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Seismic demand on nonstructural components due to frequent earthquakes

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Abstract

The seismic performance of nonstructural components is nowadays recognized as one of the most critical issues related to the performance-based earthquake engineering. In order to obtain an adequate seismic design of nonstructural components, a consistent estimation of the seismic demand is essential. Recent studies have shown that current international building codes provide unreliable approaches to the evaluation of acceleration floor spectra. This paper examines the seismic demand on light acceleration-sensitive nonstructural components caused by frequent earthquake through an accurate parametric study. In this work, a number of reinforced concrete frame structures are designed according to Eurocode 8 and subjected to a set of frequent earthquakes, i.e. 63% probability of exceedance in 50 years. In order to assess the reliability of Eurocode 8 formulations, dynamic nonlinear analysis have been performed on a set of reference structures. It is found that Eurocode 8 significantly underestimates the acceleration demand for a wide range of periods. An alternative method to estimate floor acceleration demands in multi-story buildings is presented through the proposal of a new formulation that is able to better predict the actual acceleration demand.

Keywords: Nonstructural components, Seismic demand, Eurocode 8, Seismic performance, Floor spectra.

1. Introduction

Nonstructural components are those systems and elements housed or attached to the floors, roof and walls of a building or industrial facility that are not part of the main structural system but may also be subjected to a large seismic force [1]. Nonstructural components in a building can be classified into different categories for seismic design purposes [2]: drift-sensitive components, such as windows and partitions, and acceleration-sensitive components, which are the subject of interests in this paper, like parapets and suspended ceilings. However, most of the components are categorized as both acceleration-sensitive and drift-sensitive components [3], such as fire sprinklers and heavy infill walls. For this reason considering both forces and displacements becomes crucial for all nonstructural components.

Experiences from past earthquakes has highlighted that the seismic performance of nonstructural components has become a key issue in the framework of the performance-based earthquake engineering.

- In most cases nonstructural components are highly vulnerable to a relatively low level of earthquake intensity. Consequently, they exhibit significant damage even for low seismic demand. This may critically compromise the performance of vital facilities, such as hospitals, emergency command centers and all the facilities that should remain operative immediately after the seismic event;
- The economic loss due to nonstructural components generally exceeds the one due to structural elements [3]. This statement is especially valid for commercial structures [2], in which the cost connected to the loss of inventory and to the downtime is not negligible;
- The damage to nonstructural components can result not only in major economic loss but also can pose a real threat to life safety.

In recent years a significant effort has been made through many studies for assessing the seismic demand on nonstructural components as well as their seismic capacity [4]. Studies on floor response spectra were conducted by Lin and Mahin [5] and Sewel *et al*. [6]: they inquired the effects of the primary structure's non-linearity on floor response spectra. Rodriguez et al. [7] estimated the earthquake-induced floor horizontal accelerations in cantilever wall buildings built with rigid diaphragms. Sullivan et al. [8] verified the reliability of Eurocode 8 formulation for the definition of floor response spectra on both 8-story and 20-story cantilever RC wall structure. They also provided an alternative method to define floor spectra on single degree of freedom supporting structures and encouraged further researches on the subject.

This paper summarizes the outcome of a study on the seismic demand caused by frequent earthquakes on light acceleration-sensitive components presented in [9]. The study is mainly encouraged by the questionable approach proposed by the current European code, where the displacement-sensitive nonstructural components are verified at the damage level limit state, while the acceleration-sensitive nonstructural components are verified at the life safety limit state. Also the significant nonstructural damage exhibited after low intensity earthquakes motivated such a study. A set of RC frame structures with a different number of stories are designed according to Eurocode 8. Nonlinear dynamic analyses are performed in order to compare the Eurocode floor spectra with those derived from the analyses. Some observations on the peak floor acceleration and on the component amplification have been made. Finally, a novel formulation for the definition of the design floor response spectra is proposed for an easy implementation in future building codes.

2. Methodology

Nonlinear dynamic analyses are performed on one, two, three, five and ten-story high buildings to observe the effects of different variables on the floor spectra. The structures are designed according to Eurocode 8 [10] and characterized by a 3 m interstorey height and two 5 m wide bays in each direction. A 0.25 g design ground acceleration on stiff soil a_g is considered. The horizontal elastic response spectrum is defined referring to a 5% damping ratio and a 1.2 soil factor, i.e. soil type B. The seismic design meets the ductility class "high" (DCH) requirements. The sizing of the column cross sections is strongly influenced by the restricted value of normalized design axial force; indeed, the average compressive stress over the concrete compression strength must not exceed 0.55. A halved moment of inertia is considered for the primary elements, according to Eurocode 8, in order to take into account the effect of cracking. The fundamental periods of the benchmark structures are listed in Fig. 1. The mass is lumped at each story considering the actual column and beam cross sections. The mass per square meter ranges from 0.87 t/m^2 at the 1st floor of the 1-story structure to 1.39 t/m^2 at the 1st floor of the 10-story structure. Further details are included in [9].

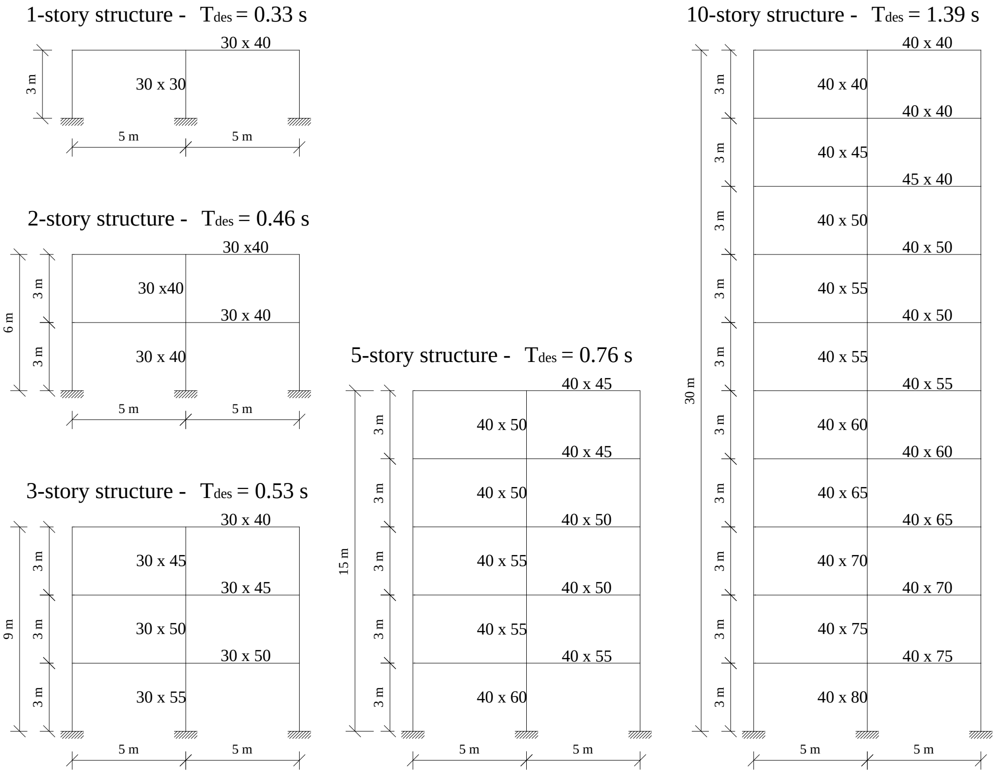

Fig. 1 - Lateral view of the considered building models and their design fundamental period (T_{des}). The dimensions of the cross sections are in cm (adapted from [9])

The analyses are carried out using the Opensees software [11] 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. The linear modeling provided that the primary elements are modeled as elastic beam-column with the gross moment of inertia. Concrete is modeled as an elastic material with a Modulus of Elasticity equal to 31476 MPa according to the C25/30 concrete class assumed during the design phase.

The nonlinear model of the structures is represented by a distributed plasticity approach in order to allow the investigation of pre- and post-cracking behavior of the elements. The primary elements are modeled as nonlinear force-based elements. At each element appropriate cross sections are assigned. The cross section is divided into fibers and a stress-strain relationship is defined for each of them considering different constitutive laws to three different kinds of fibers: unconfined concrete associated to the cover fibers, confined concrete associated to the core fibers, steel associated to the longitudinal reinforcement fibers. The stress–strain relationship for both unconfined and confined concrete are evaluated according to Mander et al. [12]. The tensile concrete strength is also considered. The class B450C for the steel is used and a bilinear with hardening relationship is adopted.

In order to perform nonlinear dynamic analysis on the benchmark structures, a set of 7 accelerograms representative of the frequent earthquake ground motion have been chosen matching the design spectrum corresponding to a frequent earthquake. In order to define this spectrum, it was necessary to refer to the Italian Building Code, according to which a frequent earthquake is characterized by a 63% probability of exceedance in 50 years, i.e. by a 50-year return period; for the selected location the peak ground acceleration on stiff soil, characterized by a 63% probability of exceedance in 50 years, is equal to 0.078 g. The 50- year return period spectrum is shown in solid thin line in Fig. 2. Further details are included in [9].

3. Results and Discussion

As a result of the dynamic analyses performed on both elastic and inelastic models, floor response spectra with a 5% damping ratio at a given story are computed as the mean of the floor response spectra evaluated subjecting the structure to the 7 selected accelerograms. These spectra provided the acceleration demand on the nonstructural component located at that floor and characterized by a natural period T. Fig. 3 compares the mean floor response spectra resulting from elastic and inelastic models.

In both cases it can be observed a significant amplification of acceleration near the fundamental period of the building. This phenomenon is due to the filtering effect of the primary structure that modifies the frequency content of the earthquake at different stories. If the nonstructural component period corresponds to one of the vibration periods of the structure, a double-resonance phenomenon occurs. The inelastic floor response spectra obtained showed that the curves exhibit peaks at periods much larger than the elastic ones, due to the different initial stiffness of the two models. Moreover, high modes effect is significant for tall buildings, whose floor spectral accelerations, associated to the higher modes, are greater than the ones corresponding to the first mode. This effect is more noticeable for inelastic models, in which the peaks corresponding to the higher modes are slightly reduced. It can be observed that the reduction of the spectral ordinates is significant despite the structural elements are not yielded; the nonlinearity due to the cracking of the elements could be significantly beneficial in terms of the seismic demand on nonstructural components, especially for long-period nonstructural components in tall structures.

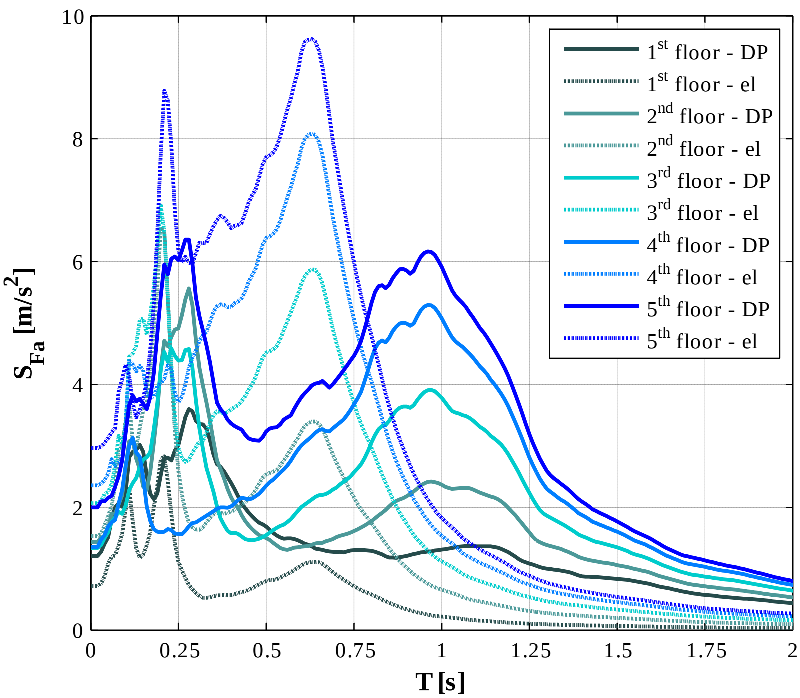

Fig. 2 - Floor response spectra in elastic (dotted lines) and distributed plasticity (DP) inelastic (solid lines) models for 5-story structure

The ratio between the peak floor acceleration (PFA) and the peak ground acceleration (PGA) is plotted in Fig. 4 versus the relative height, in order to study the floor acceleration magnification with height. Both elastic and inelastic models' diagrams show almost linear trends but in the inelastic case the amplification is smaller because of the cracking of the primary elements. The trends are also compared with EC8 and ASCE 7 [13] provisions that define a linear trend that goes from 1 at the base of the structure to 2.5 and 3 at the top for EC8 and ASCE 7 respectively. Referring to Fig. 4, it is clear that the approximation provided within ASCE 7 and EC8 provide a conservative amplification with the height for all structures.

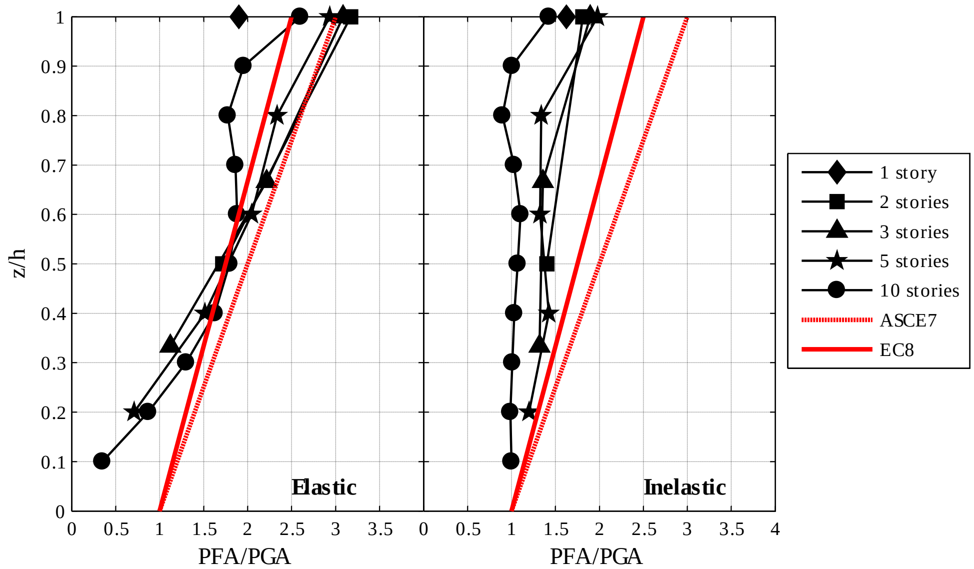

Fig. 3 - Ratio between peak floor acceleration (PFA) and peak ground acceleration (PGA), versus the relative height (z/h) compared to the provisions included in ASCE7 and EC8

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. Only small differences are observed between the elastic and the inelastic models. The component acceleration magnifications in tall buildings are generally smaller than the ones in short structures. Comparing the observed trends with the provisions included in ASCE7 and EC8 (Fig. 5), a significant underestimation of the code provisions is highlighted.

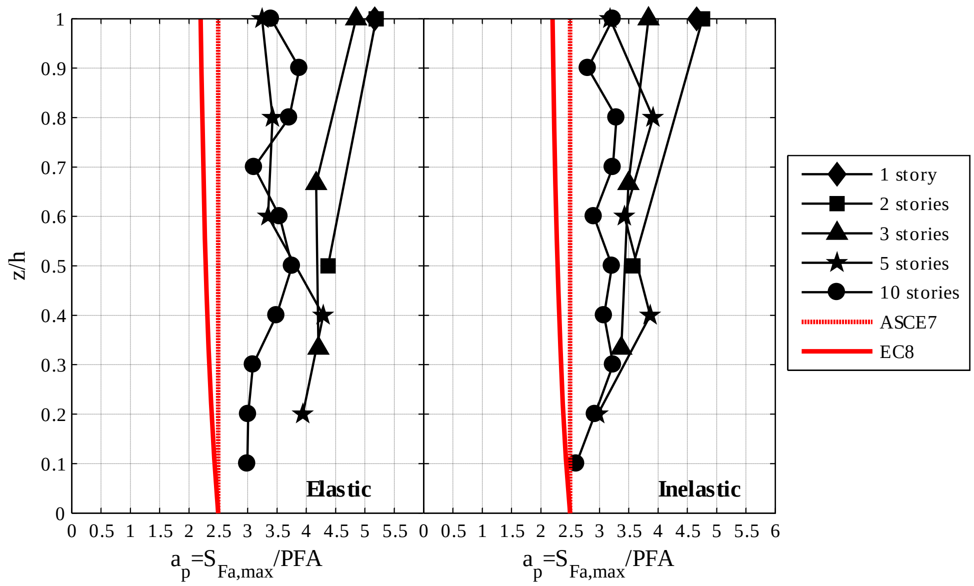

Fig. 4 - Floor acceleration magnification on nonstructural components versus the relative height (z/h) compared to the provisions included in ASCE7 and EC8

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

$$S_{Fa,EC8}\left(T\right)=\alpha\cdot S\cdot\left[\frac{3\cdot\left(1+z/H\right)}{1+\left(1-T/T_1\right)^2}-0.5\right]\cdot g\geq\alpha\cdot S\cdot g \tag{1}$$

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 Eurocode 8 floor response spectra have been compared to the floor spectra resulting from the inelastic models. This comparison highlights that Eurocode 8 formulation underestimates the acceleration demand for a wide range of periods, especially for periods close to the structural natural periods. As it is clear from Fig. 6, Eurocode floor spectra give a good approximation, typically safe-sided, of the floor spectra for period sufficiently larger than the fundamental period of the structure.

Moreover, Eurocode floor spectra do not take into account the higher mode effects: a remarkable underestimation is recorded in the range of periods close to the higher mode periods of vibration. This leads to a significant underestimation of the demand, especially for tall buildings, i.e. the 5- and the 10-story structures, in which the higher modes are predominant. An urgent need to include higher modes in the code formulation is clearly evidenced.

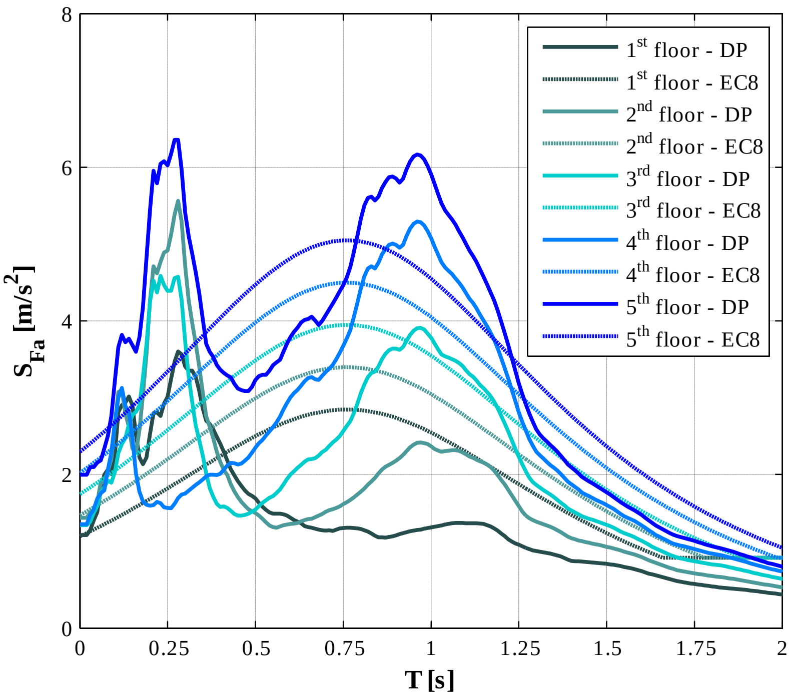

Fig. 5 – Floor response spectra (solid lines) on distributed plasticity (DP) inelastic models compared to Eurocode 8 floor spectra (dashed lines) for 5-story structure

The previous sections clearly pointed out the unreliability of the Eurocode provisions for the evaluation of the seismic demand on acceleration-sensitive nonstructural components. In this Section a novel formulation, based on the Eurocode ones, is proposed.

- A three-branch floor response spectrum is defined The first and the third branches have a shape similar to the Eurocode 8 floor spectrum. The definition of the flat branch no. 2 allows considering the peaks corresponding to both the first and the higher modes of the primary structure; it is also capable to include the uncertainty in the evaluation of the structural periods.

- The formula included in EC8 is slightly modified in order to directly distinguish the different terms, i.e. ground acceleration, floor amplification and component amplification, that influence the definition of the floor response spectrum (S_{Fa}).
- The PFA over PGA ratio trend is modified and goes from 1 at the base of the to 2 at the top.
- The amplification factor a_p is increased up to 5 for short buildings and is reduced for tall ones.

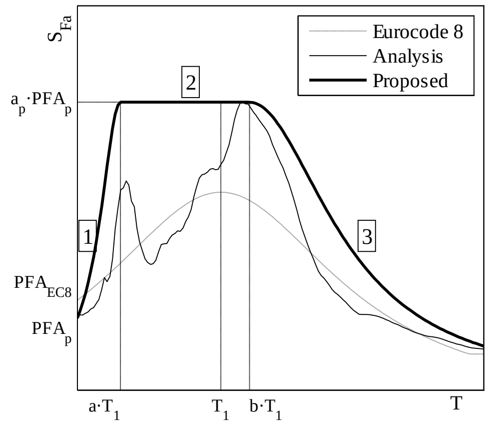

Fig. 6 - Proposed floor spectral shape compared to the Eurocode 8 floor spectral shape and to a typical analytical floor spectrum (adapted from [9])

The proposed response spectrum is defined by the following expressions:

$$S_{Fa,proposed}(T)=\begin{cases}\alpha\cdot S\cdot g\cdot(1+z/H)\cdot\left[\dfrac{a_p}{1+(a_p-1)\left(1-T/_{a\cdot T_1}\right)^2}\right]\geq\alpha\cdot S\cdot g & \text{for } T<a\cdot T_1\\ \alpha\cdot S\cdot g\cdot(1+z/H)\cdot a_p & \text{for a}\cdot T_1<T<b\cdot T_1\\ \alpha\cdot S\cdot g\cdot(1+z/H)\cdot\left[\dfrac{a_p}{1+(a_p-1)\left(1-T/_{b\cdot T_1}\right)^2}\right]\geq\alpha\cdot S\cdot g & \text{for } T>b\cdot T_1\end{cases} \tag{2}$$

The parameters a, b and a_p are defined according to the fundamental period of the structure T_1 and based on the indications included in [14]and they are then calibrated to match analytical results. Referring to Fig. 8, the floor spectra evaluated according to the proposed formulation are compared to the analytical floor spectra evaluated on the inelastic models. The proposed floor spectra are typically safe-sided with respect to the analytical results and include the peaks related to the structural higher modes; moreover, it takes into account the reduction of the seismic demand on very flexible nonstructural components.

The proposed formulation produces a conservative acceleration demand for a wide range of periods, especially for periods close to the fundamental period of the structure; however, this overestimation is beneficial since could remedy the uncertainty related to the estimation of the structural period due to, for instance, the presence of stiff infill walls and partition walls [15], as well as the uncertainty in the estimation of the nonstructural component.

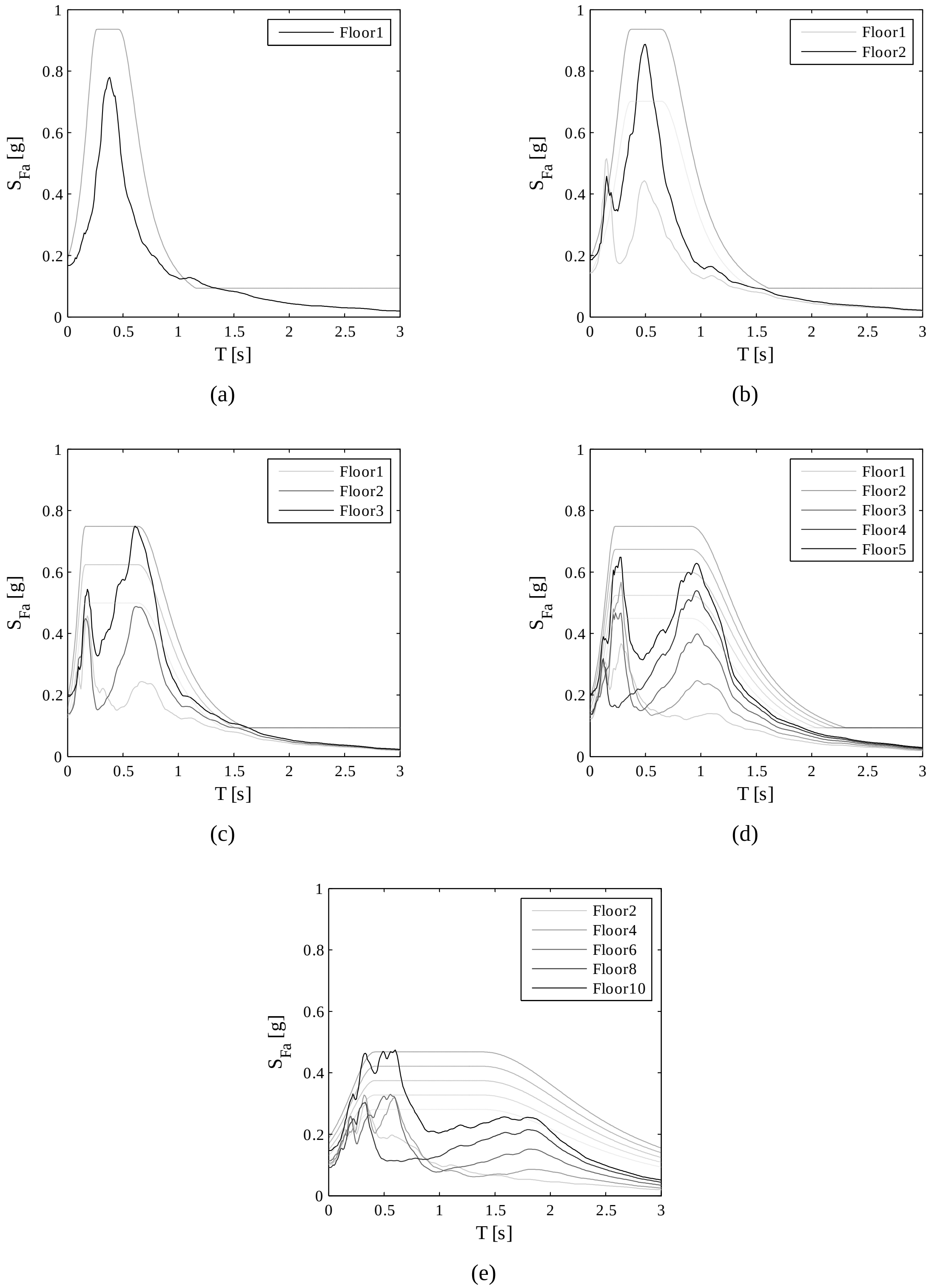

Fig. 7 -Floor response spectra (solid lines) on inelastic models compared to the proposed floor spectra according to the formulation (0) (dashed lines) for the (a) 1-story, (b) 2-story, (c) 3-story, (d) 5-story and (e) 10-story structures (adapted from [9])

4. Conclusions

A parametric study for the evaluation of the floor response spectra in European RC frame structures, i.e. 1- 2- 3- 5- and 10-story structures, was conducted. The structures, designed according to Eurocode 8, were subjected to a set of 7 accelerograms that are compatible with the design frequent seismic input, defined through a detailed analysis of the limit states definition according to the actual European and Italian building codes.

Nonlinear dynamic analyses were performed on both elastic and inelastic models of the benchmark structures. The comparison between elastic and inelastic floor response spectra showed a substantial reduction of the peak spectral ordinate associated to the first mode. Moreover, a period elongation phenomenon is clearly evidenced in floor spectra of the inelastic models; this was caused by the cracking of the primary elements. The nonlinearity due to the cracking also produces a reduction of the floor spectral ordinates associated to the first mode; it is rather less evident the peaks' reduction associated to the higher modes, and this could lead to an inaccurate prediction of floor spectra for tall buildings, where the accelerations associated to the higher modes are greater than those corresponding to the first mode.

The peak floor acceleration shows an almost linear trend with the structural relative height. The predictions included both in EC8 and ASCE 7 provide larger values of peak floor acceleration compared to the analysis results.

The peak component acceleration normalized to the peak floor acceleration, exhibits an almost constant trend with the structural relative height. Moreover, for tall structures the component amplification factor reduces. A significant unsafe-sided prediction of both EC8 and ASCE 7 provisions is shown. Comparing the floor spectra of inelastic models with the EC8 provisions, it is concluded that Eurocode typically underestimates the acceleration demand on nonstructural component for a wide range of periods. A significant underestimation is recorded in the range of periods close to the higher mode periods of vibration of the reference structures while a good approximation is provided for periods sufficiently larger than the fundamental period of the structure.

A novel formulation is then proposed for an implementation in the future building codes. The proposed formulation is able to envelope the floor spectral peaks due to the higher modes. Moreover, it produces a conservative acceleration demand for a wide range of periods, especially for periods close to the fundamental period of the structure; however, this overestimation is beneficial since could remedy the uncertainty related to the estimation of the structural and nonstructural period during the design phase.

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