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ON THE SEISMIC RISK OF MEDIEVAL ITALIAN MASONRY TOWERS

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Abstract. *The paper, after a brief discussion of the methodologies of the research project RiSEM (Seismic Risk of Monumental Buildings), a research project started in 2011 and concluded at the end of 2013, discusses on the seismic assessment of historic masonry towers according to the Italian "Guidelines for the assessment and mitigation of the seismic risk of the cultural heritage". The RiSEM project aimed at developing and testing innovative and expeditious methodologies (i.e. without direct contact with the masonry building) to evaluate all the main structural features of the monumental buildings that are required for the assessment of their seismic safety. As a relevant case study the historic towers of the city of San Gimignano (Italy) in the UNESCO list of the World Cultural Heritage was selected, and the paper summarizes the analyses and the results obtained on three of the analysed towers. The Italian Guidelines identify a methodology of analysis based on three different levels of evaluation, according to an increasing path of knowledge (or requirement) of the structure, namely: LV1 (analysis at territorial level), LV2 (local analysis) and LV3 (global analysis). The paper, summarizing the results obtained for two of the above three levels, highlights a few issue concerning the seismic risk of historic masonry tower.*

1 INTRODUCTION

The quality of the life of a community is strongly affected by the safety and the functionality of the buildings and the infrastructures that constitute the urban environment [1] [2]. In case of Italy (and Tuscany in particular), where the territory is characterized by a massive presence of historical and monumental buildings, life quality is strongly connected with the presence and the functionality of these historical structures. These buildings, as recently demonstrated by the earthquakes of L'Aquila (April 2009) [3] [4] and Reggio Emilia (May 2012) [5], are extremely vulnerable to seismic loads. In addition, although the earthquake which affected L'Aquila was a seismic event of exceptional power (the main shock had a moment magnitude equal to 6.3 MW, with PGA equal to 0.68g) not infrequently damage, and in some cases collapse, 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 developing and test methods of investigation and analysis that may allow to carry out a fairly expeditious seismic risk quantification, developing at the same time technologies that can be used with a certain repetitiveness and on a territorial scale, to provide general guidelines to establish priority of intervention to protect historical monuments.

Among the different typologies of historic monumental buildings, the masonry towers represent a hallmark of many Italian and European town centres. Their construction, in the most ancient cases, dates back to the late-medieval age, and today there are many historic towns that owe their notoriety, and even their economic welfare, to their towers. This is the case of the town of San Gimignano, a small medieval village between Florence and Siena in Tuscany included in the UNESCO World Cultural Heritage list since 1990. In its period of maximum splendour San Gimignano had over seventy towers-houses (some as high as 50 m). Today only 13 of these towers have survived.

The seismic risk of the historic towers of San Gimignano was recently analysed, as an illustrative case study, within the 2-year research project RiSEM "Seismic Risk of Monumental Buildings". The project aimed at developing and testing expeditious and innovative methodologies (i.e. without direct contact with the masonry construction) to assess the structural data needed for the subsequent evaluation of the seismic risk. The whole project, funded by the Tuscany Region, was developed by a consortium which included two Italian Universities (Florence and Siena) through four University Departments from different scientific areas. The methodology adopted in the research was based on the following elements: a) assessment of seismic hazard and soil-structure interactions; b) acquisition of the geometric characteristics and reconstruction of the historical evolution of masonry buildings; c) evaluation of the static and dynamic behaviour of towers (structural identification) through non-conventional and innovative investigation techniques; d) 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 e) evaluation of the seismic risk. The final goal of the project was therefore to develop guidance for the assessment of the seismic vulnerability of historic masonry towers, according to the Italian Recommendations (NTC2008 [6]).

The town of San Gimignano was identified as an exemplary case study due to the typological structural homogeneity of its historic tall masonry towers (in fact the presence of several buildings with a similar dynamic behaviour makes the case study particularly significant for "testing" new techniques of investigation and analysis).

The seismic assessment of the towers, was developed according the provisions of the Italian Guidelines for the assessment and mitigation of the seismic risk of the Cultural

Heritage (DPCM2011 [7]). The Guidelines propose a methodology of analysis based on three different levels of evaluation, according to an increasing knowledge of the structure. The first level of analysis (LV1, analysis at territorial scale) allows to evaluate the collapse acceleration of the structure by means of simplified models based on a limited number of geometrical and mechanical parameters (and qualitative tools such as visual inspections). The second level (LV2, local analysis) is based on a kinematic approach performed to analyse the local collapse mechanisms that can develop on several macro-elements. The identification of proper macro-elements is based on the knowledge of structural details of the building (cracking pattern, construction technique, connections between the architectonic elements, etc). The last level of evaluation (LV3, global analysis) requires a global analysis of the whole building under seismic loading by suitable numerical models. Compared to the previous two levels, the LV3 should be the most accurate but it requires a large amount of input data and, depending on the employed numerical approach, great computational effort.

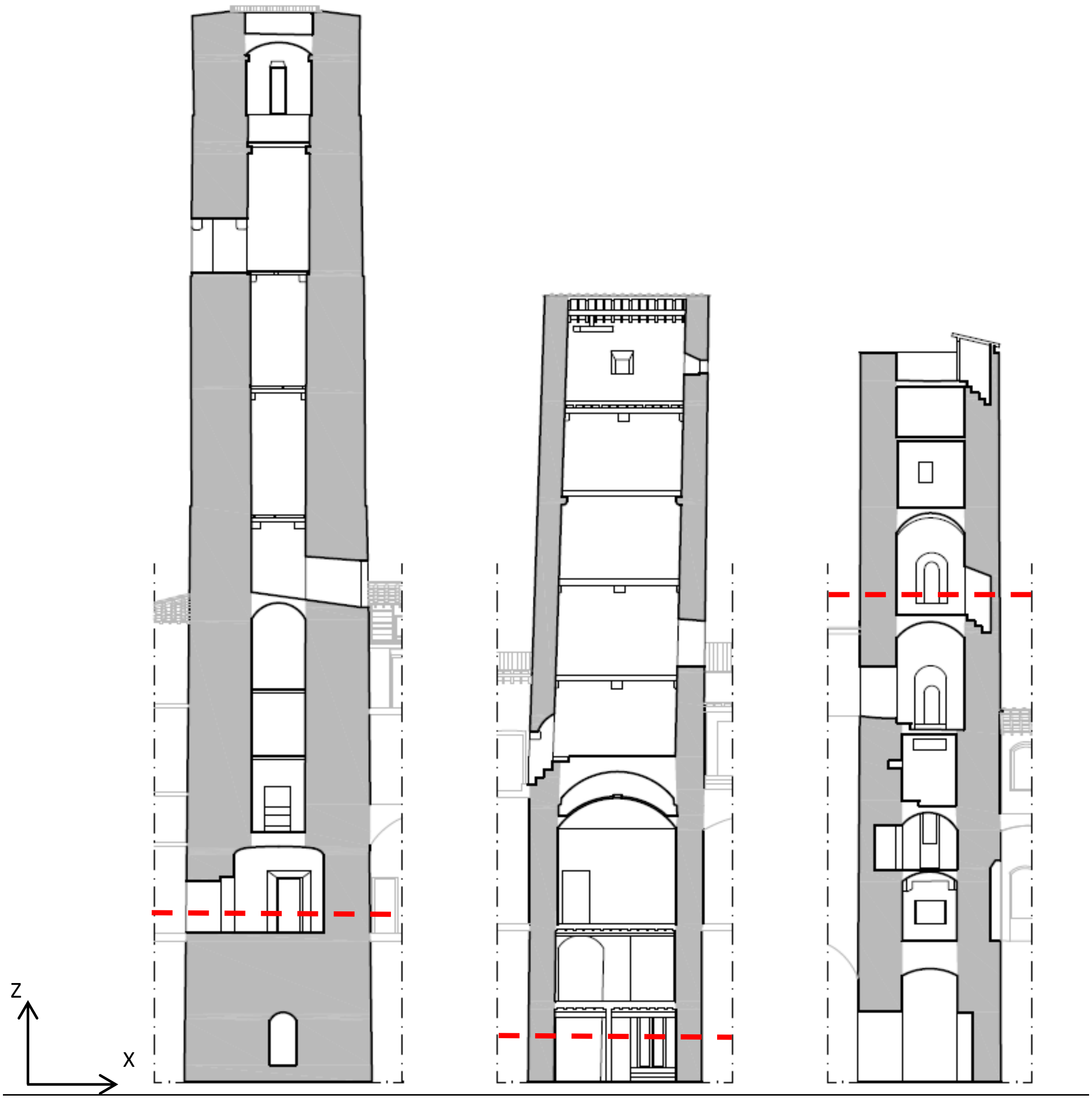
The paper, as a first step toward the synthesis of the results obtained within the project, summarizes the analysis executed and the results obtained for three of the analysed towers: the Becci tower, the Coppi-Campatelli tower and the Chigi tower (Figure 1). After a short description of the towers, the methodology employed to cover the unknowns deriving from the knowledge process and the performed parametric analyses are critically discussed.

2 THE HISTORIC TOWERS OF SAN GIMIGNANO

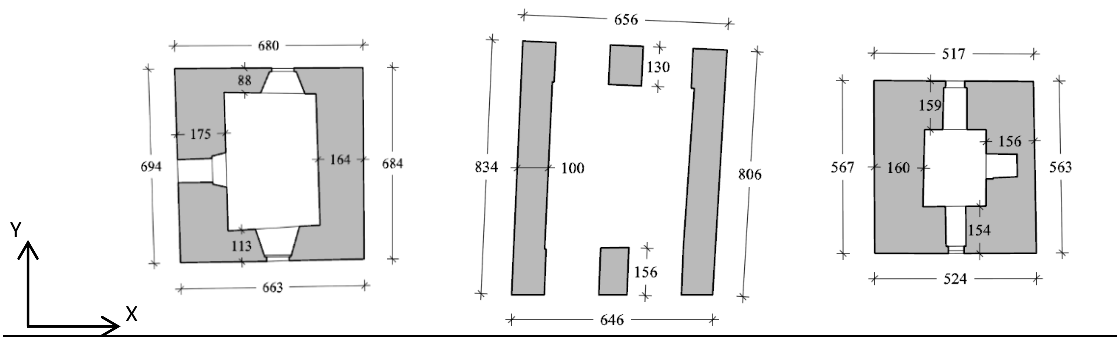
The historic towers of San Gimignano date back to XII-XIII century. The sustaining walls of the towers are multi-leaf stone masonry walls with the internal and external face usually made with the same typology of material (and also, presumably, the same thickness); the internal, and thick, core is composed of heterogeneous stone blocks tied by a good mortar.

A section of the three analysed towers, together with the base cross-section, is reported in Figure 1. Slenderness of the towers ranges between 3.4 (Coppi-Campatelli tower) and 5.9 (Becci tower) and the thickness of the walls is almost uniform for all the three towers. At the lower level the towers are largely incorporated into the neighbouring buildings and hence the lower sections present several openings (in most cases subsequent to the tower construction) to allow communication with the confining buildings.

The internal and external faces of the towers are made by a local cavernous limestone except the upper part of the Chigi tower that was built with masonry bricks (Table 1). Due to the lack of information (tests, core drilling, mineralogical surveys etc.), apart from the visual inspection, the mechanical properties of the walls were characterized by taking into account the provisions of the Circular 2009 [8]. In particular several typologies of masonry walls were considered (Table 1). The first is the scheme of uncut stone masonry with facing walls of limited thickness and infill core (USM). The second is the scheme of soft stone masonry (tuff, limestone, etc.) (SSM). The third is the scheme of dressed rectangular stone masonry (DRS). The fourth, characterizing only the upper part of the Chigi tower, is the scheme of full brick masonry with lime mortar (FBM). The reference intervals for the value of the mechanical properties reported in the table C8A.1 of the Circular 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. Furthermore, the values proposed by the Circular 2009 assume, in case of multi-leaf stone masonry, disconnected facing walls and/or lack of systematic transverse connecting elements (or interlocking between the masonry facing walls). To account for good quality mortar, for the presence of a thick inner core and for the presence of thin joints the correction factors proposed by the Circular 2009 (table C8A.2.) were employed.



H [m]	39.4	27.6	26.9
S [m]	~ 6.6 × 6.8	~ 6.5 × 8.1	~ 5.2 × 5.6
λ_x [-]	5.9	4.2	5.2
λ_y [-]	5.8	3.4	4.8



Becci	Coppi-Campatelli	Chigi

Figure 1: Towers sections and cross sections (H – height, S – base section dimension, $\lambda_{x,y}$ – slenderness).

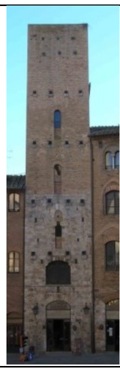
Tower	Masonry typology	Tower walls	View	
Becci	**D**: Soft stone masonry (SSM).	internal and external faces similar / thick inner core / good quality mortar.		
Coppi-Campatelli	**B:** Uncut stone masonry with facing walls of limited thickness and infill core (USM). **E**: Dressed rectangular stone masonry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

Table 1: Visual characterization of the masonry typologies.

Type of masonry	Mechanical characteristics			Correction factors		
	f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	thick or poor internal core	good quality of the mortar	thin joints
B	200 300	3.5 5.1	1020 1440	0.8	1.4	1.2
D	140 240	2.8 4.2	900 1260	0.9	1.5	1.5
E	600 800	9.0 12.0	2400 3200	0.7	1.2	1.2
F	240 400	6.0 9.2	1200 1800	0.7	1.5	1.5

Table 2: Mechanical properties evaluated according to [8]
(f_m – compressive strength, τ_0 – shear strength, E - modulus of elasticity).

Specifically, considering that a limited level of knowledge (KL1 according to the Code [6], corresponding to a confidence factor of 1.35) was reached, the standard prescribes to employ as resistance parameters (f_m and τ_0) the minimum values of the ranges given in table C8A.2.1 of the Circular 2009, while for the elastic moduli (E and G) the mean values were assumed. According to what the standards suggest, and in the absence of more accurate investigations, the correction factors (table C8A.2. Circular 2009) reported in Table 2 were applied to the mechanical parameters.

The performed estimation of the masonry mechanical parameters through literature results was in accordance with the project's goals, aiming the research to test expeditious techniques to assess the seismic risk of monumental buildings without direct contact with the masonry construction. In this respect the selected types of masonry (Table 1), similar for morphology to those visually detected in situ, were assumed as lower and upper bound for the actual masonry parameters to be employed in the subsequent analysis models.

3 RISK ASSESSMENT

The evaluation of the seismic risk of the tower was performed according to the three levels of evaluation introduced by the Italian DPCM2011 ("Guidelines for the assessment and mitigation of the seismic risk of the cultural heritage" [7]). The Guidelines, that represent an innovative tool in the European context, propose an assessment methodology organised on three levels:

- Level 1 (LV1) is a territorial risk model, 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 model, in which the spectral coordinates of the earthquake represent the input and the vulnerability is evaluated analysing the activation of partial collapse mechanisms in a single part of the structure (macroelement). The evaluation of the safety indexes still requires a few geometrical and mechanical parameters;
- Level 3 (LV3) is a global mechanical risk model, 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). The model asks for a detailed analysis of the single building, considered as a whole (or as an assembly of macroelements).

Among the three level of analysis the paper reports, for three of the thirteen analysed towers of San Gimignano, the results obtained with the models LV1 and LV3. The two level are compared through the examination of two safety indexes, evaluated with reference to the limit states of Life Safety (SLV).

The first index is the seismic safety index ($I_{S,SLV}$), the ratio between the return period of the seismic action which 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 ($T_{R,SLV}$ = 475), defined as follows:

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

A seismic safety index greater than one corresponds to a safe state for the tower (with respect to the assumed reference period V_R and for its coefficient of use c_u). A safety index lower than one highlights possible critical issues that require in-depth investigations.

The second examined index is the acceleration factor ($f_{a,SLV}$), the ratio between the acceleration which brings the tower to the Life Safety limit state (a_{SLV}) and the reference acceleration for the Life Safety limit state ($a_{g,SLV}$), both referred to a rigid ground (ground type A):

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

The acceleration factor, while considering only one of the parameters that define the seismic action spectrum, has the advantage of providing a quantitative indication of any deficiency in terms of mechanical strength of the structural system. The acceleration factor $f_{a,SLV}$ is a purely mechanical parameter, which may be useful for an evaluation of the weakness of the structure in terms of strength. The seismic safety index $I_{S,SLV}$, being based on the return periods of the seismic demand and the capacity of the structure, provides a direct evaluation of the eventually present vulnerability of the tower over time.

The indexes were evaluated according to the two principal directions of each section of the towers, in both directions, being not (usually) possible to identify in advance the most critical section. In addition the nominal life V_N (which is obtained from the value of the return period of the seismic demand that brings to the achievement of the SLV) and the corresponding return time were also calculated.

4 LV1 ANALYSES

With the aim to evaluate the seismic risk at territorial level, the expeditious model proposed by the DPCM2011 [7] schematizes the tower as a cantilever beam, subject to a system of horizontal forces, assuming that the collapse can occurs according to a combined compressive and bending stress mode. The simplified LV1 model allows to evaluate the collapse acceleration of the structure based on a limited number of geometrical and mechanical parameters (or qualitative tools such as visual test, construction features, and stratigraphic survey). It is hence mainly aimed to evaluate a "comparative ranking risk" between similar structures in order to highlight the need for subsequent in-depth investigations (or to program actions to mitigate the seismic risk).

Results of seismic vulnerability at territorial level should be a useful tool for the public administration for highlighting the most critical situations in the territory and to establish priorities for future interventions. It is hence implicitly assumed that lower LV1 safety indexes actually correspond to lower safety indexes in case of refined LV3 analyses.

In case of masonry tower, from the operative point of view, the LV1 model foresees to divide the structure into *n* sectors (blocks) having uniform characteristics. This step should be performed taking into account several aspects, among them: i) beginning and ending of the openings; ii) level of detachment of the tower from the neighbouring buildings (if the tower is not isolated); iii) levels in which there is a reduction in the thickness of the masonry walls; iv) levels where there are changes of materials and / or changes in the construction phases. To identify the sectors into which the tower must be divided responds to the need to obtain structural portions with geometrical and mechanical uniform characteristics where safety checks are to be performed. Afterwards, the safety checks are carried out by comparing, at the base of each sector and for each load direction, the seismic demand (the acting bending moments) with the seismic capacity (the correspondent ultimate resistant moment).

Since the towers (Figure 1) are largely incorporated into the neighbouring buildings, to apply the LV1 model following schemas were considered:

- Model A: the towers are analysed as an isolated construction, i.e. without considering the presence of the neighbouring buildings (it is implicitly assumed that the action offered by the neighbouring structures is ineffective);
- Models B and C: these models still considers the towers as isolated constructions, but assume as tower height the portion of the structure that emerges from the surrounding buildings.

The above models are aimed to introduce lower and upper bound within approximate the actual behaviour of the towers. The adjacency of the towers with the lower constructions can, in fact, significantly alter their structural behaviour: on the one hand, it reduces the effective slenderness (thus reducing the period); on the other hand these constructions constitute stiffeners which may produces localized areas of possible stress concentration (and pounding).

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. It was shown ([10]) that the empirical correlation (3) provided by the NTC2008 [6], in case of slender masonry towers, 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.

$$T_1 = 0.050 \cdot H^{0.75} \tag{3}$$

On the basis of experimental results concerning the main periods of historic masonry towers Ranieri and Fabbrocino [10] proposed the following empirical correlation, here also used for the estimation of the fundamental period:

$$\mathrm{T}_1 = 0.013 \cdot \mathrm{H}^{1.10} \tag{4}$$

The Eq. (4), like the empirical correlation provided by the Italian building code, provides the main period of the structure as only function of the height H of the tower. For comparative purpose the main period of the towers were also estimated by employing the classical formula of the linear elasticity:

$$\mathrm{T}_1 = 1.787 \cdot \mathrm{H}^2 \cdot \sqrt{\frac{\gamma \cdot \mathrm{A}}{\mathrm{E} \cdot \mathrm{J} \cdot \mathrm{g}}} \tag{5}$$

where A is the cross sectional area of the *i*-th sector, γ is the specific weight, E is the modulus of elasticity and J denotes the area moments of inertia (to be evaluated in the analysed load direction).

The Italian guidelines require, in order to account for the behaviour of the structure at the ultimate limit state (i.e. to consider the non-linear 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 which varies between 1.40 and 1.75.

The main periods T_1 evaluated by Eqs. (3)-(5) were thus amplified with a factor equal to 1.40 to obtain a period T_1^* representative of the damage phenomena induced by seismic load at the ultimate limit state. The estimated main periods of the three towers for the three schemas A, B and C are reported in Table 3.

Model	Eqs.	Chigi			Coppi-Campatelli			Becci		
		H [m]	T_1 [s]	T_1* [s]	H [m]	T_1 [s]	T_1* [s]	H [m]	T_1 [s]	T_1* [s]
	(3)		0.59	0.83		0.69	0.97		0.79	1.10
A	(4)	26.9	0.49	0.68	33.1	0.61	0.85	39.4	0.74	1.04
	(5)		0.63	0.88		1.09	1.52		1.47	2.06
	(3)		0.35	0.48		0.60	0.84		0.43	0.60
B	(4)	13.4	0.22	0.31	27.6	0.49	0.69	17.4	0.31	0.47
	(5)		0.16	0.22		0.89	1.25		0.30	0.42
	(3)		0.28	0.40		0.35	0.49		0.39	0.55
C	(4)	10.5	0.17	0.23	13.4	0.23	0.32	15.5	0.27	0.38
	(5)		0.08	0.11		0.75	1.05		0.24	0.34

Table 3: Main periods of the towers (empirical correlations and analytical expression).

Chigi	direction	T_1 [s]	$f_{a,SLV}$ [-]	$I_{S,SLV}$ [-]	V_N [years]	T_R [years]
Model A	N-S (X)	0.49	0.97	0.91	46	434
		0.88	1.29	2.29	115	1089
	E-W (Y)	0.49	1.04	1.12	56	530
		0.85	1.38	2.95	147	1399
Model B	N-S (X)	0.16	1.60	5.21	261	2475
		0.48	1.60	5.21	261	2475
	E-W (Y)	0.15	1.60	5.21	261	2475
		0.48	1.60	5.21	261	2475
Model C	N-S (X)	0.08	1.60	5.21	261	2475
		0.28	1.60	5.21	261	2475
	E-W (Y)	0.08	1.60	5.21	261	2475
		0.28	1.60	5.21	261	2475

Table 4: Chigi LV1 safety indexes.

Campatelli	direction	T_1 [s]	$f_{a,SLV}$ [-]	$I_{S,SLV}$ [-]	V_N [years]	T_R [years]
Model A	N-S (X)	0.85	1.39	1.62	81	771
		1.52	1.61	5.21	261	1529
	E-W (Y)	0.85	1.43	1.78	89	846
		1.52	1.71	3.37	169	1601
Model B	E-W (Y)	0.69	1.31	1.34	67	637
		1.25	1.58	2.51	126	1195
Model C	N-S (X)	0.87	1.87	4.74	238	2255
		1.05	1.87	4.75	238	2259
	E-W(Y)	0.87	2.34	5.21	261	2475
		1.05	2.34	5.21	234	2475

Table 5: Coppi-Campatelli LV1 safety indexes.

Tower	T_1 [s]	$f_{a,SLV}$ [-]	$I_{S,SLV}$ [-]	V_N [years]	T_R [years]
Chigi	0.49	0.97	0.91	46	434
	0.85	1.38	2.95	147	1399
Coppi-Campatelli	0.85	1.39	1.62	81	771
	1.52	1.71	3.37	169	1601
Becci	0.73	1.12	1.47	105	696
	0.88	1.34	2.67	191	1270

Table 6: LV1 safety indexes: acceleration factor ($f_{a,SLV}$), seismic safety index ($I_{S,SLV}$), nominal life (V_N) and return time (T_R).

Results of the analyses are summarized in Table 4 (Chigi tower) and Table 5 (Coppi-Campatelli tower) in terms of acceleration factor $f_{a,SLV}$, seismic safety index $I_{S,SLV}$, nominal life V_N and return period T_R. For all the analyzed cases, the minimum spectral acceleration is obtained in correspondence of the base section of each considered model.

The LV1 results show that no critical situations are detected (although, in the case of the Chigi tower, the A model with main period evaluated accordin to Eq. (4) without the amplification factor 1.40 provides an acceleration factor slightly less than one). Even the values of the nominal life V_N, representing the time period in which the constructions can be used with the same level of safety, do not highlight critical scenarios being the values of V_N always greater than V_R. It is possible to observe a general consistency of the results in terms of safety indexes. In addition, it also possible to observe, with respect to both $f_{a,SLV}$ and $I_{S,SLV}$, that the smaller values are obtained with the A models (isolated towers). Lower and upper bounds are summarized in Table 6.

5 LV3 ANALYSES

The third level of analysis is based on the use of numerical models able to simulate the global structural behaviour in order to evaluate the accelerations leading the structure to each analysed limit state. This level, compared with the previous one, is more demanding since it requires a deepen knowledge of the constructive techniques and the structural details, together with the material properties (tensile and compressive strength of the materials), to perform a reliable evaluation of the seismic capacity of the building. The reliability of the model, and consequently the provided results, is closely connected with 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 identifying between 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. These, and other aspects not expressly called up, were addressed in this level of investigation, and analogously to what was done in the previous one, through a parametric investigation in order to identify lower and upper bound of behaviour.

To investigate this level, linear and nonlinear analyses were performed by means of finite element (FE) models of the towers. In particular, the FE models of the Chigi and Coppi-Campatelli towers were employed to perform nonlinear static analyses (pushover) analysing the directions +/- X and +/- Y (as reported in Figure 1) while the FE model of the Becci tower was employed to perform linear time-history analyses through a simplified approach. Hereinafter, main results and the employed methodology are discussed.

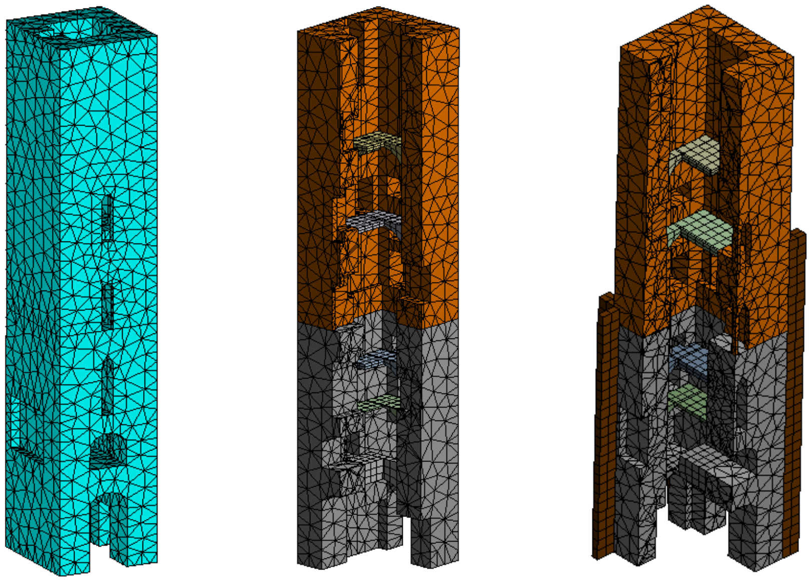


Figure 2: FE model of the Chigi tower (ANSYS).

5.1 The Chigi tower

The FE model of the Chigi tower (Figure 2) was built by using the commercial code ANSYS [11] accurately reproducing the geometry of the structure. The numerical model included the masonry vaults, while internal wooden slabs were not modelled. The major openings of the wall of the tower (doors, windows, recesses, etc.) were reproduced, and the nonlinear analyses were performed assuming a rigid ground foundation (fixed base model).

The strength parameters of the material were evaluated taking account the provision of the Italian Recommendation [8] but, in addition, due to the lack of experimental data, and within the aim of the project, an extensive parametric investigation was developed taking into account the variability of the strength parameters. The analysed configurations are reported in Table 7.

	Stone (lower part)				Brick (upper part)			
#	f_t [MPa]	f_c [MPa]	E [MPa]	γ [kg/m^3]	f_t [MPa]	f_c [MPa]	E [MPa]	γ [kg/m^3]
A11	0.106	0.493	1458	1600	0.195	0.901	2363	1800
A12	0.106	0.493	2916	1600	0.195	0.901	4726	1800
A21	0.106	0.986	1458	1600	0.195	1.801	2363	1800
A22	0.106	0.986	2916	1600	0.195	1.801	4726	1800
B11	0.212	0.986	1458	1600	0.390	1.801	2363	1800
B12	0.212	0.986	2916	1600	0.390	1.801	4726	1800
B21	0.212	1.973	1458	1600	0.390	3.603	2363	1800
B22	0.212	1.973	2916	1600	0.390	3.603	4726	1800

Table 7: Parametric values of strength parameters (f_t – tensile strength, f_c – compressive strength, E - modulus of elasticity, γ - own weight).

The seismic behaviour of the tower was analysed employing a pushover approach: the static nonlinear analyses were developed increasing monotonically the horizontal loads that were applied under conditions of constant gravity loads. The analyses were performed considering all the seismic directions (+/-X and +/-Y). Pushover curves were evaluated assuming as control point the displacement of the centre of mass of the upper section.

In addition to account for the confining effects provided by the lower buildings, the following two limit cases were considered to identify lower and upper bound of behaviour:

- model of isolated tower (IT): the first limit case considers the tower alone, without taking into account the interaction with the confining buildings (model IT);
- model of confined tower (CT): the second limit case considers the presence of the adjacent buildings (in all the directions) assuming their effects as a rigid constraint (model CT1 and model CT2).

The interactions of the tower with the lower buildings were considered as constraints for both the load directions, while the model of isolated tower takes into account 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 them from neighbouring structures).

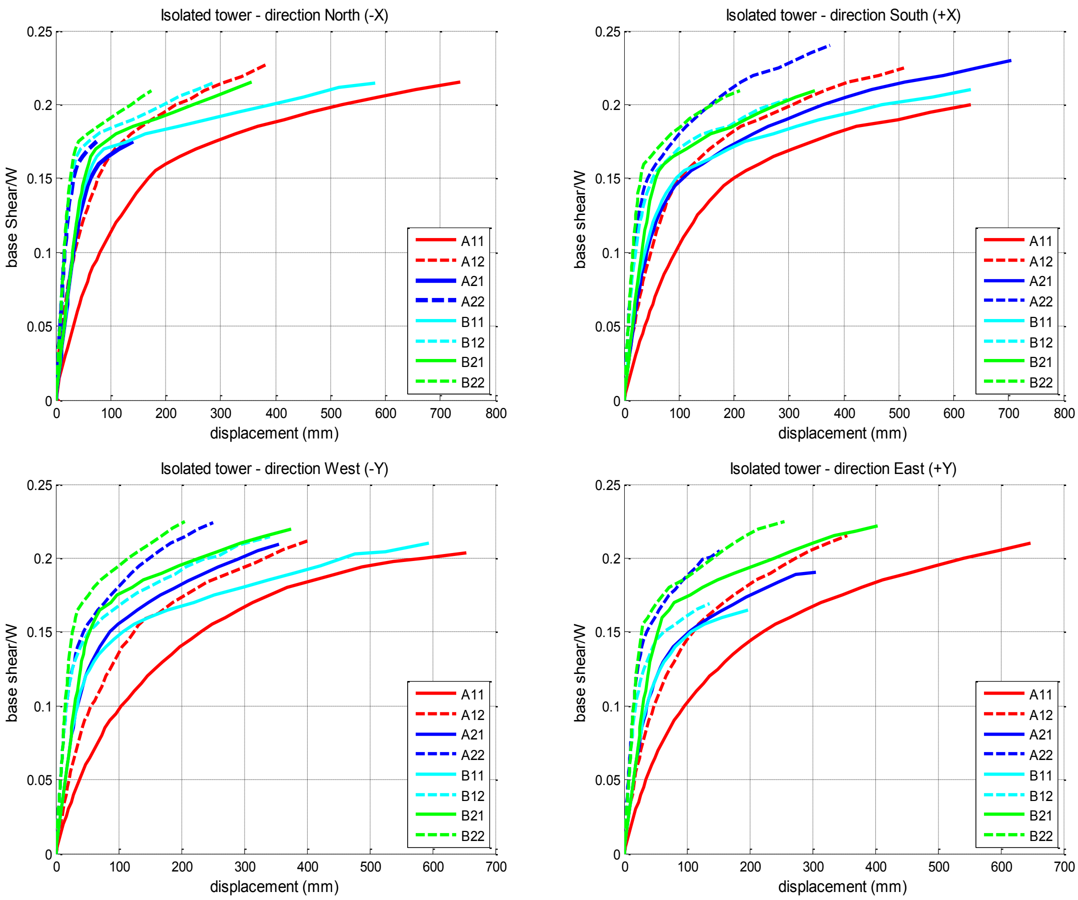


Figure 3: Pushover curves for +/-X (Up) and +/-Y directions (Down).
FE model of isolated Chigi tower.

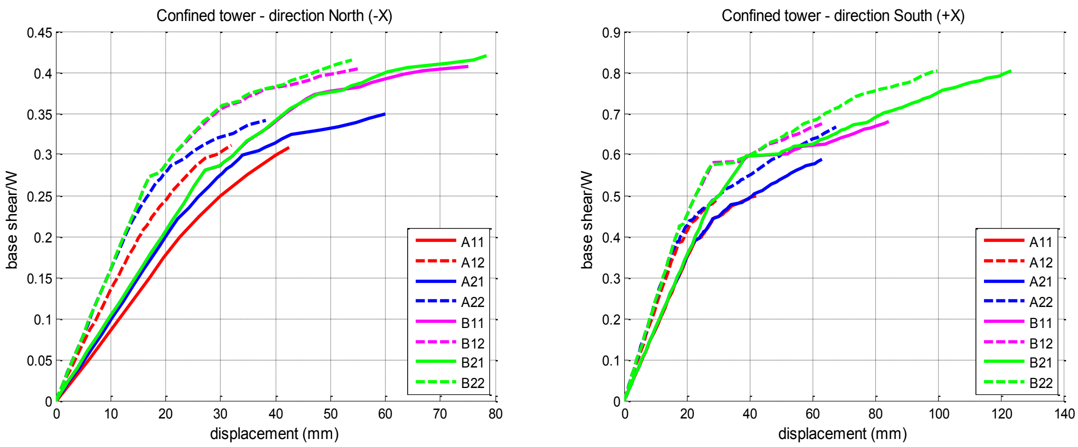


Figure 4: Pushover curves for +/-X directions.
FE model of confined Chigi tower.

The equivalent stiffness of the lower buildings was calculated by equating the displacements of the upper end of the confining wall, subjected to a unitary force, with the corresponding displacement of the elastic boundary constraints (taking into account both the flexural and shear deformation). The equivalent elastic modulus thus obtained, E_1, was employed to model the boundary elements (model CT1). As a further case, and for comparative purposes, was also analysed a value E_2 equal to 10 times the first (model CT2). This investigation can be quite significant since the presence of confining structures can be an effective constraint for the tower and the tower seismic vulnerability can be strongly influenced by the dynamic interaction with them (the confining structures reduce the slenderness of the tower and, at the same time originate, points of stresses concentrations and pounding).

The pushover curves (generalized force–displacement relationship) with respect to the case of loading acting in +/-X (the North-South) and +/-Y (the East-West) directions are reported in Figure 3 for the case of isolated tower. As control node to build the capacity curves, the horizontal displacements of the corner nodes of the top section of the tower were considered (their average value). The capacity curves of the isolated tower with the confined tower are reported in Figure 4.

Chigi	Model	T* [s]	$f_{a,\,SLV}$ [-]	T_R [years]	$I_{s,\,SLV}$ [-]
Direction South (+X)	IT	1.10	5.01	>2475	-
	CT1	0.39	2.60	>2475	-
	CT2	0.24	3.83	>2475	-
Direction North (-X)	IT	0.65	1.22	484	1.02
	CT1	0.42	1.38	979	2.04
	CT2	0.26	2.82	>2475	-
Direction East (+Y)	IT	0.79	2.82	>2475	-
	CT1	0.42	2.18	>2475	-
	CT2	0.27	4.19	>2475	-
Direction West (-Y)	IT	0.98	3.69	>2475	-

Table 8: Chigi LV3 safety indexes.

The obtained pushover curves were employed to build the curve of the equivalent bilinear single-degree-of-freedom (SDOF) oscillator employed to perform the seismic checks and to evaluate the acceleration factor, the seismic safety index and the return period T_R. Table 8 reports the safety indexes obtained for the material A22 (Table 7). Overall, the LV3 analyses have not highlighted critical situations: all the cases are verified and the results of the level of investigation LV3 are in agreement with the first level of evaluation (LV1).

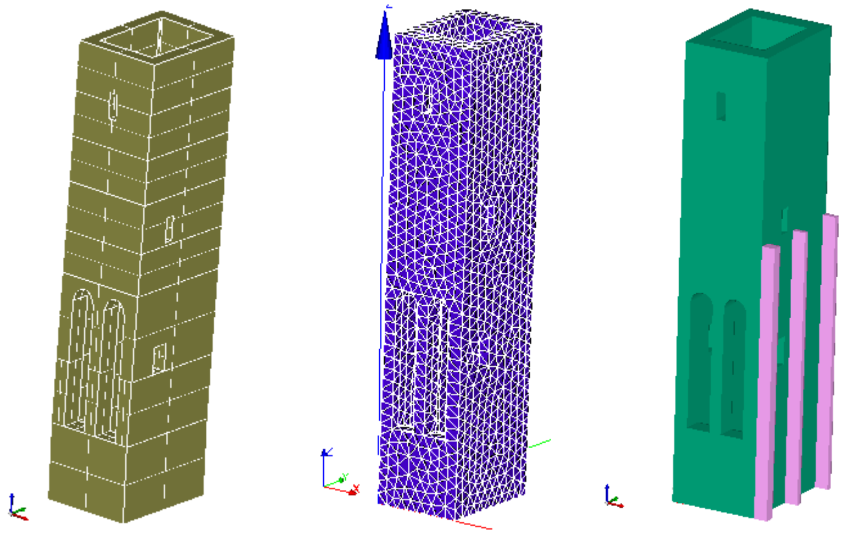


Figure 5: FE model of the Coppi-Campatelli tower (Code Aster).

5.2 The Coppi-Campatelli tower

The FE model of the Coppi-Campatelli tower (Figure 5) was built by using Code Aster, an Open Source finite element code. The code has a wide library of nonlinear material models and to reproduce the masonry nonlinear behaviour the continuum damage model of Mazars [12] was adopted. The numerical model was employed to perform pushover analyses analysing the directions +/- X and +/- Y (Figure 1) assuming a rigid ground foundation (fixed base model). The parameters required by the damage model to reproduce the masonry nonlinear behaviour were selected in order to fit the limit scheme DRS (dressed rectangular stone masonry) and the cases of confined (CT) and isolated (IT) tower were, as for the Chigi tower, analysed.

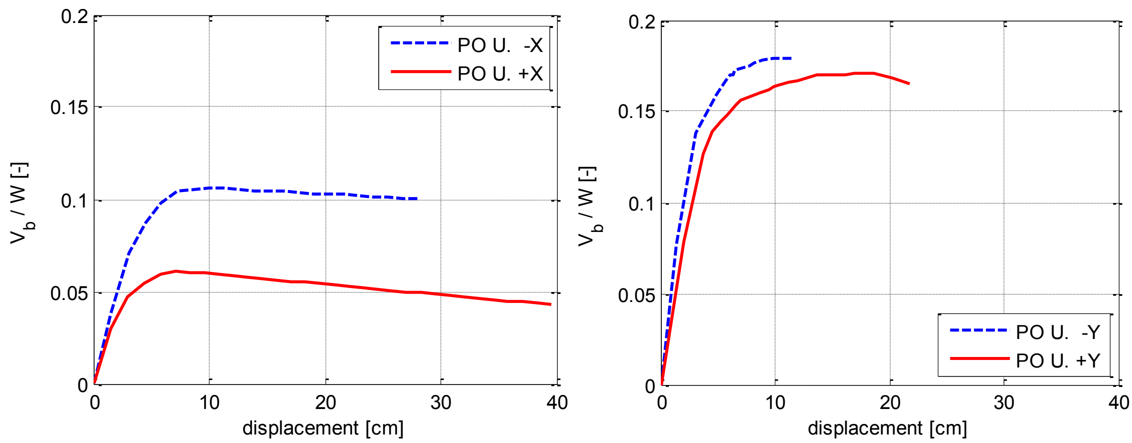


Figure 6: Pushover curves for +/-X and +/-Y directions. FE model of isolated Coppi-Campatelli tower.

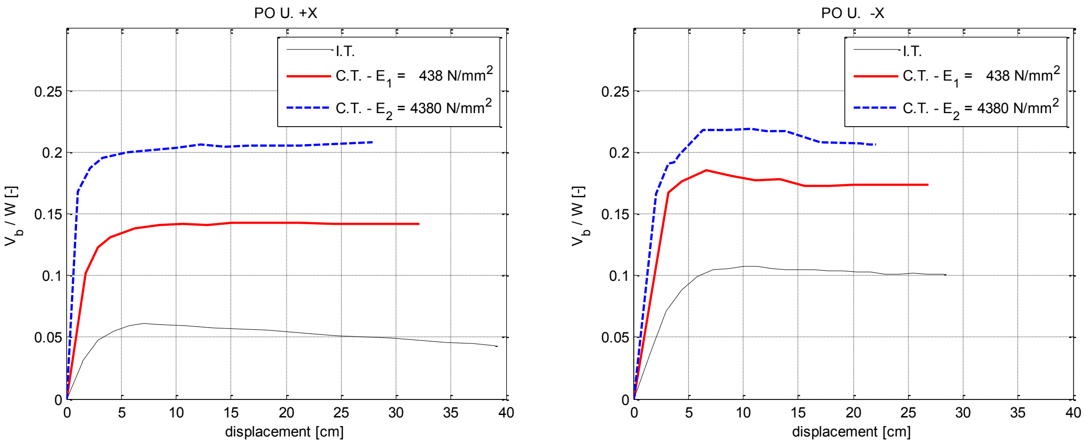


Figure 7: Pushover curves for +/-X and directions.
Comparison between capacity curves of isolated and confined Coppi-Campatelli tower.

The pushover curves with respect to the case of loading acting in +/-X (the North-South) and +/-Y (the East-West) directions are reported for the case of isolated tower in Figure 6. The comparison of the pushover curves of the isolated tower with the confined tower are reported in Figure 7 (load directions +/-X). In case of direction +X, for instance, it is possible to observe that the base shear, in the case of tower with constraint with equivalent elastic modulus E_1, shows an increase of about 135% if compared to the isolated tower case. This increase becomes about 240% if case E_2 is considered. In terms of ultimate displacement the two restrained cases show comparable values (25-35 cm), still lower than those obtained in the case of isolated tower (about 40 cm).

As in the previous case, the pushover curves were employed to characterize the equivalent bilinear single-degree-of-freedom (SDOF) oscillator employed to perform the seismic checks. Table 9 reports the safety indexes (acceleration factor, seismic safety index and return period T_R). Also for the Coppi-Campatelli tower the LV3 analyses do not highlight critical situations providing safety indexes in agreement with the one obtained with the LV1 model. It is interesting to observe that the acceleration factor obtained with the LV3 model are always higher than those obtained with the LV1 model.

Coppi-Campatelli	Model	T* [s]	$f_{a, SLV}$ [-]	T_R [years]	$I_{s, SLV}$ [-]
Direction South (+X)	IT	1.54	2.21	>2475	-
	CT1	0.84	4.40	>2475	-
	CT2	0.51	7.63	>2475	-
Direction North (-X)	IT	1.34	3.08	>2475	-
	CT1	0.89	4.33	>2475	-
	CT2	0.72	4.38	>2475	-
Direction East (+Y)	IT	1.07	2.87	>2475	-
	CT1	0.84	1.33	625	1.32
	CT2	0.47	1.57	992	2.09
Direction West (-Y)	IT	0.91	1.79	1519	3.20

Table 9: Coppi-Campatelli LV3 safety indexes.

5.3 The Becci tower

The seismic risk of the Becci tower was analysed through the simplified approach proposed by Bartoli et al. [13] performing time-history analysis. The approach, unlike the previous nonlinear one, requires only a reduced number of data (employed for the tuning of a linear numerical model). After the identification of a linear FE model of the whole structure, the model is employed to evaluate the load acting at every section of the tower due to a specified earthquake (modelled by an appropriate accelerogram). Loads acting at every section [z] of the tower are identified in global terms like shear force [T(z; t)], normal force [N(z)] and bending moment [M(z; t)]. After the evaluation of the time-history of each internal action, for a certain section of the tower, the evaluation of the seismic reliability is carried out by analysing the following two limit states:

- <u>I limit state</u>: tower over-turning (it is verified when the own weight combined with the seismic loads causes a resultant load which eccentricity is internal with respect to the cross-sectional area);
- <u>II limit state</u>: mechanical collapse of an external panel in its plane (it is verified when the seismic load acting on the tower is not able to produce a local cracking/crushing on a panel of the tower).

The proposed methodology aims to connect, for each of the above limit state (LS), an appropriate ground acceleration a_g able to assure their respect. The first limit state is identified in the whole tower, while the second one is related to the behaviour of a single masonry panel (with its actual nonlinear properties). The respect of the two limit states results by the comparison between the resisting force *R* (evaluate upon geometrical aspects for the I LS.; estimate upon the collapse behaviour of a masonry panel for the II LS) and the acting force *S* (obtained by the seismic load applied as ground acceleration time history). Despite its simplifications, the approach has the advantage that the I LS requires substantially only to evaluate the experimental frequencies of the tower that are needed for the tuning of the model. Subsequently, few mechanical data are required for the II LS (it requires to evaluate the collapse surface of elementary panels).

The Becci tower was analysed in order to evaluate the ground acceleration a_g able to assure the respect of the I LS. The FE model of the tower was built with the code SAP2000 that was tuned in order to reproduce the experimental frequencies. The earthquake loads acting at the base of the tower were modelled by artificial accelerograms generated by using SIMQKE (synthetic accelerograms) and REXEL (natural accelerograms). Different class of subsoil were considered to develop parametric analyses. The PGA of the considered input are reported in Figure 8 (according to the [6], 7 accelerograms were generated for each input), and an example of an accelerogram of each input is reported in Figure 8.

Types of accelerograms	Ground type	PGA [m/s^2]
Natural accelerograms		
INPUT 1	A	2.45
INPUT 2	B	2.21
INPUT 3	Subsoil of San Gimignano	2.56
Synthetic accelerograms		
INPUT 4	B	1.99

Table 10: Types of accelerograms.

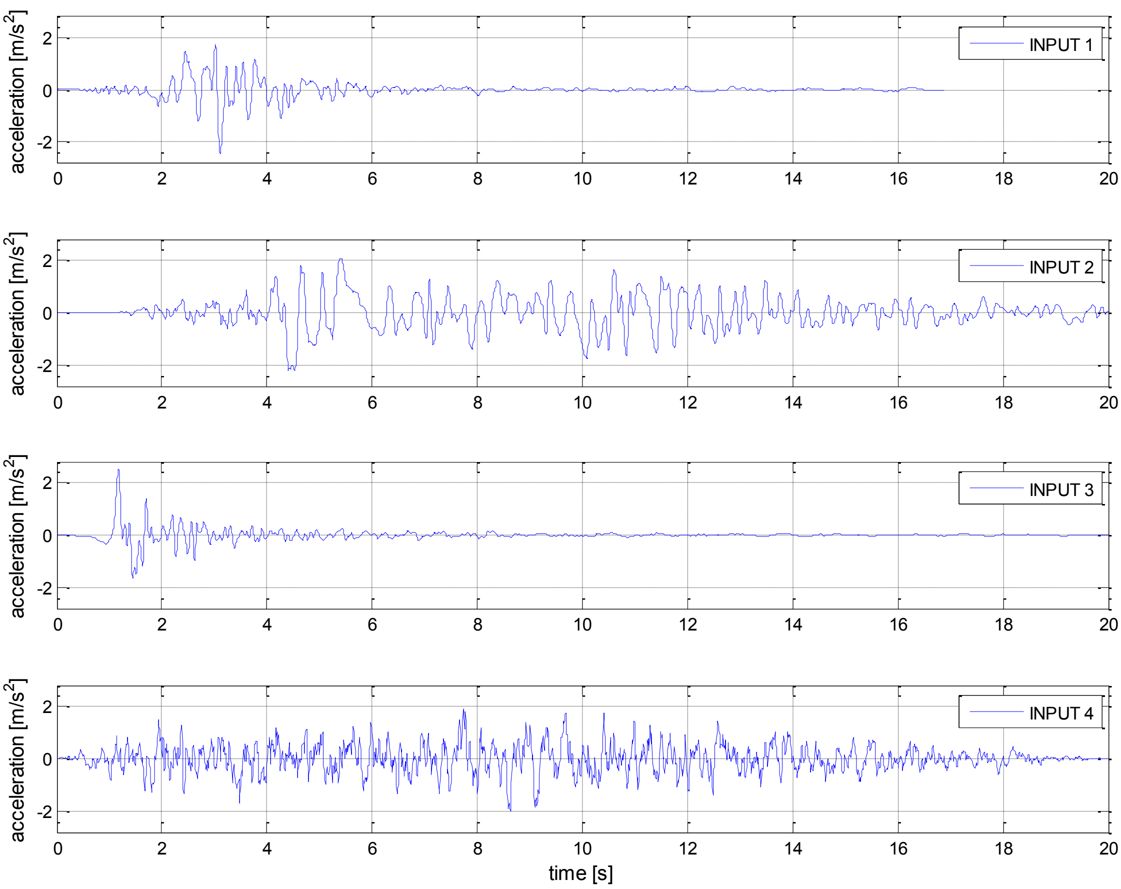


Figure 8: Time history of acceleration.

The accelerograms were applied on all restrained joints of the identified numerical model, and the time-history of the loads acting at every section [z] of the tower were evaluated: shear force [T(z; t)], normal force [N(z)] and bending moment [M(z; t)]. It is noteworthy to specify that due to the structural configuration of towers (which act structurally as cantilever beams), the internal forces acting at each level are statically determined and therefore they can be estimated by a simple dynamic linear model.

After the evaluation of the time-history of each internal action, the time history of the eccentricity was evaluated:

$$e(z,t) = \frac{M(z,t)}{N(z)} \tag{6}$$

Figure 9, as an example, reports the time-history of the eccentricity for one of the considered accelerogram. The verification of the I LS (tower over-turning) is ruled by following inequality:

$$|e_{max}| \le |e_{lim}| \tag{7}$$

where e_{max} is the maximum value assumed by e(z,t) during the loading process, while e_{lim} is the value of the eccentricity of the normal force originating the over-turning (equal to the half-length of the tower section).

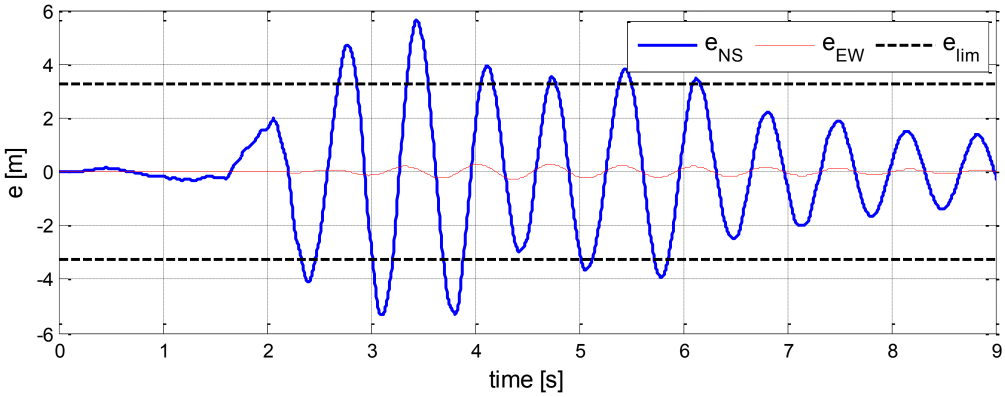


Figure 9 : Time-history of e(z,t) corresponding to a_g (INPUT 2)

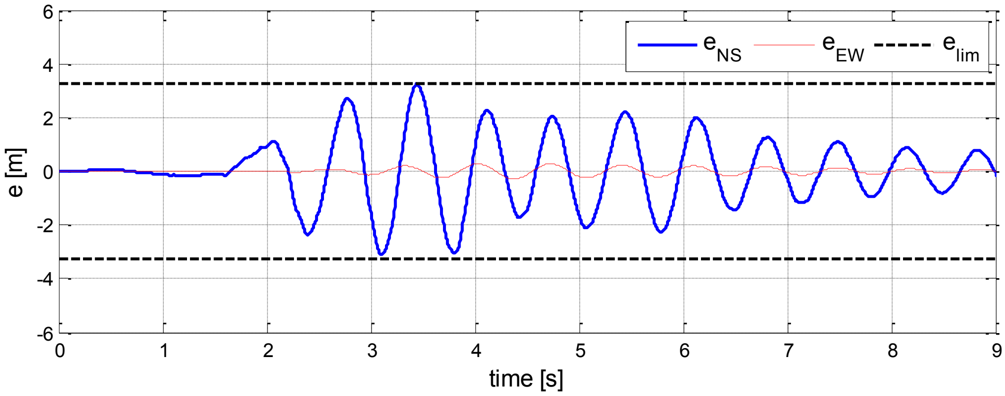


Figure 10 : Time-history of e(z,t) corresponding to reduced acceleration (INPUT 2)

The procedure leads to evaluate a coefficient of reduction of the seismic input able to assure the respect of the I LS:

$$\alpha = \frac{e_{max}}{e_{lim}} \tag{8}$$

The maximum acceleration that the tower can accept without over-turning is hence $a_{g1}=a_g/\alpha$. Figure 10 report the time-history of the eccentricity e(z,t) after the reduction of the seismic input.

The reduction coefficient was evaluated for each accelerogram of each group and the results in terms of α-factor (in both direction) are summarixed in Figure 11. Despite its semplification the index allows a synthetic and effective representation of the seismic behaviour of the tower. The value of the α-factor shows, for instance, a substantial structural simmetry of seismic response of the tower. In addition it is possibile to observe a strong dependence on the soil chacteristics. Highest α-factor are required when natural accelerograms with ground type A (INPUT 1) are considered. The α-factor obtained with the INPUT 2 (obtained assuming a ground type B) and INPUT 3 are similar. The INPUT 3 corresponds to the subsoil of San Gimignano and it is interesting to observe that the PGA of this class of input is greater than the PGA of the INPUT 1.

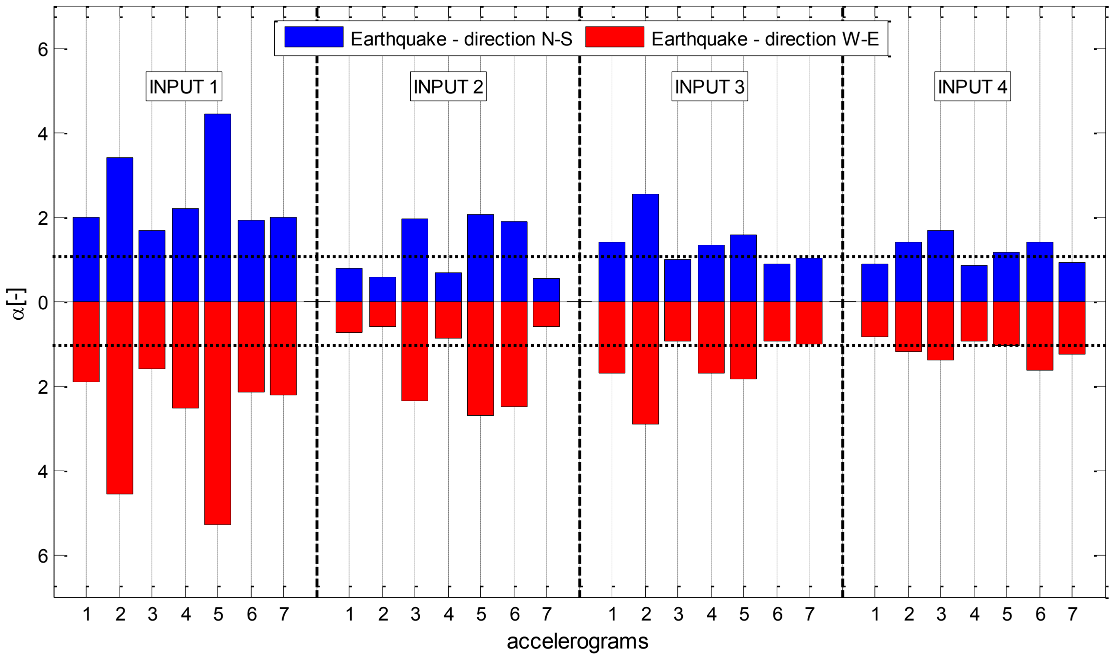


Figure 11 : α-factors.

CONCLUSION

The paper discussed about the methodology of analysis articulated on three levels of evaluation, according to an increasing knowledge of the structure, proposed by the Italian "Guidelines for the assessment and mitigation of the seismic risk of the Cultural Heritage". As a reference case, three of the masonry towers analysed within the research project RiSEM (Seismic Risk of Monumental Buildings) were discussed reporting the analyses performed to evaluate their seismic risk. The analyses were carried out by an articulated series of comparative assessments, aimed at investigating the effects of complementary and parametric hypothesis. The effects of the presence of the adjacent constructions at the lower lever, for instance, was assessed investigating the seismic response of the structures as a function of the degree of constraint provided by them.

The analysis at territorial scale (LV1) does not revealed critical situations. The last level of analysis, the LV3, was approached through global finite element models of the towers where proper damage models were employed to reproduce the masonry non-linear behaviour. The finite element models were employed to perform static nonlinear analyses and linear time-history analyses and the effects of the neighbouring buildings at the lower level of the towers were evaluated by analysing two limit cases: a) isolated tower and b) confined tower. The safety indexes (acceleration factor and seismic safety index) evaluated with the third level (LV3) confirm the results obtained with the first one and the LV3 safety indexes are always greater than those obtained with the LV1 model showing, despite the difference, a general coherence of the two models.

In addition, through a simplified approach, time-history analyses were performed in one of the case study and the results highlighted the strong dependence of the first safety index (the α-factor) on the energy content of the assumed accelerometer.

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