Comparative seismic assessment of masonry towers through nonlinear analysis: The RiSEM experience.

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Comparative seismic assessment of masonry towers through nonlinear analysis: The RiSEM experience

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ABSTRACT: Masonry towers are diffused through the whole European territory; these structures show unique typological and morphological features that might lead to severe effects under horizontal loads (such as the earthquake loads). Recently, the seismic assessment of historical masonry structures has been specifically taken into account by the Italian Guidelines. These recommendations propose a methodology of analysis based on three different levels of evaluation, according to an increasing knowledge of the structure. The last level of evaluation (LV3) requires a global seismic analysis of the whole structure by proper nonlinear numerical models. In general, the reliability of these tools depends on the achieved level of knowledge and, in this respect, a fundamental task is the estimation of the uncertain parameters (material properties, boundary conditions, etc.) affecting the structural behaviour. The paper aims to approach the effects of the uncertain parameters on the global structural response through the discussion of an illustrative case study: the historic masonry towers in the city centre of San Gimignano (Siena, Italy). The seismic risk of these towers was analysed in the framework of the research project RiSEM ("Seismic Risk of Monumental Buildings", a project granted by Tuscany Regional Administration in the period 2011–2013) and the availability of experimental data on as many as fourteen towers with similar features, make the case study particularly relevant. The results of the LV3 approach carried out on some of the historic towers are summarized in the paper and useful conclusions are drawn in order to quantify the effects of the uncertainties on the seismic risk of such structural typology when performing a LV3 analysis through nonlinear models.

Keywords: historic masonry towers, seismic assessment, nonlinear analysis, Italian guidelines, finite element modelling

1 INTRODUCTION

The assessment of the structural behaviour of historic towers under seismic loading is an important task. These structures, widely diffused through the whole European territory, were built to withstand only vertical loads and consequently, due to their typological and morphological features, horizontal loads may have severe effects on their global stability.

Indeed, their high slenderness combined with high masses make the masonry towers particularly vulnerable with respect to the seismic loads. These aspects have been taken into account by the Italian Guidelines (DPCM2011), and a chapter of these recommendations is specifically devoted to such structural typology. The analysis methodology proposed by the guidelines is based on three different levels of evaluation, according to an increasing knowledge of the structure. The last level (LV3) consists in a global analysis of the whole structures under seismic loading performed 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 a great computational effort depending on the employed numerical approach and on the typology of employed nonlinear numerical analysis (static or time-history).

The effectiveness of these tools to investigate the seismic behaviour of masonry towers was analysed by Pintucchi & Zani (2014). The authors compare nonlinear static (pushover) and time-history analyses in several case studies, and the paper highlights the advantages of the pushover approach allowing to consider the nonlinear behaviour and to preserve simplicity of the static analysis. Other comparative studies were developed by Casolo et al. (2013) on ten masonry towers in the costal Po Valley (Italy). To analyse the seismic behaviour of the towers, full nonlinear time-history analyses were performed employing two-dimensional (2D) numerical models. Results of the analyses confirm that slenderness and base shear area play a crucial role on the tower seismic vulnerability. A comprehensive study, comparing pushover analyses and full non-linear time-history analyses, was carried out by Acito et al. (2014) to analyse a clock tower in Finale Emilia (Italy), collapsed after the May 2012 Emilia Romagna earthquake sequence.

All these studies, on the one hand demonstrate the need to perform nonlinear analyses in order to obtain exhaustive information on both the failure mechanisms and the areas which undergo severe damages (needed to plan efficient interventions). On the other hand, they also highlights the need to quantify the effects on the results of the uncertainties (material properties, boundary conditions, etc.) that unavoidably affect an historic construction.

This paper moves within the last line discussing an illustrative case study, useful for parametric consideration about the effect of the uncertainties on the seismic behaviour of the towers (as evaluated by means of nonlinear static analyses). In the following, after a brief introduction of the RiSEM project (Seismic Risk of the Monumental Buildings), the analyses performed on four medieval towers on the city centre of San Gimignano (Italy) are reported. Some parametric considerations are developed in order to quantify, when performing a LV3 approach through nonlinear models, the effects of the uncertainties on the seismic risk of such structural typology.

2 THE RISEM EXPERIENCE

The seismic risk of the historic towers of San Gimignano was recently analysed, as an illustrative case study, within the research project RiSEM.

The project was 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. Two Italian Universities (Florence and Siena) through four University Departments (from different scientific areas) developed the whole project, granted by Tuscany Regional Administration in the period 2011–2013. 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 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 risk 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). The Guidelines identify a methodology of analysis based on three levels of evaluation. 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.

3 CASE STUDY

The historic towers of San Gimignano date back to XII-XIII century. San Gimignano, in its period of maximum splendour, had over seventy tower-houses (some as high as 50 m); today only 13 of these towers have survived (Figure 1). The paper reports about the seismic assessment of four of these towers: Coppi-Campatelli tower (CC), Chigi tower (CH), Becci tower (BE) and Cugnanesi tower (CU).

Figure 1. City centre of San Gimignano.

The sustaining walls of the towers are multi-leaf stone masonry, walls with the internal and external faces usually made with the same typology of material (and also, presumably, the same thickness).

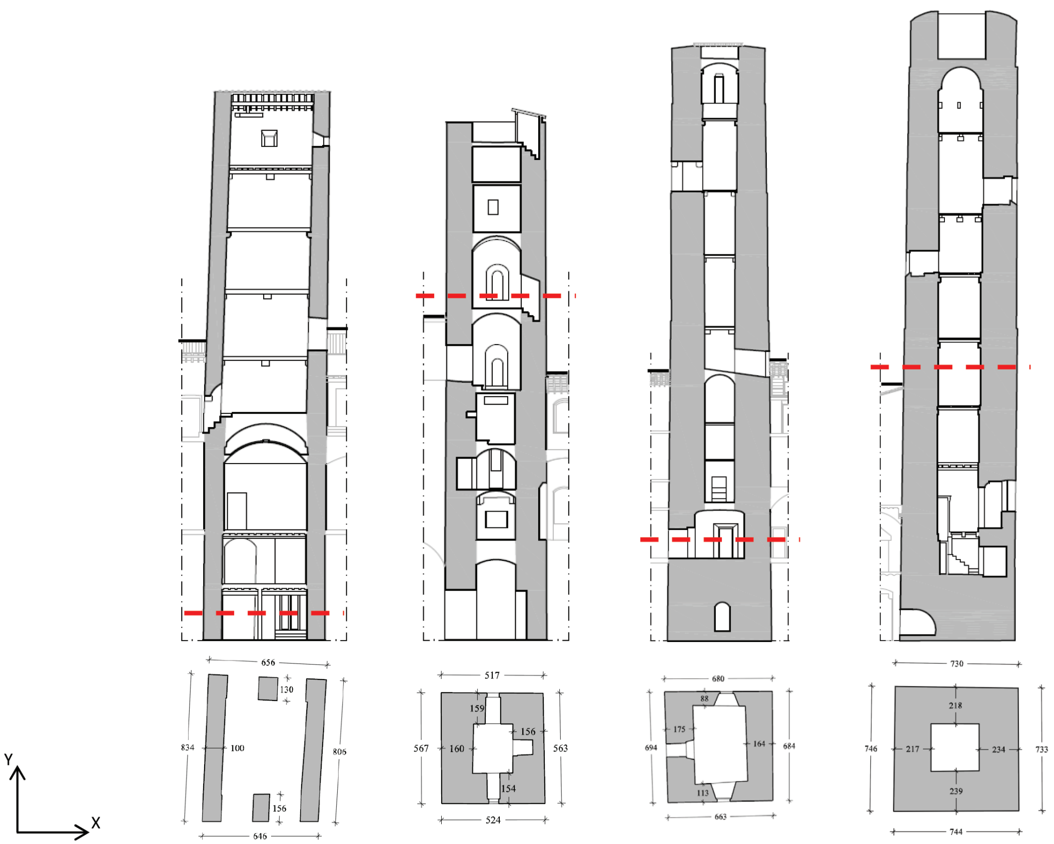
The internal core of the multi-leaf walls is composed of heterogeneous stone blocks tied by a good mortar.

Sections of the analysed towers are reported in Table 1 together with their geometric dimensions. 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 four 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.

Within the RiSEM project, experimental tests were performed and dynamic parameters, like main frequencies of some towers, were acquired. For two of the four towers herein considered, the frequencies are known and reported in Table 2.

When the experimental frequencies were available, numerical models of the towers were identified in order to fit the experimental data. Modal analyses were carried out varying the stiffness of the elastic constraints (represented as solid elements, Figure 2) that account for the neighbouring buildings until numerical and experimental frequencies they agreed. In absence of experimental results,

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



		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

Table 2. Results of dynamic survey (f—frequency).

Tower	Direction	f
		Hz
Becci	N-S (Y)	1.37
Cugnanesi	N-S (Y)	1.46
	E-W (X)	1.31

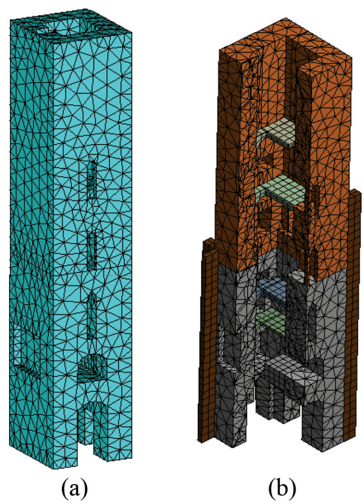


Figure 2. FE model of the Chigi tower: (a) Isolated tower; (b) Confined tower.

Table 3. Mechanical properties according to the NTC-Instruction 2009 (f_c – compressive strength, τ_0 – shear strength, E—modulus of elasticity, correction factor corresponding to (a) thick or poor internal core, (b) good quality of the mortar and (c) thin joint).

Type of masonry	Mechanical properties			Correction factors		
	f_c	τ_0	E	(a)	(b)	(c)
	N/cm^2	N/cm^2	N/mm^2			
B	200	3.5	1020	0.8	1.4	1.2
	300	5.1	1440			
D	140	2.8	900	0.9	1.5	1.5
	240	4.2	1260			
E	600	6.0	1200	0.7	1.2	1.2
	800	9.2	1800			

mechanical parameters were chosen according to the NTC Instruction 2009 (Table 3).

4 NONLINEAR ANALYSES

To investigate the global behaviour of the towers under seismic action, nonlinear static (pushover) analyses were performed by using Finite Element (FE) models of the towers.

According to the pushover approach, the analyses were developed by monotonically increasing, under conditions of constant gravity loads, a uniform profile of horizontal loads. Loads then resulted as directly proportional to the towers masses (uniform distribution), and this means that displacements at lower levels are overestimated, while the opposite happens on the displacement at the top levels. It is noteworthy 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 and then 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 distribution along the height. Hence the hypothesis of invariance of static loads could cause an overestimation in the analysis of the masonry building seismic capacity especially when a non-uniform damage or a high level of cracking are expected. 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 considering all the main directions (+/-X and +/-Y, Table 1), and the comparison between the results was performed analysing the capacity curves (generalized force–displacement relationship). Capacity curves have been built by assuming the base shear and the top displacement of each tower as control parameters.

4.1 *FE models*

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 finite element code. The numerical models were built to accurately reproduce the geometry of the structures and to include, when existing, the masonry vaults; internal wooden slabs were not modelled. Major openings of the walls of the towers (doors, windows, recesses, etc.) were also reproduced. As an example, Figure 2 reports the numerical model of the Chigi tower.

The mechanical parameters were evaluated, due to the lack of experimental data, by taking into account some existing provisions (such those provided by Italian recommendations); nevertheless, 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. Some comparative analyses were carried out to account for the effects of the adjacent structures. The interaction with the adjacent buildings was reproduced by modelling the walls perpendicular to the perimeter of the tower (see Figure 2b as an example) and as elastic stiffness. This investigation is quite important since the presence of confining structures can be an effective constraint for the tower, strongly influencing its seismic vulnerability. On the other way, the confining structures, while reducing the slenderness of the tower, can originate points of stress concentrations and pounding. The observation of the post-earthquake damages clearly shows the different seismic behaviour between isolated and constrained (i.e. connected to walls) towers (Cattari et al. 2014).

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

- Isolated Tower (IT) modelling: the tower is considered as 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 neighbouring structures);
- Confined Tower (CT) modelling: 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 analysed scenarios is to identify lower and upper bounds for the identification of the towers structural behaviour.

Two different mechanical laws were finally employed to reproduce the nonlinear behaviour of the masonry:

- the continuum damage model introduced by Mazars (1984) and Mazars et al. (1989), for the Coppi-Campatelli tower FE model;
- the combination of the Drucker-Prager yielding behaviour for compressive stresses (f_c) and the Willam-Warnke cracking behaviour for tensile stresses (f_t) for the other three towers (Chigi, Becci and Cugnanesi).

Table 4. Mechanical (strength) parameters of the investigated towers (f_c compressive strength; f_t tensile strength).

Tower	f_c	f_t
	MPa	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

Table 5. Combination of mechanical parameters used for the analysis of Chigi tower.

	f_t	f_c	E
	MPa	MPa	MPa
CH_{A1}	0.106	0.493	1458
CH_{A2}		0.493	2916
CH_{A3}		0.986	1458
CH_{A4}		0.986	2916
CH_{B1}	0.212	0.986	1458
CH_{B2}		0.986	2916
CH_{B3}		1.973	1458
CH_{B4}		1.973	2916

Table 6. Elastic and mass parameters of the investigated towers (E Young's modulus; ρ mass density).

Tower	E	ρ
	MPa	kN/m^3
CC	1230–2800	20–22
CH	1458–2916	16–18
BE	1350	16
CU	2800	22

For the Chigi tower only, 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 (fc), the tensile strength (ft) and the Young's modulus (E), according to the values reported in Table 6.

4.2 *Pushover capacity curves*

In this section, some pushover curves are reported in order to assess the effect of some uncertain parameters: restraint and material property. The first comparison was done analysing the three towers CH, BE and CU, modelled by using ANSYS, with respect to the isolated configuration. The strength parameters are reported in Table 4 and Table 5, while the elastic and mass parameters are shown in Table 6. According to the values reported in the tables, comparable strength parameters were selected.

Figure 3 reports the pushover curves (along the weakest direction), where global base shear V_b is reported as a fraction of the total weight W. Despite the towers are quite similar (in terms of geometry), a similar behaviour 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 period.

Furthermore, in order to analyse the influence of the strength parameters, in Figure 4 the pushover curves of confined Chigi tower according to parame-ters as in Table 6 are reported. In this case, it is possible to identify a common behaviour, being the dif-ference in parameters values mainly affecting the ultimate displacement.

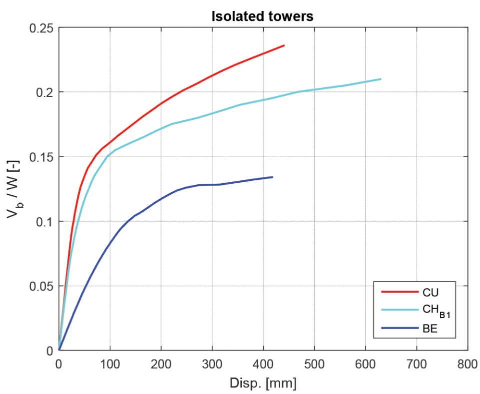


Figure 3. Comparison between isolated towers.

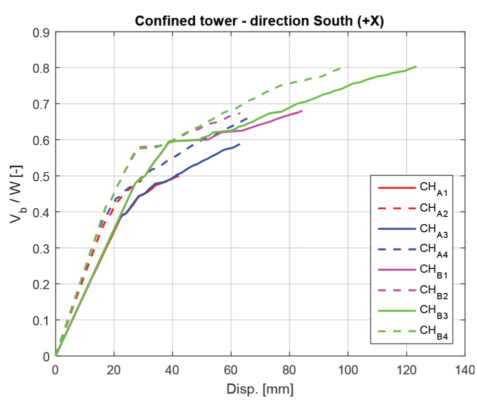


Figure 4. Pushover curves, +X direction. Comparison between capacity curves of different material property of CH tower.

In order to analyse the influence of confining buildings, in Figure 5 the behaviour of an isolated tower compared with the confined configuration is reported. In particular, the figure refers to the case of Becci tower. By analysing the pushover curves, it is to be highlighted that the ratio between the maximum values of the base shear (compared to the weight of the tower) in both cases is almost proportional with the ratio between the lengths of the unrestrained portions in these two configurations. Even if this result is quite obvious when dealing with cantilever structures whose behaviour is mainly due to their bending resistance, a similar behaviour 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 to be noticed that, when the experimental frequencies were available, 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 behaviour of the tower when a seismic event have caused the detachment of the neighbouring buildings.

Lastly, Figure 6 reports pushover curves of Coppi-Campatelli tower (by using Code-Aster). In this case, the experimental frequencies were not available; the confined configuration was done with two different set of lateral restraint (CT1, CT2) to identify limit cases.

Just as an example, the following Figure 7, reports 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 safety assessment was carried out through the Capacity Spectrum

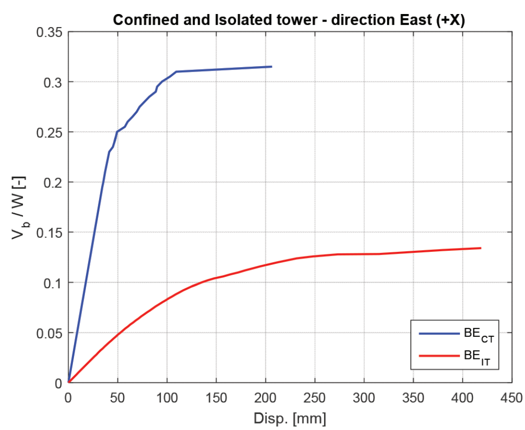


Figure 5. Pushover curves, +X direction. Comparison between isolated and confined configuration for BE tower.

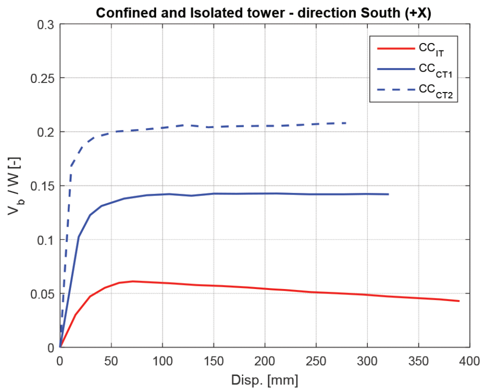


Figure 6. Pushover curves - +X direction. Comparison between confined and isolated configuration for CC tower.

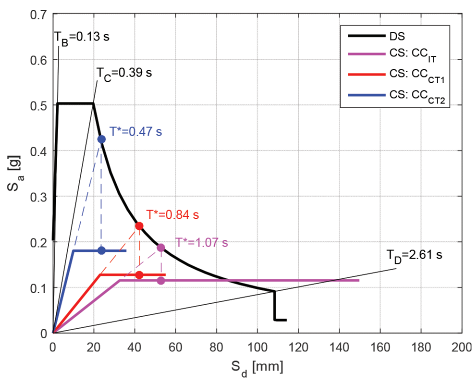


Figure 7. Comparison between the Demand Spectrum (DS) and the Capacity Spectra (CS), IT—Isolated tower, CT—confined tower. Coppi-Campatelli tower.

Method (CSM) according to the performance-based seismic analysis technique, describing the capacity curve and the response spectrum in terms of spectral acceleration and displacement in the Acceleration-Displacement Response Spectra (ADRS) format. The CSM provides an effective graphical evaluation of the seismic behaviour of the construction since the intersection of the capacity spectrum with the demand spectrum identifies a point (performance point), representing the condition for which the seismic capacity of a structure is equal to the seismic demand. Figure 7 reports, in particular, a ADRS check for Coppi-Campatelli tower.

It is quite clear that the change in the confinement given by adjacent buildings can change the structural behaviour so dramatically that it can compromise the seismic performances of the tower. When the isolated tower is capable to withstand the seismic action (thanks to its sufficiently high displacement ductility more than due to its resistance), as soon as the restraint offered by adjacent buildings is considered as effective the capacity is no more adequate to the required seismic demand. Even if Figure 7 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 towers performances.

5 CONCLUSIONS

Conventional pushover analyses have been performed on four masonry towers in the city centre of San Gimignano. Each tower has been studied in depth by considering uncertain mechanical, mass and strength parameters. The aim of the study was to identify the boundary condition of the behaviour of the towers under seismic action representing a fundamental aspect especially when the level of knowledge is low and experimental tests are neither available nor feasible.

Previous studies have already stressed the importance of some parameters such as the slenderness and the resistant shear base area. In the paper, the role of neighbouring buildings on the towers behaviour, mainly influencing the displacement ductility level, has been discussed and analysed.

Further aspects that shall be further analysed are: a) the increasing of the number of case study (to make statistically significant the comparison a wider range of examples will be included); b) the comparison of the results of static nonlinear analysis with dynamic nonlinear analysis. From the results herein presented, 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.

The structural response was investigated by using a nonlinear static analysis approach. The results of the pushover analyses in terms of capacity curves were employed to perform a safety check according to the Capacity Spectrum Method (CSM). It resulted once more how strongly the effect of confinement reflected on tower performances, then evidencing the needing of more accurate investigations about the effective portion of the tower to be considered as unrestrained with respect to adjacent buildings.

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