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Modal pushover and response history analyses of a masonry chimney before and after shortening

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ABSTRACT

The 50 m high masonry chimney located in the old industrial facility that houses the School of Engineering of the University of Ferrara, Italy, suffered severe damages during the 2012 Emilia seismic sequence. Afterward, for security reasons, the upper damaged 12.40 m were disassembled. Both before and after shortening, the ratio between the effective mass of the fundamental mode and the total mass is approximately 20%, leading standard pushover analysis methods not to be appropriate for estimating the seismic demand. Using a single, consistent 3D FE formulation, the results of a Modal Pushover Analysis (MPA) and four nonlinear Response History Analyses (RHA) for the shortened and the original chimney were presented in the paper. The ground motions considered in the simulations are accelerograms recorded during recent and less recent devastating seismic events in Italy, New Zealand, and Japan.

For both chimneys, a very good agreement between MPA and RHA was observed in terms of lateral displacements. Moreover, for the 50 m high chimney, a strong similarity was observed between the damage maps deriving from the MPA and those obtained with the RHA. All analyses confirmed a significant contribution of the higher modes. For the shortened chimney, the MPA revealed damages in the lower part of the stack (8–21 m), because of a prevailing influence of the fundamental mode. In the RHA, a more evident contribution of the higher modes was observed, probably because of the effect of the vertical component of the ground motion, not accounted for in the MPA.

1. Introduction

The seismic analysis of unreinforced masonry chimneys and towers represents a real challenge for the Earthquake Engineering, especially because of the difficulties in developing predictive computational models for reproducing the nonlinear masonry response in the dynamic field. The problem complexity is often increased by the strong influence of the higher modes of vibration. One of the first studies on the possible failure mechanisms of masonry chimneys under earthquake excitation is reported in [1], where an attempt to explain earthquake-induced damages observed in some chimneys in California and China in the Eighties was presented. In [2], based on a simplified model using elastic beam elements connected through nonlinear joints, and accounting for the soil–structure interaction, a seismic vulnerability assessment for a 97.2 m high masonry tower in the historical center of Bologna, Italy, was presented. More recently, an increased computing power allowed for more advanced analyses using two- or three-dimensional finite elements. In [3,4], typical first-mode failure mechanisms, with collapse being triggered by masonry cracking at the chimney base, were described. In [5], an experimentally-calibrated numerical model is used to verify the effectiveness of the seismic strengthening of an industrial masonry chimney using CFRP strips. A 70 m high stone masonry minaret was analyzed in [6], where a critical comparison between nonlinear static and dynamic analysis methods was reported. In particular, in that work it was concluded that the pushover analysis is not able to reproduce the change in the dynamic properties of the structure typically occurring during severe ground motions, leading to unacceptable underestimations of lateral displacements and drifts. An interesting state-of-the-art review of seismic assessment and strengthening techniques of masonry chimneys is reported in [7]. The effects of wind and seismic actions on a damaged chimney in Alicante, Spain, were analyzed in a recent work [8] through a numerical model that includes the longitudinal cracks observed along the stack. With regard to the in-situ characterization of masonry chimneys, an experimentally-based model updating

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technique was developed in [9]. The procedure proposed can detect the stiffness variations along the chimney stack due to possible damages, resulting to be a useful tool for assessing the structural stability [10].

In the present paper, a seismic damage assessment for an unreinforced masonry chimney built at the beginning of the 20th century in the service of a sugar factory in Ferrara, Italy, is presented. The chimney is no longer in use since the end of the Fifties, when electric power replaced steam in the industry, but represents a symbol in the industrial history of the Po River Plain and is protected by the Italian Ministry of Cultural Heritage. In the aftermath of the 2012 earthquake sequence in Northern Italy (Emilia earthquake), a survey campaign put in evidence diagonal cracks in the outer surface of the stack at an altitude of approximately 40–45 m. Preliminary numerical analyses in both linear and nonlinear field highlighted the role played by the higher modes of vibration in determining the damages [11]. The behavior factor estimated on the basis of a Modal Pushover Analysis (MPA) was in line with the value $q = 1.5$ recommended by [12], indicating a very low dissipative capacity. In the same work, a number of nonlinear Response History Analyses (RHA) were conducted using a numerical model with two-dimensional rigid finite elements connected through nonlinear interfaces, that allowed for partially explaining the vulnerability of the upper part of the stack.

The geology of the Po River Plain is characterized by very deep alluvial deposits. This feature, although the distances of the chimney from the epicenters of the two mainshocks exceed 30 km, might have likely amplified the seismic shake, contributing to the damages. Anyway, the observed damage pattern was associated with a significant mortar deterioration in the same part of the structure (i.e., at elevation $z = 38–50$ m), that certainly weakened the stack and probably triggered the crack propagation. This situation, because of the proximity of the chimney to buildings containing university classrooms and offices, suggested, for security reasons, a shortening of approximately 12 m to remove the damaged part. The current chimney (Fig. 1a) is 37.6 m high, and is not substantially damaged. In view of a possible reassembling of the removed part to recover the original, 50 m high configuration (Fig. 1b), the principal objective of the present investigation is to compare the seismic response of the shortened chimney with that of the original structure. With this aim, MPA and RHA of the two structures were carried out using a single consistent numerical formulation with three-dimensional nonlinear finite elements. Because the effective mass associated with the fundamental mode of vibration is only approximately 20% of the total mass for both the 37.6 m and the 50 m high chimney, it was necessary to include in the MPA four and five "modal" force distributions, respectively. In these analyses, the target displacements were computed from the inelastic response spectrum, i.e., the response spectrum evaluated for elasto-plastic systems, of the ground motion recorded the May 29th, 2012, by the seismic station in Mirandola, at an epicentral distance of approximately 5 km. In the RHA, both horizontal and vertical components of the same, and of other three real ground motions were alternatively applied at the base of the two chimneys. Described and comparatively discussed in the paper are lateral displacement profiles and damage maps obtained for the chimneys from the static and dynamic analysis methods.

2. Numerical models

2.1. Geometry of the chimneys

The geometry of the original, 50 m high chimney resulting from a preliminary inspection was first reported in [11]. Afterward, an accurate survey with 3D laser scanner technique allowed for an overall geometry update. The resulting new values of the outer diameter and the corresponding thicknesses are reported in Table 1 for ten cross sections. Note that section S7, located at $z = 37.6$ m, corresponds to the top of the shortened chimney. The analyses of the shortened chimney reported hereinafter assumed an initially undamaged state, because earthquake-induced damages were observed only in the removed part of the stack. Moreover, in view of a possible reassembling of this part in the near future, also the 50 m high chimney was assumed to be initially undamaged.

2.2. FE discretization

For the numerical analyses, FE models using 8-node solid (brick) elements were developed. In particular, for the shortened chimney, 75 and 16 equal subdivisions along the vertical axis and in the planes of the cross sections, respectively, were used, resulting in a total of 1200 finite elements and 2432 nodes. The choice of this FE discretization, presenting one single subdivision through the thickness, was motivated by the need for enabling accurate numerical solutions while limiting the computational effort. A second, refined FE mesh was implemented using two subdivisions through the thickness, leading to 2400 finite elements and 3648 nodes. The comparison between the two numerical models with regard to the frequency and pushover analyses is presented in the next section. Herein we can state in advance that the coarser numerical model proved to be sufficiently accurate. Therefore, using the same meshing criterion, the original, 50 m high chimney was discretized using 100 and 16 equal subdivisions along the vertical axis and in the planes of the cross sections, respectively, resulting in 1600 solid elements and 3232 nodes.

The influence of the soil–structure interaction on the seismic response of slender structures such as chimneys and towers may be relevant [13], and a suitable analysis of this phenomenon should be based on extensive in-situ investigations [14]. In particular, in [15,16] the dynamic characterization of Medieval brickwork bell towers was presented. In that works, masonry properties such as Young's and transverse shear elastic moduli E and G, as well as shear wave velocity V_s in the surrounding soil, were identified by minimizing a cost function depending on experimentally determined natural frequencies and corresponding numerical estimates. To excite the towers, impulsive loading was used. In the FE models developed, the soil–structure interaction was accounted for by defining, at the tower base, suitable translational and rotational spring stiffnesses that were related to the estimated V_s according to the formulation presented in [17]. Very recently, dynamic identification analysis and model updating of the Ghirlandina tower in Modena, Italy, were published [18]. Mode shapes and corresponding vibration frequencies and damping ratios were obtained using ambient excitation. Some vibration mode was found particularly sensitive to the soil deformability.

It is worth observing that the towers studied in [15,18] have a relatively complex geometry and present a structural connection with the adjacent church, that certainly gives rise to a global stiffening effect. This undoubtedly justifies an accurate assessment of the dynamic behavior.

For the 50 m high chimney investigated in the present paper, no dynamic characterization was carried out until shortening occurred in 2012, above all because the structure was no longer in use since the Fifties. Nevertheless, for the shortened chimney a series of dynamic tests has already been planned for the near future, together with an accurate calibration of soil–structure interaction parameters. Actually, through a simplified numerical model with beam elements, the study presented in [11] showed that, for the 50 m high chimney, the soil–structure interaction has a negligible effect on vibration frequencies and mode shapes, at least from a technical point of view. In fact, by adopting typical