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EVALUATION OF THE SEISMIC DEMAND ON ACCELERATION-SENSITIVE NONSTRUCTURAL COMPONENTS IN RC FRAME STRUCTURES

Crescenzo PETRONE[1], Gennaro MAGLIULO[2], Maddalena CIMMINO[3], Gaetano MANFREDI[4]

ABSTRACT

A parametric study for the evaluation of the floor response spectra in RC moment-resisting frame structures, i.e. 1- 2- 3- 5- and 10-story structures, is conducted. The benchmark structures are designed according to Eurocode 8 provisions. A 0.25 g design ground acceleration a_g is considered. The horizontal elastic response spectrum is defined referring to a 5% damping ratio and to a 1.2 soil factor, i.e. soil type B. The structures are then subjected to a set of earthquakes that matches the design response spectrum. Time-history analyses are performed both on elastic and inelastic models of the benchmark structures, in order to evaluate the influence of the inelasticity on the definition of the floor response spectrum.

The floor response spectra are compared to Eurocode 8 formulation; some considerations on the peak floor acceleration and the maximum floor spectral ordinate are also included. It is concluded that the Eurocode formulation for the evaluation of the seismic demand on nonstructural components does not fit well the analytical results. The urgent need to include both the structural ductility demand and the influence of higher modes in code formulas for the evaluation of floor spectra is claimed.

INTRODUCTION

Nonstructural components (NSC) are those systems and components attached to the floors, roof and walls of a building or industrial facility that are not part of the main load-bearing structural system, but may also be subjected to large seismic actions (Villaverde, 1997). Recent earthquakes pointed out that nonstructural component damage gives the largest contribution to the earthquake economic loss. For instance, the damage to cladding panels was the most common damage in precast structures in 2012 Emilia earthquake (Magliulo et al., 2014). The economic impact could be much more severe if loss of inventory and downtime cost are considered: the cost related to nonstructural components failure could exceed the replacement cost of the building (Earthquake Engineering Research Institute (EERI), 1984). Moreover, the failure of nonstructural components may also threaten the life safety. These motivations encouraged several analytical (e.g., (Petrovčič and Kilar, 2012)) and experimental (e.g., (Badillo-Almaraz et al., 2007; Magliulo et al., 2012) among many others) studies on nonstructural components.

Nonstructural components should be subjected to a careful and rational seismic design, in order to reduce the economic loss and to avoid threats to the life safety, as well as what concerns the

[1] PhD, University of Naples Federico II, Naples, crescenzo.petrone@unina.it
[2] Assistant Professor, University of Naples Federico II, Naples, gmagliul@unina.it
[3] PhD Student, University of Naples Federico II, Naples, maddalena.cimmino@unina.it
[4] Full Professor, University of Naples Federico II, Naples, gamanfre@unina.it

structural elements. Nonstructural components are subjected to severe seismic actions due to the dynamic interaction with the primary system. The design of nonstructural components is based on the evaluation of the maximum inertia force, which is related to the floor spectral accelerations. Several research studies were conducted in the past concerning the evaluation of the floor acceleration and the floor response spectra.

Very limited studies were performed concerning the Eurocode 8 (CEN, 2004b) formulation for the evaluation of the floor spectral acceleration, according to which the seismic demand on a given nonstructural component is evaluated. Moreover, past studies were usually focused on steel buildings or wall structures. For this reason a set of benchmark RC frame structures are selected and designed according to Eurocode 8. Dynamic nonlinear analyses are performed on the benchmark structures in order to validate the Eurocode formulation; a set of accelerograms compatible with the Eurocode 8 design spectrum is defined. Dynamic analyses are performed both on elastic and inelastic models of the benchmark structures, in order to evaluate the influence of the inelasticity on the definition of the floor response spectrum. The floor response spectra are compared to Eurocode 8 formulation; some considerations on the peak floor acceleration and the maximum floor spectral acceleration are also given.

METHODOLOGY

A parametric study is conducted to investigate the seismic demand to which a light acceleration-sensitive nonstructural component may be subjected in multi-story RC frames. 2D frame structures are considered: they are representative of a tridimensional structure with a double symmetric plan and with three frames arranged in each direction (Fig. 1 and Fig. 2). Benchmark structures with different number of stories are considered: one-, two-, three-, five- and ten-story buildings, with a 3 m interstory height and two 5 m wide bays.

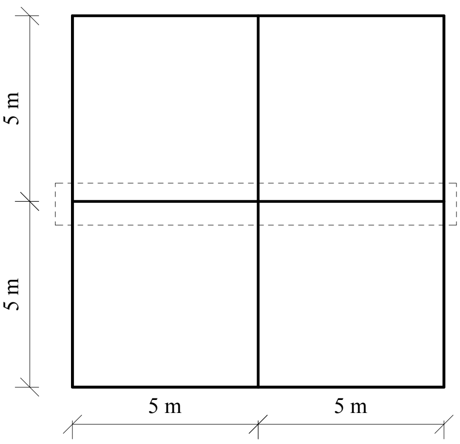

Figure 1. Plan view of the benchmark structures.

The benchmark structures are designed according to Eurocode 8 (EC8) (CEN, 2004b) provisions. A 0.25 g design ground acceleration a_g is considered. The horizontal elastic response spectrum is defined referring to a 5% damping ratio and to a 1.2 soil factor, i.e. soil type B.

The seismic design meets the ductility class "high" (DCH) requirements: the behavior factor is equal to 4.95 for one-story building and 5.85 for multi-story frames. The sizing of primary elements is strongly influenced, especially for tall structures, by the restricted value of normalized design axial force, i.e. the ratio between the average compressive stress and the concrete compression strength, which must not exceed 0.55. Moreover, the seismic detailing requirements in terms of longitudinal and transversal reinforcements provide an amount of reinforcement which is larger than the one strictly required by the design analysis. They produce high overstrength ratios which influence the structural response, as discussed in the following sections. A halved moment of inertia is considered for the primary elements during the design phase, according to EC8, in order to take into account the effect of cracking. The fundamental period of the benchmark structures, evaluated according to such a "reduced" flexural stiffness, are listed in Fig. 2.

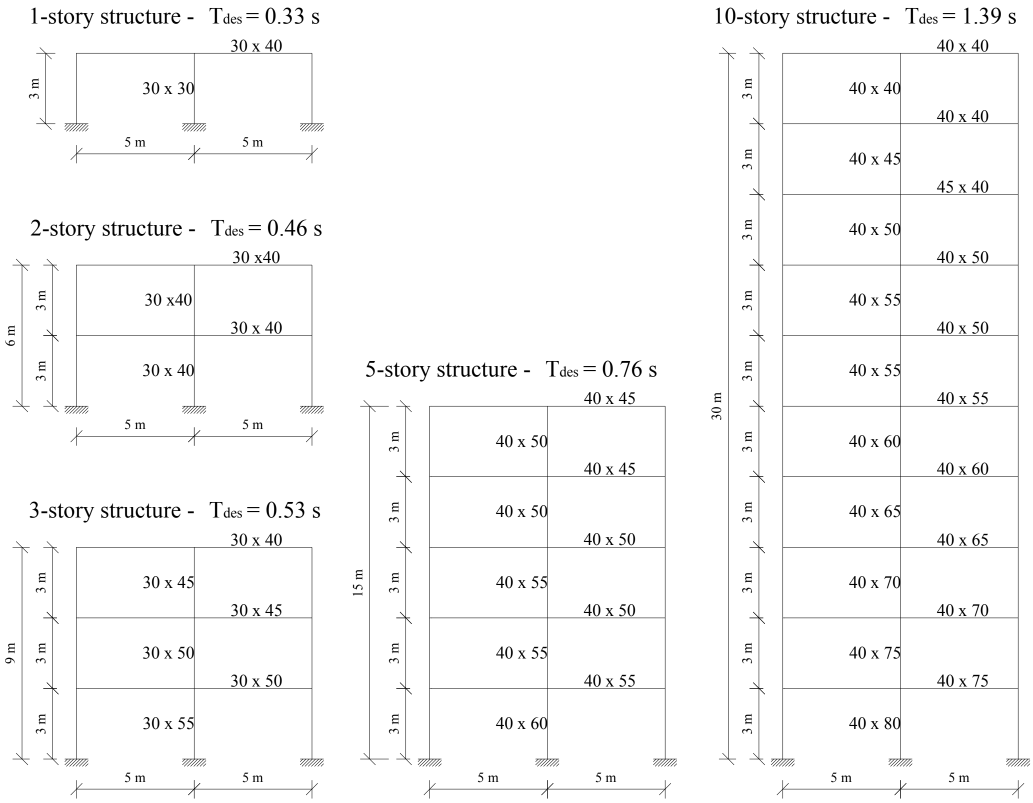

Figure 2. Lateral view of the considered building models and their design fundamental period (T_{des}). Dimensions of the cross sections are in cm.

Both elastic and inelastic structural responses are investigated. Dynamic analyses are carried out for a set of seven earthquake records, on both linear and nonlinear models. Rigid diaphragms are considered for each floor; a third of the seismic mass of the corresponding 3D building is assigned to a master joint at each floor. Analyses are performed using the OpenSees program (McKenna and Fenves, 2013).

The linear modeling provides that the primary elements are modeled as elastic beam-column elements with the gross moment of inertia. Concrete is modelled as an elastic material with a Modulus of Elasticity equal to 31476 MPa according to the C25/30 class concrete assumed during the design phase.

A lumped plasticity nonlinear approach is also considered: it is assumed that the primary elements have an elastic behavior and that any inelasticity source is lumped in plastic hinges at their ends. Moment–rotation envelopes in the plastic hinges are defined according to the formulation suggested by Haselton (2006). The nonlinear behavior of the plastic hinges is defined by peak-oriented hysteretic rules, which simulate the modified Ibarra-Medina-Krawinkler (Ibarra et al., 2005) deterioration model. The cracking point is neglected, i.e. the initial stiffness is equal to the yielding secant stiffness. In order to determine the moment-curvature diagrams, appropriate cross sections are defined for each element considering the actual geometry and steel reinforcement. The cross section is divided into fibers and a stress-strain relationship is defined for each fiber. Different constitutive laws are applied to three different kinds of fibers: unconfined concrete law is associated to cover fibers, confined concrete law is associated to core fibers and steel law is associated to the longitudinal reinforcement fibers. The stress–strain relationship proposed by Mander et al. (1988) is used both for unconfined and confined concrete. The B450C steel class is adopted with a bilinear with hardening relationship. The steel mechanical characteristics are calculated according to Eurocode 2 (Table C.1, “Properties of reinforcement”) (CEN, 2004a).

The structural response is investigated through time history analyses. Therefore, a suitable set of 7 accelerograms (Table 1) is provided, matching the design spectrum at the life safety limit state, i.e. 475 years return period earthquake, according to the EC8 recommendations (Maddaloni et al., 2012):

- the mean of zero-period spectral response acceleration values, that is equal to 3.69 m/s^2, is larger than the design value, i.e. $a_g \cdot S$;
- the mean elastic spectrum of the selected ground motions is larger than 90% of the design elastic response spectrum in the range of periods between $0.2T_{1,min}$ and $2T_{1,max}$, where $T_{1,min}$ and $T_{1,max}$ are, respectively, the minimum and the maximum fundamental period of the benchmark 2D structures (Fig. 3).

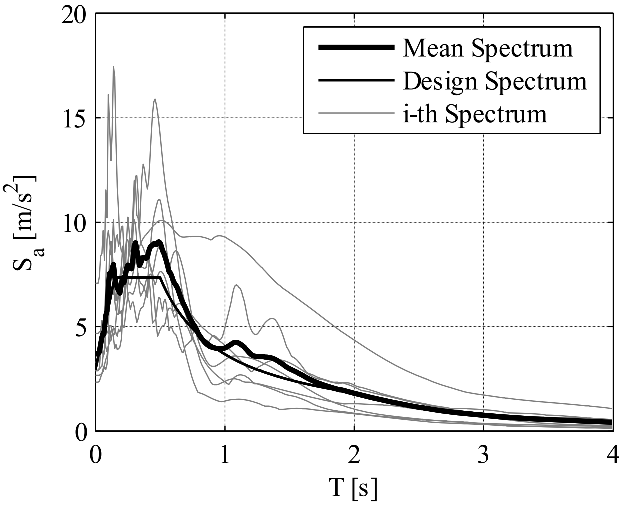

Figure 3. Comparison between the mean acceleration response spectrum of the adopted set of accelerograms and the design spectrum according to EC8.

Table 1. Waveform ID, earthquake ID (Eqk ID) and name, date, moment magnitude (MW), epicentral distance (R), horizontal direction (Dir.) and peak ground acceleration (PGA) of the accelerograms selected for dynamic analyses (Ambraseys et al., 2002).

Waveform	Eqk ID	Earthquake Name	Date	M_W [-]	R [km]	Dir.	PGA [m/s^2]
146	65	Friuli (aftershock)	15/09/1976	6.0	14	y	3.296
197	93	Montenegro	15/04/1979	6.9	24	x	2.880
413	192	Kalamata	13/09/1986	5.9	10	y	2.910
414	192	Kalamata	13/09/1986	5.9	11	x	2.354
414	192	Kalamata	13/09/1986	5.9	11	y	2.670
4673	1635	South Iceland	17/06/2000	6.5	15	y	4.677
6334	2142	South Iceland (aftershock)	21/06/2000	6.4	11	y	7.070

RESULTS AND DISCUSSION

Dynamic analyses on both elastic and inelastic models are performed and the horizontal acceleration time-histories at different levels are recorded for each selected accelerogram. A lumped plasticity approach is adopted in the inelastic models. Floor response spectra are obtained for each floor accelerogram with a 5% damping ratio and a mean response spectrum is plotted for each floor (Fig. 4). These spectra provide the acceleration demand of nonstructural components that are connected to the floor and exhibit a fundamental period T. Fig. 4a shows the mean floor response spectra, evaluated on both the elastic (dotted line) and inelastic (solid lines) models for the 5-story structure.

Due to the dynamic interaction, the primary structure modifies the frequency content of the earthquake so that the floor accelerogram, amplified with respect to the base accelerogram, has a large frequency content for periods close to the vibration periods of the elastic model. If the nonstructural component period corresponds to one of the vibration periods of the structure, a double-resonance phenomenon occurs; the floor response spectra exhibit peaks which may exceed five times the acceleration of gravity, i.e. about 20 times the base acceleration, at the top floor of the structure.

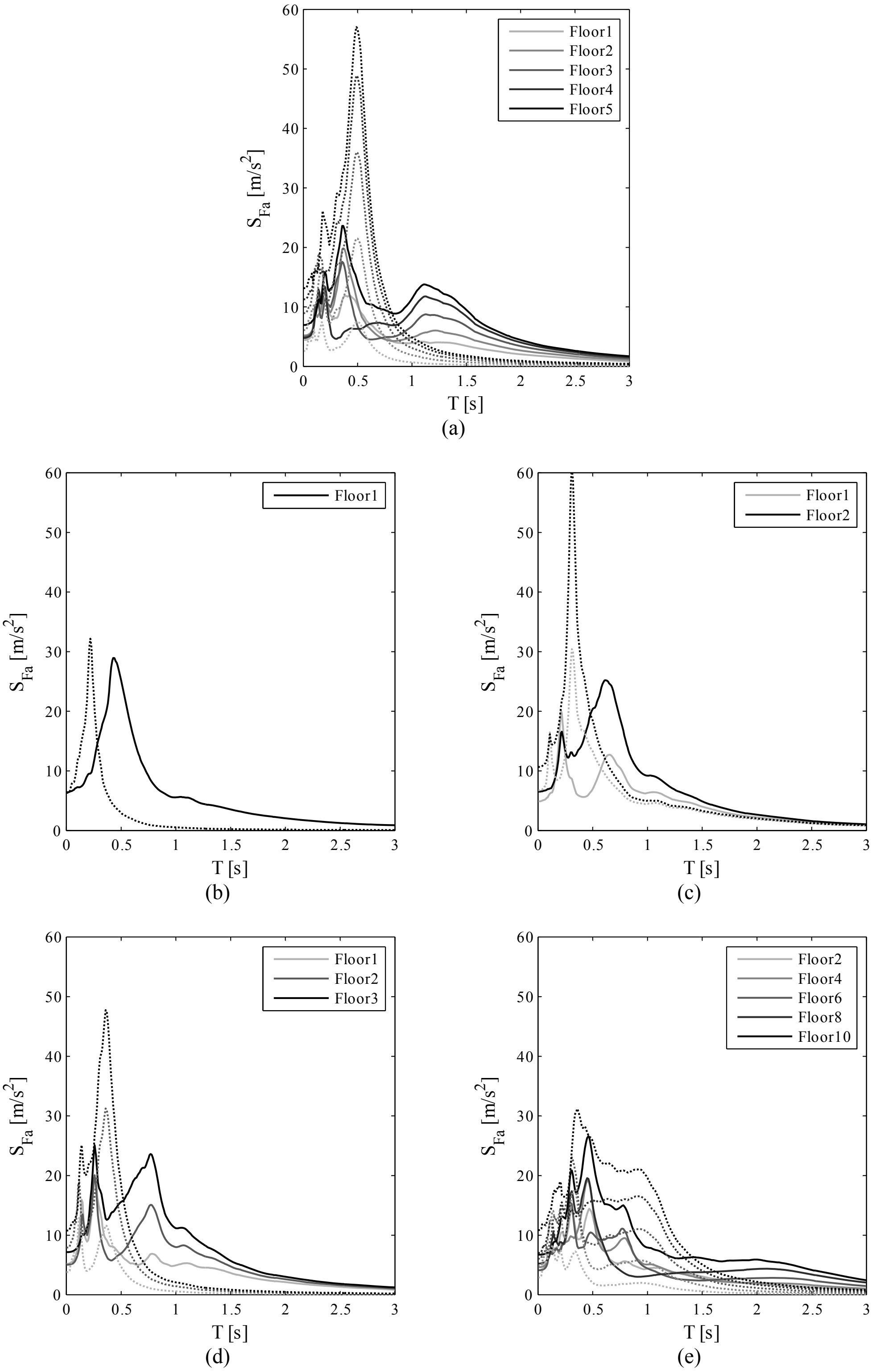

Figure 4. Floor response spectra of the (a) 1-story, (b) 2-story, (c) 3-story and (d) 10-story structures evaluated on both the elastic (dotted line) and inelastic models (solid line).

The inelastic floor response spectra (solid lines in Fig. 4), show that the curves exhibit peaks at periods, i.e. $T_{1,nl\text{-}eff}$ and $T_{2,nl\text{-}eff}$, much larger than the elastic ones, due to the different initial stiffness of the two models.

Fig. 4 shows the comparison between elastic and inelastic floor response spectra for the remaining structures. The following comments can be drawn:

- a significant period elongation is exhibited, comparing the peak related to the first structural mode of the elastic model with the inelastic one;
- the comparison of the peak related to the first structural mode of the elastic model with the inelastic one also shows a substantial reduction of the peak spectral ordinate: the maximum spectral values of the inelastic model are less than 3 g for the different structures. The reduction is caused by both the period elongation phenomenon and the ductility demand experienced by the structure;
- higher modes effect is significant in the 10-story structure. Moreover, the peak spectral values associated with the higher modes are slightly reduced in the inelastic model. At lower stories, the spectral values associated with higher modes can be even larger than the elastic ones, as also pointed out by (Chaudhuri and Villaverde, 2008) in a research study on steel moment-resisting frames. This phenomenon confirms that the higher mode influence becomes more significant in the inelastic range (Fischinger et al., 2011; Rejec et al., 2012).

It can be observed that the inelastic spectral acceleration demand reduction is significantly far from the assumed behavior factor, due to the large structural overstrength. It is also confirmed that the energy dissipation is mostly related to the first mode; indeed the peak value associated to $T_{2,nl\text{-}eff}$ may exceed the peak value associated to $T_{1,nl\text{-}eff}$. It can be concluded that in case inelastic models are considered, higher modes give a larger contribution to the definition of the floor spectral ordinates.

The ratio between peak floor acceleration (PFA) and peak ground acceleration (PGA) is plotted versus the relative height in Fig. 5 for the benchmark structures, in order to study the floor acceleration magnification with height. The PFA over PGA trend with the relative structural height is shown for both elastic and inelastic models.

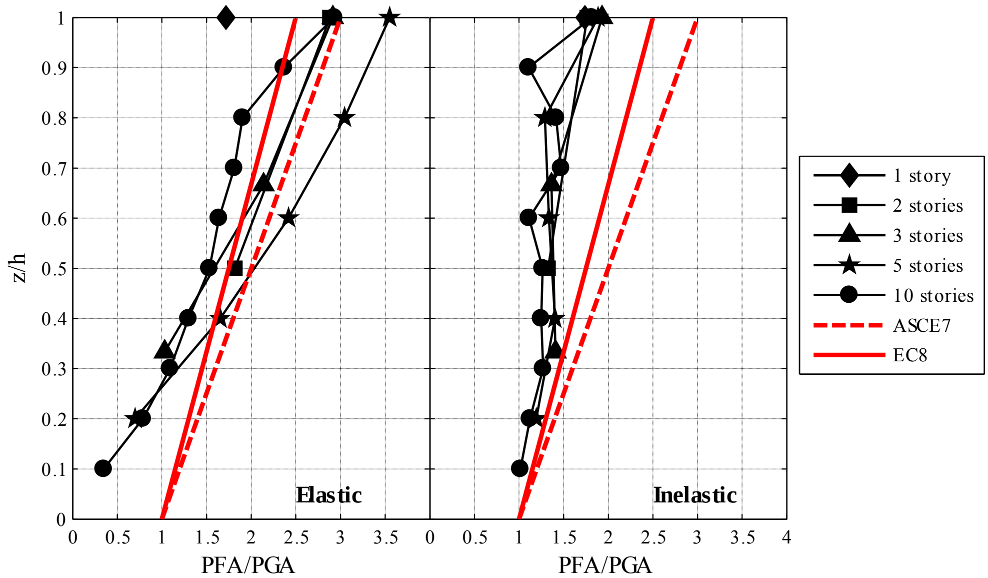

Figure 5. Ratio between peak floor acceleration and peak ground acceleration, versus the relative height (z/h) for the different considered structures compared to the provisions included in ASCE7 and EC8.

The elastic model diagrams, which represent the average response of each structure, show an almost linear trend and they reach values of PFA/PGA close to three at the top floor. At the same relative height, the values of the ratio PFA/PGA are larger for structures with a larger number of floors, except for the tallest structure. At the lower stories of tall structures, PFA values are smaller than PGA values.

The inelastic model diagrams also show a linear trend. In this case the amplification is smaller than the one of the elastic models: the PFA/PGA values are always greater than one and they reach the maximum value, close to 2, at the top story. As pointed out by Wieser et al. (2013) and Ray-Chaudhuri and Hutchinson (2011), the yielding of the structure and the period elongation cause a significant reduction of the peak floor accelerations.

Both the elastic and inelastic trends are compared to ASCE7 (American Society of Civil Engineers, 2010) and Eurocode 8 provisions (Fig. 5). Such a comparison shows that both the ASCE7 and EC8 provisions are safe-sided for the inelastic diagrams, which are the most realistic ones. Finally, a linear trend that goes form 1 at the base to 2 at the top would better fit the outcomes of the nonlinear analyses.

The ratio between the maximum floor spectral acceleration and the PFA, i.e. a_p, is plotted versus the relative floor height for each floor of the analyzed structures in order to study the floor acceleration magnification on the component (Fig. 6). This ratio represents the amplification of the floor acceleration demand for a nonstructural component that is in tune with the primary structure.

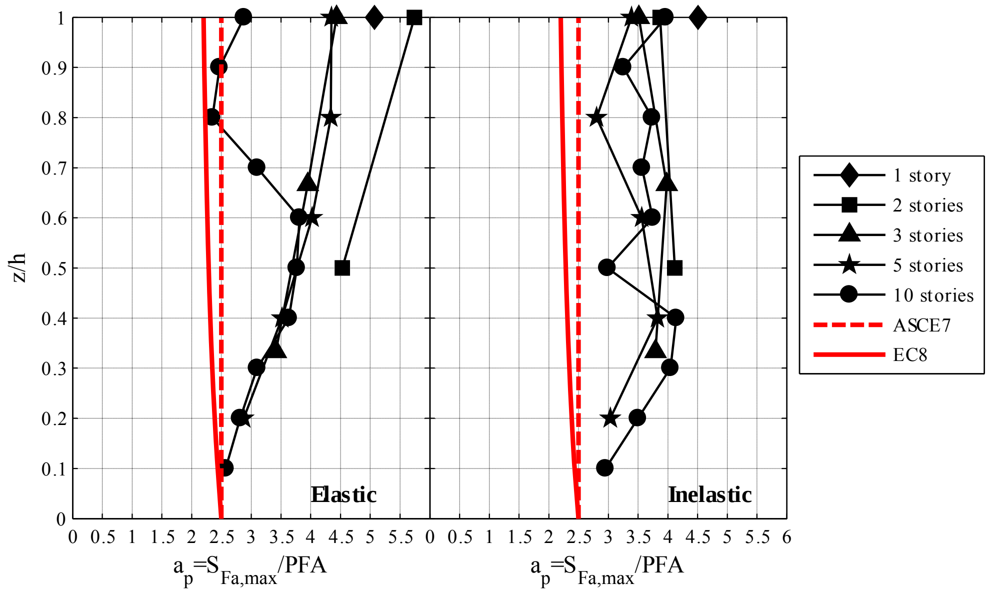

Figure 6. Floor acceleration magnification on nonstructural components.

A significant underestimation of the a_p values in the current building codes is clearly evidenced, confirming the results included in Medina et al. (2006).

In order to take into account the realistic behavior of the primary structures, inelastic floor spectra should be considered. These diagrams are compared with the ones obtained by Eurocode 8 formulation (CEN, 2004b) for the evaluation of the floor response spectrum acceleration S_a acting on a nonstructural component:

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5 \right] \cdot g \geq \alpha \cdot S \cdot g \tag{1}$$

where:

- α is the ratio between the ground acceleration and the gravity acceleration g;
- S is a soil amplification factor;
- z/H is the relative structural height at which the component is installed;
- T_a is the nonstructural component period;
- T_1 is the fundamental period of the primary structure, assumed during the design phase.

The design floor response spectrum depends on the ratio between the nonstructural component period and the structural period, as well as by the level at which the nonstructural component is installed. The formulation does not identify separately the different factors that affect the floor spectral accelerations, as provided, instead, by ASCE7 formulation (American Society of Civil Engineers, 2010). However, it implicitly assumes that the PFA linearly ranges from PGA at the base to 2.5 times

PGA at the top of the structure, whereas a_p linearly ranges from 2.5 at the base to 2.2 at the top of the structure. Moreover, the maximum S_a value is equal to 5.5 times the PGA, i.e. the spectral acceleration acting on a component placed at the top floor which is in tune with the structure.

For different values of T_a and for each floor, the Eurocode formulation provides a curve that shows the maximum value for T_a equal to T_1. In Fig. 7 both inelastic floor spectra and design Eurocode 8 floor spectra are plotted for the benchmark structures.

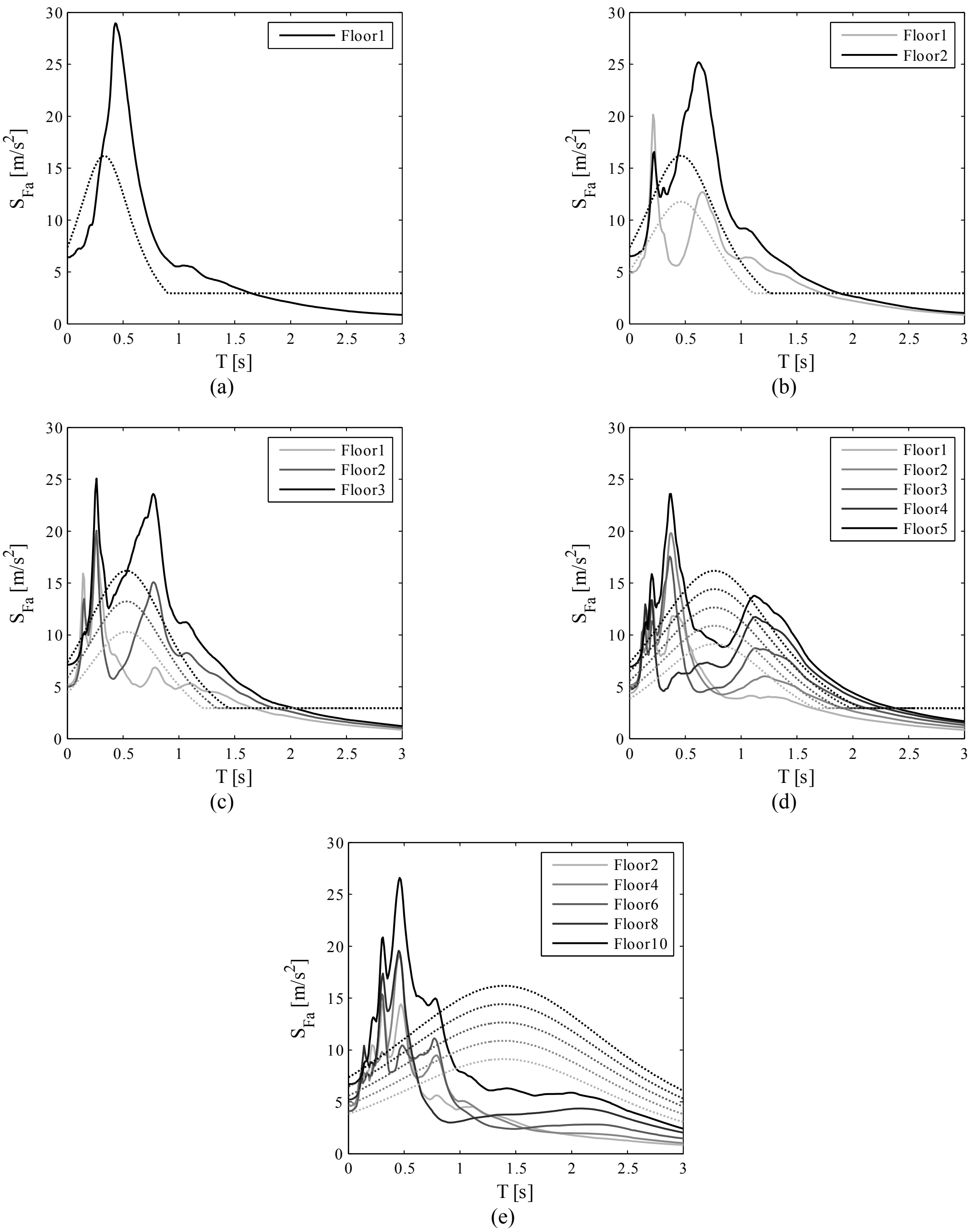

Figure 7. Comparison between effective inelastic floor response spectra (solid lines) and floor response spectra evaluated according to Eurocode 8 (dashed lines) for the (a) 1-story, (b) 2-story, (c) 3-story, (d) 5-story and (e) 10-story structures.

This comparison underlines that Eurocode formulation underestimates the maximum floor acceleration demand for a wide range of nonstructural component periods, whereas it may overestimate the acceleration demand on nonstructural components with a period close to the design period of the structure. Moreover the peak of the Eurocode curve is reached at the design period, which is lower than the effective one for all inelastic models.

Eurocode formulation does not take into account higher modes: a significant underestimation is recorded in the range of periods close to the higher modes periods of vibration. The effective floor spectrum acceleration can be significantly underestimated, especially for tall structures, e.g. the 10-story structure in Fig. 7e, in which higher modes are predominant.

The approach proposed by Fathali and Lizundia (2011), who considered a constant floor response spectral acceleration in a wide range of periods, could be adopted. It would allow removing both the issue related to the uncertainty in the definition of the structural fundamental period and the non-inclusion of the higher modes effects in the floor response spectra.

The effect of the higher modes in the floor response spectra is clearly influenced by the nonlinear excursion that the structure experiences during the earthquake motion. However, both European and US codes do not explicitly take into account the reduction of the floor response spectra due to the nonlinear behavior of structures.

It would be better to explicitly include the ductility demand level in code formulas for the evaluation of floor spectra, as mentioned in Medina et al. (2006). The ductility level experienced by a structure, subjected to the design earthquake motion, is strongly influenced by the structural overstrength, which is in turn related to the prescriptions included in the code itself. Hence, the definition of a formula that includes the structural ductility demand level would certainly be code-dependent.

The structural overstrength of a given building cannot be easily assessed during the design phase. Moreover, it is related to many factors, e.g. the bay width, the presence of irregularities in plan or elevation and the design peak ground acceleration among others, that are not considered in this research study. Hence, a very wide parametric study is required to define a code formula that explicitly takes into account the ductility level that the structure experiences.

Alternatively, the code formulation for the evaluation of the nonstructural component demand could be referred to the elastic floor response spectrum. This approach would be too conservative, i.e. acceleration on components could be up to 20 times the acceleration at the base; moreover, it would not reflect the realistic behavior of the structure in terms of both the fundamental period and the attitude of the structure to dissipate energy.

Finally, it is concluded that the Eurocode could not adequately address the design of acceleration-sensitive nonstructural components, as pointed out by Velasquez et al. (2012) who analyzed the floor time-history accelerations recorded during a shake-table test campaign.

CONCLUSIONS

A parametric study for the evaluation of the floor response spectra in RC frame structures, i.e. 1- 2- 3- 5- and 10-story structures, is conducted. The structures, designed according to Eurocode 8, are subjected to a set of earthquakes that are compatible with the design response spectrum.

Preliminary nonlinear static analyses show that the benchmark structures are characterized by a significant overstrength, due to some geometric limitations included in the Eurocode 8.

Time-history analyses are performed both on elastic and inelastic models of the benchmark structures. The comparison between elastic and inelastic floor response spectra indicates a substantial reduction of the peak spectral ordinate associated to the first mode; moreover, the peak occurs at a longer period due to the period elongation phenomenon. The peak spectral values associated with the higher modes are only slightly reduced in the inelastic model. At lower stories, the spectral values associated to higher modes can be even larger than the elastic ones.

The ratio between PFA and PGA trend with the relative structural height shows that both the ASCE7 and EC8 provisions are safe-sided. A linear trend that goes form 1 at the base of the structure to 2 at the top would better fit the outcomes of the analyses. The yielding of the structure gives a significant contribution to the peak floor acceleration reduction.

The component amplification, i.e. the ratio between the maximum floor spectral value and the PFA, is almost constant with the height for both elastic and inelastic models. An unsafe-sided estimation of the ap values in the actual building codes is clearly evidenced: the component amplification ap values are significantly greater than 2.5, which is the value recommended by ASCE7 and EC8, and close to 4.5.

It is found that Eurocode formulation for the evaluation of the seismic demand on nonstructural components does not fit well the results of the analyses. It underestimates the maximum floor acceleration demand for a wide range of nonstructural component periods, whereas it overestimates the acceleration demand on nonstructural components with a period close to the design period of the structure. The underestimation is significant for nonstructural component periods close to the higher modes structural periods, since the Eurocode formulation does not include higher modes effect.

The urgent need to include the structural ductility demand in code formulas for the evaluation of floor spectra is claimed. However, it is underlined that the ductility level is influenced by the structural overstrength, which is in turn related to the prescriptions included in the reference building code. Hence, the definition of a formula that includes the structural ductility demand level would certainly be code-dependent.

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