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Static and Seismic Safety of the Inclined Tower of Portogruaro: A Preliminary Numerical Approach

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Abstract: Masonry towers are peculiar structures with complex structural behavior despite biased conclusions deriving from their geometrical regularity and simplicity. Their geometrical features and the epistemic uncertainty that masonry material bears strongly influence their static and seismic behavior. This paper investigates a remarkable and representative case study. The bell tower of Portogruaro (Italy) is a 57 m high tall construction, built in the XII-th century, and has a notable inclination. The Italian Guideline for the safety assessment of masonry towers is a key focus in this paper, highlighting the pros and cons of different suggested approaches. Some relevant proposals are presented in this paper in order to address the seismic safety assessment of masonry bell towers. The findings show that very slender structures do not meet the guidelines recommendations due to limitations in their current stress state. In addition, in similar cases, the recommended values for the mechanical properties of masonry material led to predicting non-withstanding structural behavior, questioning the correct choice of the adapted material properties. Advanced pushover analysis has been conducted in order to investigate the results of the simplified approach in terms of failure patterns and seismic safety estimation. The simulations are implemented for four different hypothetical scenarios of the existing masonry mechanical properties. The results obtained for the case study tower reflect a different perspective in the seismic assessment of masonry towers when specific approaches are defined. The preliminary results on the safety of Portogruaro Tower show a significant variability of seismic safety based on the adopted scenario, highlighting the necessity to pay attention to the preservation state of the present case and of similar ones.

Keywords: masonry; towers; seismic vulnerability; structural safety; inclination; pushover

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1. Introduction

Masonry structures are iconic structures spread around the world, standing for centuries and being an essential part of the architectural and cultural heritage. The towers have been built for different purposes: medieval defense towers, chimneys, minarets, bell towers, civic towers, lighthouses, etc. Previous studies on their structural performance have highlighted many of their current vulnerabilities, from material deterioration, lack of proper construction techniques, inclination, creep effect on highly compressed masonry, etc. [1–10]. One of the greatest impacts on this structural typology are the earthquakes, which have caused severe damage during the last registered events [11–19].

Current regulations require that a properly designed structure should be able to fulfill a multi-level performance requirement, i.e., likely to withstand frequent earthquakes without significant damage and not collapse during rare events. This whole approach implies considering many uncertainties and different seismic scenarios to approximate the structural performance closer to its reality. The goal would be to achieve the most probable failure mechanisms and to investigate the structure's limit state. For a simplified geometry, typical failure mechanisms of unreinforced masonry bell towers, as presented in literature [20–23], could fairly approximate their ultimate state under seismic loads. These approaches are fundamentally based on simplified geometries of towers, by considering the shaft of the tower as a hollow polygon geometric shape, which will be able to withstand safe for the design loads. This approach has turned out to be quite accurate in providing reliable results on how these structures will respond to vertical and lateral loads. Many normatives that are relatively important in the scientific and engineers' community provide the best current code of practice to address these structures, like the Italian and Eurocodes [24–28].

On the other hand, a simplified approach cannot always predict complex structural behavior; henceforth, the main strategy of the current and most cutting-edge state of research is focused on advancing numerical strategies. Some relevant contributions on the topic worthy to mention are [29–37].

This paper confronts the results obtained from a simplified approach of seismic analysis of masonry towers with a numerically advanced approach, i.e., pushover analysis, implemented for a specific case study. The selected case study is a typical Venetian medieval bell tower in Portogruaro (Italy); see Figure 1. The considerable height of masonry towers makes these constructions subjected to high vertical stresses, comparable to their compressive strength, which are not often encountered for medium-high and short towers. In this light, the simplified approaches provide some limitations to describe the stress state adequately and estimate the tower's capacity to withstand lateral forces. Therefore, according to classical and simplified approaches, it is highly prone that would lead in the identification of a failure pattern that will not mimic a realistic failure mechanism. On the other hand, relevant uncertainties make the problem even more complicated, such as the shape pattern of lateral forces, material properties, quality of the construction, etc.

(**a**) (**b**) (**c**) (**d**)

Figure 1. Photo of the Portogruaro Tower, (**a**,**b**) context view of the tower and Duomo's apse, (**c**) entrance side view of the tower, (**d**) vault of the tower.

The main aims of this work are the following: (1) to exploit the limitations of the simplified approach in assessing the seismic performance of masonry towers; (2) to propose modifications on the simplified numerical approach for towers subjected to high levels of vertical stresses; (3) to correlate the present inclination with the seismic capacity reduction; (4) to use detailed 3D FE models and pushover analyses for advanced estimation of seismic performance; and (5) to compare the results from the two approaches to investigate the corresponding effectiveness.

This article will focus on the material properties and how their principal features affect the expected results, studying different scenarios. Some relevant findings have been observed and addressed to address seismic safety accurately.

2. The Tower of Portogruaro

2.1. Geometry

The Portogruaro Tower stands on the left side of the Cathedral of Saint Andrea. Its construction dates back to the 12th–13th centuries, contemporaneous with the old church of Saint Andrea, which was demolished in 1793 to make way for the new cathedral. The tower is about 57 m high and has a square footprint with a base side of approximately 7.3 m. Based on available historical sources, the foundations are about 2.5 to 3 m wide and set about 3 m deep from ground level, widening by about 0.8 m compared to the tower’s base footprint. From the available documentation and conducted surveys, it is hypothesized that the foundation consists of natural stone, extending about 2.80 m deep from ground level, resting on a wooden platform supported by a pile foundation of wooden stakes 1.5 to 2 m long. The construction technique is similar to the traditional construction technique of Venetian towers.

The tower has undergone different interventions. The first significant intervention was building the spire at the end of the 1800s. This elevation intervention made the current tower have two vaults, one at the base of the current belfry and one at a lower floor, in correspondence to the previous location of the belfry.

Significant interventions, mainly involving general masonry consolidation through cement injections and reinforced concrete rings at various levels, were performed from 1962 to 1975. During the 1975 intervention, the base part of the tower was reinforced with transverse anchoring.

2.2. Inclination Trend

The peculiarity of this structure is its significant tilt towards the northeast. The inclination, believed to be due to a rigid rotation of the tower’s foundation, has very early origins. It is evident that the masonry near the bell chamber, besides having a different texture, also presents a different inclination, being more vertical than the underlying part due to an elevation between 1877 and 1879. The spire, obviously supposed to have been conceived vertically, also shows a slight deviation from the plumb line. This observation indicates a progressive phenomenon over time. A geometric survey of the entire structure was conducted in 2001, which shows a noticeable general inclination compared to the vertical axis, confirming the above observations. During the survey at the northeast corner of the balcony above the bell chamber, located about 37 m from the ground, the deviation from the plumb line was about 1.2 m. Figure 2 shows the section view of the tower.

The municipal administration has therefore commissioned a group of researchers from the University of Trento to conduct permanent monitoring of the tower, which has been ongoing for several years. Summarizing the findings based on data acquired between 2003 and 2023, it is confirmed that the global displacement of the tower has a northeast direction with a displacement trend of 2 mm/year.

2.3. Masonry Material

Identifying the compressive and tensile strengths is even more challenging and uncertain in the absence of experimental tests. Italian code practice strongly suggests investigating masonry material properties by excessive material testing. However, it is acknowledged that due to the difficulties and complexities of the tests, such investigations are not always successful, and many parameters, are indirectly obtained from those tests. Considering these difficulties, and particularly to avoid the invasiveness of destructive tests in architectural and historical heritage structures, the code predicts different levels of knowledge and recommends using the referenced values in Table C8A.2.1 of [25]. The table provides minimum and maximum values for masonry's most important mechanical parameters, subdivided into categories based on various construction typologies.

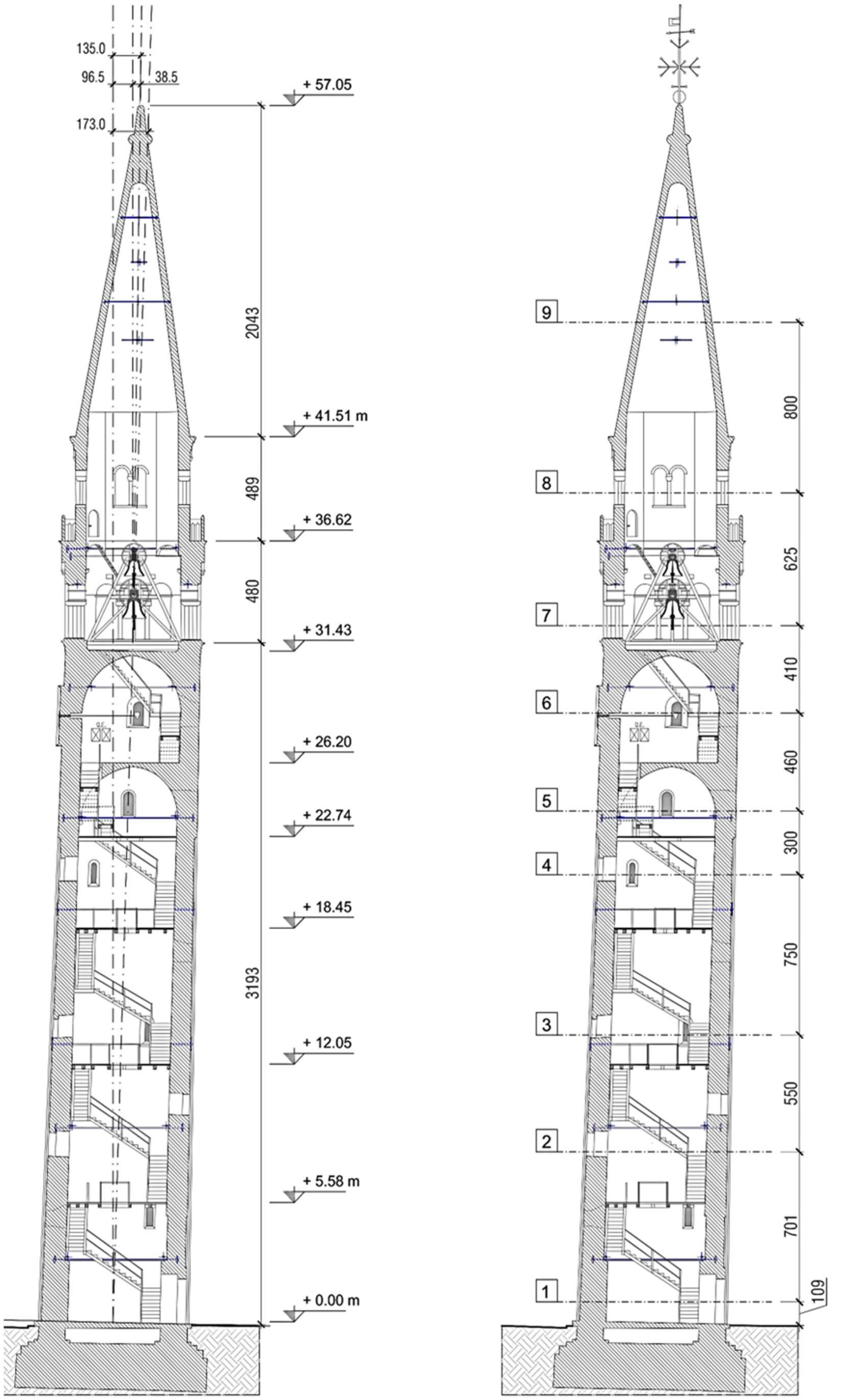


Figure 2. Section view of the Tower. The measurement of the inclination (**left** image). The assignments of sections (**right** image).

Briefly, the characterization of masonry material, based on Italian code of practice, can be assigned as follows based on Table C8A.2.1 values:

LC1 (limited knowledge):

$f_m = f_{min(\text{Table})}$; $f_{vo} = f_{vo,min(\text{Table})}$; $\tau_o = \tau_{o,min(\text{Table})}$; $E_m = \mu'_{(\text{Table})}$; $FC = 1.35$.

LC2 (average knowledge):

$f_m = \mu'_{(\text{Table})}$; $f_{vo} = \mu'_{(\text{Table})}$; $\tau_o = \mu'_{(\text{Table})}$; $E_m = \mu'_{(\text{Table})}$; $FC = 1.2$.

LC3 (extensive knowledge):

$f_m = \mu''$; $f_{vo} = \mu''$; $\tau_o = \mu''$; $E_m = \mu''$; $FC = 1$.

Where the mean values ($\mu'_{(\text{Table})}$ and $\sigma'_{(\text{Table})}$) have the following meanings:

$$\begin{aligned}\mu'_{(\text{Table})} &= \frac{X_{max} + X_{min}}{2} \\ \sigma'_{(\text{Table})} &= \frac{X_{max} - X_{min}}{2}\end{aligned} \tag{1}$$

And the corrected mean value μ'' is estimated according to:

$$\mu'' = \frac{n\overline{X} + k\mu'}{n + k} \tag{2}$$

where $\overline{X}$ is the average of the n direct tests and k is a coefficient that accounts for the ratio between the dispersion (variance) of the estimation obtained through the tests, X_{max}, X_{min} are the characteristic values, respectively the minimum and maximum specified by the normative on Table C8A.2.1. FC stands for the so-called factor of confidence, which is applied to reduce the mechanical parameters utilized based on the level of knowledge achieved. As can be noted, there is a significant difference between the corresponding material parameters allowed by the LC1 and LC2 approaches. An improvement of approximately $1.4 \approx (FC_{LC3}/FC_{LC2}) \cdot (\mu'/X_{min})$, by improving the level of knowledge and performing some tests without explicitly considering the respective experimentally obtained values. This means that despite the obtained test results (obviously excluding those that show very poor strength), the level of knowledge strongly influences the expected level of structural safety. In this regard, it is suggested that professionals be conscientious about the level of knowledge they accept to adopt.

On the other hand, reaching the LC3 is not without drawbacks, related to the test results interpretation and their correlation to the assigned mechanical parameters. One of the classical tests performed in towers is the dynamic identification test, giving a reference value for the elastic modulus. Then, the elastic modulus can be used to indirectly estimate the compressive strength of the masonry. In specific cases, the flat jack test or double flat jack test cannot be correctly executed and does not reach the masonry failure. These tests, generally speaking, limit the accuracy of the results as they describe the first branch of the loading curve, where the elastic modulus of masonry is normally higher than the one expected. The elaborated results could overestimate the corresponding elastic modulus and consequently the strength resistance. It is also worth noting that despite the level of knowledge, the same level of safety is not applied for different load configurations. At the limit state, the partial safety factor, $\gamma = 3$ for static conditions and $\gamma = 2$ for dynamic loading conditions should be applied.

2.4. Case Study Scenarios

This study has considered different scenarios related to the mechanical properties of the masonry material, which are reported in the following table. Four scenarios have been considered in total, see Table 1. Scenarios M1 and M2 are related to the LC1 and LC2 levels of knowledge, strictly following the code guidelines. Scenario M3 adopts the maximum values of the mechanical properties recommended in the Italian code, Table C8A.2.1 of [25]. It is a hypothetical case where we consider the best performance characteristics. The scenario M4 is similar to scenario M3, but increases the mechanical properties by the correction factor, Cc = 1.3, taken from Table C8A.2.2 of [25], in cases when the quality of the masonry material is good. A good masonry quality refers to: the mortar's thickness is uniform, the bricks are properly interlocked and assembled according to the state of art, and no critical cracks are present. In the considered tower, such conditions are met for most of the tower's volume.

Table 1. Mechanical properties of masonry.

Scenario M1	Scenario M2	Scenario M3	Scenario M4
LC1	LC2	LC3	LC3
FC = 1.35	FC = 1.20	FC = 1.0	FC = 1.0
$f_m = 26$ daN/cm^2	$f_m = 34.5$ daN/cm^2	$f_m = 43$ daN/cm^2	$f_m = 43$ daN/cm^2
$f_{vo} = 1.3$ daN/cm^2	$f_{vo} = 2.0$ daN/cm^2	$f_{vo} = 2.7$ daN/cm^2	$f_{vo} = 2.7$ daN/cm^2
$\tau_o = 0.5$ daN/cm^2	$\tau_o = 0.9$ daN/cm^2	$\tau_o = 1.3$ daN/cm^2	$\tau_o = 1.3$ daN/cm^2
Cc = 1.0	Cc = 1.0	Cc = 1.0	Cc = 1.3

3. Simplified Numerical Simulations

3.1. Section Capacity According to the Italian Guidelines for Culturally Built Heritage

According to the Italian Guideline of 2011 for the built heritage risk reduction [28], the capacity of a masonry tower under bending moment with a hollow rectangular shape, considering that the acting normal force is lower than $0.85 \cdot a_i \cdot t_i \cdot f_d$, is given by the expression:

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

where $M_{u,i}$ is the resisting moment, $\sigma_{0i} = W/A_i$ median compressive tension in the section, A_i area of the section without the area of the opening, b_i is the length of the rectangular base parallel to the application of lateral forces, a_i is the length of the rectangular base orthogonal to the application of lateral forces, t_i is the thickness of the wall, f_d is the compressive strength of the masonry material, taking into account the factor of confidence (FC) for the knowledge level achieved and the partial safety factor γ. This formulation is not brought to the updated Italian Code [24,25], where there is specified only the capacity of the rectangular section under (i) bending moment and shear forces, which for the last one are foreseen two ways of failure: (ii) sliding and (iii) diagonal cracking, represented in the following equation in the mentioned order.

$$\begin{cases} M_u = \frac{l^2 \cdot t}{2}\left(1 - \frac{\sigma_0}{0.85 \cdot f_d}\right) \\ V_t = l' \cdot t(f_{vmo} + 0.4 \cdot \sigma_0) \\ V_t = l \cdot t \cdot \frac{1.5 \cdot \tau_{0d}}{b}\sqrt{1 + \frac{\sigma_0}{1.5 \cdot \tau_{0d}}} \end{cases} \quad (4)$$

where l is the length of the rectangular section, l' compressed length of the rectangular section assuming a rectangular shape of stresses, t in the thickness of the rectangular section, $1 \le b = h/l \le 1.5$ is a correction factor based on the distribution of tensions in the section and varies by the slenderness of the wall, and for the case of masonry towers can be assumed to be 1.5, τ_{0d}, f_{vmo} and f_d are the shear resistance of masonry without compressive forces and compressive resistance, respectively.

3.2. Corrected Section Capacity

As mentioned in the previous section, Equation (3) is valid only for certain conditions, where one single wall of the cross-section can withstand the tower's total weight. For very slender structures, as a vast number of masonry towers are, this condition very often is not fulfilled; henceforth, it is required to analyze case by case the section's capacity to withstand an overturning moment. In the following formulation, Equation (3) of the guideline is extended to the general case, according to the ultimate state depicted in Figure 3.

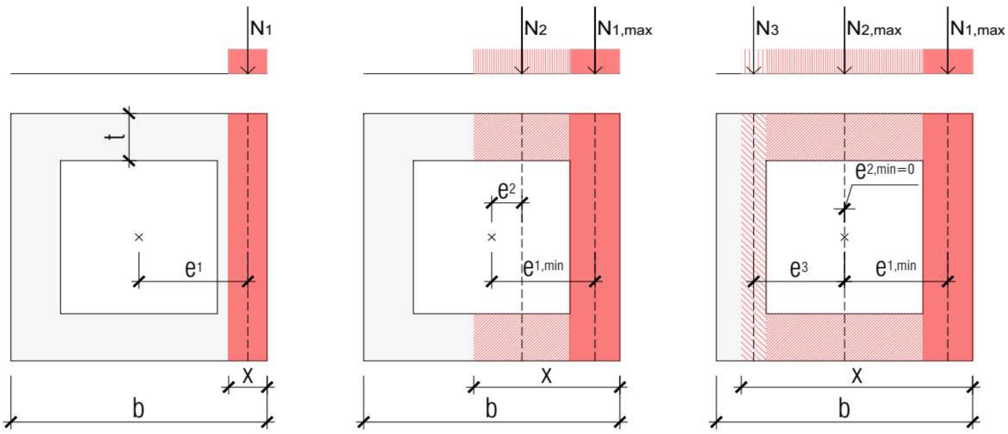
$$\begin{cases} x < t & M_u = N_1 \cdot e_1 \\ t < x < b - t & M_u = N_{1,max} \cdot e_{1,min} + N_2 \cdot e_2 \\ b - t < x < b & M_u = N_{1,max} \cdot e_{1,min} - N_3 \cdot e_3 \end{cases} \tag{5}$$

It can be noted that Equation (5) is able to track the capacity of the cross-section in accordance with Equation (4) of NTC 2018, obtaining a decreasing trend when the axial load is relatively high until the complete annulment. Such behavior cannot be captured by Equation (3), as it is valid only for a limited compressed zone of the section. Each contribution to the overturning moment, N_i, is calculated based on the compressed area of the section A_i multiplied by the reduced strength of the masonry, $0.85 \cdot f_d$. e_i is estimated as depicted in Figure 3.

$$N_i = A_i \cdot 0.85 \cdot f_d \tag{6}$$

The weight of the tower is distributed among the contributing parts, N_i. It is evident that the greater the weight of the tower is, the more diminutive the effect it has in contributing to the overturning capacity.

$$W_{Tower} = \sum N_i \tag{7}$$



(**a**) (**b**) (**c**)

Figure 3. Regular hollow section ultimate state under vertical compressive force for three different stades (**a**–**c**).

3.3. Tower Discretization and Preliminary Capacity Estimation

The towers of Portoguaro have been discretized in different sections through the height. The sections of interest of this study are from cross section 1 to 6, see Figures 2 and 4, as they have the shape of a hollow regular cross-section, as described in the Italian code. The geometrical properties of each section are reported in Figure 4. The capacity of the section to withstand bending moments is analyzed according to Sections 3.1 and 3.2 of the present paper. The ultimate bending moment capacity is initially analyzed by comparing different scenarios and aforementioned approaches.

Figure 5 shows the numerical results of the cross-section capacity. Each sub-image has four graphs, respectively: (i) LS (Guideline)—section capacity for dynamic loading according to Equation (3) without applying any check or correction; (ii) LS (Ne)—section capacity for dynamic loading according to Equation (5); (iii) ULS (Guideline)—section capacity for static loading according to Equation (3) without applying any check or correction; (iv) ULS (Ne)—section capacity for static loading according to Equation (5). The first observation is that the LS capacity is higher than the ULS capacity due to the utilization of a lower partial safety factor, $\gamma = 2$, instead of $\gamma = 3$.

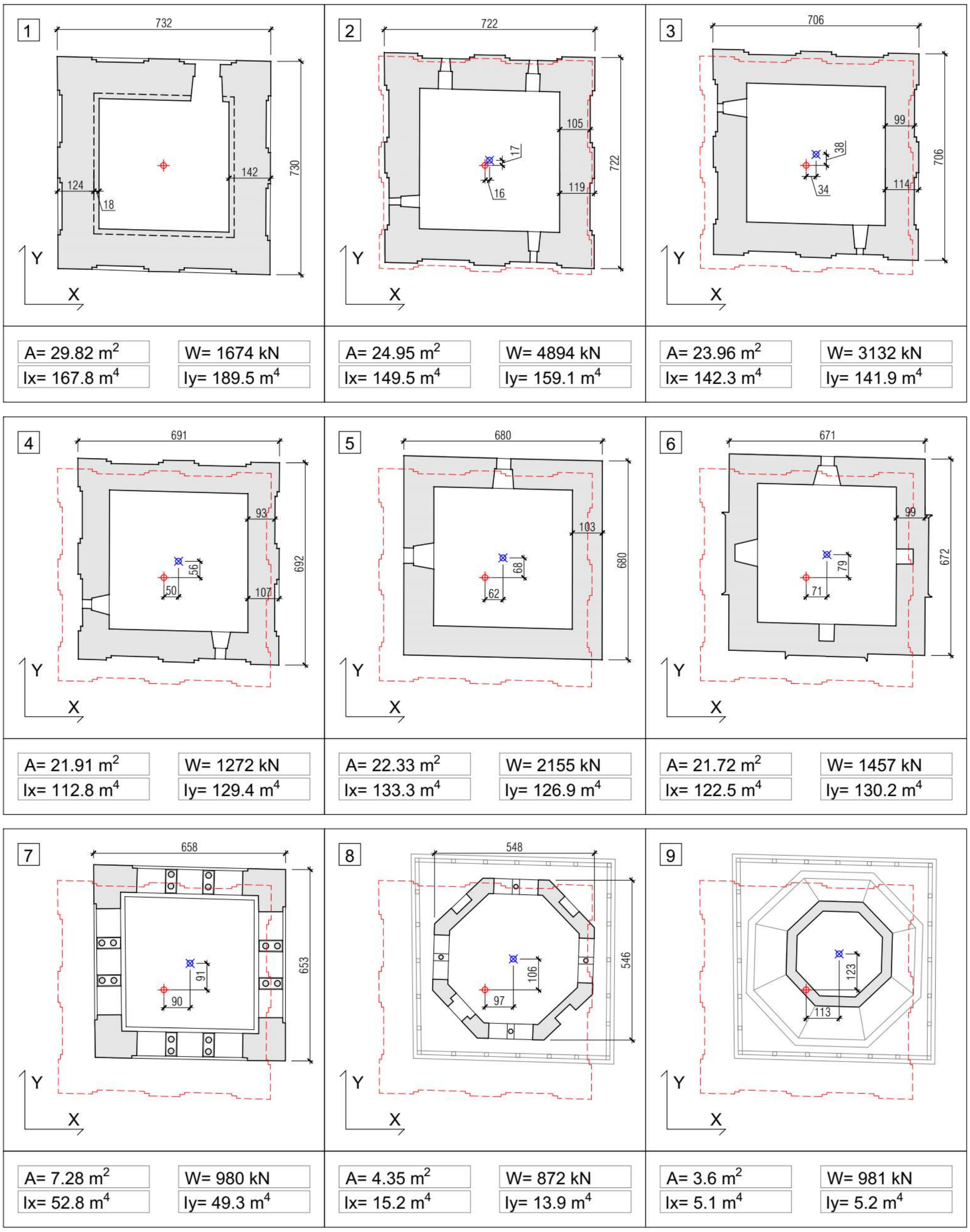


Figure 4. Geometrical properties of the cross sections of the tower.

The most notable observation is that the capacity of the section, estimated according to Equation (5), is lower compared to Equation (3), and this is more highlighted when the masonry strength capacity exhibits lower values. The most important result is that for lower compressive strengths of masonry, respectively corresponding to the LC1 and LC2 approaches, the current tower, according to the Italian normative, would not be safe or without an overturning capacity. The slightest lateral force (wind) would cause the collapse of the tower, but as the tower has resisted such conditions, it means that those approaches regarding the compressive strength are too conservative. For dynamic loadings, the present case shows better performance; despite the reduced capacity to withstand overturning moments, the capacity is positive instead of negative.

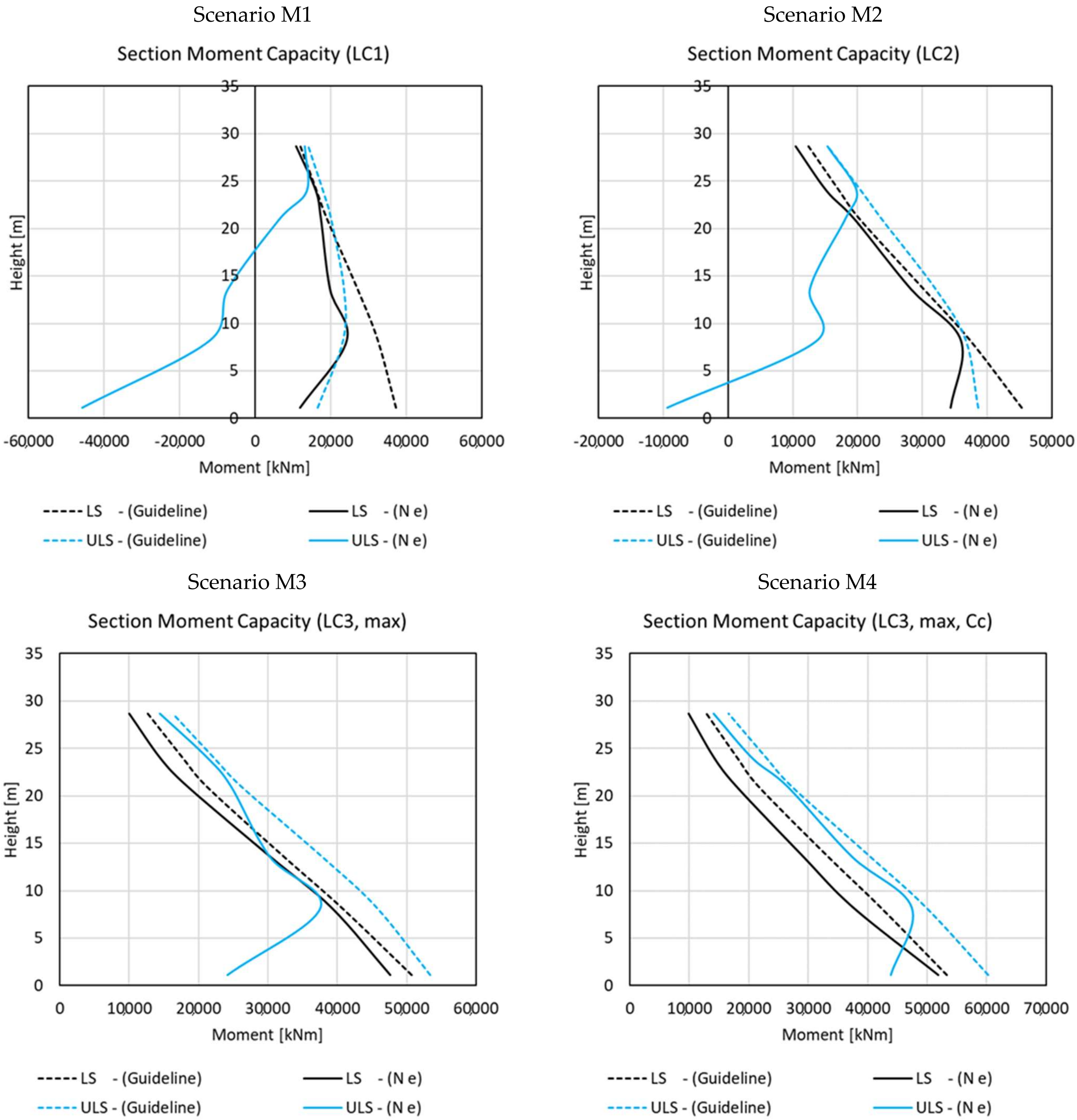


Figure 5. Section capacity of the Portogruaro Tower for different scenarios.

These preliminary results indicate that it is important to pay attention in the selection of the resistance parameters of the material. Generally speaking, these kinds of structures are very vulnerable to seismic actions, and their capacity under static lateral loads is often neglected. Based on the current simulation, adopting the parameters recommended for the LC1 and LC2 knowledge levels leads to non-physically admissible states. However, for advanced numerical simulations, the partial safety factor is not applied; therefore, this effect is barely encountered in many numerical simulations.

3.4. Simplified Seismic Risk Estimation

Based on the capacity of masonry towers under bending moments, the Italian Guideline of 2011 for the built heritage risk reduction [28] recommends estimating the peak ground acceleration on site that would cause the ultimate limit state according to the following equation.

$$\frac{a_{ULS,i}}{g} = \frac{0.4 \cdot S_{d,ULS,i}}{g} \cdot \frac{T_1}{T_c} = 0.4 \cdot \frac{M_{ULS,i}}{0.85 \cdot W \cdot Z_{fi}} \cdot \frac{T_1}{T_c} \tag{8}$$

$M_{ULS,i}$ is the ultimate moment capacity of the section, T_1 first fundamental period of the tower, T_c the characteristic period of the spectrum where the plateau of the spectrum ends, W is the total weight of the tower, Z_{fi} is the corresponding height where the lateral force is applied to cause the ultimate bending moment. In our case, the verification is done for the base of the tower, and results are reported in Table 2. A row considering the present inclination has been introduced in this preliminary consideration. Due to the geometrical configuration, a maximum overturning moment of around 7800 kNm is present and deduced from the ultimate base section capacity. Based on the following results, the inclination factor is very relevant when the overturning capacity is relatively low. The tower's inclination is a crucial factor that significantly reduces the tower's capacity to withstand lateral forces [38–41].

Table 2. Simplified seismic performance of Portogruaro Tower for four scenarios.

	Scenario M1		Scenario M2		Scenario M3		Scenario M4	
	LS-(Guideline) [kNm]	**LS-(Ne)** [kNm]	**LS-(Guideline)** [kNm]	**LS-(Ne)** [kNm]	**LS-(Guideline)** [kNm]	**LS-(Ne)** [kNm]	**LS-(Guideline)** [kNm]	**LS-(Ne)** [kNm]
w/o Inclination	37,282	11,883	45,331	34,334	50,744	47,602	53,264	51,911
w Inclination	29,425	4026	37,474	26,477	42,887	39,745	45,407	44,054
	Sd,$_{SLU}$, base/g		**Sd,$_{SLU}$, base/g**		**Sd,$_{SLU}$, base/g**		**Sd,$_{SLU}$, base/g**	
w/o Inclination	0.090	0.029	0.109	0.083	0.122	0.114	0.128	0.125
w Inclination	0.071	0.010	0.090	0.064	0.103	0.096	0.109	0.106
	$\frac{a_{ULS,base}}{g}$		$\frac{a_{ULS,base}}{g}$		$\frac{a_{ULS,base}}{g}$		$\frac{a_{ULS,base}}{g}$	
w/o Inclination	0.103	0.033	0.125	0.094	0.140	0.131	0.147	0.143
w Inclination	0.081	0.011	0.103	0.073	0.118	0.109	0.125	0.121

Table 3 summarizes the seismic safety consideration by the estimated peak ground acceleration ratio for the ultimate limit state and the seismic demand, $a_{g,\,ULS}/PGA$, considering a return period of 712 years as mandatory for class III structures, according to the Italian normative. For the specific site, with the considered return period, the PGA results equal to 0.118 g. As noted, the tower's seismic vulnerability would result quite high for scenarios M1 and M2. For scenarios M3 and M4, the results highlight that, from a seismic risk point of view, the tower is verified. It is worth noting that according to the Italian Code, these kinds of structures can also be verified for reduced seismicity to 60% [24]. That means that if the ratio between structural capacity and seismic demand is higher than 0.6, the structure is considered to pass the security check for the given seismic hazard.

Table 3. Seismic safety of Portogruaro tower for four scenarios.

$(a_{g,\,ULS}/PGA)$ PGA = 0.118 g	Scenario1 **LS-(Guideline)**	**LS-(Ne)**	Scenario2 **LS-(Guideline)**	**LS-(Ne)**	Scenario3 **LS-(Guideline)**	**LS-(Ne)**	Scenario4 **LS-(Guideline)**	**LS-(Ne)**
w/o Inclination	87%	28%	106%	80%	118%	111%	124%	121%
w Inclination	69%	9%	87%	62%	100%	93%	106%	103%

4. FE Numerical Simulation

The main focus of the research is to investigate the expected seismic damage for the tower, considering different material properties. The induced damages are expected to depict in a reliable way the vulnerabilities and pathologies of the current state of the tower. The obtained damage pattern would be more reliable than the previously discussed failure pattern estimation. This approach imposed the use of an advanced numerical approach and simulation using the non-linear properties of the material. The literature offers different approaches towards modeling structures where heterogeneity is emphasized and the limitations regarding the non-linear behavior of masonry are highlighted, as referred to [42–45]. To mimic the non-linear behavior of the masonry material under seismic cyclic loads and characterize material damage, the Abaqus CAE software is used, and the Concrete Damage Plasticity material model (CDP), proposed by Lubliner [46], and later modified by introducing a distinct damage parameter for compression and tension [47], has been adopted. The constitutive modeling of masonry in the non-linear phase is assumed according to a CDP model in Abaqus and is described by isotropic elastoplastic behavior constitutive laws defined by different ultimate stress, damage, and softening in tension and compression, and a three-dimensional behavior obeying a Drucker-Prager failure criterion assuming and non-associated flow rule. The typical stress-strain relationship implemented in the finite element solver environment is depicted in Figure 6. Detailed information on the model parameters can be found in the theory manual of Abaqus CAE [48]. This approach is a commonly used approach in the current research practice [40,41]. The full 3D numerical model is depicted in Figure 7, and as can be noted, it implements a detailed representation of the geometry by means of openings, inclination of face walls, inclination of the tower, the vaults, the belfry, etc. The mesh contains approximately 290 000 tetrahedron elements. The C3D4 type of Abaqus elements are best for meshing with any irregular shape and faster numerical processing.

4.1. Capacity Curves

Pushover analyses have been conducted on the tower for all four principal directions: dir. +X, dir. −X, dir. +Y, and dir. −Y. It should be noted that according to Italian Code [24,25], when dealing with pushover analyses, the structure's response should be investigated along the geometrical orthogonal axes X and Y, in both the positive and negative directions. In the present case, directions +X and + Y are towards the inclination of the tower; thenceforth, as expected, the capacity of the tower to withstand lateral loads is lower compared to the other directions, respectively dir. −X and dir. −Y, see Figure 8.

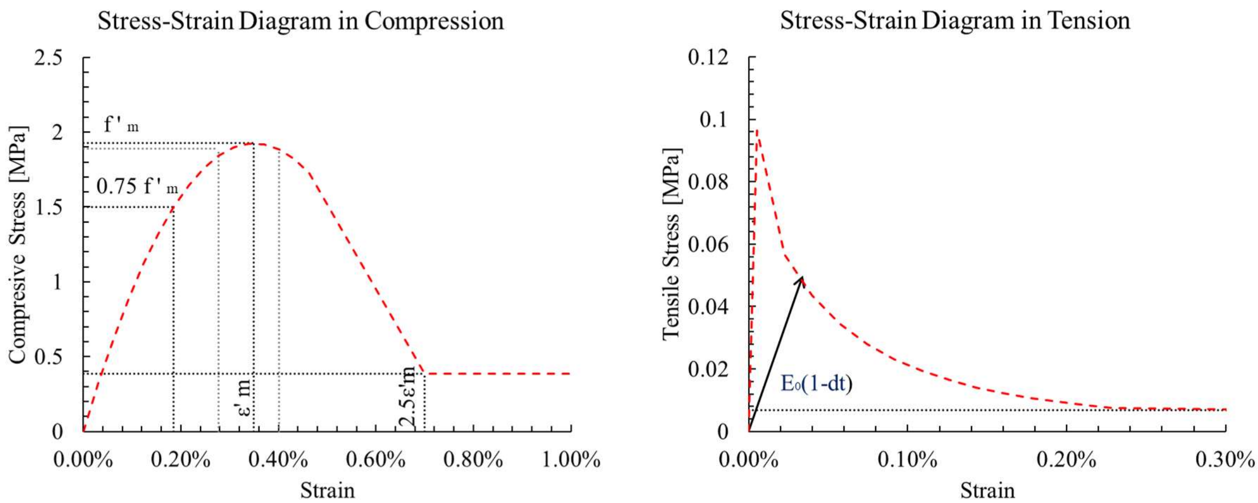


Figure 6. Representative stress-strain relationship of the masonry material adopted for pushover analyses [48].

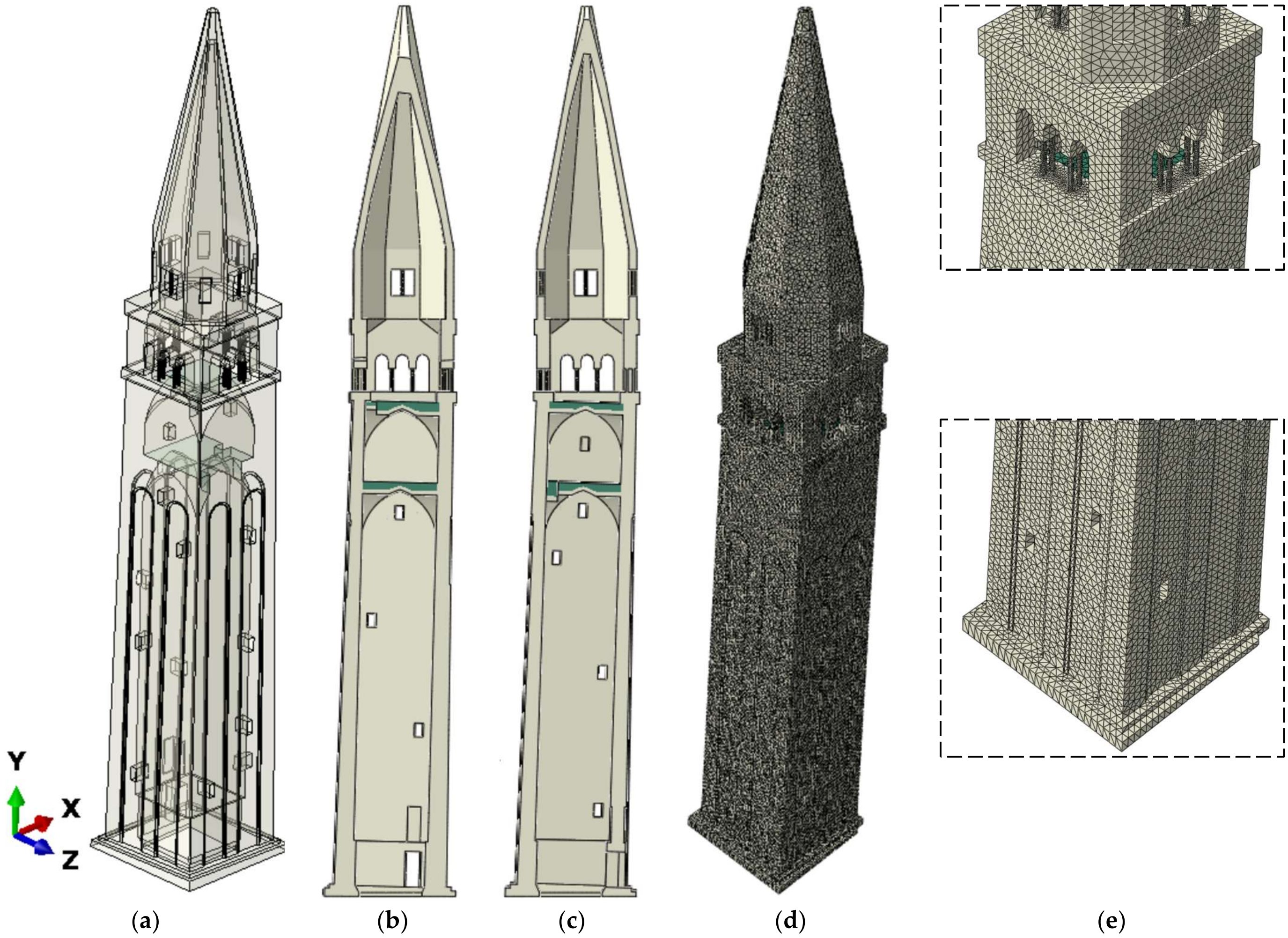


Figure 7. (**a**) 3D view of the numerical model; (**b**) section view in XY plane; (**c**) section view in ZY plane; (**d**) 3D mesh view; and (**e**) mesh details.

Here are two load patterns implemented for damage pattern investigation: (i) force control (uniformly distributed body forces); and (ii) displacement control (imposed displacement at the belfry of the tower). The first approach is the classic one to understand the diffusion of forces in the shaft of the tower. The second approach overcomes the limitation of the first one in not obtaining the softening branch, which is a key feature for assessing a maximum allowable displacement and the maximum resisting lateral force. The pushover analysis results are reported in terms of capacity curves, the total base shear versus the top displacement of the tower; see Figures 8 and 9.

Based on the results reported in Figure 8, it is noted that the tower responds differently if the forces are parallel to the inclination or against the inclination of the tower. The lowest performance is achieved when the forces are parallel to the tower's inclination, and these directions are further studied to assess the seismic capacity. Such a result is trivial considering that an inclination angle around 1.7° is really significant and in accordance with the expected behavior. The displacement control is then reported only for the case where the forces (displacement) increase in the direction of the current inclination; see the graphs in Figure 8.

A significant difference between the force control and displacement control approaches of the pushover analyses can be noticed. The main difference is the softening branch, as mentioned above; however, it is worth noting that the softening is due to the local failure under compression. For cases M3 and M4, the softening is not emphasized in both direction of the applied loads. For cases M1 and M2, the softening branch is very clear. The other difference can be observed in the maximum lateral force, which varies approximately between 50% (M1 case) and 65% (M4 case). It can be concluded that the choice on the material properties and load pattern could overestimate the maximum lateral force capacity and the correlated damage pattern, with an error of up to 100%.

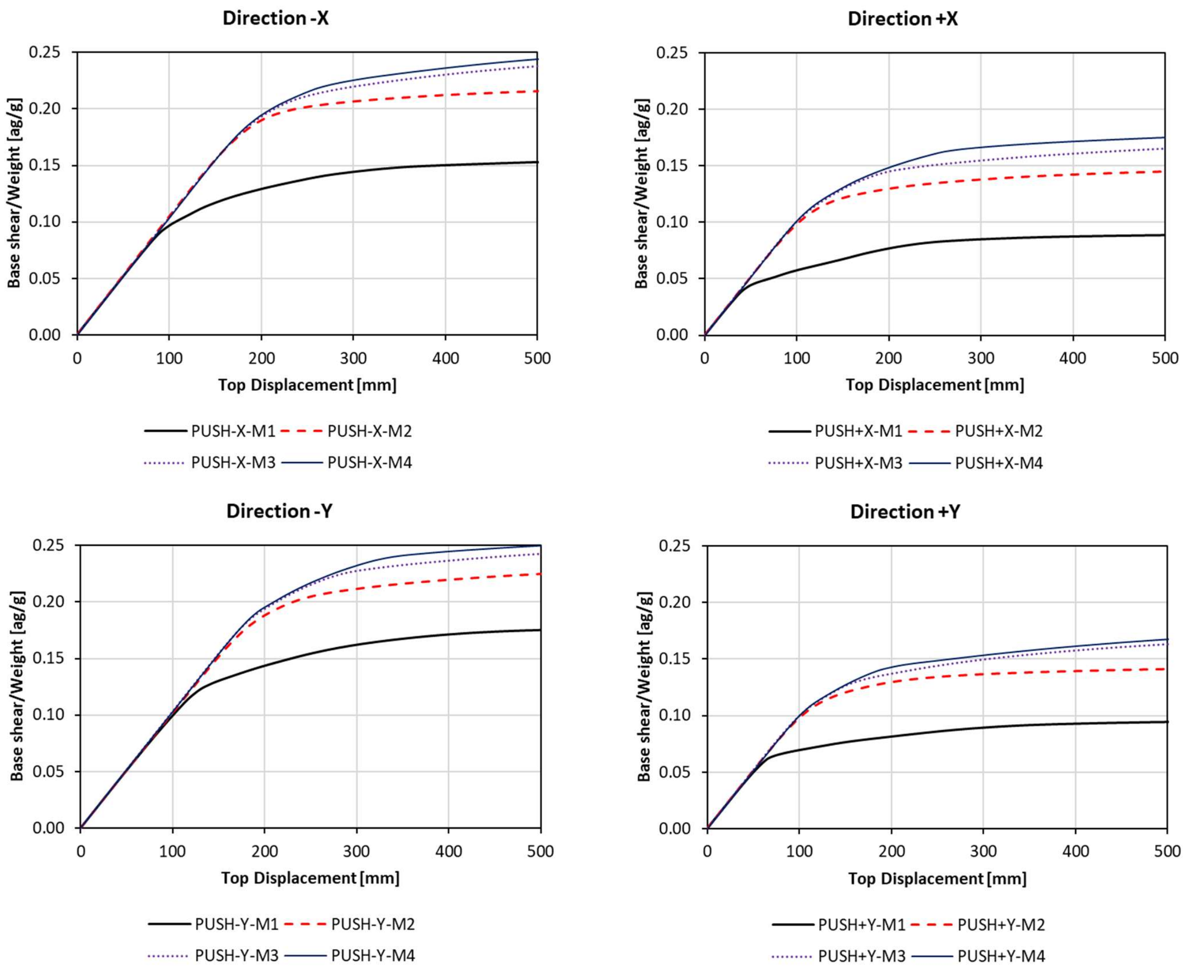


Figure 8. Pushover capacity curves with a force control pattern of Portogruaro Tower.

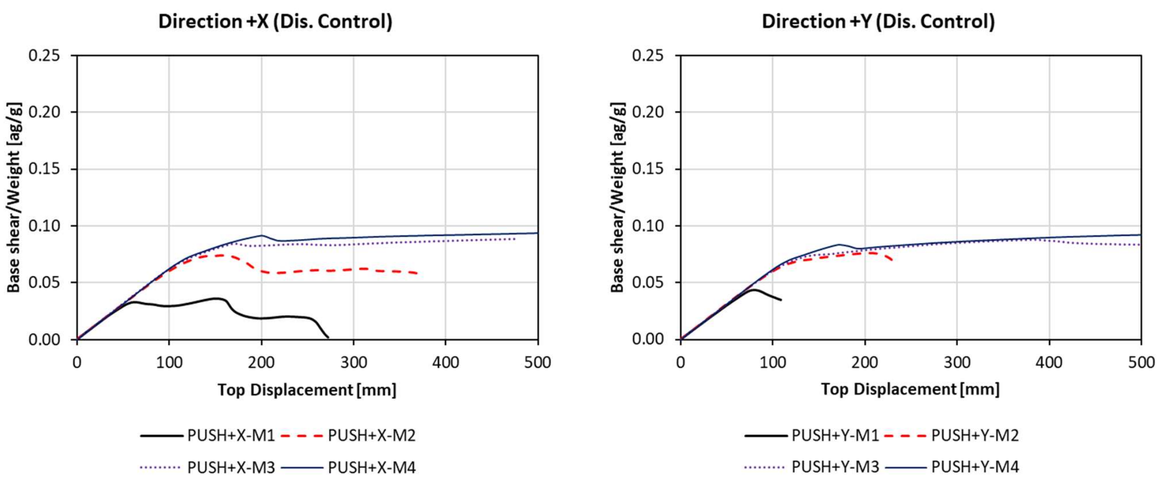


Figure 9. Pushover capacity curves with displacement control pattern of Portogruaro Tower.

For the cases of M3 and M4, the strength provided in compression seems to guarantee enough overturning capacity for the tower, such that the ruling aspect is the tensile strength. The tensile strength is not zero; see Figure 6 for numerical convergence issues. Overall, the flat part of the curve is more evident for these cases compared to cases M1 and M2.

4.2. Damage Pattern Analysis

The damage patterns obtained from the analyses conducted by implementing a displacement control approach are reported in the following images. This choice is based on the fact that displacement control is more accurate in providing a correlation between displacement, base shear, and failure pattern. However, it is worth noting that similar patterns have been encountered for both approaches, with minor differences, which are understandable considering that the load patterns are not identical. The most notable conclusion worth highlighting is the formation of damage patterns in the compressed side, likely diagonally diffused or even forming a cross; see Figures 10 and 11 for cases M1 and M2. Such a damage pattern shows the vulnerability of towers when they both exhibit low compressive strength and are inclined. It can be noticed that these crack diffusion patterns are influenced by the opening or reductions in sections, as occurred above the door of the tower.

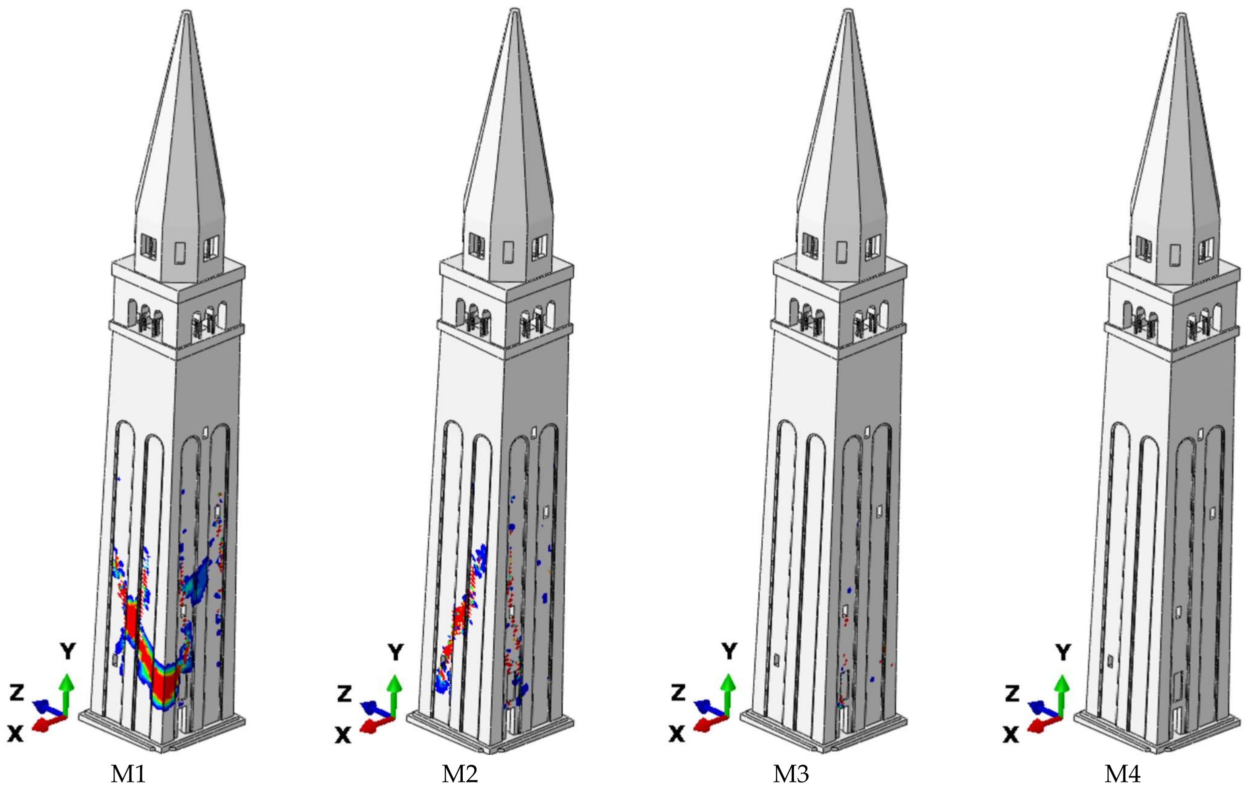


Figure 10. Damage in compression from pushover analyses in direction +X (compressed side view of the tower).

A notable difference is also related to the damages in tension, as shown in Figures 12 and 13, where it is possible to form vertical cracks on the most compressed side for low-strength masonry capacity. On the other hand, more performing masonry materials impose damage patterns more similar to classical failure patterns, with a crack at the tower's base, accompanied by a second failure plane approximately inclined at 45°. This is the expected failure pattern for non-crushable masonry with limited resistance in tension.

The formation of cracks on the compressed side from crushing of the material or shear failure means that in those cases, it is more likely to perform poorly under cyclic loadings, indicating a low structural behavior factor value. This behavior cannot be detected from static analysis; therefore, non-linear dynamic analyses are recommended. However, other studies with similar geometry and material features have obtained the same damage pattern, even from dynamic analyses, as seen in [41].

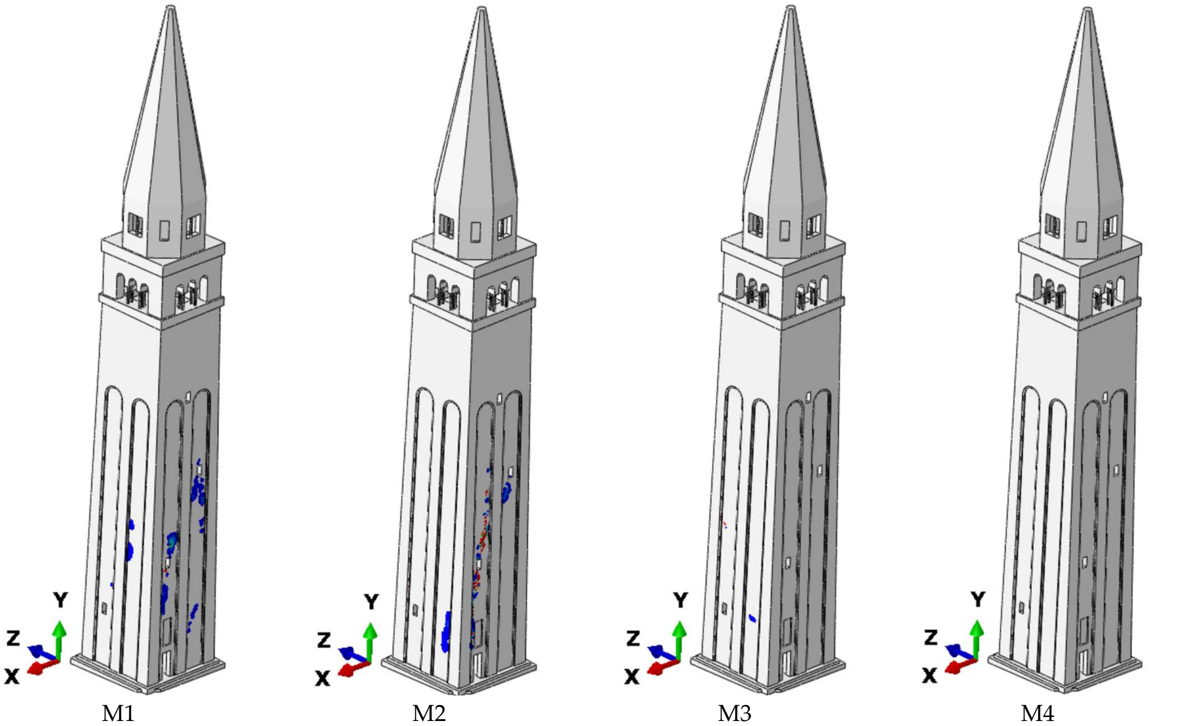


Figure 11. Damage in compression from pushover analyses in direction +Y (compressed side view of the tower).

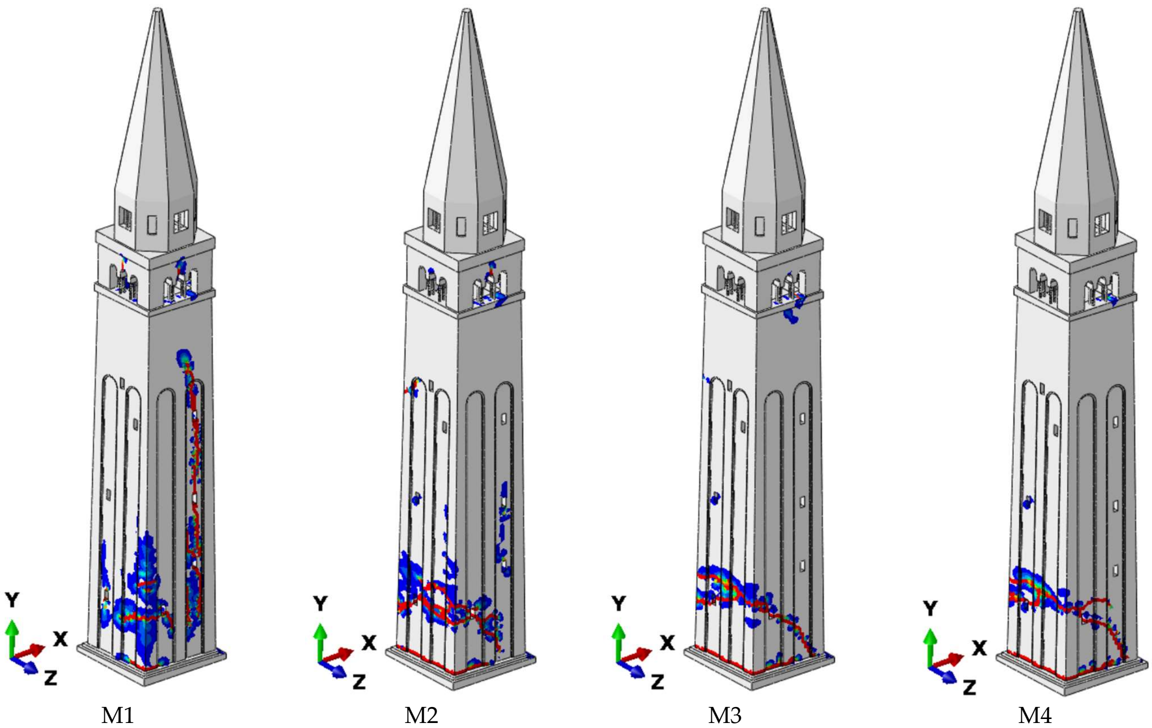


Figure 12. Damage in tension from pushover analyses in direction +X (in tension side view of the tower).

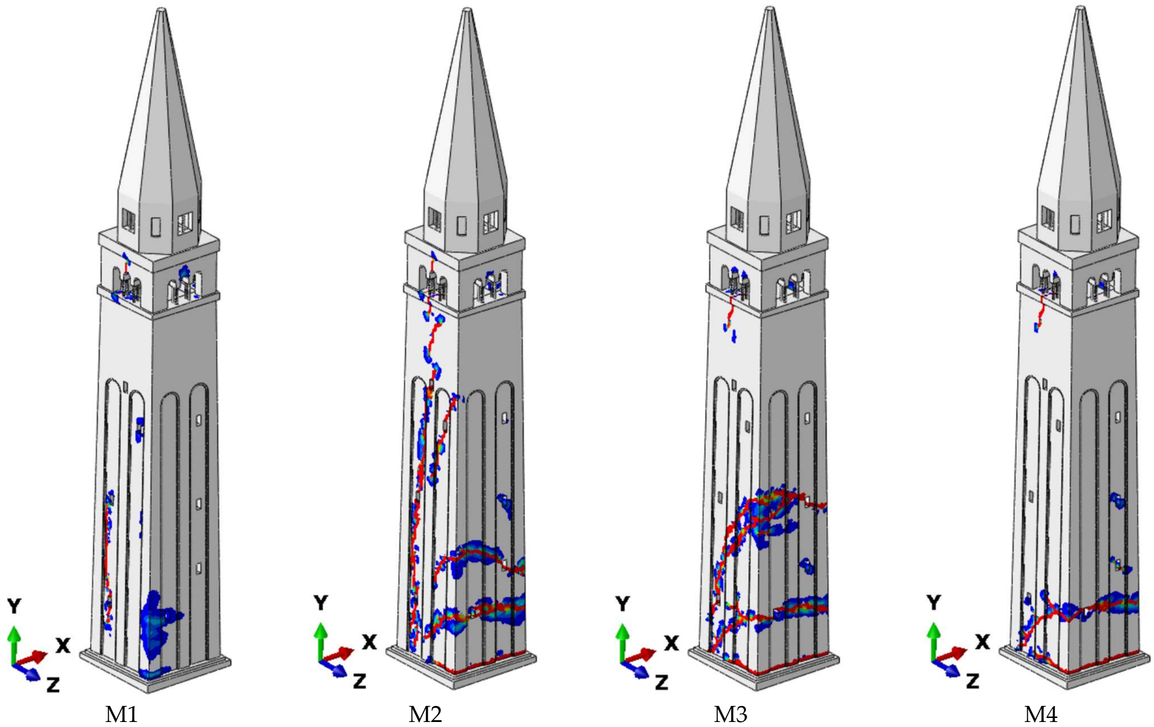


Figure 13. Damage in tension from pushover analyses in direction +Y (in tension side view of the tower).

4.3. Preliminary Seismic Risk Estimation

A simplified assessment procedure is also adopted for the seismic verification of the global structural behavior of the tower under study. The so-called N2 method developed by Fajfar [47] and adopted by different codes is based on pushover analyses and inelastic demand spectrum. The demand spectrum is estimated according to the Italian Code [24], without taking into consideration advanced studies on the local response spectrum. The method is formulated in the acceleration-displacement (AD) format, which enables the visual interpretation of the results. By means of a graphical procedure, the capacity of a structure is compared with the demand of an earthquake ground motion on the same structure, Figure 14. The capacity curve of the tower is transformed into a bilinear capacity curve of an equivalent SDOF system by means of the transformation factor $\Gamma = \sum m_i \varphi_i / \sum m_i {\varphi_i}^2$. The elastic acceleration S_{ae} and the corresponding elastic displacement demand S_{de} are computed by intersecting the radial line corresponding to the elastic period of the idealized bilinear system with the elastic demand spectrum. Due to the geometrical features of slender towers, as in the present case, the elastic period of the bilinear system is larger than T_c (the upper limit of the period of the constant spectral acceleration branch), and the inelastic displacement demand S_d is equal to the elastic displacement demand S_{de}. The displacement capacity corresponds to the end point of the bilinear curve. From the obtained results, the M1 case represents a capacity lower than the demand. In the other cases, from M2 to M4, results show that the tower would be verified.

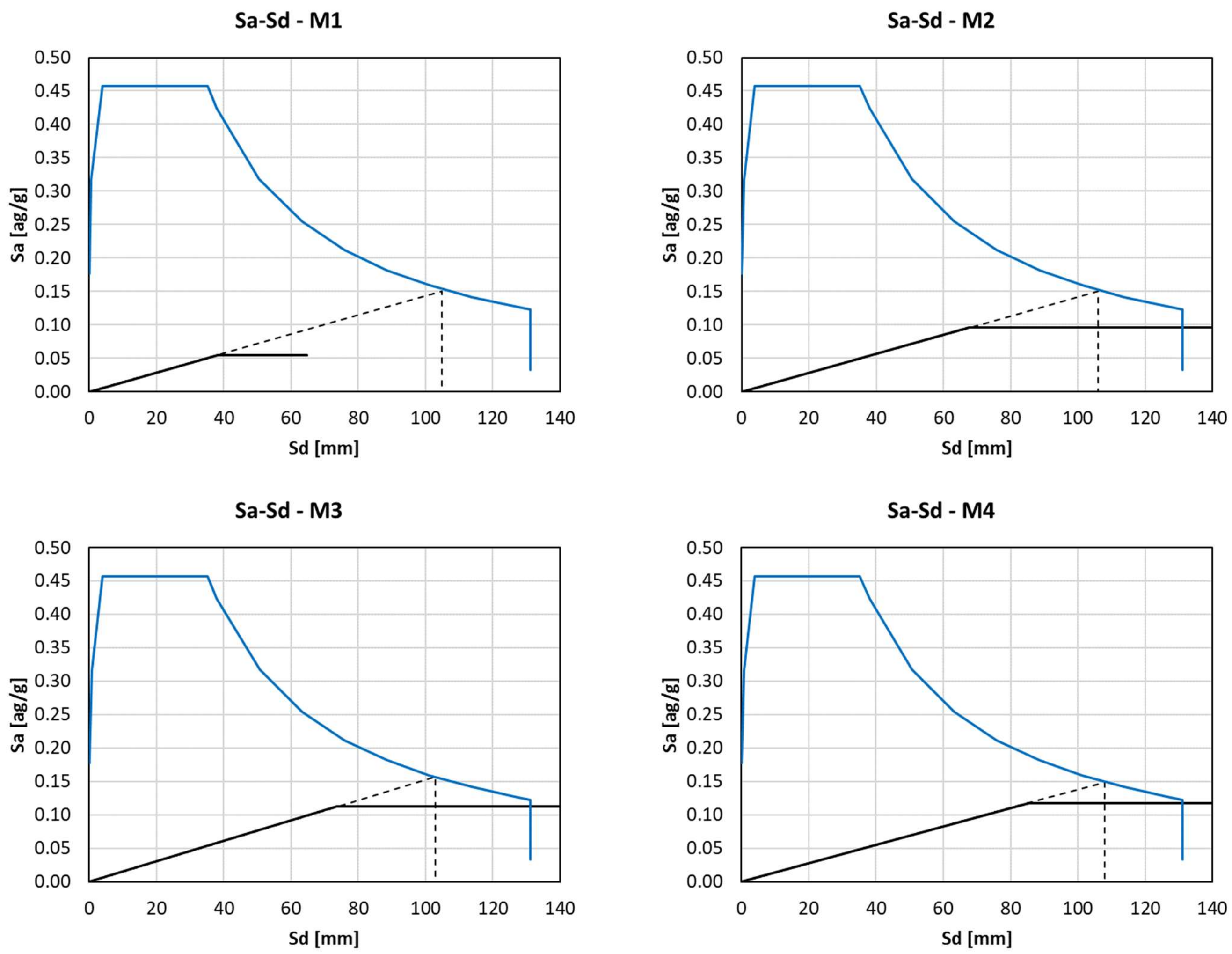


Figure 14. Seismic performance of Portogruaro Tower estimated in the ADRS format of the capacity and demand curves.

5. Conclusions

A comprehensive numerical study is presented using advanced FE simulations (pushover analyses) and simplified numerical estimation based on Italian guidelines for modeling towers as hollow polygonal bodies. The selected case study is deliberately chosen from the Venetian bell towers in Portogruaro. The tower exhibits particular features of geometry, material construction technique, seismicity level of the zone, and inclination that are comparable to hundreds of case studies. It is believed that the obtained results could be extended to a vast cultural heritage manifesting these features, especially highlighting the need to assess their seismic and static state of conservation.

The following conclusions may be drawn from an overall analysis of the results obtained in this study.

- Masonry towers are very slender, which causes the concentration of very high compressive stresses at the base. Based on the ratio between the average stresses and the masonry's compressive strength, the section's capacity to resist lateral forces drops rapidly. According to the current guideline [28], a similar case is not explicitly described to what was described and what was encountered in this study.
- The assumed and achieved level of knowledge adopted for slender masonry towers is a complex problem because it influences the estimation of structural safety much more than a random structure. A lower level of knowledge adopted for simplified

analyses underestimates the capacity, while for advanced simulations, it imposes the activation of complex failure patterns.

- A better masonry material in terms of mechanical properties results in the activation of classic failure patterns, i.e., a horizontal plane and an inclined plane with 45° located at the base of the tower.
- Poor masonry materials in combination with inclination are prone to activating complex failure patterns in combination of compression, tension, and shear. Diagonal cracking is prone to happen in compressed faces, passing through the opening. A detachment of the compressed face is probable to occur due to the activation of vertical cracks in tension.
- The seismic performance of Portogruaro Tower is strongly influenced by the material properties assumed. From a preliminary estimation, the seismic vulnerability varies from 10% to 60% based on the assumed analysis approach. This indicates the necessity and importance of implementing advanced studies for similar complex cases.

The present work is focused on the limitations that the here-treated assumptions influence the seismic assessment. Summarizing, it highlights the complexity of the problem and the importance of starting from an advanced level of knowledge of similar structures obtained from destructive and non-destructive tests. Dynamic tests to identify the vibration characteristics are crucial for seismic analysis. One of the most important things to be carefully considered is the soil-structure interaction, as a significant factor in the dynamic characteristic of towers and as a direct influence in the cause of inclination. Choosing a proper simulation tool to model and approximate a realistic behavior of the aforementioned aspects—material, geometry, and interaction—is important. Despite the assumption made in the numerical simulations, it is imperative to take any displacement results with reserve, at the discretion of the researcher or the engineer, to avoid false estimations.

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