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SEISMIC VULNERABILITY OF DIFFERENT IN GEOMETRY HISTORIC MASONRY TOWERS

Vasilis Sarhosis[1], Francesco Fabbrocino[2], Antonio Formisano[3], and Gabriele Milani[4]

[1] Newcastle University
NE1 7RU, Newcastle upon Tyne, UK
vasilis.sarhosis@ncl.ac.uk

[2] Università Telematica Pegaso
Piazza Trieste e Trento 48, Naples, Italy
francesco.fabbrocino@unipegaso.it

[3] Università di Napoli Federico II
Piazzale Tecchio 80, Naples, Italy
antoform@unina.it

[4] Technical University of Milan
Piazza Leonardo da Vinci 32, Milan, Italy
francesco.fabbrocino@unipegaso.it

Keywords: Masonry Towers, Predicting formulas for seismic vulnerability, UDEC and 3Muri, Simplified approach, Acceleration factor.

Abstract. *In the present paper, a simple predictive approach for the seismic vulnerability of existing masonry towers is proposed evaluating it on a series of "idealized" benchmark cases using different simplified approaches, namely the procedure proposed by the Italian code and pushover conducted with two commercial codes (UDEC and 3Muri). In UDEC the geometry is intentionally idealized into quadrilateral elements with different thickness, in order to properly reproduce the hollow square cross-section. The utilization of a 2D approach drastically reduces the computational effort required in carrying out medium scale systematic computations. In 3Muri macro-elements are used, providing very fast predictions as well. Within such simplified frameworks, 16 different cases that can be encountered in practice are critically analyzed, changing two key parameters that proved to be important for the vulnerability determination, namely slenderness and transversal shear cross area.*

The simplifications introduced in the modelling phase allow for fast sensitivity analyses in the inelastic range and an estimation of the acceleration factor in that range of slenderness that is useful for practical purposes. Simplified formulas fairly representing the obtained seismic vulnerability are also reported and put at disposal to any practitioner interested in a preliminary estimation of the behavior of the towers before doing any calculation.

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For validation purposes, the results obtained previously by one of the authors by means of refined full 3D Abaqus discretization on 25 existing towers located in the Northern Italy are also reported. Good agreement between the predictions provided by the simplified method here proposed and previously presented reference data is obtained.

1 INTRODUCTION

The preservation of the architectural heritage is a task of great societal importance for developed countries in Europe and technically a very challenging aim, especially in seismic area. Masonry towers in form of medieval defense, clock and churches bell towers are quite diffused all over Europe and are an important part of the historical and architectural heritage to be preserved. Recent seismic events have highlighted that ancient masonry towers are particularly susceptible to damage and prone to partial or total collapses under earthquake excitations. The safety assessment of such unique masterpieces against horizontal loads is therefore paramount and this paper deals with such particular topic. As a matter of fact, old masonry towers usually show peculiar morphologic and typological characters, which are at the base of all the difficulties encountered in the recent past to find a standardized methodology to predict their behavior under horizontal loads and hence give a reliable safety assessment.

In ancient times, towers were exclusively conceived to be able to withstand vertical loads. Recently, however, national [1]-[3] and international standards [4] have imposed the evaluation of the structural performance in presence of horizontal loads, which simulate earthquake excitations, encouraging the use of sophisticated non-linear methods of analysis.

According to the previous remarks, it is pretty clear that the most accurate approach to deal with the analysis of masonry towers under horizontal loads should require specific ad hoc FE devices [5]-[19] in order to deal with the complexity of the problem through a suitable level of accuracy.

However, in engineering practice, the utilization of non-linear methods and full 3D Finite Element models is not so common, because commercial codes with advanced material models should be adopted by users that are supposed to have a strong mechanical background and deep knowledge on sophisticated non-linear analyses conducted with FEs.

To cope with this key issue, the Italian code for the built heritage [3] allows evaluating the seismic vulnerability of masonry towers by means of a simple cantilever beam approach, where only flexural failure is taken into consideration. Such procedure is very straightforward and can be tackled even by unexperienced practitioners without the need of using any FE code. The drawback is represented by the impossibility to account for a combined shear and flexural failure of the towers, which in practice is common in case of low slenderness.

In order to put at disposal to practitioners some formulas to preliminarily estimate the seismic vulnerability of an existing tower (without the need to perform any calculation), in the present paper we analyze a series of "idealized" benchmark cases using different simplified approaches, namely the procedure proposed by the Italian code and pushover conducted with two commercial codes (UDEC [20]-[26] and 3Muri [27]). The geometry is intentionally idealized into parallelepiped blocks with hollow square cross-section, thus favoring the utilization of 2D approaches, in order to drastically reduce the computational effort required in carrying out medium scale systematic computations. Within such simplified framework, 16 different cases that can be encountered in practice are critically discussed, changing two key parameters that proved to be important for the vulnerability determination, namely slenderness and transversal shear cross area.

The simplifications introduced in the modelling phase allow for fast sensitivity analyses in the inelastic range and an estimation of the vulnerability in that range of slenderness that is useful for practical purposes. Simplified formulas fitting the obtained seismic vulnerability are also reported and put at disposal to any practitioner interested in a preliminary estimation of the behavior of the towers before doing any calculation.

For validation purposes, the results obtained previously by one of the authors [15]-[18] by means of refined full 3D Abaqus [28] discretization on 25 existing towers in Northern Italy are also reported. Good agreement between the predictions provided by the simplified method here proposed and real data is obtained.

2 SENSITIVITY ANALYSIS CONDUCTED

The sensitivity analysis conducted in the present paper is aimed at covering the majority of the real cases that can be encountered in practice. It relies into the investigation of the structural behavior of 16 “ideal” masonry towers, with different geometric features, such as a variety of heights, thicknesses and transversal cross sections, as illustrated in Figure 1. Intentionally, the ideal towers do not exhibit any form of irregularity, such as changes of thickness of the perimeter walls, presence of perforations of any kind (doors, windows, bell cells, etc.) and internal walls, stairs or vaults. The aim is indeed to simplify the approach to a great extent, in order to provide results in terms of seismic vulnerability that are dependent on only two geometric parameters, namely slenderness and cross shear area.

REGION	B (m)	b (m)	H (m)	t (cm)
	Base edge length	Base edge length	Height	thickness
Abruzzo	4–10	4–10	20–50	130–150
Campania	6–13	5–13	30–75	60–100
Emilia-Romagna	2–12	2–12	16-87	45-160
Marche	2.50-9	2.50-8	16-45	60-120
Molise	5-6.50	5-7	20-35	100-200
Toscana	5-10	6-10	27-55	130-260
Veneto	4-15	4-10	20-58	80-200

Table 1: Initial survey conducted in different Italian regions to investigate the typical geometrical features most diffused in the national territory

According to a preliminary survey conducted in some Italian regions and synoptically reported in Table 1, the ideal towers exhibit typical slenderness λ and normalized shear area ξ (defined respectively as λ=H/B and $\xi = \left[B^2 - \left(B - 2t \right)^2 \right] / B^2$, as depicted in Figure 2, where also minimum and maximum values of λ and ξ found during the survey are represented with green circles.

As can be noted, the ideal towers fit well the general geometric characteristics of the real towers, meaning that they can be used to have a rough prediction of real cases under seismic loads.

Each ideal tower is represented with its own symbol, differing in shape and color, so towers having small ξ s are depicted with cold colors (blue and cyan), whereas those with large ξ s with cool colors (yellow and red). Different λ s are represented with different symbols, namely squares, triangles, circles and diamonds. Each tower belonging to the same series (denoted with A, B, C and D) is characterized by the same ξ.

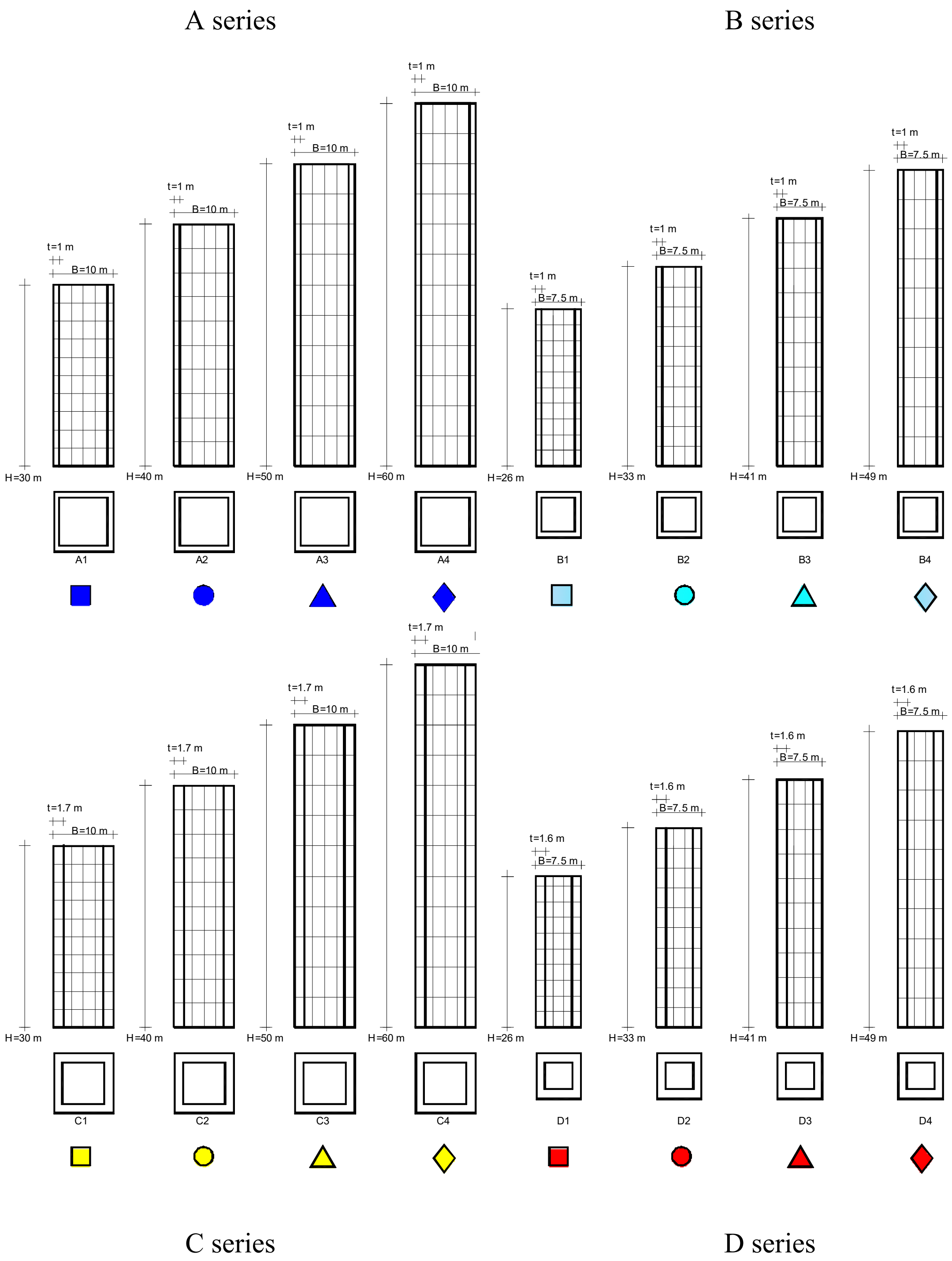

Figure 1: Geometric properties of the "ideal" case studies analyzed in the present paper. Each tower is labeled with a different symbol. Warm colors indicate large equivalent shear cross areas (>0.5), whereas cold colors indicate small equivalent cross areas.

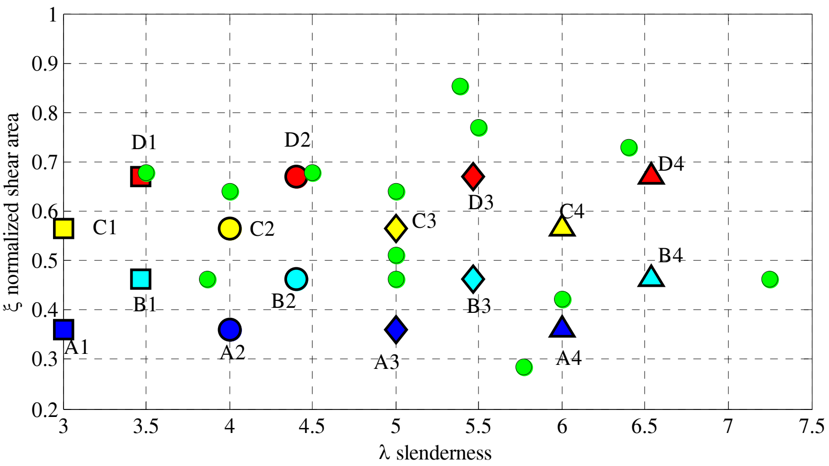

Figure 2: Relation between normalized shear cross area and slenderness for the different "ideal" towers analyzed for comparison purposes (green dots represent maximum and minimum values in different Italian regions, according to a survey by the authors).

3 METHODOLOGY OF EVALUATION OF THE SEISMIC VULNERABILITY

Three different numerical methods are utilized in this Section to have a quantitative insight into the seismic vulnerability of the towers, namely the simplified approach according to Italian code specifics (also known as Equivalent Static Analysis ESA), a distinct element approach carried out within UDEC code and pushover analyses with 3Muri.

3.1 Italian code simplified approach (Equivalent Static Analysis ESA)

According to the Italian Guidelines for the built heritage, equivalent static analyses (ESAs) should be carried out to estimate the seismic vulnerability of a masonry tower. They are conducted according to § 5.4.4 of the Guidelines [3], subdividing the tower in blocks with horizontal cross sections and adopting a distribution of horizontal forces on the blocks proportional to the product $W_i z_i$, being W_i the weight associated to the i-th block and z_i the vertical position of its barycenter. When evaluating the resultant horizontal force as F_h = 0.85 $S_e(T_1)$ W/(qg), reference is made to the elastic response spectrum S_e provided by the NTC 2008 [1], reduced by the behavior factor q = 3.6 suggested by the above Guidelines in the case of geometry and mass regularity along the height. The spectral ordinate corresponding to the fundamental period T_1 is here referred for a seismic zone Z_1 by EC8 with soil D. Italian code is not utilized in this case because the spectrum is given there only knowing the latitude and longitude of tower location instead giving distinct seismic zones.

Period T_1 can be evaluated rigorously in this case using well known results on vibration of Euler-Bernoulli beams. In particular the frequency, assuming a cantilever beam hypothesis is given by the following simple formula:

$$f_i = \frac{\alpha_i}{2\pi L^2}\sqrt{\frac{EI}{\mu A}} \qquad (1)$$

Where μ is the density of the structure, E is the Young modulus, A and I the cross section area and inertia moment, L the height and α_i =3.5156.

According to the Italian Guidelines, it is necessary to compare the acting bending moments on different transversal sections, within the application of equivalent static loads and under the hypotheses of class use and soil done, with the resisting ones.

For towers with rectangular section, FEM may be avoided and simplified formulas could be adopted according to Italian Guidelines specifics [22]. Under the hypothesis that the normal pre-compression does not exceed $0.85 f_d A$, the ultimate bending moment of a masonry rectangular sections is:

$$M_u = \frac{\sigma_0 A}{2}\left(b - \frac{\sigma_0 A}{0.85 a f_d}\right) \tag{2}$$

Where a is the transversal edge length of the section, b the longitudinal length edge, A the section area, $\sigma_0 = W/A$ the average pre-compression (W: tower weight above the section considered) and f_d the design compressive strength.

External moments, within a cantilever beam hypothesis (subdivided into n elements), may be evaluated at the generic section j as:

$$M_j = F_e z_j$$
$$z_j = \frac{\sum_{i=1}^{j} z_i^2 W_i}{\sum_{k=1}^{n} z_k W_k} \tag{3}$$

With $F_e = 0.85 S_d(T_1) W/g$ (S_d spectrum, T_1 first period of the structure, g gravity acceleration).

In order to evaluate the seismic vulnerability of the tower, the Italian code suggests the evaluation of the so-called acceleration factor $f_{a,SLV}$.

The acceleration factor is the ratio between soil peak accelerations corresponding to the capacity and the expected demand:

$$f_{a,SLV} = \frac{a_{SLV}}{a_{g,SLV}} \tag{4}$$

where a_{SLV} is the soil acceleration leading to the SLV ultimate state and $a_{g,SLV}$ is the acceleration corresponding to the reference return period. The acceleration factor is a purely mechanical parameter, which may be useful for an evaluation of the weakness of the structure in terms of strength.

The evaluation of the acceleration of the response spectrum corresponding to the instant where SLV limit state is reached on the i-th section can be obtained taking into account the reduction induced by the confidence factor as follows:

$$S_{e,SLV,i}(T_1) = \frac{q g M_{R,i} \sum_{k=1}^{n} z_k W_k}{0.85 W F_C \left(\sum_{k=1}^{n} z_k^2 W_k - z_i \sum_{k=1}^{n} z_k W_k\right)} \tag{5}$$

Where q is the behavior factor, g the gravity acceleration, $M_{R,i}$ is the resistant bending moment on the i-th section, z_k and W_k are the height and the weight in correspondence of the k-th section, respectively, W the total weight, FC the confidence factor (here assumed equal to 1.35), z_i the height of the i-th section with respect to the base and *n* the number of cross sections.

3.2 UDEC pushover analyses

UDEC (Universal Distinct Element Code), see for reference for instance [20]-[24], is a very general code, capable of modelling many types of jointed systems, ranging from assemblies of many discrete blocks to extended continua crossed by a few major fractures.

Among the most important capabilities of UDEC that make it very suitable for masonry, we could mention the possibility to choose rigid or deformable blocks, the ability to simulate progressive failure associated with crack propagation, the capability of simulating large displacements/rotations between blocks, the possibility to use rounded corners to overcome interlocking, etc.

Each deformable block is independently discretised into an internal element mesh. An explicit, large deformation, Lagrangian formulation with constant-strain triangles is used. While the original formulation follows the finite-difference approach, these triangular zones are equivalent to finite elements. General constitutive relations can be assumed for the block material.

The soft contact approach is used, so a finite normal stiffness is taken to represent the measurable stiffness that exists at a contact or joint. A joint is represented numerically as a contact surface formed between two block edges. The representation of the interface between blocks relies on sets of point contacts. For each pair of blocks that touch (or are separated by a small gap), data elements are created to represent point contacts. Adjacent blocks can touch along a common edge segment or at discrete points where a corner meets an edge or another corner. No special joint element or interface element is defined.

A point contact hypothesis is used. In the point contact approach, the interaction force at each contact is a function of solely the relative displacement between blocks at that location. This assumption implies that a larger number of contact points is needed to get an accurate contact stress distribution on the joint surface.

When two blocks come into contact, a force develops between them which can be resolved into normal and shear components, as shown in Figure 3 (left).

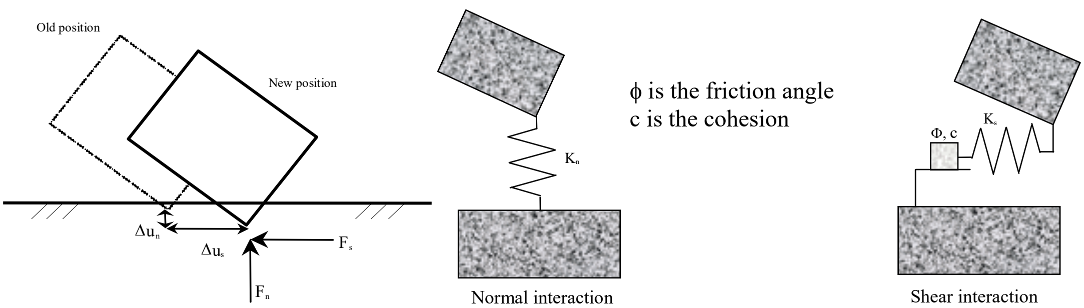

Figure 3: Forces between blocks (left) and Representation of joints within DEM (blocks are in contact, separation is shown for clarity)

Towers under consideration are discretized in UDEC as shown in Figure 1. Such discretization has the following characteristics:

1) It is two-dimensional, but takes into account the actual geometry assuming for the flanges (lateral walls) a thickness equal to B and for the core a thickness equal to 2t.
2) By means of the discretization adopted, the code can provide failure modes under a pure flexural behavior, pure shear, vertical cracks or a combination of the previous typical failure modes observed in practice.

A typical series of pushover curves obtained with UDEC is depicted in Figure 4 (only D typology is shown for the sake of conciseness).
Mechanical properties assumed for the masonry material require some discussion. As a matter of fact, According to Italian Code NTC2008 [1], Chapter 8, and subsequent Explicative Notes [2], the mechanical properties to assume for the masonry material depend on the so called knowledge level LC, which is related to the so called Confidence Factor Fc. There are three LCs, labeled from 1 to 3, related to the level of knowledge that one has on the mechanical and geometrical properties of the structure. LC3 is the maximum, whereas LC1 is the minimum. For the case at hand, obviously in absence of specific in-situ test results and with the aim of providing straightforward preliminary estimations, a LC1 level is assumed.
FC summarizes the level of knowledge regarding the structure and the foundation system, from a geometric and mechanical point of view. It can be determined defining different partial confidence factors FCk (k=1,4), on the base of some numerical coefficients presents in the Italian Code (Table 4.1 Italian Line Guides). Due to the limited level of knowledge achieved in this case, the highest confidence factor is adopted FC = 1.35.
Values adopted for cohesion and masonry elastic moduli are taken in agreement with Table C8A.2.1 of the Explicative Notes [2], assuming a masonry typology constituted by clay bricks (approximate dimensions 210x52x100 mm^3) with very poor mechanical properties of the joint and quite regular courses. Such kind of masonry is typical for towers located in the Northern Italy, but calculations can be repeated also assuming different mechanical characteristics according to Table 4.1.
Elastic and inelastic material properties utilized in all the simulations within the different analysis strategies are summarized in Table 2.

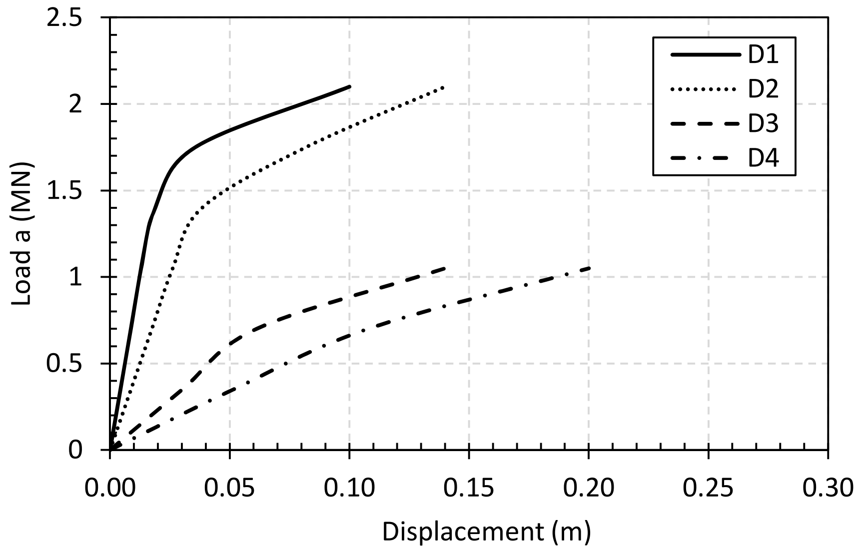

Figure 4: Typical pushover curves obtained with the software UDEC (D typology).

	f_m	τ_0	E	G	w
	MPa	MPa	MPa	MPa	kN/m^3
Masonry with clay bricks and poor mortar	2.4	0.06	1500	500	18

Table 2: Mechanical properties adopted for masonry and vaults infill

3.3 3Muri macro-elements analysis

The four types (A, B, C and D) of towers under investigation have been also modelled by means of the 3Muri macro-elements analysis software [27]. Four masonry macro-elements have been assembled all together with effective joints at their intersection in order to create the box structure of the towers, which have been covered with a plane bi-directional rigid floor at the top (Figure 5-a). Therefore, towers are susceptible to undergo in-plane mechanisms only under the formation of shear and compression-bending failures, whereas local out-of-plane collapses have not been taken into account.

The same mechanical properties assumed for the previous two models have been adopted in 3Muri, see Table 2. A cracked condition has been assumed for Young and Shear moduli of the masonry, which however does not affect the calculation of the acceleration factor. Linear dynamic (to estimate the first vibration mode) and non-linear static analyses have been performed on the towers considered with a spectrum as per EC8, according to previous computations.

The analysis of the collapse mechanisms detected from pushover analyses shows that, when seismic accidental eccentricity is considered, in all cases towers with the lowest slenderness (A1, B1, C1 and D1) exhibit the strongest coupling between translational displacement and torsion rotation (Figure 5-b). On the other hand, all the remaining towers have a less pronounced torsion behavior (Figure 5-c). However, in all investigated cases, towers show compression-bending plastic behaviour and collapses only, without exhibiting shear failures.

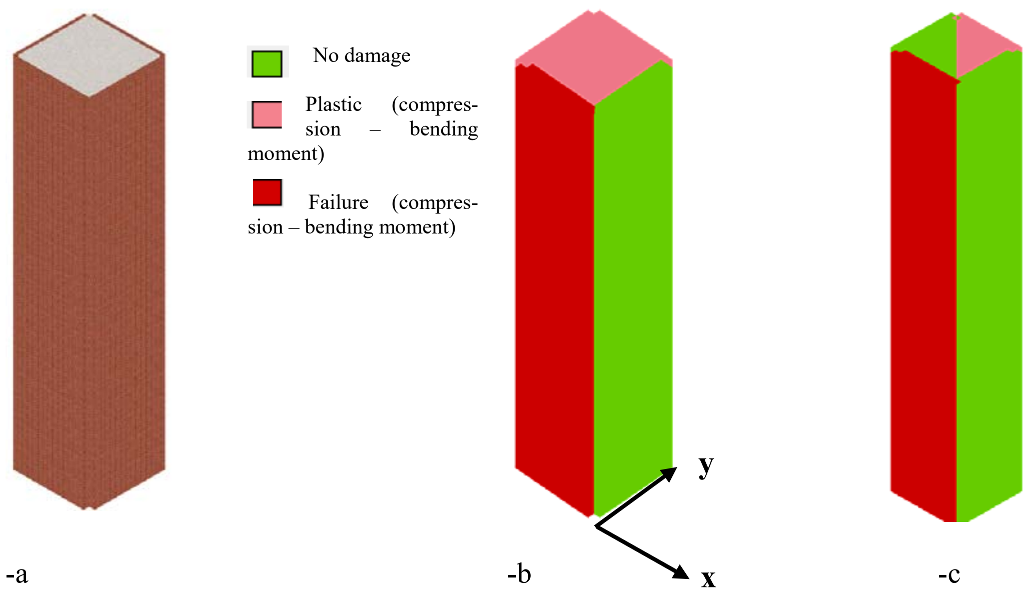

Figure 5: Seismic pushover analysis in direction x with accidental eccentricity: the 3Muri model (a) and collapse mechanisms for the shortest tower (b) and the remaining ones (c)

4 DISCUSSION OF THE RESULTS OBTAINED

Results obtained with the three models proposed in terms of acceleration factor (Z1 according to EC8) are depicted from Figure 6 to Figure 8. In particular, Figure 6 refers to Italian Guidelines, Figure 7 to UDEC and Figure 8 to 3Muri. In the horizontal axis, slenderness is represented.

In the same figures, the results obtained in previous studies by one of the authors [15]-[18] are reported for the sake of comparison and in order to benchmark the approach proposed against real cases that can be encountered in common practice. Such results, represented in the figures with green diamonds, have been obtained by means of sophisticated non-linear (both static and dynamic) analyses performed on detailed full 3D FE discretizations by means of the commercial code ABAQUS [28], assuming for masonry a Concrete Damage Plasticity (CDP) model. The reader interested in further details is referred to [15][16][29].

A fitting exponential function is also reported with the corresponding equation, in order to give the possibility to any practitioner interested to enter into the diagrams and predict an acceleration factor on a real tower without the need to perform any computation. As a matter of fact, only the value of slenderness is needed.

From an overall analysis of the obtained results, the following considerations are worth noting:

1) Italian Guidelines and 3Muri outputs are almost completely independent from the normalized cross shear area, as shown by Figure 6 and Figure 8, where blue symbols almost superimpose with the corresponding red ones. Such results are quite obvious, because the observed failure mechanisms in 3Muri are flexural and the Italian Guidelines a priori exclude shear failures. In addition, as far as the Italian Guidelines are concerned, the evaluation of the resistant bending moment by means of formula (2) is little influenced by walls thickness, and this explains the small differences observed between series D (large shear area) and A (small shear are).
2) UDEC results are quite sensible to shear area (see Figure 7), especially and as expected for low slenderness, i.e. where a shear failure is more likely. When slenderness increases, the two fitting curves (one for large shear areas the other for small shear areas) tend obviously to coincide, a clear indication that failure is purely flexural.
3) The vulnerability of the real 25 towers is generally well predicted by the fitting curves provided by all models. Italian Guidelines curve slightly overestimates the acceleration factor, clearly because it does not take into account the presence of irregularities.
4) Fitting functions provided by all models are almost superimposable, with probably the most accurate prediction provide by UDEC analyses, which also are sensitive to the different shear areas of the towers, thus providing also an implicit indication on the failure mode.

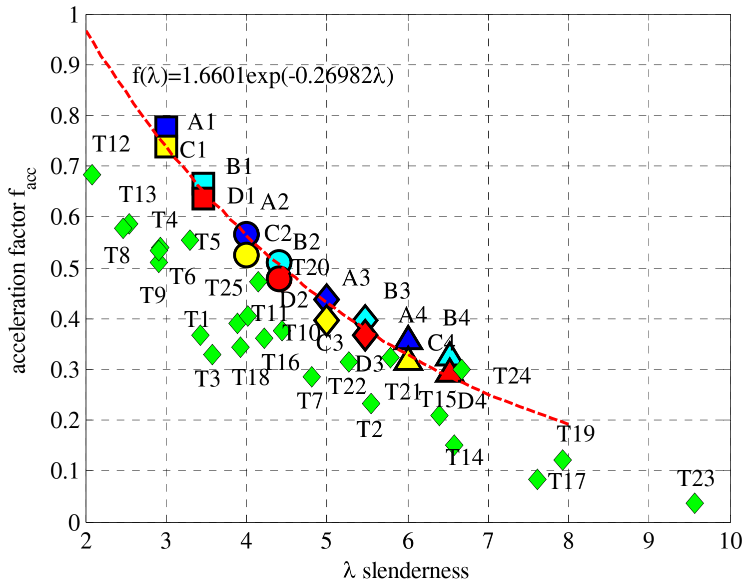

Figure 6: Italian Guidelines for the Built Heritage. Acceleration factor vs slenderness, comparison between idealized approach and real case studies.

5 CONCLUSIONS

The paper has presented a simplified fitting formula to predict, without any computation, the vulnerability of an existing masonry tower, with the only knowledge of the most important geometrical parameters of the structure, namely slenderness and cross shear area. Such formula has been calculated conducting a sensitivity analysis on 16 “idealized” towers without any irregularity and with different slenderness and shear area. Three different simplified approaches have been used to cross-check results, namely the procedure proposed by the Italian code and pushover conducted with two commercial codes (UDEC and 3Muri). In UDEC the geometry is intentionally idealized into quadrilateral elements with different thickness, in order to properly reproduce the hollow square cross-section. In 3Muri macro-elements are used and again very fast computations of the pushover curves have been provided.

The results have been also benchmarked using previously presented vulnerability studies conducted on 25 real case-studies, showing a very good agreement between the predictions provided by the simplified method and real data.

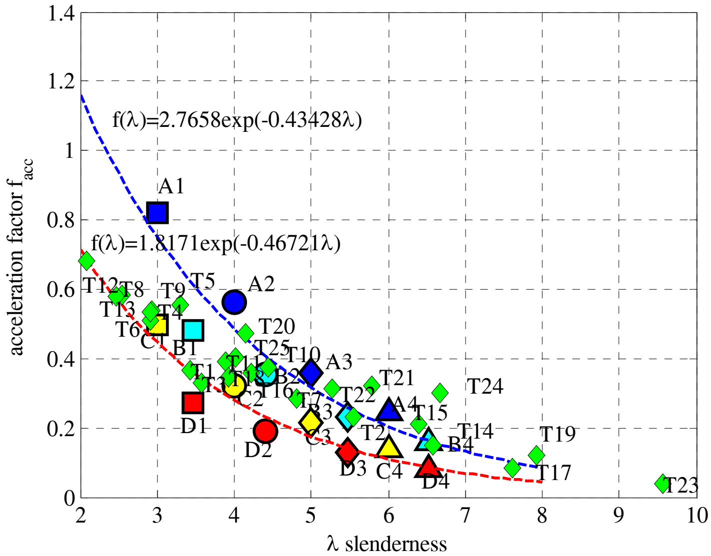

Figure 7: UDEC software. Acceleration factor vs slenderness, comparison between idealized approach and real case studies.

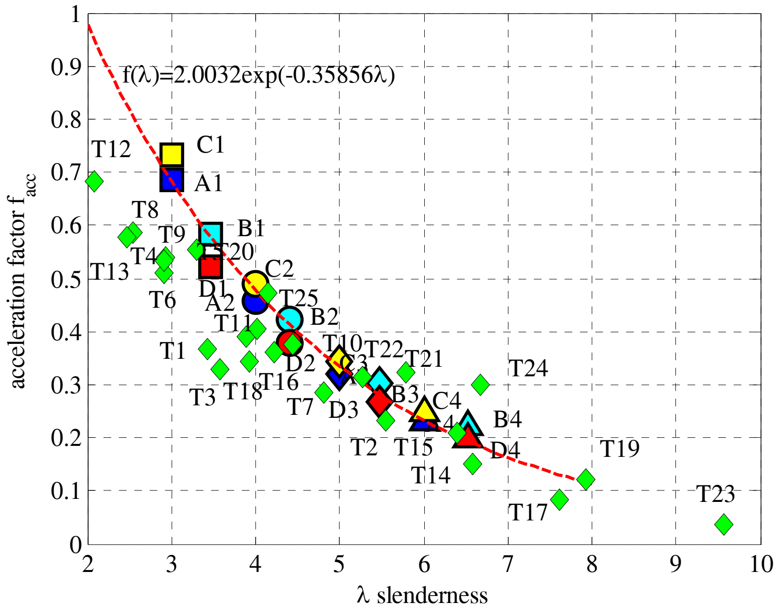

Figure 8: 3Muri software. Acceleration factor vs slenderness, comparison between idealized approach and real case studies.

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