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Seismic Assessment of a Historical Tower with Advanced Numerical Model Tuned on Ambient Vibration Data

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Abstract This paper deals with the dynamic characterization and the evaluation of the seismic response of the medieval civic tower of Soncino (Cremona, Italy). The dynamic characteristics and the mechanical properties of the masonry tower are evaluated through ambient vibration tests, which provide results in a fast and non destructive way with respect to the traditional methods such as forced vibration tests. Nonlinear static and dynamic analyses are performed on a finite element model of the tower calibrated on the results of the dynamic identification. The damage levels and the seismic capacity of the structure are also evaluated. The obtained results allow to predict the seismic behaviour of the tower and to define possible strengthening and restoration interventions.

Keywords: Historical masonry tower, dynamic identification, ambient vibration test, pushover analysis, nonlinear dynamic analysis.

Introduction

The safeguard of historical constructions from seismic actions is a predominant issue in Italy because of the richness of its architectural heritage. An effective seismic assessment of such structures can be achieved only through nonlinear static and dynamic analyses. However these structures can not be investigated through invasive tests, therefore the dynamic identification by ambient vibration tests represents a valid alternative tool to define accurate numerical models, useful to perform nonlinear seismic analyses.

In the following the actual seismic capacity of the medieval civic tower of Soncino (Cremona, Italy) is evaluated by means of nonlinear static and dynamic analyses performed on a finite element model (FEM) of the masonry tower. The parameters of the model are calibrated on the results of a dynamic identification based on ambient vibrations input. The dynamic identification allows to evaluate the mechanical parameters of the masonry and the boundary constraints of the tower.

Structural Identification

Description and Historical Survey The civic tower of Soncino (Cremona, Italy) is a brick masonry structure having a square plant with a 6 m side (Fig. 1). At the time of its construction (12^{th} century) the tower was 31.5 m tall, but in 1575 it was heightened at the present height of 41.8 m. After the 1802 earthquake the tower has undergone several interventions that gave it the present shape (Galantino 1869).

The masonry walls of the structure have a thickness varying from 1.55 m at the base to 0.98 m at the top. The tower is surrounded on three sides by other constructions up to the height of about 11 m. These constructions are constituted by masonry walls in the east-west direction (X direction) and by frames with masonry columns and concrete beams in the north-south direction (Y direction), on the southern side only. At the height of 6.7 m there is a barrel masonry vault, while at the height of 35 m there is a reinforced concrete slab.

The tower shows a good preservation status even though some surface damages due to its age and to the atmospheric exposition, as well as to past earthquakes, are present (Fig. 2). Therefore its safe-

Figure 1: North-east view of the medieval civic tower of Soncino (Cremona, Italy)

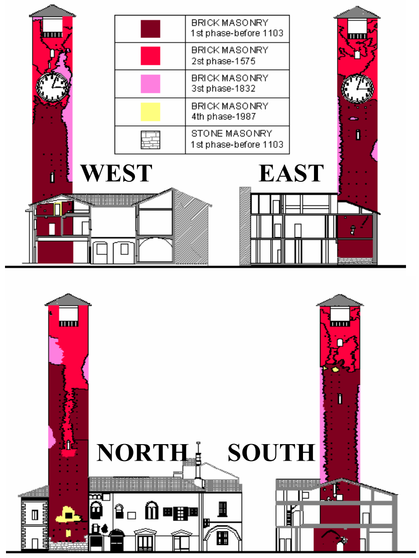

Figure 2: Construction phases of the tower

guard against possible future seismic actions is of primary importance.

Ambient Vibration Data Historical constructions can not be tested with destructive methods; for this kind of structures the dynamic identification by ambient vibration data is the most appropriate test. The civic tower of Soncino has been the object of an experimental campaign performed by the Milano IDPA-CNR institute (Dusi et al. 2006, Dusi et al. 2007). The aim of the performed structural identification was the evaluation of the Young modulus E of the masonry; two values of E were estimated: one for the masonry of the original construction (E1), another for the masonry used to heighten the tower in 1575 (E2).

The signals recorded during the test have been analyzed in the frequency domain. For each recording station and for each component of acceleration the power spectral density function (PSD) has been extrapolated. The peaks of the PSD highlight the fundamental frequencies of the structure; the peak value of the PSD is related to the displacement and then to the modal shape. The two fundamental frequencies resulted 1.08 Hz in the X direction and 1.11 Hz in the Y direction. The gap between the two fundamental frequencies, although minimal, points out the significant constraint effect in the X direction given by the masonry walls of the constructions surrounding the tower.

Identification of the Young's Modulus and Evaluation of the Compressive Strength Two 2D models have been set up, considering the symmetry of the tower and the independence of the two fundamental frequencies. The identification process is based on an iterative procedure that performs the modal analysis until an objective function is minimized; this function describes the gap between experimental and analytical frequencies and contains the modal assurance criterion (MAC), an index that measures the correspondence between experimental and analytical modal shapes.

The definition of the degree of constraint given to the tower by the surrounding constructions was of significant importance to properly reproduce its structural behavior. Different hypothesis have been made and the one best fitting with the experimental results has been obtained restraining the tower up the height of 11 m.

The structural identification led to a value of E1=1000 MPa and of E2=15-30 MPa. The values obtained for E2 are not significant from a physical point of view, but they evidence the degradation of the masonry in the upper part of the tower and the bad quality of its connection with the lower part. In the hypothesis that the damaged masonry in the upper part of the tower be repaired, a single value of E=1000 MPa has been assigned in the following to the whole structure.

The elastic modulus identified from ambient vibrations is a dynamic modulus; a ratio of 1.2 has been assumed between the dynamic and the static modulus. This assumption led to a value of 833

MPa for the static modulus. A reasonable value of the compressive strength has been deduced from the identified elastic modulus. Considering a parabolic-rectangular stress-strain relation for the masonry and taking into account the stress state due to vertical loads, at which the test was performed, a value of 1.1 MPa has been estimated for the masonry compressive strength.

Linear Elastic Analysis

A preliminary linear elastic analysis has been performed on a FEM model of the tower with the computer program SAP2000. The interaction of the tower with the surrounding constructions has been simulated utilizing springs in the FEM model; the stiffness of the springs in the X direction has been assumed equal to the shear stiffness of the walls in this direction, while the stiffness of the springs in the Y direction has been assumed equal to that evaluated with reference to shear-type models of the frames in this direction. A response spectrum analysis has been performed referring to a return period of 975 years according to the current Italian code (NTC 2008), that for existing buildings suggests a behaviour factor $q = 2 \cdot \alpha_u / \alpha_1$, where α_u / α_1 is the overstrength factor. In the case of lack of specific studies a value of $\alpha_u / \alpha_1 = 1.5$ leading to a behaviour factor $q = 3$ can be assumed. To investigate the dissipative capacity of the tower and support the assumption of a behaviour factor $q = 3$ a pushover analysis on a simplified numerical model, made up of linear elements with plastic hinges at the ends, has been performed, obtaining a value of 1.65 for α_u / α_1, that lead to a strength reduction factor of 3.3. The response spectrum analysis has been then performed assuming the value $q=3$ suggested by the current Italian code (NTC 2008).

The performed linear elastic analysis allowed to preliminarily investigate the stress levels in the tower under the design seismic action. In particular, the stress level has been defined at the most critical sections of the tower: at the base, at the height of 12 m where the constrain effects of the surrounding constructions are negligible, and at the height of 19.1 m in correspondence of the first reduction of thickness of the walls. The calculated stress levels are generally compatible with the masonry compressive strength, even if at the base of the tower the principal compressive stress reaches the value of 1.06 MPa, that is very close to the masonry compressive strength. These results have been obtained considering a behaviour factor q=3, that is in the hypothesis that the tower dissipates energy when structural damages occur. The actual capacity of the tower has been than defined by performing a nonlinear analysis on an advanced numerical model.

Nonlinear Analysis

FEM Model and Material Nonlinearity The nonlinear static and dynamic analyses of the masonry tower have been performed with the computer program DIANA (Diana 2008). For the compressive regime the Drucker-Prager failure surface with a cohesion $c = 0.45MPa$ and a friction angle $\phi = 11°$ has been considered, while for the tensile regime a constant tension cut-off with smeared cracking, a tensile strength of 0.1 MPa and a linear softening until the maximum tensile strain of 1‰ have been considered. A pushover analyses has been first performed, both in the X and the Y direction, applying along the tower forces proportional to the masses for the displacements of the normalized deformed shape in the considered direction. Successively, a nonlinear dynamic analysis has been carried out applying at the tower base the El Centro recorded accelerogram of the 1940 Imperial Valley earthquake. The analysis has been conducted varying the peak acceleration from a value of 0.05 g up to the value determining the collapse of the structure.

Pushover Analysis The pushover analysis has been carried out up to the collapse, in correspondence of which the structure shows an extended cracking and crushing. Figs. 3(a) and 3(b) show the principal stress-strain relation in the section at the height of 3 m at the compression and the tension side, respectively; the reported diagrams evidence that the principal stresses reach the strength limit of the masonry. They are also representative of the constitutive behaviour of the implemented material, that shows a hardening in compression (Fig. 3(a)) and a linear softening in tension, approximately reaching the value of the limit strain (1‰) (Fig. 3(b)). Fig. 4(a) reports the vertical stress in the east façade of the tower at the collapse load step, while Figs. 4(b) and 4(c) show the crushing and cracking patterns in the critical zones at the base of this façade for the same load step.

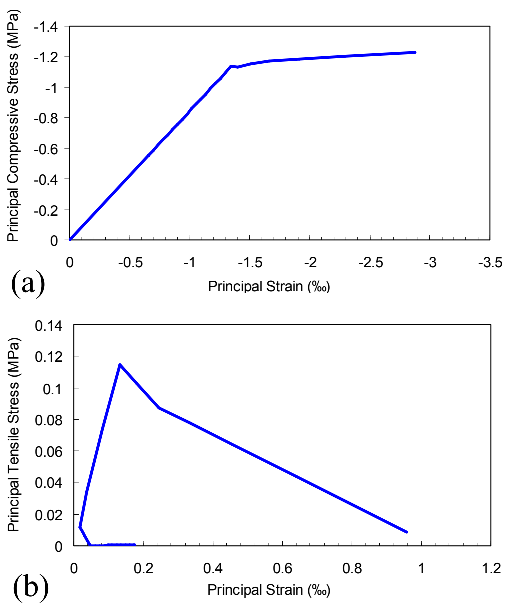

Figure 3: Principal stress-strain relation at the (a) compression and (b) tension side of the horizontal section at the height of 3 m

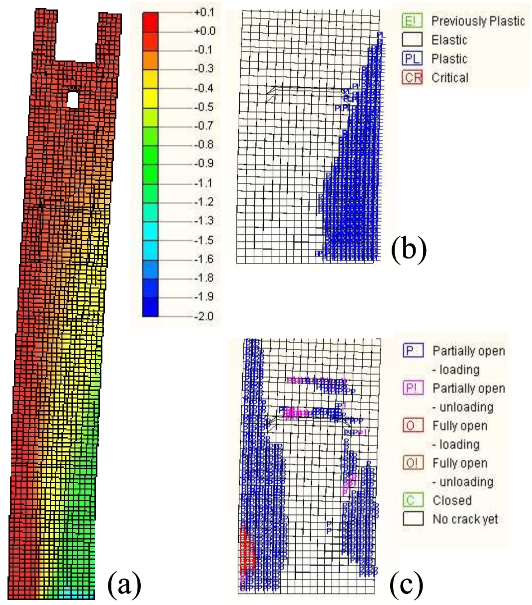

Figure 4: (a) Vertical stresses in the east façade at the collapse load step; (b) crushing and (c) cracking patterns in the critical zones at the base of the east façade at the collapse load step

The performed analysis has allowed to define the pushover curve of the tower. The curve has been then compared with the one obtained from the simplified initial analysis on the model including plastic hinges (Fig. 5). The comparison shows a good correlation between the two curves, in spite of the different models adopted.

The pushover curve obtained from the MDOF model has been reduced by the modal participation factor of the fundamental mode to obtain the capacity curve of the equivalent SDOF system (Fig. 6, green line), that has been then transformed in a bilinear curve (Fig. 6, red line). The collapse limit state has been verified by means of a graphical procedure in the acceleration-displacement response spectrum (ADRS) plane: the capacity of the structure has been compared with the seismic demand represented by the ultimate state elastic spectrum (Fig. 7). The intersection of the radial line corresponding to the elastic period T^* of the bilinear system with the demand spectrum identifies the

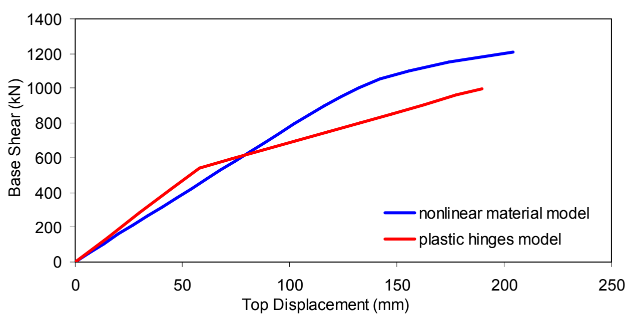

Figure 5: Comparison between the capacity curves of the nonlinear material model and of the plastic hinges model

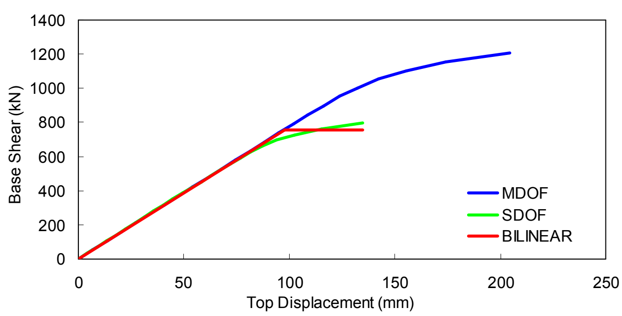

Figure 6:Pushover curve of the MDOF model, capacity curve of the equivalent SDOF system and bilinear curve

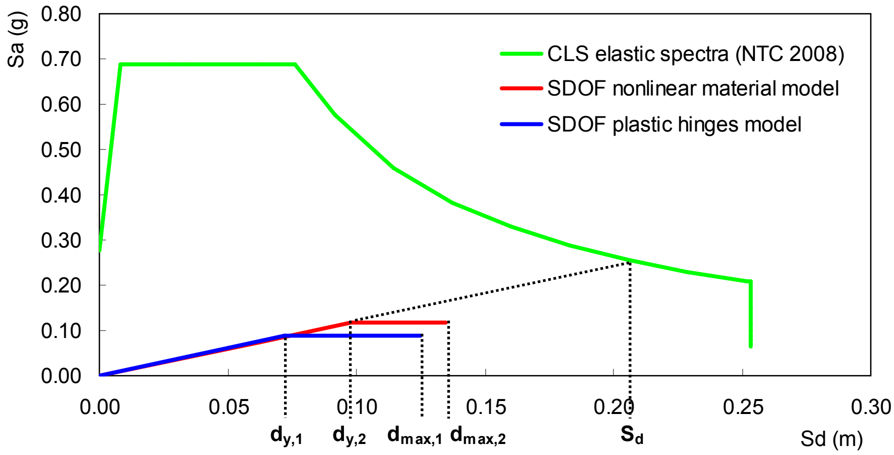

Figure 7: Capacity curves of the two considered models and seismic demand spectrum

displacement demand S_d. From the two bilinear pushover curves associated to the two different considered models (Fig. 7, blue and red lines) it was possible to determine the value of the behaviour factor q as the ratio between the maximum displacement d_{max} and the yield displacement d_y. The two considered models give a value of q_1=1.74 and q_2=1.38, respectively; both these values are lower then the value suggested by the current italian code (NTC 2008). Moreover, the different values obtained evidence the conventional nature of the factor q, which only gives an indication about the dissipative capacity of the structure, that can be actually assessed only through more sophisticated models, taking into account the material nonlinearity.

Nonlinear Dynamic Analysis Four nonlinear dynamic analyses have been carried out utilizing as seismic input the El Centro accelerogram with a peak acceleration of 0.05g, 0.10g, 0.15g and 0.20g. Figs. 8(a) and 8(b) report the obtained top displacement and base shear time histories, respectively, that evidence how the base shear oscillates at a different frequency with respect to the top displacement. In particular the qualitative comparison between the two curves (Fig. 8(c)) shows

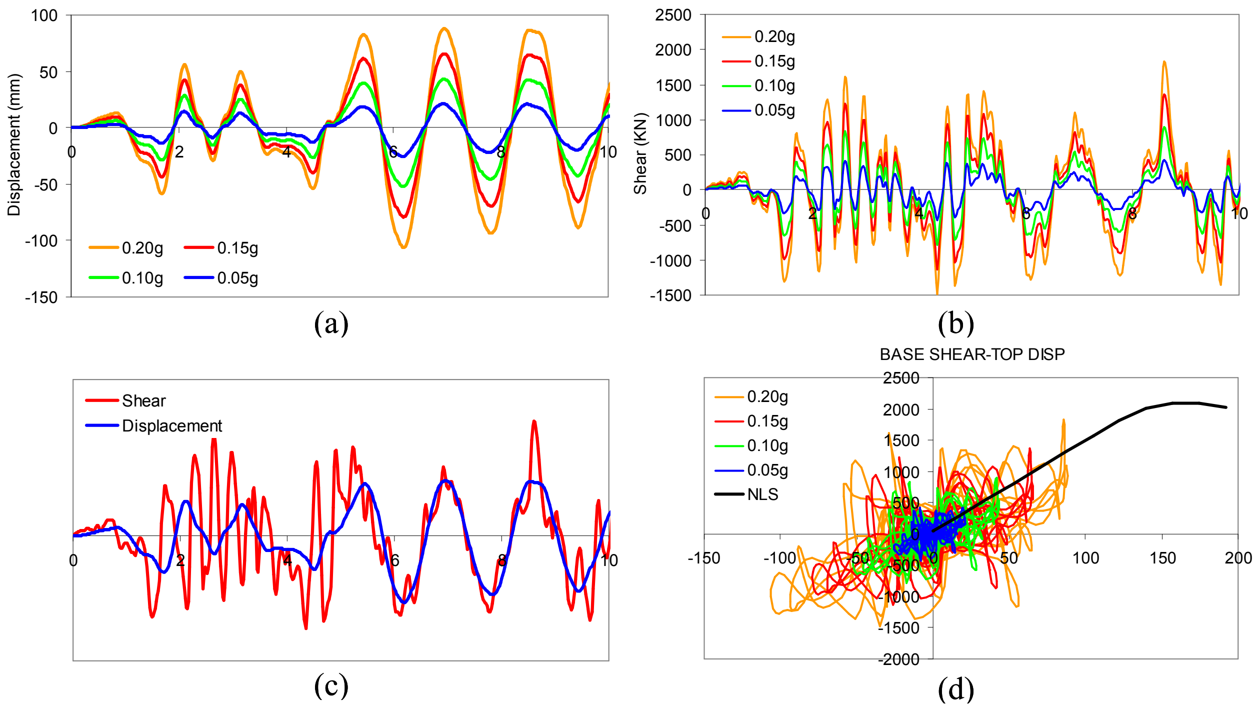

Figure 8: (a) Top displacement time history, (b) base shear time history and (c) relative qualitative comparison; (d) base shear-top displacement curves and pushover curve

that, with the exclusion of the initial transitory, the base shear oscillates at two frequencies: a principal frequency corresponding to the first fundamental frequency of the structure (the same at which the top displacement oscillates), and a secondary frequency, corresponding to the frequency of the second vibration mode of the structure. Therefore, while the top displacement is dominated by the first modal shape, the base shear is also conditioned by the effects of the higher modes. The base shear-top displacement curves are very irregular (Fig. 8(d)) since the sign inversions of the base shear are not followed by the top displacement; this effect becomes less relevant after the initial transitory. All the base shear-top displacement curves have as a backbone the static pushover curve (Fig. 8(d)), evidencing a good correlation between the results obtained with the nonlinear static analysis and those obtained with the nonlinear dynamic analysis. However, the nonlinear dynamic analysis provides levels of displacement capacity lower than those resulting from the nonlinear static analysis; this evidences how the nonlinear static analysis is unable to capture the inertial effects associated with a dynamic input that can lead to an early collapse of the structure.

Conclusions

The performed study highlights the importance of experimental campaigns based on ambient vibration testing. In the examined case the dynamic identification tuned on ambient vibration data gave important information about the material properties and the boundary constraints. The tower FEM model, calibrated on the results of the experimental campaign, has been the object of response spectrum analyses and nonlinear static and dynamic analyses that evidenced, among other aspects, the conventional nature of the behaviour factor q: different modeling approaches can lead to different values of q. The analyses have also evidenced that the q value suggested by the current italian code overestimates the actual dissipative capacity of the tower of a factor ranging between 1.7 and 2.2.

The results obtained with the nonlinear dynamic analysis show a good correlation with those obtained with the nonlinear static analysis; all the base shear-top displacement curves have as a backbone the static pushover curve. However, the nonlinear dynamic analysis provides levels of displacement capacity lower than those resulting from the nonlinear static analysis; moreover they evidenced the influence of the higher frequencies on the response in terms of base shear. The study underlines the importance of an advanced modeling and analysis, taking into account the material nonlinearity and the effect related to a dynamic input, to evaluate the actual seismic capacity of a masonry structure, also when it should apparently not be so influenced by the nonlinear behaviour as the analyzed slender tower.

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