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SEISMIC ASSESSMENT OF HISTORICAL MASONRY TOWERS IN THE NORTH-EAST REGION OF ITALY

Marco Valente[1], **Gabriele Milani**[1]

[1] Department of Architecture, Built Environment and Construction Engineering (A.B.C.)
Technical University of Milan,
Via Ponzio 31, I-20133, Milan, Italy
e-mail: {marco.valente, gabriele.milani}@polimi.it

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Abstract. *In the present paper a comprehensive seismic vulnerability assessment of several historical masonry towers, mainly built during the medieval age and located in the North-East region of Italy, is carried out by means of simplified and advanced FE approaches. Three-dimensional finite element models of the masonry towers are created on the basis of geometrical data deduced from both existing available documentation and in-situ surveys. Nonlinear static (pushover) and dynamic full 3D analyses are carried out to investigate the seismic performance of the towers. A comparison between non-linear dynamic analyses and non-linear static procedures, even at a global level, can provide useful information about the seismic vulnerability of such type of structures. Numerical results provide an accurate understanding of the behavior of the towers under horizontal loads and highlight the effects of some geometrical features, such as slenderness, thickness of the perimeter walls, presence of perforations, internal vaults, irregularities and inclination.*

1 INTRODUCTION

The protection of historical masonry constructions against seismic actions is of strategic importance in many European countries, especially in Italy, which are prone to earthquakes. Ancient masonry towers are widely disseminated in Italy and represent one of the main elements of the local cultural heritage. In ancient times these structures were usually conceived mainly to resist vertical loads and are particularly vulnerable to earthquakes because of the limited ductility of masonry combined with the slenderness of the towers that are sometimes characterized by complex geometry and irregularities. The axial stresses due to gravity loads could be of the same order of magnitude as the compression strength of the old masonry and, when combined with the dynamic loads induced by earthquakes, can produce heavy damage and even local collapse. The prediction of the seismic response of historical masonry buildings represents a crucial issue and a challenging research item. An effective seismic vulnerability assessment of such structures can be obtained through non-linear dynamic and static analyses by means of suitable finite element (FE) models. In recent times, national and international codes have imposed the evaluation of the structural performance under horizontal loads, encouraging the use of sophisticated non-linear methods of analysis.

The paper, which can be considered as a thoughtful collection of case studies useful to infer general considerations, presents a comprehensive numerical study on the seismic performance assessment of eight historical masonry towers located in the North-East region of Italy. The towers exhibit different geometrical characteristics in terms of slenderness, cross-section area, openings, wall thickness and internal irregularities, but they are built with similar technologies and masonries presenting similar mechanical properties. Their structural behavior under horizontal loads may be therefore thought to be influenced mainly by geometrical issues.

Detailed three-dimensional finite element (FE) numerical models are created through the software package Abaqus to represent the geometry of the towers. The main geometrical features of the towers are deduced from both existing available documentation and in-situ surveys. The evaluation of the seismic response of the historical masonry towers is carried out through non-linear dynamic analyses. A damage plasticity material model, exhibiting softening in both tension and compression, already available in the commercial code Abaqus, is used for masonry. The seismic performance assessment of the towers is carried out in terms of displacement time-history and tensile damage distribution. The effects of different geometrical characteristics and local irregularities on the seismic response of the towers are investigated.

A non-linear static procedure based on pushover analysis is also used for the seismic vulnerability assessment of the masonry towers. The results obtained by the two methods are compared in order to verify whether the simplified approach may represent the seismic behavior of the towers. A comparison between non-linear dynamic analyses and non-linear static procedures, even at a global level, can provide useful information about the seismic vulnerability of the towers.

2 TOWERS UNDER STUDY AND FE MODELS

This section provides a concise overview of the main geometrical features of the towers under study, along with some rough details on the FE discretization adopted. The elevation views and the FE models of the eight towers are presented in Figure 1 and Figure 2. The FE models of the eight towers (bell, clock or battle towers) are created directly in the commercial code Abaqus. Three-dimensional elements are used to model masonry, with different material properties where necessary, to suitably take into account the presence of infill over the vaults or the possible central layer in multi-leaf walls. The choice of the element size is done in order to share the advantages of sufficiently reliable results and numerical efficiency during the non-linear dynamic analyses that usually needed very long time to be performed, even in workstations with large RAM. A preliminary size equal to 0.4-0.5 m is chosen for the sides of the 3D elements, with local or global refinements, depending on the specificity of the structure. Still, reasonable values of the mesh distortion are obtained, with a worst aspect ratio ranging between 2 and 3.

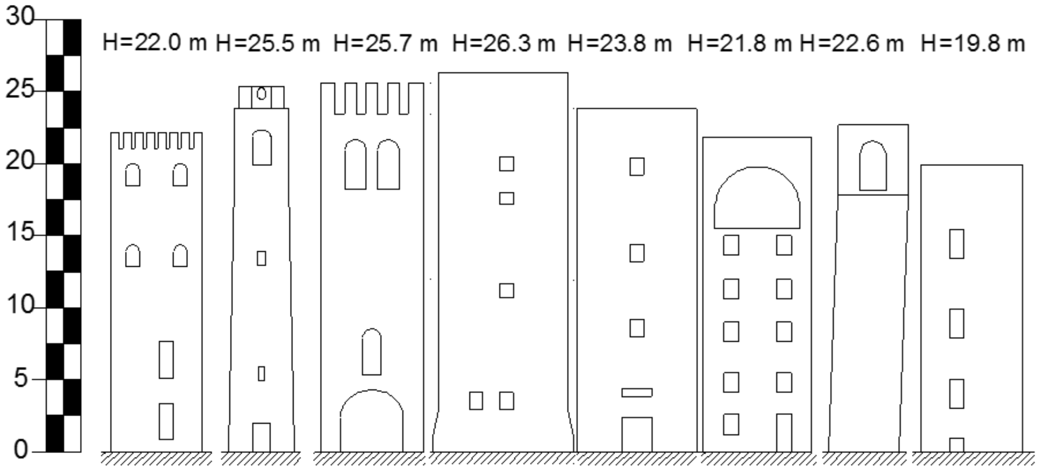

Figure 1: Schematic elevation views of the towers under study and indication of the height.

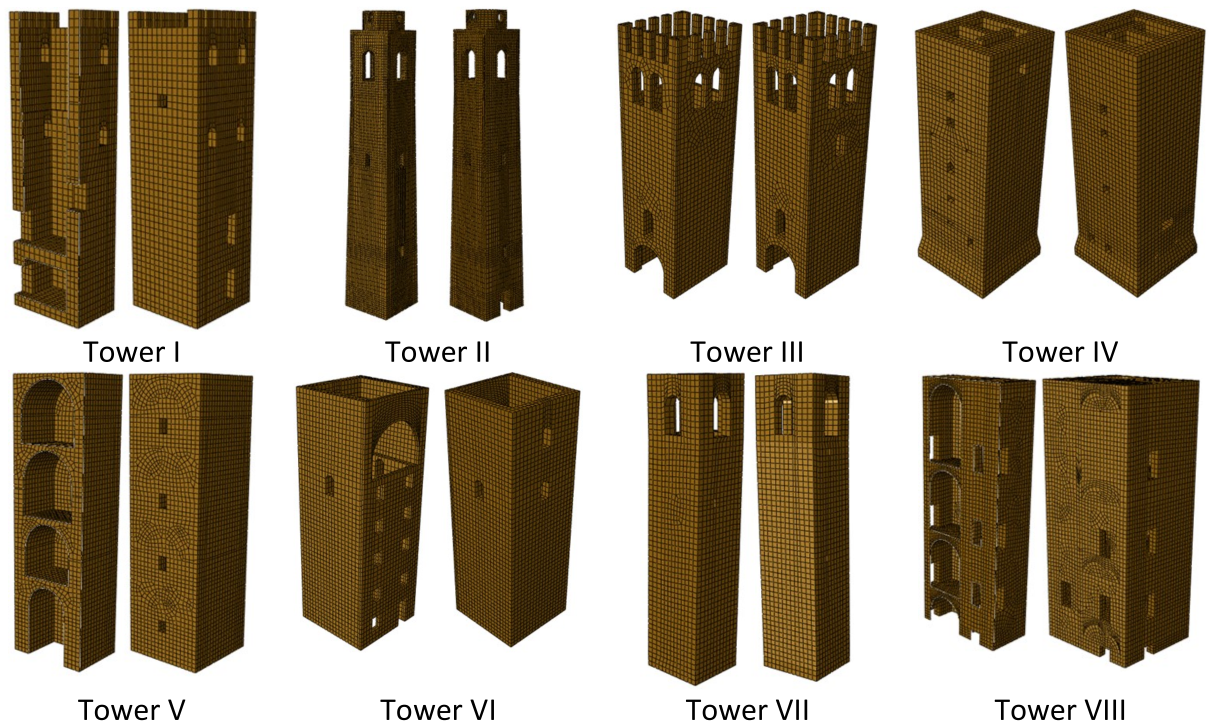

Figure 2: Finite element model of the towers under study

3 MATERIAL MODEL

The 3D FE models of the towers are implemented into Abaqus taking into consideration geometrical (large displacement effects) and material non-linearity by means of the Concrete Damage Plasticity (CDP) model, fully available in the standard package.

Concrete Damage Plasticity model is based on the assumption of a scalar isotropic damage with distinct damage parameters in tension and in compression. It is particularly suitable for applications in which the material exhibits damage, especially under loading-unloading conditions, and therefore for seismic analyses. A different inelastic behavior in tension and compression is then introduced.

To describe the multi-dimensional behavior in the inelastic range, masonry is assumed to obey a Drucker-Prager strength criterion with non-associated flow rule. The strength domain is a standard Drucker–Prager surface modified by means of the so-called K parameter, representing the ratio between the second stress invariant on the tensile meridian and that on the compressive meridian. This parameter is set equal to 0.666 in the numerical simulations.

A value equal to 10° is adopted for the dilatation angle, which seems reasonable for masonry subjected to a moderate-to-low level of vertical compression. This value is in agreement with experimental evidences available in the literature. To avoid numerical convergence issues, the tip of the conical Drucker-Prager strength domain is smoothed using a hyperbola. Abaqus code allows for smoothing the strength domain by means of the so-called eccentricity parameter, which represents the distance between the points of intersection with the p-axis of the cone and the hyperbola in the p-q plane, where p is the hydrostatic pressure stress and q is the Mises equivalent stress. A value equal to 0.1 is adopted for the eccentricity parameter in the numerical simulations.

The available experimental results on regular masonry wallets show a moderate orthotropy ratio (around 1.2) under biaxial stress states in the compression-compression region. Obviously, such feature cannot be taken into account when an isotropic model, like the present one, is utilized. However, it is commonly accepted in the literature the utilization of isotropic models (like concrete smeared crack approach available in both Ansys and Adina) after an adaptation of the parameters to fit an average behavior between vertical and horizontal compression. A suitable model should also take into account the ratio between the ultimate compression strength in biaxial stress states and in uniaxial conditions. Such a ratio, which exhibits similar values for concrete and masonry, is reasonably set equal to 1.16.

The final stress–strain relationship in tension adopted for the dynamic analyses follows a linear-elastic branch up to the peak stress σ_{t0}. Then, micro-cracks start to propagate within the material, leading to a macroscopic softening. In compression, the response is linear up to the yield stress σ_{c0}. Then, a linear hardening is assumed up to the crushing stress σ_{cu}, followed by a linear softening branch.

The damage variables in tension d_t and compression d_c are defined by means of the following standard relationships:

$$\begin{aligned} \sigma_t &= (1-d_t)E_0\left(\varepsilon_t - \varepsilon_t^{pl}\right) \\ \sigma_c &= (1-d_c)E_0\left(\varepsilon_c - \varepsilon_c^{pl}\right) \end{aligned} \tag{1}$$

where $\sigma_t(\sigma_c)$ is the uniaxial tensile (compressive) stress, E_0 is the initial elastic modulus, ε_t (ε_c) is the uniaxial total strain in tension (compression), $\varepsilon_t^{pl}(\varepsilon_c^{pl})$ is the equivalent plastic strain in tension (compression). In the present study, only tension damage is assumed to be active, because the adopted tensile strength of the material is significantly lower than the compressive strength. When the strain reaches a critical value, the material starts to degrade showing, in the unloading phase, a modulus equal to $E < E_0$. In particular, in the numerical simulations conducted in this study a reduction equal to 5% of the Young modulus with respect to the initial value is assumed for a plastic deformation equal to 0.003.

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4 ANALYSES PERFORMED

Two different numerical approaches, namely non-linear static procedure based on pushover analyses and non-linear dynamic analyses, are carried out in this study, with a level of numerical complexity that can be considered ranging from moderately high to high in common design practice.

4.1 Non-linear dynamic analyses

The real accelerogram registered in Mirandola on the 20th of May 2012 during the Emilia-Romagna seismic event is used to investigate the seismic response of the towers under study. The accelerogram is appropriately scaled in order to obtain two acceleration time histories with different values (0.1g and 0.2g) of the peak ground acceleration (PGA). Figure 3 shows the acceleration time history with PGA=0.1g along with some meaningful instants identified with different letters and colors.

The dynamic analyses are performed applying the accelerogram, separately, along the two principal (X and Y) directions of each tower. The horizontal displacement time histories of some control points, normally the nodes at the top of the towers, along with the damage state at the end of the simulations, are used to qualitatively determine if the structure is in a state of incipient collapse or not.

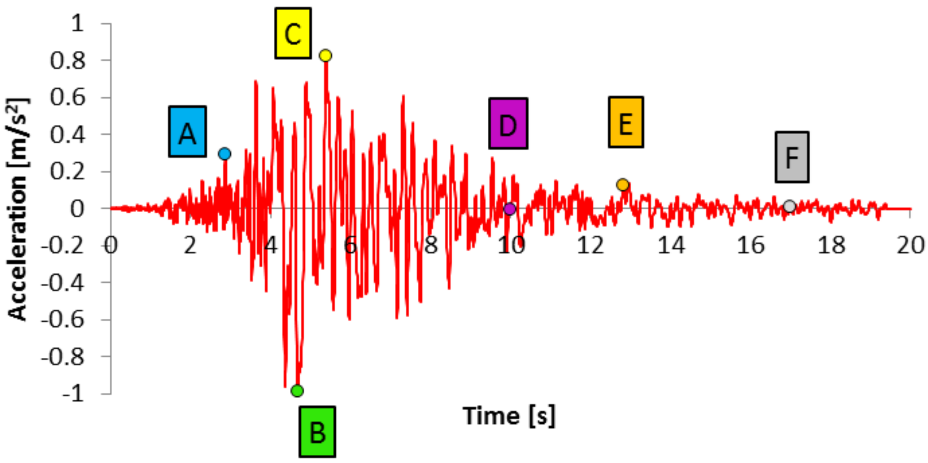

Figure 3: Scaled real accelerogram used in the non-linear dynamic analyses and meaningful points used to evaluate the state of damage of the towers under study.

4.2 Simplified procedure based on pushover analysis

A simplified assessment procedure is also adopted for the seismic verification of the global structural behavior of the towers under study. The procedure was developed at the University of Ljubljana by Fajfar and is based on pushover analyses and on inelastic demand spectrum. This simplified method of analysis is an effective technique for the seismic assessment of existing structures and combines pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) model. 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 4.

The capacity of the structure is represented by a force-displacement curve obtained by non-linear static analysis. The capacity curve of the structure is transformed into the capacity curve of an equivalent SDOF system by means of the transformation factor $\Gamma = \sum m_i \phi_i / \sum m_i \phi_i^2$, where ϕ_i is the i^{th} component of the eigenvector $\mathbf{\Phi}$ deduced from modal analysis and m_i is the mass of the node i. The capacity curve of the equivalent SDOF system is then reduced to a bilinear elastic-perfectly plastic force-displacement diagram on the basis of the equal energy concept (the areas underneath the actual and bilinear curves are approximately the same, within the range of interest).

The displacement capacity corresponds to the end point of the bilinear curve. The inelastic demand in terms of accelerations and displacements is provided by the intersection point of the capacity curve with the demand spectrum corresponding to the ductility demand μ, as schematically shown in Figure 4. In this study, the seismic demand is computed with reference to the Eurocode 8 response spectrum (soil type C). The theoretical predictions are performed for Sa_g levels equal to 0.1g and 0.2g. The displacement demands refer to the equivalent SDOF system. The displacement demands of the structure are obtained by multiplying the displacement demands of the SDOF system by the transformation factor Γ. Seismic assessment is performed by comparing displacement capacity and demand. The main results of the non-linear static procedure are reported for a direct comparison with the results of the non-linear dynamic analyses.

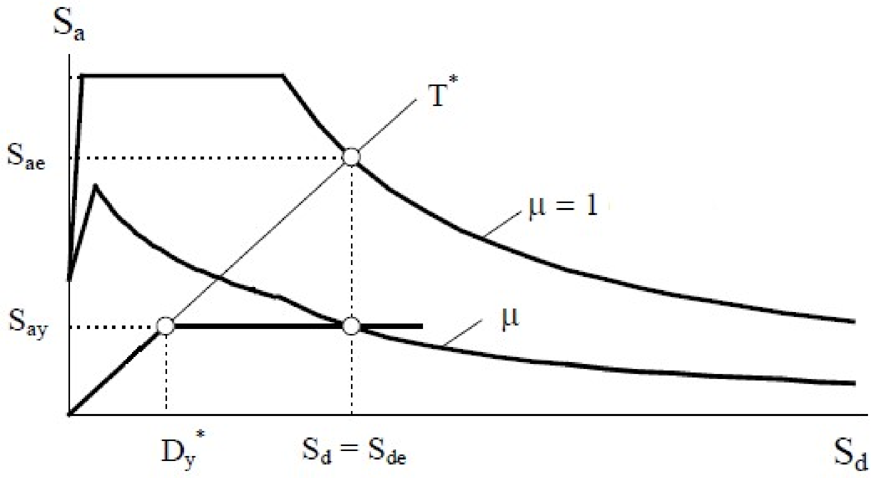

Figure 4. Elastic and inelastic demand spectra versus capacity diagram in acceleration-displacement (AD) format.

5 RESULTS OF NON-LINEAR DYNAMIC ANALYSES

It can be roughly stated that if a residual deformation, defined as the ratio between the horizontal inelastic residual displacement and the height of the tower, ranging between 0.4% and 0.8% is reached, the structure may be reasonably considered in incipient state of collapse. Values of 0.4% and 0.8 % of residual deformation are taken with reference to masonry piers behavior at failure under in-plane bending and shear, respectively, in agreement with Italian code specifics. While such a choice is rather debatable, because masonry towers can be hardly thought to behave as single piers, it is probably the only quantitative indication that can be attempted in this case.

The inelastic residual deformations obtained for all the cases investigated are synoptically shown in Figure 5, where threshold values of 0.4% and 0.8% are also indicated. The maximum values of the top displacements normalized to the height of the towers are summarized in Figure 6.

From an overall analysis of the results of the dynamic simulations with two different PGAs, the following remarks may be drawn.

Under seismic excitation with PGA=0.1g, it can be noted that Tower I, Tower II, Tower III, Tower IV and Tower VIII exhibit values of residual deformations smaller than 0.4% of the height. It can be observed that the values of residual deformation are critical (within 0.4%-0.8%) for Tower VI and Tower VII; Tower V can be considered as prone to collapse. These results are confirmed in terms of non-dimensional top displacements.

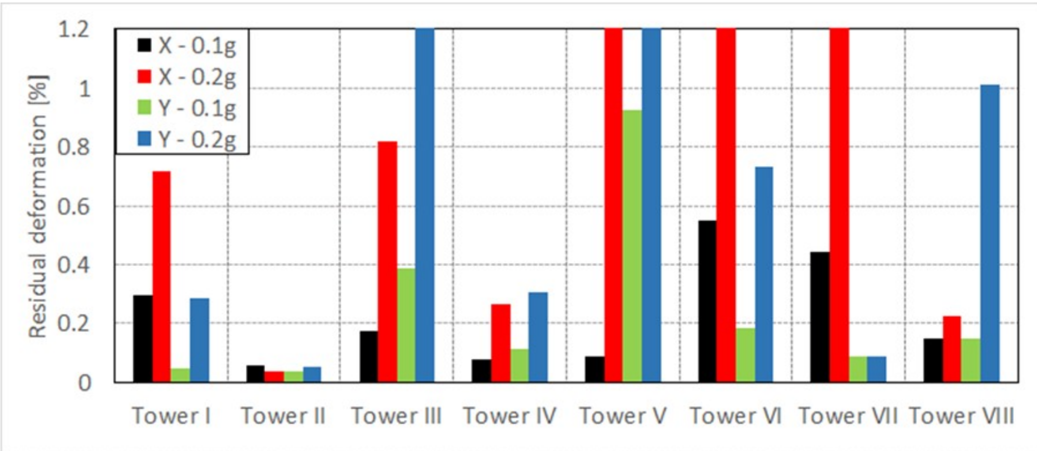

Figure 5: Non-linear dynamic analyses: residual deformations at PGA=0.1g and PGA=0.2g in the X and Y directions for the different towers.

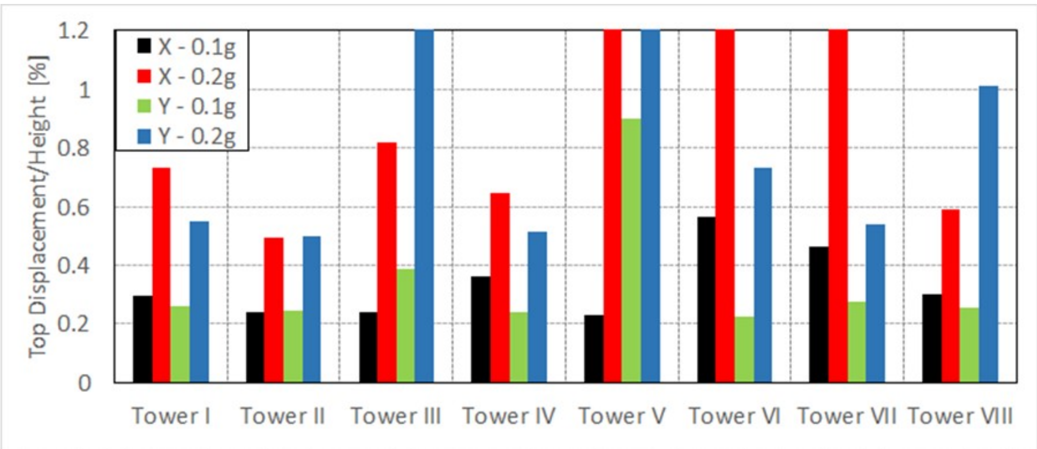

Figure 6: Non-linear dynamic analyses: maximum normalized top displacement (top displacement/height) at PGA=0.1g and PGA=0.2g in the X and Y directions for the different towers (values of normalized top displacements are interrupted at 1.2%).

Under seismic excitation with PGA=0.2g, basing on the criterion of inelastic residual deformations, a collapse mechanism is activated for Tower III, Tower V, Tower VI, Tower VII and Tower VIII. The values of residual deformations are critical (within 0.4%-0.8%) for

Tower I; Tower II and Tower IV exhibit acceptable residual deformations (smaller than 0.4% of the height). These results are confirmed in terms of normalized top displacement, but also Tower II and Tower IV present non-dimensional top displacement within 0.4%-0.8%.

6 SIMPLIFIED PROCEDURE BASED ON NON-LINEAR STATIC ANALYSES

The results of the non-linear static procedure in the acceleration-displacement response spectrum plane are illustrated in Figure 7. The capacity curve of the equivalent SDOF system, transformed in a bilinear curve, is reported and the seismic demand corresponds to two values of the effective peak ground accelerations equal to Sa_g=0.1 and Sa_g=0.2g; for the sake of clarity, only the elastic demand spectrum is shown. The seismic vulnerability is evaluated by comparing the displacement demand and the displacement capacity obtained through the pushover analyses. The displacement capacity and demand in the X and Y directions are then summarized for the towers under study in Figure 8 and Figure 9 for two different seismic intensity levels.

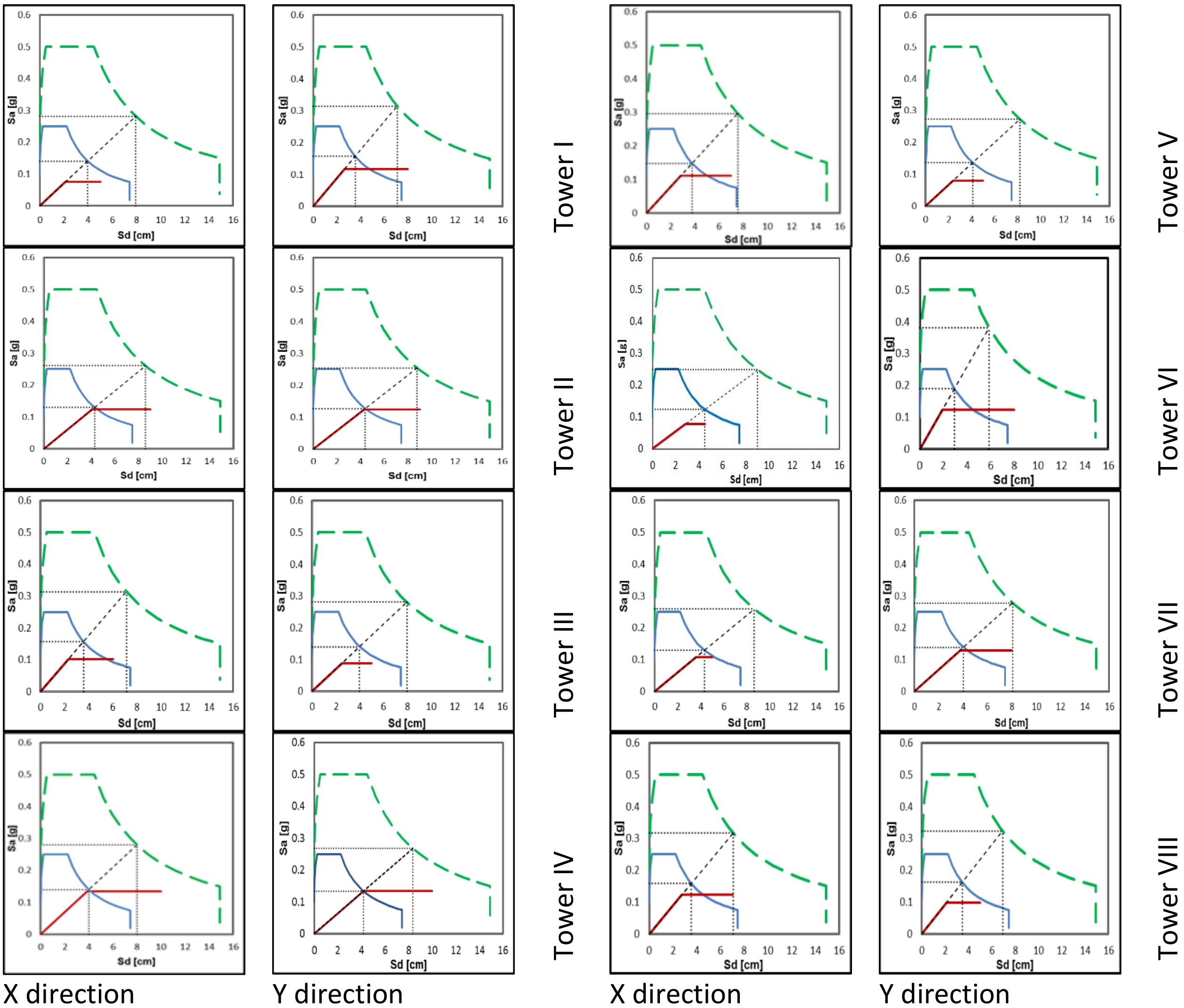

Figure 7: Non-linear static procedure in the acceleration-displacement response spectrum plane (X and Y directions) for different seismic intensity levels: Sa_g=0.1g (continuous blue line) and Sa_g=0.2g (dashed green line).

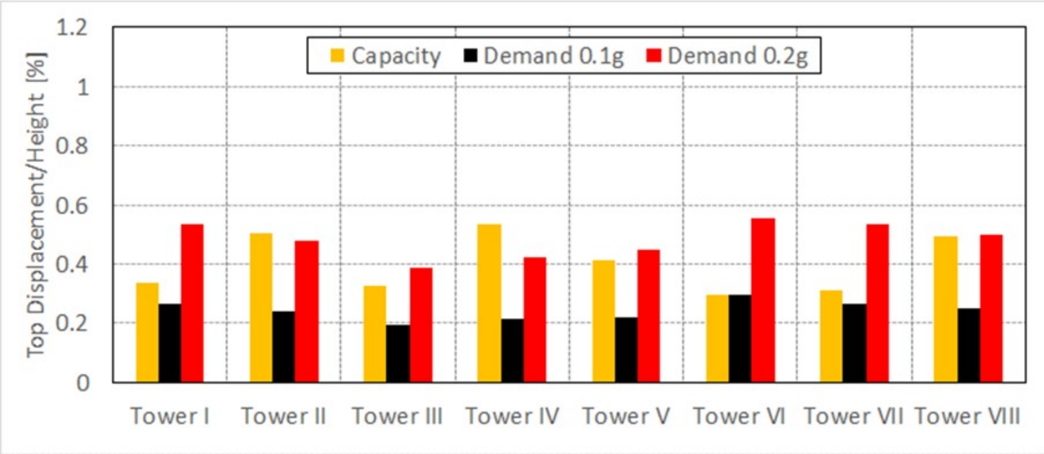

Figure 8: Non-linear static procedure, X direction: displacement capacity and displacement demand at different seismic intensity levels for the different towers.

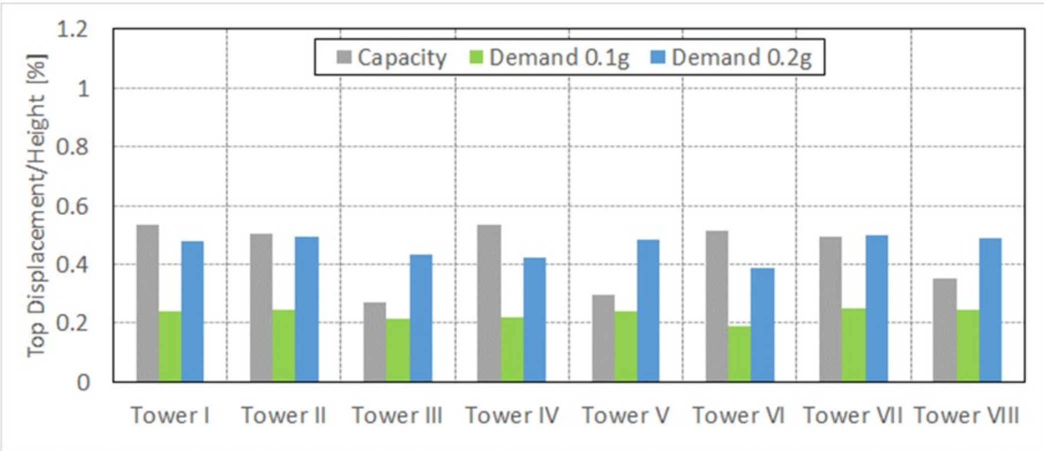

Figure 9: Non-linear static procedure, Y direction: displacement capacity and displacement demand at different seismic intensity levels for the different towers.

The results obtained from the non-linear static procedure are also synoptically summarized in Table 1 for a direct comparison with the outcomes of the non-linear dynamic analyses. As can be noted, the seismic safety assessment provided by the two methods is very similar for almost all the towers, in some cases with a very satisfactory agreement. The non-linear static procedure may provide reasonable synthetic predictions of the seismic vulnerability of the towers. A slightly more conservative trend for the non-linear dynamic simulations can be noticed after a detailed analysis of individual cases. The non-linear static procedure provides smaller values of displacement demand than those resulting from the non-linear dynamic analyses. The simplified static approach is not able to capture the inertial effects associated with seismic excitation that can lead to heavy damage and premature collapse of the structure.

7 DISCUSSION OF RESULTS

7.1 Comparison between the procedures

The displacement demands obtained by the non-linear static procedure are in a good agreement with those obtained by the non-linear dynamic analyses when the check of the structural safety is verified. When the check is not satisfied according to the non-linear static procedure, the seismic demand is smaller than the maximum value of the top displacement experienced by the structure during the non-linear dynamic analyses. In the majority of cases this result can be explained by the large increase of the displacements registered in the non-linear dynamic analyses when a collapse mechanism occurs. From an overall analysis of the results, it can be noted that the non-linear static procedure generally gives reliable results in terms of displacements. Large differences are observed in the cases of Tower V and Tower VI, where local collapse mechanisms are registered during the non-linear dynamic analyses and for this reason the displacement distribution differs significantly. In the case study of Tower V, the non-linear static procedure does not catch the collapse of the upper vaults, which is registered at the end of the non-linear dynamic analyses. In the case study of Tower VI, a local collapse of the upper part of the thin wall is clearly observed during the non-linear dynamic analysis: moreover the presence of this type of irregularity triggers non-uniform displacement demands at the top of the structure. Non-linear dynamic analyses are able to account for higher mode effects that may be introduced by local irregularities.

Table 1. Comparison on the collapse state of the towers evaluated through the non-linear static procedure and the non-linear dynamic analyses: X and Y directions, PGA=0.1g and PGA=0.2g.

	X direction				***Y direction***			
Collapse √ YES × NO √/× Borderline	Non-linear static procedure		Non-linear dynamic analysis		Non-linear static procedure		Non-linear dynamic analysis	
Tower	PGA 0.1g	PGA 0.2g	PGA 0.1g	PGA 0.2g	PGA 0.1g	PGA 0.2g	PGA 0.1g	PGA 0.2g
I	×	√	×/√	√	×	×	×	×/√
II	×	×	×	×	×	×/√	×	×
III	×	√	×	√	×	√	×/√	√
IV	×	×	×	×/√	×	×	×	×/√
V	×	√	×	√	×	√	√	√
VI	×/√	√	√	√	×	×	×	√
VII	×	√	×/√	√	×	×/√	×	×
VIII	×	×/√	×/√	×/√	×	√	×	√

The tensile damage distribution corresponding to the displacement demand is generally similar to the damage pattern observed at the end of the non-linear dynamic analyses, but the numerical values of damage are always smaller. The only exception is the severe damage observed at the base of the towers during the pushover analyses: it is less evident at the end of the non-linear dynamic analyses. However, the damage pattern detected by the pushover analysis can represent a good indicator of the vulnerable parts of the structure.
The choice of the control point, representative of the global behavior of the structure, is a fundamental issue for the application of the non-linear static procedure. The results of the procedure significantly depend on the selection of the control point and, in some cases, show quite large variations. Therefore, the results obtained by the non-linear static procedure should be evaluated with caution in function of the control point chosen for the procedure. This result is proven by the study of the Tower VI, where torsional effects are evident. It is shown that the results obtained with a control point which is not part of the local collapse overestimate the capacity of the structure. On the contrary, if a point belonging to the vulnerable part of the structure is chosen as a control point, the results underestimate the capacity of the structure.

The non-linear static procedure can give a satisfactory representation of the seismic behavior of the towers in terms of structural safety verification and displacement demands when significant irregularities are not present. In slender towers with presence of irregularities the contribution of higher modes to the global response may be considerable. Structures with higher mode effects exhibit a complex dynamic response that necessitates the use of more sophisticated methods of analyses.

7.2 Seismic safety assessment and failure modes of the towers

The check of the seismic safety of the analyzed towers is satisfied according to the non-linear static procedure under Sa_g=0.1g, with the exception of Tower VI that is a borderline case. Similar results are obtained through the non-linear dynamic analyses. Only Tower V and Tower VI present large residual deformations, indicating the activation of local failure mechanisms, mainly due to the geometrical irregularities, even for peak ground acceleration equal to 0.1g.

According to the non-linear static procedure, the analysed towers are not able to accommodate the seismic demand under Sa_g=0.2g, with the exception of Tower IV and Tower II. These results are supported by the outcomes of the non-linear dynamic analyses.

The peculiar geometrical characteristics and configurations are the main reasons of the larger seismic resistance of these two towers than the one of the other towers. In the case of Tower IV, the thickness of the four perimeter walls is larger than the one of the other towers and it remains constant along the height of the structure. In addition, at the top of the tower there are no large openings, which can represent a vulnerable upper part, when compared to the majority of the other towers. Tower II is symmetrical both in plan and in elevation and presents a regular internal distribution. The tower has a square plan and its weight is much smaller than the one of the other towers, leading to a great reduction of the seismic forces.

For the other towers under study, different failure modes can be observed. The role played by both the geometrical characteristics and the presence of irregularities on the possible collapse mechanisms is highlighted by the results of the analyses. Problems relative to the structural configuration, especially asymmetry and inadequate arrangement of openings, can affect the level of damage in the towers.

A non-uniform stiffness and strength distribution, in plan and elevation, and torsional effects can be some of the main causes for a widespread damage and even collapse of the towers. Tower VI is characterized by a wall with small thickness and a large opening at the top. A failure mechanism involving the detachment of the wall is registered.

For Tower V and Tower VIII, the role played by the internal vaults in modifying the load path for gravity and earthquake loads is evident and unexpected stress concentrations may arise. Localized and severe damage as a result of a redistribution of internal actions transferred by the vaults is highlighted by the analyses. Moreover, Tower V exhibits sudden variations of the walls thickness along the height and Tower VIII presents some irregularities within the perimeter walls.

The quite marked inclination, the high slenderness and the plan asymmetry are the main causes for the widespread damage of Tower VII.

A significant reduction of stiffness and strength of a wall is observed when multiple openings are present, like in the cases of Tower I and Tower III. Tower I has several openings on two parallel walls and Tower III presents large openings both at the base and at the top. A damage concentration with cracks propagating vertically is observed near the openings.

8 CONCLUSIONS

A comprehensive numerical study conducted by means of advanced FE simulations (non-linear dynamic and static analyses) on eight historical masonry towers located in the North-East region of Italy is presented. From an overall analysis of the results obtained in this study, the following conclusions may be drawn.

-The results of the non-linear dynamic simulations show the high vulnerability of historical masonry towers under horizontal loads. It can be roughly stated that if a residual deformation ranging between 0.4% and 0.8% is reached, the structure may be reasonably considered near collapse.

-Some geometrical characteristics, such as plan and elevation irregularities, presence of belfry, large openings, sudden variation of cross-section, internal vaults and tower inclination, play a crucial role on the seismic performance of the towers. The correlation between local geometrical issues and possible failure modes of the towers is clearly highlighted by the numerical analyses.

-The seismic demands obtained by the non-linear static procedure are generally in a good agreement with those obtained by the non-linear dynamic analyses, above all when the check of the structural safety is verified according to the non-linear static procedure. When the check is not satisfied according to the non-linear static procedure, the seismic demand is smaller than the maximum value of the top displacement experienced by the tower during the dynamic simulations. The simplified static approach is not able to capture the inertial effects associated with seismic excitation that can lead to heavy damage and premature collapse of the structure.

-The non-linear static procedure may provide reasonable synthetic predictions of the seismic vulnerability of the towers. The two approaches provide similar results in terms of seismic safety assessment, with slightly less conservative predictions for the non-linear static procedure. A comparison between the results obtained by the two approaches shows that the non-linear static procedure is able to assess the structural safety only when local collapse failures are not involved.

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