), were addressed in this level of analysis through a parametric investigation aimed at identifying lower and upper bounds of the structural behavior.

Performing an exhaustive literature review of all the contributions regarding both the numerical and the experimental analyses of masonry towers is almost impossible. However, the partial review reported in previous sections shows that the greater part of historic structures has been modeled with a macromodeling strategy (Lourenço et al. 1995). The heterogeneous masonry walls are substituted by an equivalent material with average mechanical properties (either orthotropic or isotropic) globally representing the structural response under increasing loads. In some cases, sophisticated material models have been used, ranging from elastic-plastic with softening to damaging models, which are the only ones suitable to have an insight into the nonlinear behavior of a masonry structure making use of general purpose commercial software.

This paper, in line with the discussed studies, investigates the third level of analysis employing FE models of the towers and performing nonlinear static analyses (pushover). According to the pushover approach, the analyses make use of monotonically increasing uniform profile of horizontal loads, under constant gravity loads. It is significant to point out the conventionality of the pushover approach assumed in the study, as the load profile does not change with the progressive degradation that occurs during loading; thus, it does not account for the progressive changes in modal frequencies due to yielding and cracking on the structure. This is a critical point for the application of conventional pushover to the analysis of historic masonry buildings, because it is predictable that the progressive damage of the building may also lead to period elongation, and therefore to different spectral amplifications and load distributions along the height. However, also in its conventional form, the pushover approach can provide an efficient alternative to more expensive computational inelastic time-history analyses and can offer useful and effective information on the damage that the building can develop under dynamic seismic loads.

The analyses were carried out along the main directions ($+/-$X and $+/-$Y, Fig. 3), and the comparison between results was performed by analyzing the capacity curves (generalized force–displacement relationships). Capacity curves were built by assuming the nondimensional base shear and the displacement of the center of mass of the upper section of each tower as control parameters. The FE models of the Chigi (CH), Becci (BE), and Cugnanesi (CU) towers were built by using the commercial code *ANSYS*, while the FE model of the Coppi-Campatelli (CC) tower was built by using *Code Aster*, an open-source FE code. Two different FE codes were used for comparative purposes in order to account for the different techniques employed to reproduce the masonry inelastic behavior. The numerical models, illustrated in Fig. 5, were built to accurately reproduce the geometry of the structures and to include, when existing, the internal masonry vaults (internal wooden slabs were not modeled). Major openings of the walls of the towers (doors, windows, recesses, etc.) were also reproduced. Two different mechanical laws were employed to reproduce the nonlinear behavior of the masonry:

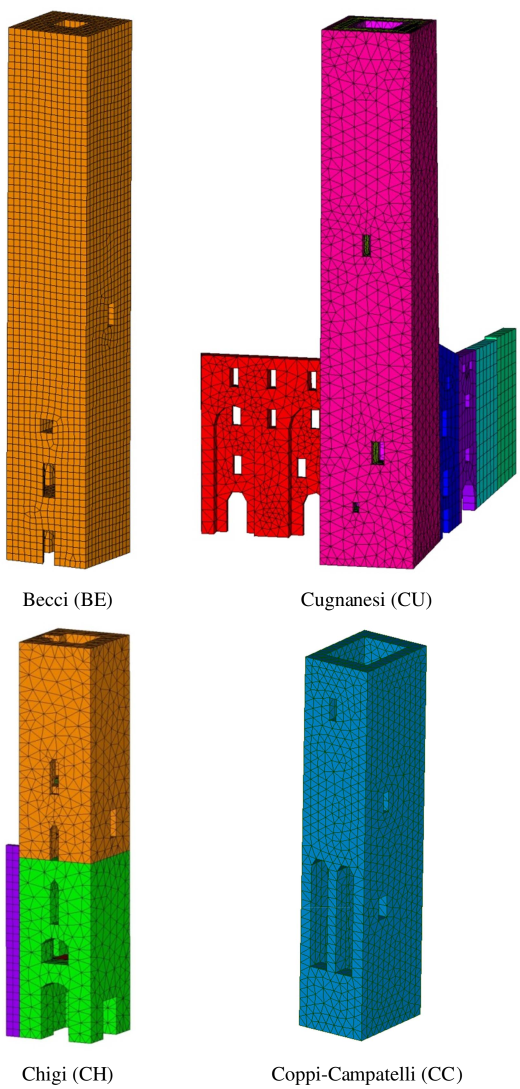


Fig. 5. FE models of analyzed towers

- The continuum damage model introduced by Mazars (1984) and Mazars and Pijaudier-Cabot (1989), for the CC tower FE model.
- The combination of the Drucker-Prager yielding behavior for compressive stresses (f_c) and the Willam-Warnke cracking behavior for tensile stresses (f_t) for the other three towers (CH, BE, and CU); parameters of the two constitutive models have been chosen in order to have a proper coupling between them, as already proposed by several authors (Betti et al. 2016).

As mechanical parameters were evaluated by taking into account existing provisions only, parametric investigations have been developed in order to take into account the variability of the strength parameters. As far as the boundary conditions are concerned, the base of all the FE models of the towers was always supposed to be fixed (due to the high stiffness of San Gimignano soil) and comparative analyses were carried out to account for the effects of the adjacent structures. The interaction with the adjacent buildings was reproduced by modeling the walls perpendicular to the perimeter of the tower and assigning them an appropriate equivalent elastic stiffness. This investigation is quite important since the presence of confining structures can be an effective constraint for the tower but, at the same time, can localize stress concentrations and pounding: The observation of the postearthquake damages clearly shows the different seismic behavior between isolated and constrained (i.e., connected to walls) towers (Cattari et al. 2014). The identified linear and nonlinear parameters are reported in Tables 7 and 8, respectively. For the Chigi tower, a parametric study has been performed in order to check for the influence of the three main parameters involved in the analysis; i.e., the compressive strength (f_c), the tensile strength (f_t), and the Young's modulus (E), according to the values reported in Table 9.

Table 7. Elastic and Mass Parameters of the Investigated Towers

Tower	E (MPa)	γ (kN/m^3)
CC	1,230–2,800	20–22
CH	1,458–2,916	16–18
BE	1,350	16
CU	2,800	22

Note: E = Young's modulus; γ = specific weight.

Table 8. Mechanical (Strength) Parameters of the Investigated Towers

Tower	f_c (MPa)	f_t (MPa)
CC	3.000	0.200
CH	0.493–1.973	0.106–0.212
BE	1.099	0.220
CU	0.729-1.370	0.298

Note: f_c = compressive strength; f_t = tensile strength.

Table 9. Combination of Mechanical Parameters Used for the Analysis of CH Tower (Lower Stone Part)

Material	f_t (MPa)	f_c (MPa)	E (MPa)
CH_{A1}	0.106	0.493	1,458
CH_{A2}		0.493	2,916
CH_{A3}		0.986	1,458
CH_{A4}		0.986	2,916
CH_{B1}	0.212	0.986	1,458
CH_{B2}		0.986	2,916
CH_{B3}		1.973	1,458
CH_{B4}		1.973	2,916

Note: E = modulus of elasticity, f_c = compressive strength, f_t = tensile strength.

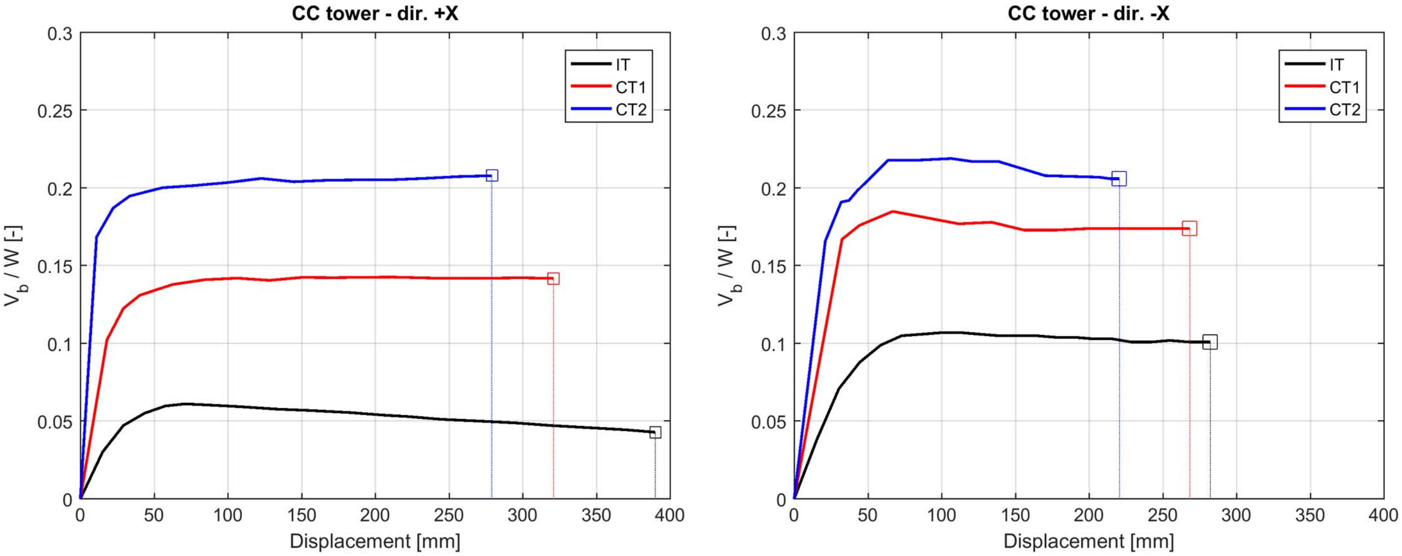


Fig. 6. CC tower: pushover curves along ±X-directions (comparison between IT and CT models)

In the examined scenarios, when the experimental natural frequencies were available, the stiffness of adjacent wall constraints was evaluated in order to reproduce the experimental results; otherwise equivalent criteria were employed by evaluating the stiffness of the confining walls. The following two opposite and complementary cases were then considered:

- Isolated Tower (IT) modeling: the tower is considered as standing alone, without taking into account the interaction with the confining buildings; the IT scenario considers the configuration where the connections with the confining buildings are not effective (i.e., ideally the situation where the tower, in case of earthquake, starts to oscillate, detaching itself from neighboring structures).
- Confined Tower (CT) modeling: the presence of the adjacent buildings (in all the directions) has been taken into account, representing their effects as elastic restraints.

The aim of the two analyzed scenarios is to identify, analogously to the LV1 analyses, lower and upper bounds for the towers' structural behavior. Accordingly, main results and the employed methodology are subsequently discussed:

- The Coppi-Campatelli (CC) tower: The FE model of CC tower (Fig. 5) was built by using the FE program *Code Aster*. To reproduce the masonry nonlinear behavior the continuum damage model by Mazars (1984; Mazars and Pijaudier-Cabot 1989) was chosen. The parameters required by the damage model were selected in order to fit the limit scheme of dressed rectangular stone (DRS) masonry and the cases of isolated (IT model) and confined (CT1 and CT2 models) towers were analyzed. The interaction with the adjacent buildings was reproduced by modeling the walls perpendicular to the perimeter of the tower, and assigning them an appropriate elastic stiffness evaluated according to elastic equivalence criteria (CT1 model). As a further case, and for comparative purposes, an additional model was taken into account, where a stiffness value equal to 10 times the previous one was employed (CT2 model). The pushover curves with respect to the case of loading acting in $+/-$X (North-South) directions are shown in Fig. 6 for the case of isolated and confined towers. With respect to the $+$X direction, the maximum nondimensional base shear range between 0.061 (IT) and 0.208 (CT2). In terms of ultimate displacement the two restrained cases show comparable values (0.25–0.35 m), still lower than those obtained in the case of the isolated tower (about 0.40 m). When the $-$X-direction is analyzed, it is possible to observe that the maximum nondimensional base shear range is from 0.107 (IT) to 0.219 (CT2); in terms of ultimate displacement, the two restrained cases and the isolated case show comparable values (0.20–0.25 m), which are still lower than those obtained in the case of isolated tower (about 0.40 m). The pushover curves were employed to characterize the equivalent bilinear single-degree-of-freedom (SDOF) oscillator to be used to perform seismic checks. Table 10 reports the obtained safety indexes (acceleration factor, seismic safety index, and return period T_R). The LV3 analyses do not emphasize any critical situations, providing safety indexes higher than and in agreement with those obtained with the LV1 approach; it is interesting to observe that the acceleration factors obtained with the LV3 model are always higher than those evaluated at LV1; an exhaustive discussion of the CC seismic risk is reported in Bartoli et al. (2016).
- The Chigi tower (CH): The FE model of the CH tower (Fig. 5) was built by using the commercial code *ANSYS*. A 3D model was defined to accurately reproduce the geometry, and the analyzed configurations are reported in Table 9. The seismic behavior of the tower, as for the CC tower, was analyzed by a pushover approach considering all the seismic directions (±X, corresponding to North-South, and ±Y, corresponding to East–West). The equivalent stiffness of the adjacent buildings was calculated by equating the displacements of the upper end of the confining wall, when subjected to a reference force, with the corresponding displacement of elastic boundary constraints

Table 10. CC Tower: LV3 Safety Indexes

CC	Model	T^* (s)	$f_{a,\mathrm{SLV}}$	T_R (years)	$I_{s,\mathrm{SLV}}$
Direction South (+X)	IT	1.54	2.21	>2,475	—
	CT1	0.84	4.40	>2,475	—
	CT2	0.51	7.63	>2,475	—
Direction North (−X)	IT	1.34	3.08	>2,475	—
	CT1	0.89	4.33	>2,475	—
	CT2	0.72	4.38	>2,475	—
Direction East (+Y)	IT	1.07	2.87	>2,475	—
	CT1	0.84	1.33	625	1.32
	CT2	0.47	1.57	992	2.09
Direction West (−Y)	IT	0.91	1.79	1,519	3.20

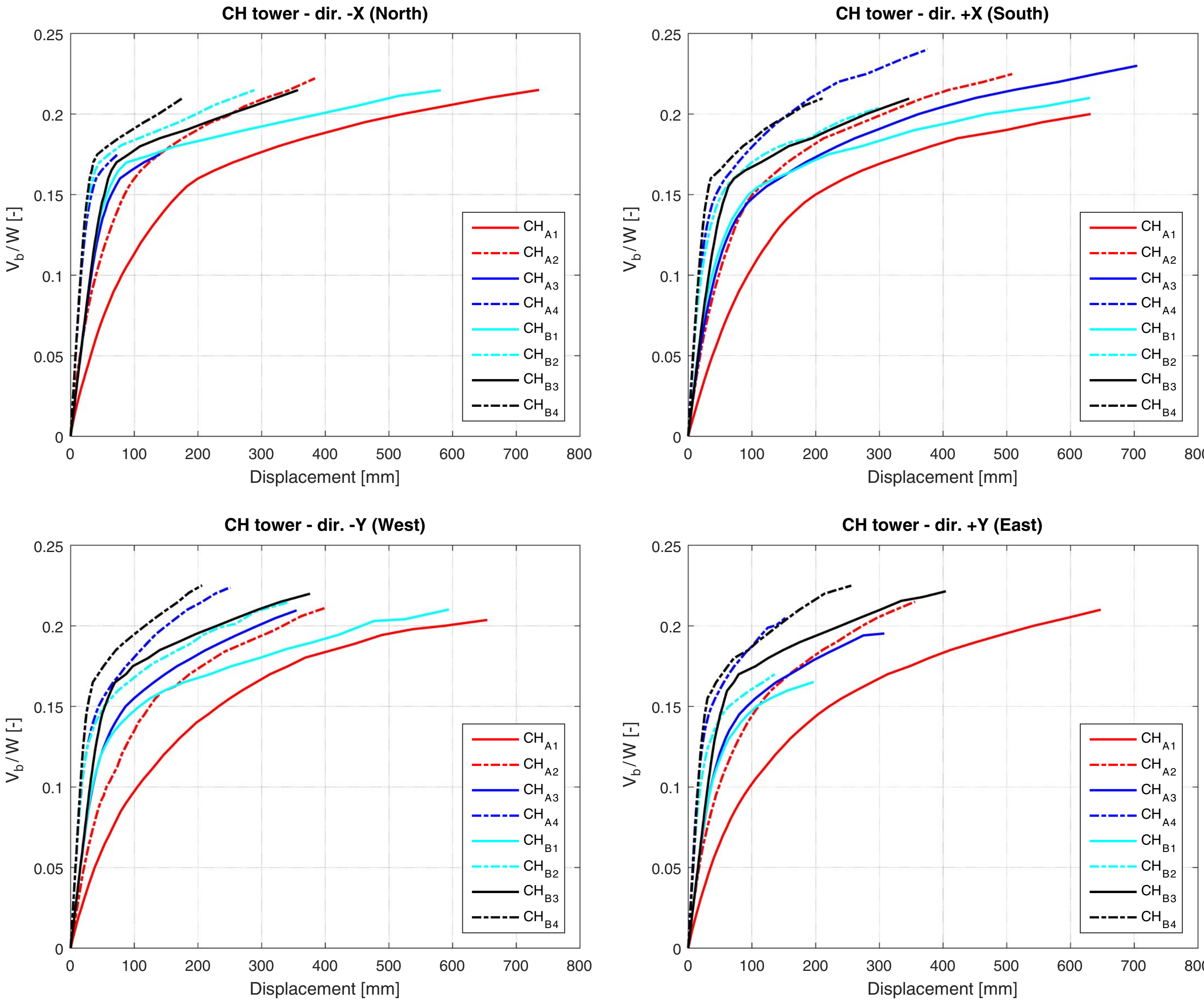


Fig. 7. CH tower: pushover curves for IT models along ±X and ±Y-directions

(taking into account both flexural and shear deformations). The equivalent elastic modulus of the confining walls thus obtained was employed to model the boundary elements (CT1 model). Due to the low trustworthiness of the obtained value, and in order to make some considerations about the reliability of the obtained results, a further case has been taken into account, where an equivalent elastic modulus equal to 10 times the previous one has been adopted (CT2 model). The first case (CT1) aims to analyze a weak interaction configuration, while the second one (CT2) analyzes a stronger interaction between the tower and the confining structures. The obtained pushover curves are reported in Fig. 7 for the case of IT. It is possible to observe that, within the analyzed mechanical configurations reported in Table 9, in almost all the four directions the nondimensional base shear is about the same, ranging between 0.16 (+Y-direction, material parameters CH_{B1}), and 0.24 (+X-direction, material parameters CH_{A4}). A great variability was instead observed with respect to the ultimate displacement that ranges between 0.06 m (−X-direction, material parameters CH_{A4}) and 0.74 m (−X-direction, material parameters CH_{A1}). The capacity curves of the confined case (CT1 model) are reported in Fig. 8 with respect to the ±X-directions. Also in this case it is possible to observe that while the nondimensional base shear remains almost the same, great variations are on the ultimate displacement that range between 0.03 m (−X-direction, material parameters CH_{A2}) and 0.12 m (+X-direction, material parameters CH_{B3}). The obtained pushover curves were employed to build the equivalent bilinear SDOF oscillator response curve to be used when performing the seismic checks, in order to evaluate the acceleration factor, the seismic safety index, and the return period T_R. Table 11 reports the safety indexes obtained for the case CH_{A4} (Table 9). On the whole, LV3 analyses have not highlighted critical situations: all the index values are higher than unity and the results obtained at the level of investigation LV3 are in agreement with those related to LV1 approach.

- The Cugnanesi tower (CU): The FE model of the CU tower (Fig. 5) has been defined by using the commercial code *ANSYS*. Masonry walls were modeled by means of *Solid65* elements, and two types of materials were considered: type E [DRS] and type B [USM]. The values of mechanical parameters to be used in the nonlinear numerical models have been chosen after a

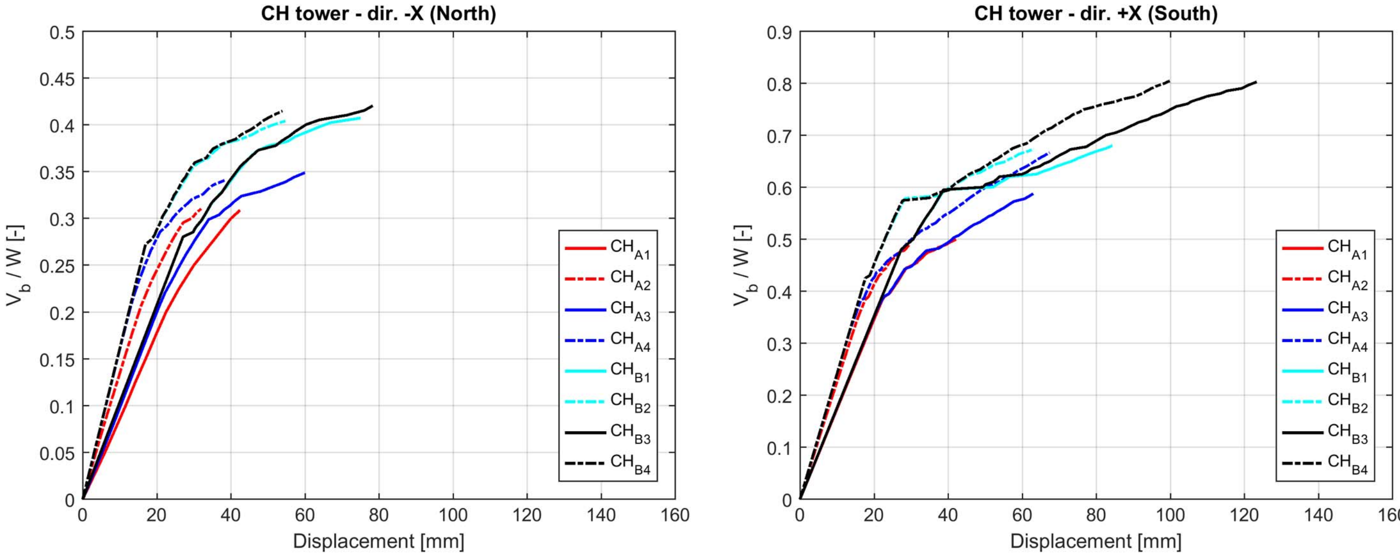


Fig. 8. CH tower: pushover curves for CT1 model along the ±X-directions

Table 11. CH Tower: LV3 Safety Indexes

CH	Model	T^* (s)	$f_{a,\mathrm{SLV}}$	T_R (years)	$I_{s,\mathrm{SLV}}$
Direction South (+X)	IT	1.10	5.01	>2,475	—
	CT1	0.39	2.60	>2,475	—
	CT2	0.24	3.83	>2,475	—
Direction North (−X)	IT	0.65	1.22	484	1.02
	CT1	0.42	1.38	979	2.04
	CT2	0.26	2.82	>2,475	—
Direction East (+Y)	IT	0.79	2.82	>2,475	—
	CT1	0.42	2.18	>2,475	—
	CT2	0.27	4.19	>2,475	—
Direction West (−Y)	IT	0.98	3.69	>2,475	—

critical analysis of those values experimentally assessed by other authors for similar masonry types. This choice has been made since the visual investigation of the masonry, together with the comparison with the parameters experimentally determined for the coeval *Torre Grossa* by Bartoli et al. (2013), suggests the use of strength parameters values higher than those proposed by the Italian standard (MIT 2009). As a matter of fact, it is to be emphasized that the parameters suggested by the standards are usually meant to be valid for ordinary existing masonry structures, while they can be too conservative for nonconventional structures realized with a particular care, as often monuments are. Selected values for the strength parameters are reported in Table 8. The numerical models were realized by modeling the isolated configuration (IT model) and the confined one (CT) taking into account the geometric disposition of the neighboring buildings (adjacent walls were modeled as indefinitely elastic solids). The capacity curves obtained for each different analysis are shown in Fig. 9; the figure reports the results obtained with the IT model and the CT model. It is possible to observe that in the isolated case almost all the analyses offer comparable values of both the collapse load and the ultimate displacement (about 0.23 and 0.48 m, respectively). The isolated model exhibited rather good displacement capacities along all the investigated directions, except for the South load direction, where the ultimate displacement of the top of the tower has been found to be about 0.15 m. The two CT models, as expected, showed a greater global stiffness than the IT one, and a

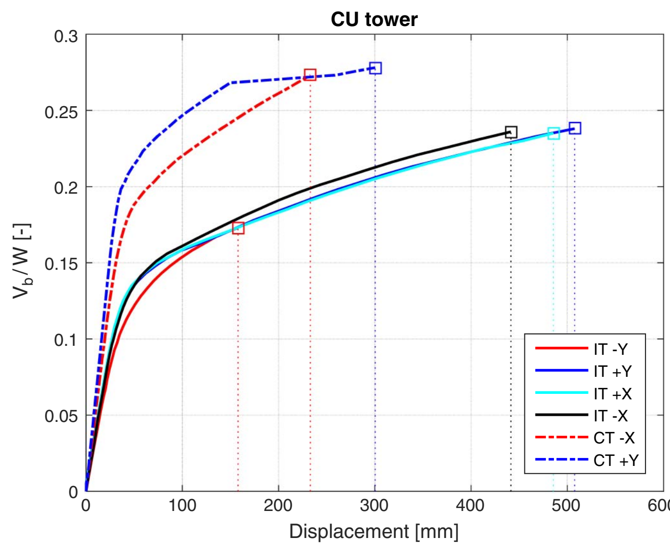
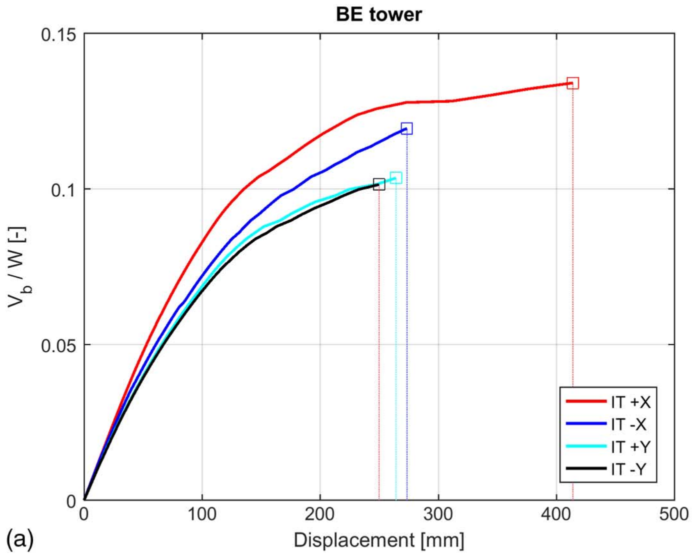


Fig. 9. CU tower: pushover curves along the ±X and ±Y-directions (comparison between IT and CT models)

Table 12. CU Tower: LV3 Safety Indexes

CU	Model	T^* (s)	$f_{a,\mathrm{SLV}}$	T_R (years)	$I_{s,\mathrm{SLV}}$
Direction South (−Y)	IT	1.15	1.52	921	1.94
Direction North (+Y)	IT	1.32	4.22	>2,475	—
	CT	0.82	4.06	>2,475	—
Direction East (+X)	IT	1.25	4.27	>2,475	—
Direction West (−X)	IT	1.26	4.04	>2,475	—
	CT	0.93	2.91	>2,475	—

slightly lower displacement capacity for West (−X) and North (+Y) load directions (about 0.25 and 0.30 m). The pushover curves were, also in this case, employed to build the equivalent bilinear SDOF oscillator response in order to evaluate the acceleration factor, seismic safety index, and return period T_R. The obtained safety indexes are reported in Table 12: all the indexes are higher than unity and in agreement with those obtained with the simplified LV1 approach.



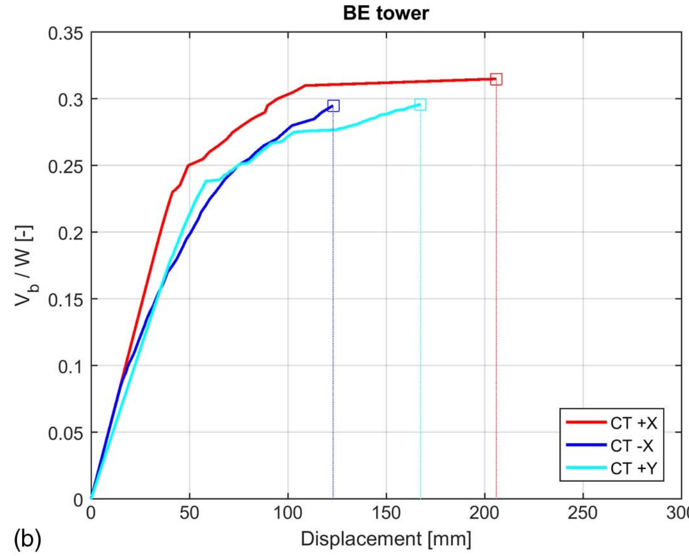


Fig. 10. BE tower: pushover curves along the ±X and ±Y-directions, for (a) IT model; (b) CT model

- The Becci tower: The *ANSYS* FE model of BE tower (Fig. 5) accurately reproduces the geometry of the structure, including the internal walls' thickness reductions and masonry vaults (while internal wooden slabs were not modeled). The nonlinear analyses were performed assuming a rigid ground foundation (fixed-base model) and the strength parameters of the materials were evaluated by taking into account the provision of the Italian standard (MIT 2009)with proper corrective factors. As in the previous cases, the numerical models of the BE tower were realized by modeling the isolated configuration (IT model) and the confined model (CT). Adjacent walls were modeled as indefinitely elastic solids and the equivalent stiffness of the confining building was evaluated in order to reproduce the experimental frequency of the tower. Obtained capacity curves have been reported in Fig. 10. In the isolated case the analyses offer comparable values of the collapse load in all four directions, ranging between 0.11 (−Y-direction) and 0.13 (+X-direction), while more significant variations are observed with respect to the ultimate displacements that range between 0.25 m (−Y-direction) and 0.42 m (+X-direction). The isolated model exhibited rather good displacement capacities along all the investigated directions; the +X-direction offers the highest values of both the base shear and the ultimate displacement. When the confined configuration is considered the collapse load increases and varies between 0.29 (+Y-direction) and 0.31 (+X-direction); the ultimate displacement, lower than the IT model, ranges between 0.13 m (−X-direction) and 0.21 m (+X-direction). The equivalent bilinear SDOF oscillator was evaluated for both IT and CT models, and the obtained safety indexes (acceleration factor, seismic safety index, and return period T_R) are reported in Table 13. The LV3 analyses have not highlighted critical situations (all the index values are higher than unity) and, also in this case, a general agreement between LV3 and LV1 approach is observed.

Table 13. BE Tower: LV3 Safety Indexes

BE	Model	T^* (s)	$f_{a,\mathrm{SLV}}$	T_R (years)	$I_{s,\mathrm{SLV}}$
Direction East (+X)	IT	1.85	3.49	>2,475	—
	CT	0.64	5.23	>2,475	—
Direction West (−X)	IT	1.98	2.12	>2,475	—
	CT	0.77	2.62	>2,475	—
Direction North (+Y)	IT	1.98	2.05	2,321	4.89
	CT	0.77	3.46	>2,475	—
Direction South (−Y)	IT	2.01	1.91	1,894	3.99

Discussion of the Obtained Results

In this section, some of the obtained results in term of pushover curves are compared in order to assess the effect of some uncertain parameters such as the restraint level and the choice of the material properties discussed in the previous section.

- The first comparison can be done by analyzing the three towers CH, BE, and CU, modeled by using *ANSYS*, in their isolated configuration (IT). The strength parameters are reported in Table 8, while the elastic and mass parameters are shown in Table 7. According to the values reported in the tables, results obtained with comparable strength parameters were selected. In Fig. 11, the pushover curves (along the weakest direction) have been reported, where global base shear V_b is evaluated as a fraction of the total weight W of each single tower. Although the

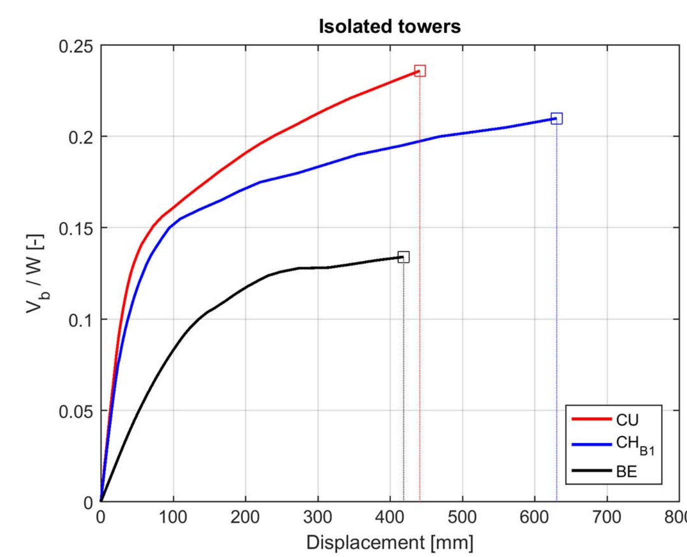


Fig. 11. Comparison among isolated towers

towers are quite similar (in terms of geometry), a similar behavior under seismic actions is not easily identifiable, as far as the maximum base shear and the maximum displacement are concerned. The only clear role is played by the different values of the Young's modulus driving to different initial stiffness (and periods).

- In order to analyze the influence of the strength parameters, Fig. 8 shows the pushover curves of the confined CH tower based on the parameters reported in Table 8. In this case, it is possible to identify a common behavior, the difference being that the parameters' values mainly affecting the ultimate displacement.
- The behavior of an isolated tower compared with the confined configuration is reported in Fig. 12 in order to analyze the influence of confining buildings; in particular, the figure refers to the case of BE and CH towers. By analyzing the pushover curves, it is made clear that the ratio between the maximum values of the base shear (compared to the weight of the tower) in both cases is almost proportional to the ratio between the lengths of the unrestrained portions in these two configurations (i.e., to the weight of the *free* portion of the tower itself). Even if this result is quite obvious when dealing with cantilever structures whose behavior is mainly due to their bending resistance, a similar behavior has not been identified for the ultimate displacement, where a simple rule for the ratio of the two obtained values is not straightforward. It is noticed that, when the experimental frequencies were available (as in the case of the BE tower), the structural identification was done by modal analysis considering confined configuration. Hence, the model of the isolated tower has not been tuned according to the first natural frequencies but it only represents the behavior of the tower when a seismic event has caused the detachment of the neighboring buildings.
- Just as an example, Fig. 13 shows the use of the obtained capacity curves to perform a safety check. The analysis has been done with the aim of understanding the level of performance of the towers. The capacity spectrum method (CSM) approach has

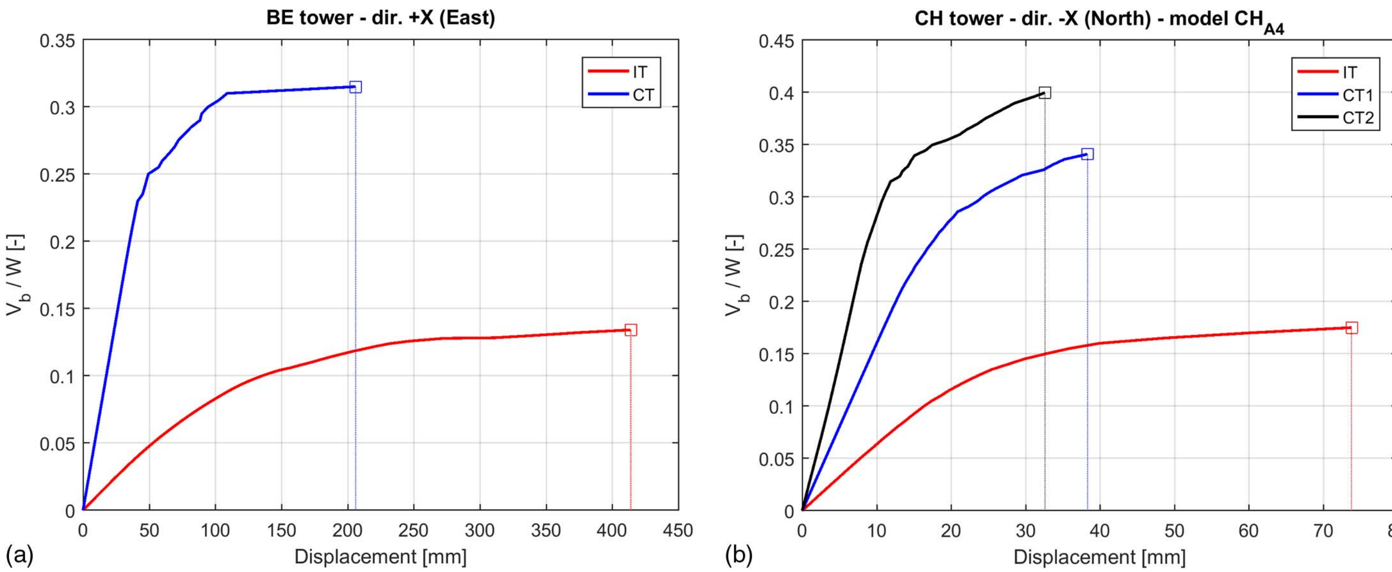


Fig. 12. (a) BE; (b) CH towers: pushover curves, along the X-direction, for CT and IT models

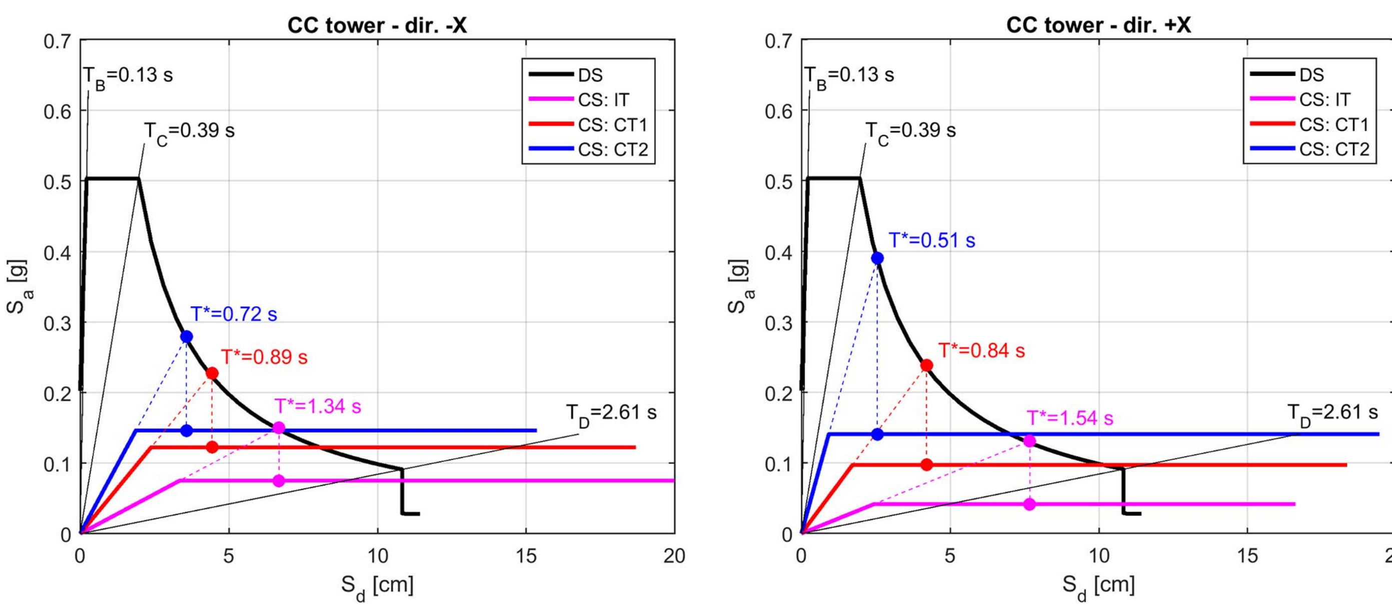


Fig. 13. CC tower: comparison between demand spectrum (DS) and capacity spectra (CS), for CT and IT models

been used for seismic risk assessment by describing both the capacity curve and the response spectrum in terms of spectral acceleration and displacement in the so-called acceleration-displacement response spectra (ADRS). The CSM provides an effective graphical evaluation of the seismic behavior of the construction, since the intersection of the capacity spectrum with the demand spectrum identifies the performance point that represents the condition for which the seismic capacity of a structure is equal to the seismic demand. The graphs shown in Fig. 13 refer to the CC tower. It is quite clear that the change in the confinement given by adjacent buildings can change the structural behavior so dramatically that it can compromise the seismic performances of the tower. The isolated tower is capable of withstanding the seismic action (thanks to its sufficiently high displacement capacity more than due to its resistance); as soon as the restraint offered by adjacent buildings is considered as effective, the capacity can become no more adequate to the required seismic demand. Even if Fig. 13 refers to a specific case, results are quite common to all the examined towers, once again underlining the possible (often negative) influence of the surrounding constructions on the towers' performances; and

- Despite the general coherence obtained between the LV1 and LV3 approaches (with respect to the towers' case studies), it should be observed that the simplified LV1 method of analysis introduced by the Italian Guidelines is primarily aimed at a comparative assessment of the safety indexes at the territorial level. According to the approach, in fact, the tower is designed as a cantilever beam and the failure can only occur due to a flexural collapse mode. Different collapse modes (such as the one due to shear failure for short towers) are not taken into account by the approach. In addition, the approach does not account for the presence of geometric irregularities or belfry. The coherence obtained here between LV1 and LV3 results must then be associated with the typological homogeneity of the towers' case studies.

Conclusive Remarks

The paper summarized the results of the LV1 and LV3 approach on four historic masonry towers in the city center of San Gimignano (Italy). LV1 analyses were performed assuming simplified mechanical models; at LV3, conventional pushover analyses were performed to assess the seismic vulnerability of the towers through global FE models where proper damage models were employed to reproduce the masonry nonlinear behavior. Each tower was analyzed in depth by considering uncertain mechanical, mass, and strength parameters and the aim of the study was to identify the effects of the boundary conditions (mainly the restraint offered by adjacent constructions at the lower level) on the seismic behavior of the towers. Two limit cases for each tower were specifically analyzed: (1) the isolated tower and (2) the confined tower.

The analysis at the territorial scale (LV1) did not reveal any critical situation. The capacity curves obtained with the last level of analysis, the LV3, were employed to perform a safety check according to the CSM. The obtained safety indexes (acceleration factor and seismic safety index) confirm the results obtained with the LV1, and the LV3 safety indexes are always greater than those obtained with the LV1 model showing, despite the difference, a general coherence between the two models. Concerning the effects of the neighboring buildings, the results once more showed how strongly the effects of confinement reflected on tower performances, thus evidencing the need of more accurate investigations regarding the effective portion of the tower to be considered as unrestrained with respect to adjacent buildings. Accordingly, from the results presented here, it can be confirmed that the slenderness is a parameter of paramount importance on the tower seismic vulnerability, but more attention should be paid to the definition of its effective value, as it can strongly depend on the lateral restraint represented by adjacent buildings.

Acknowledgments

The authors kindly acknowledge the Region of Tuscany, which financially supported the research (theme PAR FAS 2007-2013 - CIPE No. 166/2007 - line 1.1.a.3: Science and Technology for the preservation and enhancement of cultural heritage).

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