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Displacement requirement in the nonlinear kinematic procedure for masonry structures

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ABSTRACT: The present paper deals with the safety assessment of existing masonry buildings, as far as the analysis of local mechanisms is regarded. Damage pattern surveys after earthquake events have proven that such mechanisms can very frequently activate under horizontal actions, often involving out-of-plane loss of equilibrium. The relevant safety verifications alternatively consist, according to Italian code indications, in linear or non linear kinematic procedures, the latter one being an adapted implementation of the capacity spectrum method. Thus, the conventional spectral displacement capacity is to be compared with the displacement demand, evaluated through a specific response spectrum, whose analytical expression is given by the code. This paper is focused at checking the reliability of such expression in representing the out-of-plane displacement demand on single elements, deriving from earthquake excitation. In this aim, displacement spectra are derived from time history analyses of sample 2-story unreinforced masonry buildings, in which selected walls are monitored.

1 INTRODUCTION

Analysis and safety assessment of masonry structures under seismic loads are still a controversial issue, due to both the well known difficulties in defining the non-linear mechanical properties of materials, and the selection of suitable models and procedures. In this regard, the recently issued Italian seismic code OPCM/3431 (2005) requires that the collapse acceleration for the whole construction be calculated even for strengthening intervention, at least for determining the priorities. On the other hand, when local intervention do not substantially modify the original behaviour, the Italian Guidelines on the evaluation and reduction of cultural heritage seismic risk (Linee Guida, 2006) suggest to carry out the safety assessment through simplified procedures, since complete analysis of the whole structure might be excessively complex from a computational point of view, or scarcely representative due to insufficient modelling.

As recently enlightened in works by Magenes (Magenes 2006), the safety assessment of existing masonry buildings requires the analysis of global mechanisms as well as local ones, since partial collapses due to the loss of equilibrium in masonry portions can frequently occur (Figures 1–2) and precede or trigger global mechanism. In such cases, the safety verifications generally imply to study parts of the

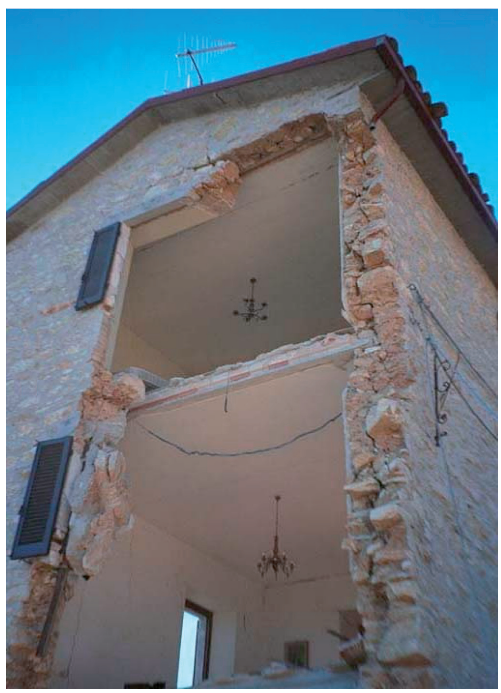

Figure 1. Partial collapse of masonry building.

Figure 2. Partial collapse of masonry building.

structure, characterized by substantially autonomous seismic behaviour (in literature regarded as "macroelements") subjected to in-plane or out-of-plane loading. In this latter case, the verification should be carried out evaluating the failure load by means of the limit equilibrium analysis, since the collapse is characterized mainly by loss of equilibrium rather than exceeding material strengths. Accordingly, the stability is to be assessed through the comparison between displacement capacity and displacement demand, rather than through conventional strength check (Priestley 1985, Doherty et al. 2002, Griffith et al. 2003).

The reliability of kinematic procedures for analysing the seismic behaviour of masonry structures has been only recently subjected to researches and studies. In (Lagomarsino et al. 2004), with reference to the transverse section of the St. Maria del Mare church in Barcelona, the results obtained with limit analysis are compared with the ones of finite element analyses, showing how the two methods can provide similar indications on the seismic vulnerability of the analysed structural scheme. In (Giovinazzi et al. 2006), the same kinematic approach is applied to the facades of earthquake-damaged churches, and the results checked against dynamic analyses. In (Guadagnuolo et al. 2007, Guadagnuolo 2008) linear and non-linear kinematic analyses are carried out with reference to the perimeter walls of a tobacco factory, characterised by repetitive structural schemes with slender walls, verifying the results with non linear finite element analyses.

This paper is focused at checking how far the analytical expression of the displacement demand given by the seismic code OPCM 3431 can be considered representative of actual out-of-plane displacement demand on single elements, rising up from earthquake loading. In this aim, several time history analyses have been carried out on a simple idealised 2-story unreinforced masonry building, in which selected walls are monitored. The derived displacement demand spectrum is then compared with the one provided by OPCM 3431.

2 THE OUT-OF-PLANE RESPONSE OF WALLS

The dynamic response of unreinforced masonry walls to out-of-plane seismic excitation is a complex and not much studied issue, since it involves the dynamic filtering effect of building and floors, the dynamic response of walls, the evaluation of walls strength and out-of-plane displacement capacity. Most difficulties are due to the complicated and convoluted seismic load path from the ground to the face-loaded upper walls of masonry building.

The in-plane loaded walls respond to the ground acceleration with response accelerations that depend on geometry and masses of the building, showing the well-known amplification with height.

At the floor height the in-plane loaded walls response accelerations act as input accelerations for the floor diaphragms. Displacements and accelerations at the floor points depend on the diaphragm stiffness and damping, being equal at all points if the diaphragm is rigid, while are modified if the floor is flexible. The second condition usually characterizes the seismic response of existing masonry buildings. The previous dependence can be theoretically appraised referring to the ratio of the natural floor frequency to the fundamental frequency of building in the in-plane walls direction. For ratios close to unity, large response amplification is likely, while for very flexible floors, the response accelerations are small.

Finally the floor diaphragm response becomes the input acceleration for the face-loaded walls. Specifically, the inertial response of such walls is excited by the floors accelerations below and above, and the response acceleration is usually larger at the walls mid-height. The magnitude of the maximum response acceleration depends on the ratio of the natural frequency of the face-loaded wall to the floor excitation frequency, as well as on damping. If cracks and rocking lead to a moderate lengthening of the face-loaded wall period, the wall response shows an amplification of the input acceleration; on the contrary, for larger

period shifts lower accelerations than the input ones characterize the face-loaded wall response.

Therefore, the ground accelerations are twice modified before loading out-of-plane walls. The consequence of the interaction between in-plane loaded walls and floor diaphragms response and between floors and face-loaded walls is a probable large increase in the ground acceleration magnitude.

Paulay and Priestley (1992) recommend an amplification of the floor acceleration ranging between 1.5 and 2.5 to obtain the ones experienced by the face-loaded walls. Simple numerical evaluations allow assessing that points of second-story walls of 2-story masonry buildings can experience out-of-plane accelerations of about five times the peak ground acceleration also. Therefore, the out-of-plane acceleration can exceed the in-plane one (Kanit & Atimtay 2006). Since the response acceleration increases with height and the upper walls are subjected to lowest gravity loads, it is consistent to assume that masonry buildings may fail initially by out-of-plane collapse of the upper stories walls.

The issue of the out-of-plane response of masonry walls subjected to seismic excitation is also not opportunely taken into account in many seismic codes, so that seismic loading or displacement demand are not clearly outlined, and frequently reference to the non structural elements behaviour is done (OPCM/3431 2005, Tomaževiè 1999).

Even though code requirements and limitations on geometry and connections as well as suitable designs can prevent out-of-plane failures, further in depth studies on the seismic response seem to be necessary.

In this paper, the analysis of the dynamic response of a sample 2-story unreinforced masonry building has allowed to assess the displacement demand in face-loaded walls. Such demand can be also assumed as reference in the safety verification of walls analysed by the nonlinear static procedure described in the following section.

3 THE NONLINEAR STATIC PROCEDURE

The so-called nonlinear kinematic procedure, as implemented in the code approach of O.PCM 3431 (OPCM/3431 2005), allows to evaluate the displacement capacity of masonry elements up to collapse, once an admissible mechanism has been chosen. Starting from the initial configuration, a series of varied configurations with incremental displacement is assigned, in order to simulate the mechanism development. For each configuration, which is uniquely defined by a lagrangian parameter Ψ_i, the horizontal loads multiplier λ_i can be derived through the application of the principle of virtual works with reference to the varied geometry. The analysis can be analytically

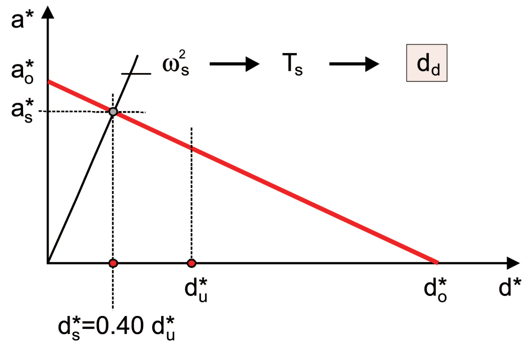

Figure 3. Evaluation of secant period T_S on the equivalent SDOF capacity curve.

performed by considering a sequence of virtual finite rotations and progressively updating the system geometry. As the lagrangian parameter is incremented, the load multiplier, which is only function of Ψ_i, decreases down to zero value, which corresponds to the loss of bearing capacity. In such manner, a response curve, representing the equilibrium path of the element in terms of load multiplier – displacement curve of a control point, is defined. This curve is then converted into the capacity curve a*-d* of a single degree of freedom system. The safety verification for the ultimate limit state is conventionally performed by comparing the ultimate displacement d_u^* (assumed equal to 40% of the zero bearing capacity value d_o^*) with the displacement demand, derived from a specific spectrum, evaluated at the period T_S where the spectral displacement d_s^* equals 40% of the ultimate one (Figure 3).

The analytical expression of the demand spectrum here reported is given by the seismic code OPCM 3431 and is defined similarly to the one adopted for the analysis of the non-structural elements:

$$
\begin{aligned}
&T_S < 1.5T_1 && d_d = a_g S \frac{T_S^2}{4\pi^2}\left(\frac{3\cdot(1+Z/H)}{1+(1-T_S/T_1)^2}-0.5\right)\\
&1.5T_1 \le T_s < T_D && d_d = a_g S \frac{1.5T_1T_s}{4\pi^2}\left(1.9+2.4\frac{Z}{H}\right)\\
&T_D \le T_s && d_d = a_g S \frac{1.5T_1T_D}{4\pi^2}\left(1.9+2.4\frac{Z}{H}\right)
\end{aligned}
\qquad (1)
$$

where a_g is the peak ground acceleration, S and T_D depend on the soil characteristics, T_1 is the fundamental vibration period of the structure along the direction considered, Z is the height from the building foundation to the centre of the weight forces whose masses generate horizontal forces, H is the total height of the building from the foundation.

How far that expression can be considered representative of the actual out-of-plane displacement demand

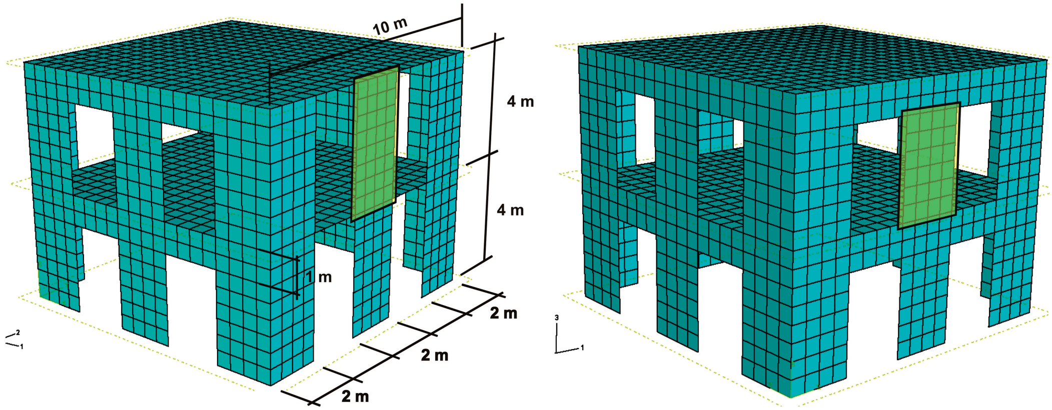

Figure 4. Finite element model of the sample masonry building.

on single elements, rising up from earthquake loading, can be difficult to asses. This represents, in the opinion of the authors, a main issue in the reliability of the non-linear kinematic procedure, at least in the application to local mechanisms verification.

4 ANALYSES AND RESULTS

In order to asses the foregoing mentioned issue, several time history analyses have been carried out on a sample 2-story building, in which selected masonry walls are tracked. A finite element model based on the use of shell elements is assumed in modelling the building, as shown in Figure 4 where the two tracked walls are highlighted. Such walls are positioned at the opposite sides of the building and are excited out of plane by the input acceleration. The difference between the two walls considered consists in the presence or absence of the spandrel beam. In this manner a single analysis can provide two sets of response data, since the influence of the walls difference on the overall behaviour is negligible. The floor diaphragms are modelled assuming r/c slabs at both the two levels. The first vibration period of the building in the direction concerned is equal to 0.17 sec.

For each analysis, the maximum displacement of a control point located at the pier mid-height is extracted. In order to build a displacement spectrum, the dynamic characteristics of only the selected walls are then changed by varying the elastic modulus. In this way, a sufficiently wide range of periods is taken into account, and consequently the displacement spectrum can be determined.

Therefore, the computed response spectrum is at least representative of the displacement demand in the considered masonry walls when subjected to out-of-plane action. Specifically, it provides displacement demand in local vertical bending mechanisms involving the wall portion between floor and roof (Figure 5). Obviously, such an evaluation does not cover the entire class of possible failure mechanisms, since the ones involving simple overturning or different vertical bending mode of piers are also conceivable.

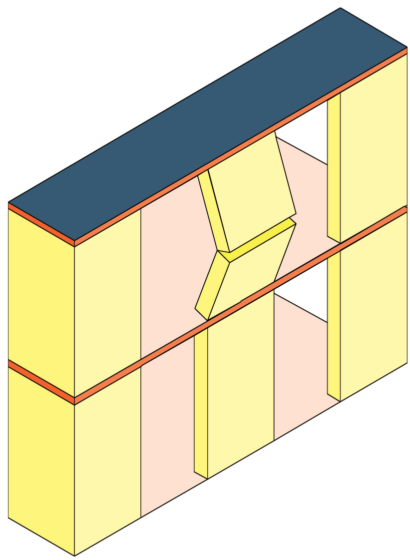

Figure 5. Vertical bending mechanism in upper piers.

The analyses are performed using generated acceleration records. The accelerograms make up a suite of ground motions that comply with the requirements of the Italian seismic code. Specifically, the acceleration histories set taken into account is a spectrum-compatible ensemble (peak ground acceleration $a_g = 0.25$ g, type B subsoil having the constant

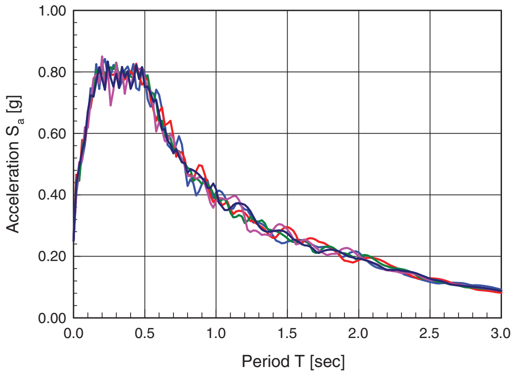

Figure 6. Response spectra of the five generated acceleration records.

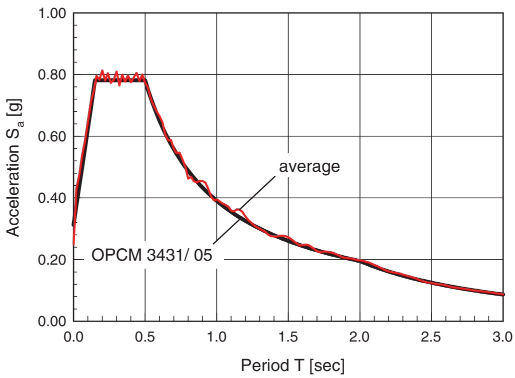

Figure 7. Average spectrum of the five generated acceleration records and OPCM 3431 code spectrum.

branch of the spectrum located between 0.15 and 0.50 seconds, soil factor $S = 1.25$). Figure 6 shows the response spectra of the five generated accelerograms (5% damping). Figure 7 shows the average response spectrum of the five records, closely matching the elastic OPCM 3431 code spectrum.

The computed displacement demands and the corresponding accelerations are plotted in Figure 8 in the ADRS format, which is the customary format for the safety verification by means of capacity spectrum methods. Obviously, the plotted values are obtained averaging the maximum (spectral) displacements and the corresponding accelerations obtained using the five different accelerograms.

The spectra are computed assuming damping ratios of 0.05 and 0.03. The first value is typically used in the computation of code spectra; the second one is mostly representative instead for masonry buildings at low cracking conditions. The demand spectrum

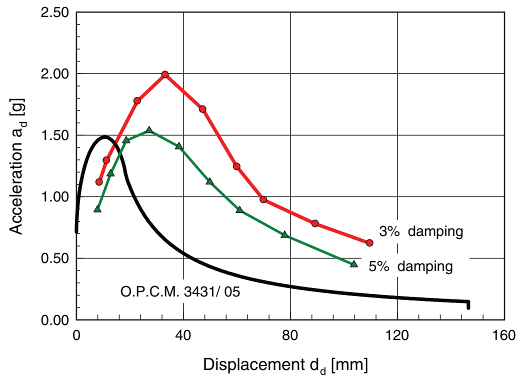

Figure 8. Computed and code displacement and acceleration demand spectra.

provided by the Italian OPCM 3431/2005 through Eqn. (1) assuming $S = 1.25$, $T_1 = 0.17$ sec, $Z = 6.0$ m, $H = 8.0$ m, is also drawn in Figure 8.

The comparison between the code and the computed demand spectrum at 5% damping shows similar acceleration amplification, but large differences in the displacement demand. The maximum acceleration amplification with respect to the peak ground acceleration is of about six times (1.5/0.25). Wider differences in the spectral values characterize the 3% damping spectrum as compared with the code ones.

It is of interest deriving the amplification factor of the out-of-plane acceleration of the upper walls with respect to the floor below. In the examined case, a mean acceleration of 7.12 m/sec^2 (0.73 g) is experienced at the base of the considered wall at the instant of the achieved maximum out-of-plane acceleration. Therefore, being this latter acceleration equal to 15.08 m/sec^2 (1.54 g), the amplification factor is equal to 2.11, close to the conservative factor of 2.5 advised by Paulay and Priestley (1992).

However, to evaluate the possible repercussion on the safety verification it needs to compare the displacement demand directly in a feasible and meaningful range of secant periods T_S. Based on a preliminary analysis, it is assumed that T_S can mostly vary from 0.2 to 0.5 seconds and, therefore, Figures 9 and 10 show the variation in displacement demand at such periods. Figure 9 shows that at the period $T_S = 0.2$ sec the displacement demand provided by the code ($d_d = 14$ mm) is larger than the one evaluated using the computed spectrum ($d_d = 10$ mm). On the contrary, Figure 10 shows that for higher periods T_S the difference in displacement demand is larger. In these cases, the computed spectrum leads to large values of d_d and the increments respect to the code ones are greater than 50%.

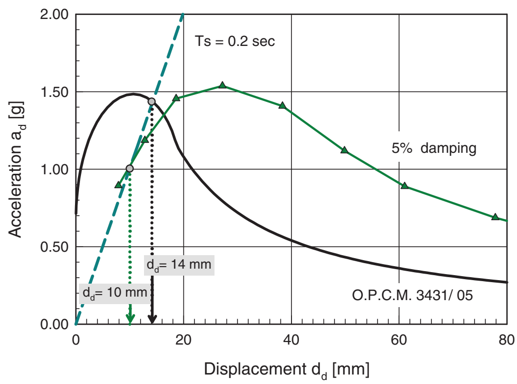

Figure 9. Displacement demand at $T_S = 0.20$ sec.

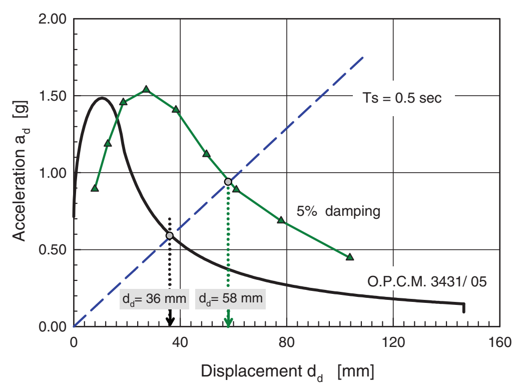

Figure 10. Displacement demand at $T_S = 0.50$ sec.

5 CONCLUSIONS

The dynamic response of a sample 2-story unreinforced masonry building has allowed to analyze the face-loaded walls seismic excitation and to assess the displacement demand. At 5% damping, it has been computed an acceleration amplification factor of the upper walls out-of-plane response with respect to the floor below about equal to 2, close to the conservative factor of 2.5 advised in literature. The comparison of the demand spectrum computed at 5% damping with the one provided by the OPCM 3431 seismic code has shown similar acceleration amplification, but variations in displacement demand also greater than 50%. Significantly larger differences in the face-loaded walls response has been instead appraised at 3% damping that is mostly representative for masonry building.

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