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On the behavior factor of masonry towers

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ABSTRACT

A recently proposed numerical algorithm, for the pushover analysis of masonry towers, is here adopted for the evaluation of the behavior factor, entering the simplified seismic analysis of masonry towers, suggested in the Italian Directive for the assessment and reduction of the seismic risk of the cultural heritage. In order to consider, within a probabilistic context, the uncertainty of the mechanical and structural parameters involved, Monte Carlo method is adopted. The study indicated that the reduction factor of the seismic forces depends mainly on the acting stress over compressive resistance ratio. It is shown that the actual value proposed in the Italian Directive may be unsafe for high values of this ratio. Finally, an empirical formula based on the different Monte Carlo simulations is calibrated for the prediction of the behavior factor.

1. Introduction

Masonry historical buildings are often characterized by a high seismic vulnerability. These structures were designed with respect to the gravity loads only and do not possess adequate resistance and ductility against horizontal loads, such as those induced by an earthquake [1–4].

The seismic analysis of masonry structures poses many difficulties, in view of their complexity [5–7] and of the nature of the constituent materials. The mechanical behavior of masonry is characterized by negligible strength and brittleness in tension, and dissipative with softening behavior in compression [8,9].

Within the architectural cultural heritage of a Country, masonry towers have a prominent place and for this reason their protection against earthquakes appears to be of primary importance.

The dynamic behavior of masonry towers has been often studied in literature, see e.g. [10], to investigate their seismic vulnerability, which sometimes led them to collapse in case of earthquake, as shown by some recent events, see e.g. [11].

According to [12], masonry towers can be analyzed under seismic actions, in a preliminary and simplified approach, as cantilever structures subjected to a global bending failure mechanism at the critical section (LV1 analysis). In relation to this mechanism, a behavior factor q is adopted to reduce the seismic forces that the structure would experience if its response was completely elastic to the minimum seismic forces, which may be used in the analysis, taking into account the displacement and energy dissipation capacity of the structure.

Therefore, the behavior factor represents a fundamental parameter in the seismic vulnerability assessment of a structure, and a wrong estimation of it may lead to non-conservative results. In [12] the value of the behavior factor is assumed, by analogy with conventional masonry buildings, between 3.6 for regular towers and 2.8 in the presence of abrupt changes in stiffness along the height or in contact adjacent structures. However, the peculiar structural behavior of a tower can hardly be compared to a masonry building, especially if the assumed failure mechanism is that of a cantilever structure failing in bending. For this reason, a tower oriented definition of q is sought in what follows.

Clear definitions of the behavior factor are hard to find in seismic codes. According to [13], the q factor is generally defined in terms of the ratio of the peak ground acceleration producing collapse of the structure to that at which first yielding occurs. Then there exist different studies in literature for the identification of the behavior factor, depending mainly on the material properties and structural configuration, usually based on the definition of indices accounting for damage accumulation, see [14]. Fathi, Castiglioni et al. proposed a cumulative damage model for the estimation of the behavior factor for steel structures, see [15–17]. Kappos, Chryssanthopoulos et al. and Gómez-Martínez et al. for a reinforced concrete structure, see [18–20], and Gattesco for a timber structure, see [21], computed the q factor on the basis of the ductility and overstrength capacity. For masonry structures, Tomaževič adopted an experimental based procedure, see [22], while cyclic lateral resistance tests combined with numerical modeling have been used to assess the load reduction factors of clay masonry walls in [23].

In this paper, a recent numerical approach for the pushover analysis of masonry towers, presented and validated in [24], was exploited to assess the behavior factor q. Consistently with the global failure

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mechanism assumed in [12], the pushover algorithm considers a global beam model of the tower and the ultimate limit state is reached in the critical section under axial and bending actions.

Nonlinear static analyses have been increasingly used for the seismic vulnerability assessment of masonry structures [25–27]. However, as evidenced in design codes and literature [28,29], the use of nonlinear static analyses can generally present some limitations. In fact, this type of analysis cannot accurately account for the changes in dynamic response and in inertial load patterns that develop in a structure as it degrades in stiffness. These pushover-based methods may be inaccurate also when applied to structures whose failure mechanism is influenced by the higher modes of vibration [30–32]. However, in the present study, the above limitations are of limited concern since the aim of the paper is to determine the behavior factor related to the global bending failure mechanism only, i.e. that induced by the first mode of vibration. Local failure mechanisms induced by peculiar crack patterns, the presence of vaults and/or large openings or the effect of higher modes, which may characterize the collapse mechanisms of masonry towers, see [33], are not considered in the adopted pushover analysis and have still to be checked by proper methods.

In order to take into account the uncertainties of the material and structural parameters involved and provide a statistical characterization of the q factor, its value has been computed in a probabilistic context, through the execution of different Monte Carlo simulations, based on several non-linear static analyses of different standardized tower structural configurations. The robustness and numerical efficiency of the adopted pushover algorithm turned out to be very useful in these probabilistic analyses, which required results from a large number of simulations, to reach conclusions valid from a statistical point of view.

2. Numerical procedure

The estimation of the behavior factor is based on the numerical approach for the pushover analysis of masonry towers, proposed in [24]. This numerical model is able to consider the main features affecting the global structural response: hollow arbitrary sections, nonlinear material behavior with dissipation and softening and nonlinear geometric effects. The shape of the load distribution is varied according to different formulations, and an ad hoc algorithm is proposed, in order to follow the post peak softening branch of the structural response. To avoid curvature localization, caused by the material softening behavior, a plastic hinge is introduced at the critical section and the effect of its length is dealt with in a probabilistic context as for the other material parameters.

Masonry is modeled by the stress-strain relationship represented in Fig. 1, already adopted in other studies (see [9,24,34]). It is assumed that, beyond the elastic limits, masonry behavior is characterized in compression by limited ductility followed by a softening branch, representing macroscopically material crushing; and in tension by a

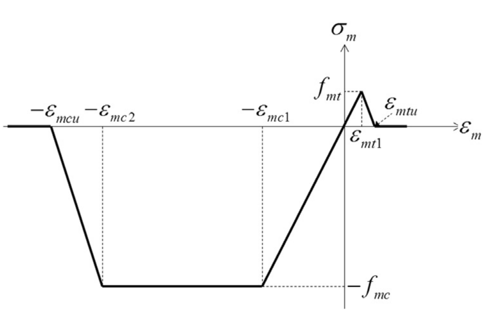

Fig. 1. Stress-strain relationship assumed for masonry. f_{mc} is the compressive strength; $\varepsilon_{mc2} = \mu_1\varepsilon_{mc1}$ and $\varepsilon_{mcu} = \mu_2\varepsilon_{mc2}$ are the strain at the end of the plateau and at the end of the softening branch in compression, respectively; $\varepsilon_{mt1} = f_{mt}/E_m$ and ε_{mtu} represent the strain at the tensile peak stress f_{mt} and the ultimate tensile strain, respectively. Young modulus E_m is assumed equal in tension as in compression.

brittle linear softening curve representing the formation and propagation of micro cracks in the material.

2.1. Load distribution

The more general expression of the seismic horizontal force per unit volume acting on a masonry tower can be expressed as follows:

$$\lambda(z) = \alpha W \frac{\psi(z)}{\int_V \psi(z) dV} \quad (1)$$

where: W is the total weight, $\Psi(z)$ provides the shape of the load distribution, usually taken as the first mode; and z is the coordinate along the height of the tower. According to seismic codes, see e.g. [12,35], $\alpha = \lambda S_e(T_1)/qg$ and the resultant base shear force is equal to:

$$F_{\max} = \int_V \lambda(z) dV = \frac{\lambda S_e(T_1)}{qg} W \quad (2)$$

where: q is the behavior factor, λ is a parameter assumed equal to 0.85, according to [12] and $S_e(T_1)$ is the elastic spectrum value in correspondence of the first natural period T_1 of the tower. The force per unit length $q(z)$ acting on the tower modeled as a beam reads:

$$q(z) = \lambda(z) A(z) = \frac{\lambda S_e(T_1)}{qg} W \frac{\psi(z) A(z)}{\int_V \psi(z) dV} \quad (3)$$

In the following analyses $\Psi(z)$ is assumed both as the first mode of a cantilever beam or as a polynomial function governed by an exponent n, namely:

$$\psi(z) = \begin{cases} \bar{\psi}\left[\cos az - \cosh az - \dfrac{\cos aH_T + \cosh aH_T}{\sin aH_T + \sinh aH_T}(\sin az - \sinh az)\right] \\ \qquad\qquad (aH_T = 1.875) \\ \left(\dfrac{z}{H_T}\right)^n \quad \text{with } n = 0, 1 \end{cases} \quad (4a,b)$$

with H_T being the height of the tower and $\bar{\psi}$ computed imposing $\Psi(H_T) = 1$.

2.2. LV1 analysis

As suggested in [12], the seismic vulnerability of masonry towers can be analyzed, in a preliminary and simplified approach, assuming a global bending failure mechanism at the critical section (say $z = z_{crit}$), where the acting bending moment turns out to be equal to:

$$M_{Ed}(z = z_{crit}) = \int_{z_{crit}}^{H_T} q(z) z dz = \lambda \frac{S_e(T_1)}{qg} W \cdot \frac{\int_{z_{crit}}^{H_T} \psi(z) A(z) z dz}{\int_V \psi(z) dV} \quad (5)$$

According to LV1 analysis the ultimate limit state is reached when the bending moment induced by the seismic action equals the corresponding resisting action, i.e. $M_{Ed}(z = z_{crit}) = M_{Rd}(z = z_{crit})$.

It follows that according to LV1 analysis the critical seismic action reads:

$$S_e^{LV1}(T_1) = \frac{\overbrace{M_{Rd}(z = z_{crit})}^{M_{Rd,crit}} g \overbrace{\int_V \psi(z) dV}^{I_V}}{\lambda W \underbrace{\int_{z_{crit}}^{H} \psi(z) A(z) z dz}_{I_{Z,crit}}} = \frac{M_{Rd,crit} g}{\lambda W} \frac{I_V}{I_{Z,crit}} \quad (6)$$

2.3. N2 analysis

Starting from the pushover curve, the seismic vulnerability of a masonry tower can also be evaluated by means of a comparison between the displacement capacity and the displacement demand, both referring to the same control point, placed at the top section of the