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Seismic force demands on acceleration-sensitive nonstructural components: a state-of-the-art review

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Abstract: Nonstructural components (NSCs) are parts, elements, and subsystems that are not part of the primary load-bearing system of building structures but are subject to seismic loading. Damage to NSCs may disrupt the functionality of buildings and result in significant economic losses, injuries, and casualties. In past decades, extensive studies have been conducted on the seismic performance and seismic design methods of NSCs. As the input for the seismic design of NSCs, floor response spectra (FRS) have attracted the attention of researchers worldwide. This paper presents a state-of-the-art review of FRS. Different methods for generating FRS are summarized and compared with those in current seismic design codes. A detailed review of the parameters influencing the FRS is presented. These parameters include the characteristics of ground motion excitation, supporting building and NSCs. The floor acceleration response and the FRS obtained from experimental studies and field observations during earthquakes are also discussed. Three RC frames are used in a case study to compare the peak floor acceleration (PFA) and FRS calculated from time history analyses (THA) with that generated using current seismic design codes and different methods in the literature. Major knowledge gaps are identified, including uncertainties associated with developing FRS, FRS generation methods for different types of buildings, the need for comprehensive studies on absolute acceleration, relative velocity, and relative displacement FRS, and the calibration of FRS by field observations during earthquakes.

Keywords: nonstructural components; peak floor acceleration; floor response spectra; component amplification factor

1 Introduction

Nonstructural components (NSCs) are parts, elements, and subsystems that are not part of the primary load-bearing system of building structures but are subject to the same seismic loading environment. Damage to NSCs may disrupt the functionality of buildings and result in significant economic losses, injuries, and casualties. (Taghavi and Miranda, 2003a; Devin and Fanning, 2019). Damage to NSCs may significantly affect the functionality of building structures. In previous earthquakes, many buildings have entirely lost their functionality not because of structural damage but nonstructural damage (EERI, 1984; Villaverde, 1997; Miranda *et al*., 2018a, 2018b; Dhakal, 2010; Ju, 2011; Dhakal *et al*., 2016; Phipps *et al*., 2017; Devin and Fanning, 2019). Examples of strong earthquakes that resulted in significant damage to NSCs are the 1971 San Fernando earthquake (Jennings and Housner, 1971), the 1994 Northridge earthquake (OSHPD, 1995; Ayres and Ezer, 1996), the 2010 Chile earthquake (Moehle *et al*., 2010), the 2011 Tohoku Pacific Earthquake (Mizutani *et al*., 2012), the 2013 Lushan earthquake (Wang *et al*., 2013, 2016), and the 2017 Mexico earthquake (Malkin and Semple, 2017). Research on NSCs was initiated and driven by the significant effects of damage to NSCs (Lim and Chouw, 2015). During the last two decades, numerous studies on NSCs have been conducted. Various seismic detailing and protection techniques have been developed and applied to NSCs to achieve multiple objectives of performance-based earthquake engineering (Filiatrault *et al*., 2018), particularly the seismic resilience of buildings (Bruneau *et al*., 2003; Myrtle *et al*., 2005). However, the compatible performance between structural components and NSCs is difficult to achieve because of complex interfaces and the distinct mechanical properties of the two types of components.

The first step in the study of the seismic performance of NSCs is to determine the input, i.e., the floor response or the floor response spectra at the position where the component is attached to the building. Although an NSC

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may be displacement and/or acceleration sensitive, most recent research has focused on the floor acceleration response spectra (abbreviated as floor response spectra, FRS herein). FRS are generated from the absolute acceleration response of a floor in a building that is excited by the input ground motion, as shown in Fig. 1(a). Different from the ground acceleration spectra, FRS reflect the dynamic characteristics of the building structures. That is, the supporting structure filters out the vibrational components with the frequencies different from the building's natural frequencies, whereas the vibrational components with frequencies close to the natural frequencies are amplified (Sullivan *et al*., 2013b). Numerous studies were conducted to establish the general FRS for the seismic design of NSCs using the fundamental principles of structural dynamics. These studies demonstrated that the FRS were highly dependent on different parameters related to the building's characteristics and the NSC characteristics, including the location of the NSCs in the structure (e.g., Chaudhuri and Hutchinson, 2004; Chaudhuri and Villaverde, 2008; Uma *et al*., 2010; Clayton and Medina, 2012; Jayamon *et al*., 2015; Petrone *et al*., 2016; Anajafi and Medina, 2018c), the ratio of the NSC period to the building's modal periods (e.g., Medina *et al*., 2006; Pavlou and Constantinou, 2006; Mohammadi and Mohammadi, 2012; Clayton and Medina, 2012; Anajafi and Medina, 2018a, 2018b, 2018c, 2019a; Kehoe and Hachem, 2003; Sankaranarayanan and Medina, 2008; Pinkawa *et al*., 2014a), the damping ratio of the supporting structures and the NSCs (e.g., Marsantyo *et al*., 2000; Singh *et al*., 2006b; Clayton and Medina, 2012; Sullivan *et al*., 2013b; Obando and Lopez-Garcia, 2018; Anajafi and Medina, 2019b), the structural nonlinear behavior (e.g., Adam and Fotiu, 2000; Taghavi and Miranda, 2003b; Medina *et al*., 2006; Sankaranarayanan and Medina, 2007; Chaudhuri and Villaverde, 2008; Politopoulos, 2010; Ray-Chaudhuri and Hutchinson, 2011; Pollino, 2012; Dantanarayana *et al*., 2012; Adam *et al*., 2013; Wieser *et al*., 2013, Kazantzi *et al*., 2018; Anajafi and Medina, 2018c and 2019a; Surana, 2019), the interaction between the NSCs and the supporting structure (e.g., Sackman and Kelly, 1979; Asfura and Kiureghian, 1986; Taghavi and Miranda, 2008; Adam and Furtmüller, 2008a and 2008b; Adam *et al*., 2013; Pinkawa *et al*., 2014b; Pardalopoulos and Pantazopoulou, 2015), the torsional response of the supporting structures (e.g., Qu *et al*., 2014; Anajafi and Medina, 2019a), the diaphragm flexibility of the supporting structures (e.g., Qu *et al*., 2014; Kollerathu and Menon, 2017; Anajafi and Medina, 2019a), the type of lateral load resisting system in the supporting building (e.g., Miranda and Taghavi, 2005; Taghavi and Miranda, 2006; Anajafi and Medina, 2018c, 2019a; Chalarca *et al*., 2019), the soil-structure interaction (SSI) (e.g., Kennedy *et al*., 1981; Chaudhuri and Gupta, 2003; Raychowdhury and Ray-Chaudhuri, 2015; Zhang and Jiang, 2017a, 2017b), and the nonlinear behavior of NSCs (e.g., Toro *et al*., 1989; Igusa, 1990; Adam and Fotiu, 2000; Villaverde, 2006; Vukobratović and Fajfar, 2017; Anajafi, 2018; NIST, 2018; Chaudhuri

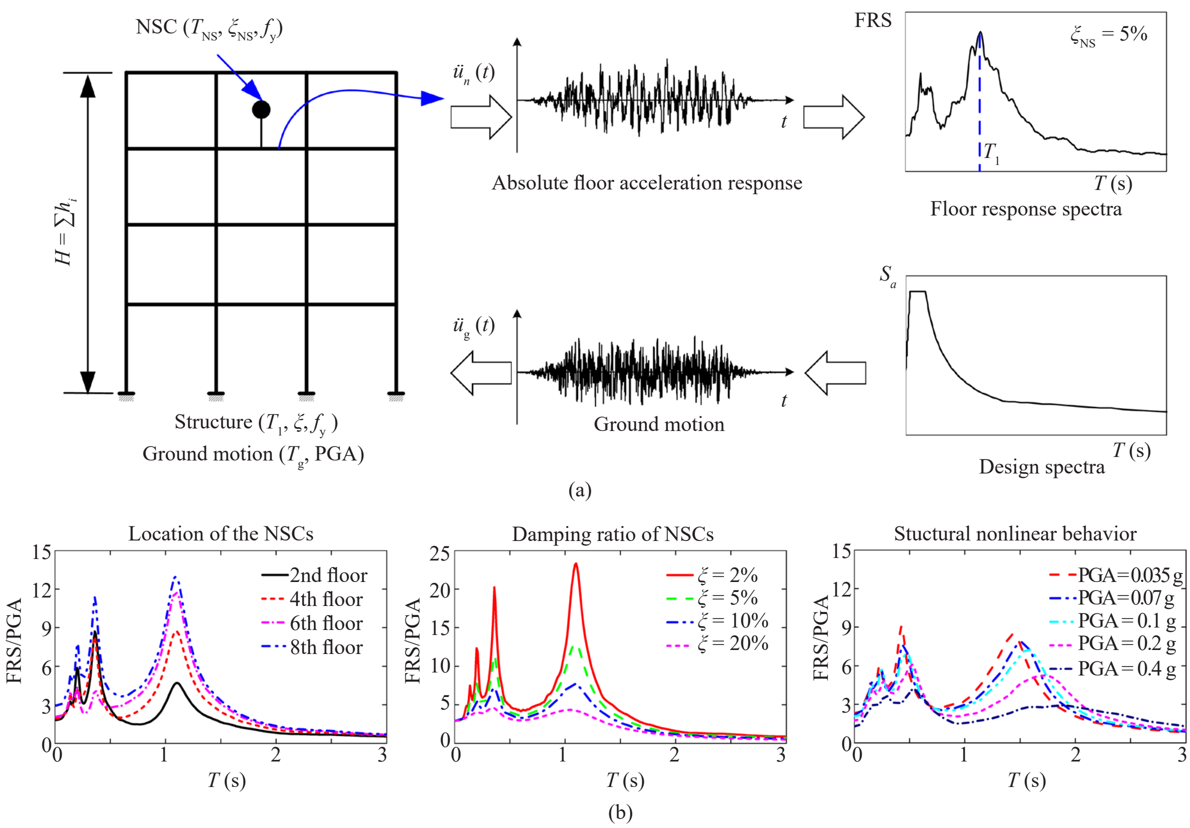


Fig. 1 Definition and development of FRS: (a) concept and definition of FRS, (b) influential factors

and Villaverde, 2008; Obando and Lopez-Garcia, 2018; Kazantzi *et al*., 2018; Anajafi *et al*., 2020). The effects of several crucial influential factors on the FRS are demonstrated by the time history analyses (THA) results of an eight-story RC frame (Wang *et al*., 2020), as shown in Fig. 1(b). The shape and value of the FRS can be quite different for different floors. A low NSC damping ratio will result in a large FRS and vice versa. As the earthquake intensity increases, the supporting structure yields, and its floor spectral accelerations are capped by the lateral force capacity of the supporting structure, so that the FRS is reduced accordingly. A detailed discussion of these influential factors and other factors, such as the nonlinear behavior of NSCs and the soil-structure interaction, is provided below.

Although several methods for generating FRS have been described in the relevant literature, none can consider all influential factors, which usually result in under- or over-estimation of the FRS. Moreover, achieving seismic resilience of buildings requires highly accurate FRS for the seismic design of the NSCs. Numerous researchers have investigated NSCs and FRS. Figure 2 (a) shows the distribution of keywords related to FRS or floor acceleration spectra in recent years. A total of 410 research papers were retrieved from the Web of Science (SCI-EXPANDED database, updated on May 10, 2019). Note that most of the studies are numerical analyses. The most common keywords are FRS, NSCs, seismic analysis, and earthquakes. As depicted in Fig. 2(b), the number of research papers on FRS has increased rapidly over time.

Several reviews on the seismic design of NSCs have been conducted, including the work by Chen and Soong (1988), and Villaverde (1997). NSCs are mentioned as a secondary structure or secondary systems in their review papers. There has been significant development in the design of either structure or NSCs during recent years, and many attempts have been made to generate FRS for the seismic design of NSCs. The existing state-of-the-art reviews are relatively old, and there is a need for an updated review that includes recent achievements. This review paper summarizes the progress made to date on the generation of FRS, which can be used to determine forces for the seismic design of acceleration-sensitive NSCs. Different methods for generating FRS based on single-degree-of-freedom (SDOF) models and multiple-degrees-of-freedom (MDOF) models are presented first, followed by a review of the amplification factor methods. Directly defined FRS methods and some newly developed methods are summarized next, and those included in current seismic design codes are compared. Subsequently, detailed investigations of research on the critical factors affecting FRS are outlined, including the nonlinear structural behavior, the vertical location of NSCs, and the interaction between structural components and NSCs, such as infill walls, the soil-structure interaction, the damping ratio of NSCs and the nonlinear behavior of NSCs. The floor acceleration response and FRS obtained from experimental studies and field observations during earthquakes are discussed. In addition, the effects of vertical components of input ground motions and near-fault ground motions

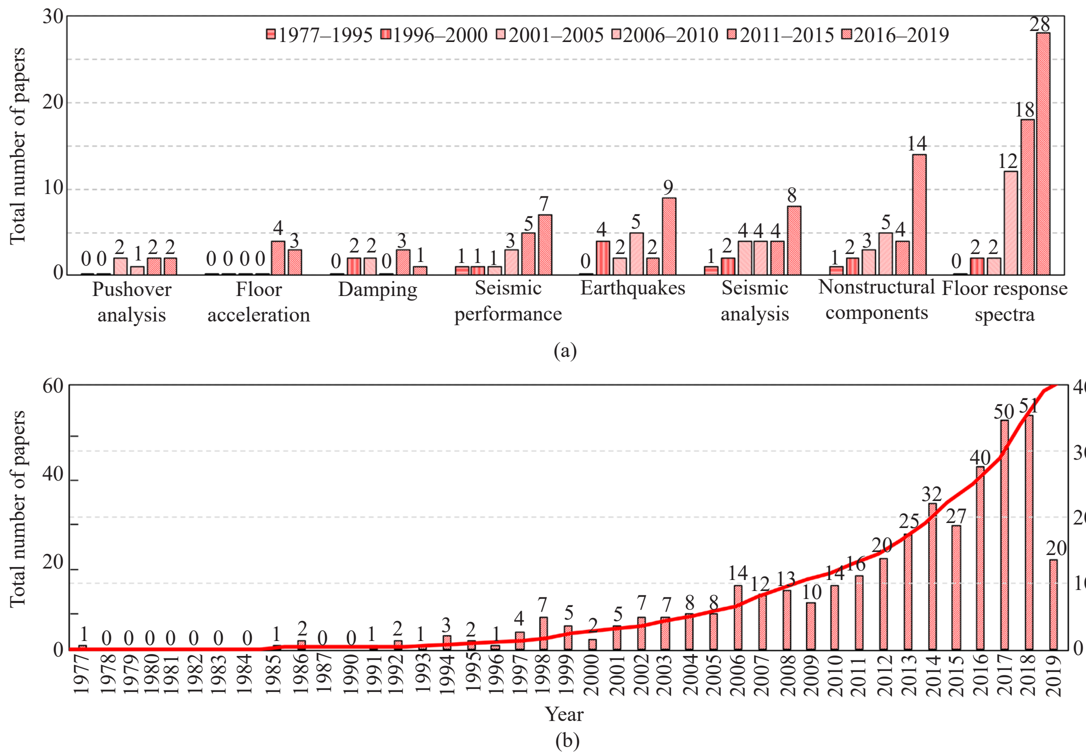


Fig. 2 Distribution of keywords related to FRS in recent years in the Web of Science (SCI-EXPANDED database, updated on May 10, 2019); (a) keyword frequencies; (b) number of papers related to FRS

(NFGMs) on the FRS are presented. Finally, the major knowledge gaps to be filled are identified, and possible future research challenges are discussed. Note that most of the results presented herein were generated for linear NSCs with a 5% damping ratio, except when otherwise indicated. Moreover, the literature related to NSCs with multiple supports is not covered in this review.

2 Development of floor response spectra

The methods to develop FRS are summarized in this section and are categorized into four groups, i.e., FRS based on SDOF models, FRS based on MDOF models, amplification factor methods, and directly defined FRS. The definitions are briefly illustrated in Fig. 3. Most early methods fall in the first category, where either the supporting structure or NSCs are treated as SDOF models. Basic mechanical parameters, such as the fundamental period, the damping ratio, and the yield strength ratio, are considered. Modal superposition methods are used in methods based on MDOF models to consider the effect of different vibration modes. FRS for each mode are first developed and then combined using a modal superposition technique to obtain the final FRS. Some methods have been developed based on the ground acceleration response spectrum (GRS) or the peak ground acceleration (PGA) to facilitate the application of FRS; these methods are categorized as the third and fourth methods, respectively. Details of these methods are discussed in the following sections. The methods adopted by some seismic design codes are reviewed at the end of this section.

2.1 FRS based on SDOF models

Research on FRS generation methods began in the 1970s. Early methods usually treated the supporting structure and NSC as SDOF systems. Penzien and Chopra (1965) and Kapur and Shao (1973) were among the first to generate the FRS from the response of a supporting structure using time history analysis (THA). Yasui *et al*. (1993) derived a direct generation method, in which a smooth design FRS can be generated using the design spectra or GRS. Because of the application of the Duhamel integration, this method does not require an empirical dynamic amplification factor, which represents the spectral acceleration of the NSCs normalized by the peak floor acceleration (PFA) of the supporting structure. The obtained FRS generation formula is expressed as Eq. (1).

$$\mathrm{FRS}\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)=\frac{1}{\sqrt{\left[1\text{-}\left(T_{\mathrm{S}}/T_{\mathrm{NS}}\right)^2\right]^2+4\left(\xi_{\mathrm{S}}+\xi_{\mathrm{NS}}\right)^2\left(T_{\mathrm{S}}/T_{\mathrm{NS}}\right)^2}}\cdot\sqrt{\left\{\left(T_{\mathrm{S}}/T_{\mathrm{NS}}\right)^2 S_a\left(T_{\mathrm{S}},\xi_{\mathrm{S}}\right)\right\}^2+S_a\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)^2} \quad (1)$$

where T_{NS} and ξ_{NS} are the period and damping ratio of the NSCs, T_{S} and ξ_{S} are the period and damping ratio of the supporting structure, $S_a\left(T_{\mathrm{S}},\xi_{\mathrm{S}}\right)$ and $S_a\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)$ are the values at the specific period and damping ratio in the elastic ground acceleration spectrum, $\mathrm{FRS}\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)$ are the FRS at the specific period T_{NS} and the damping ratio ξ_{NS}.

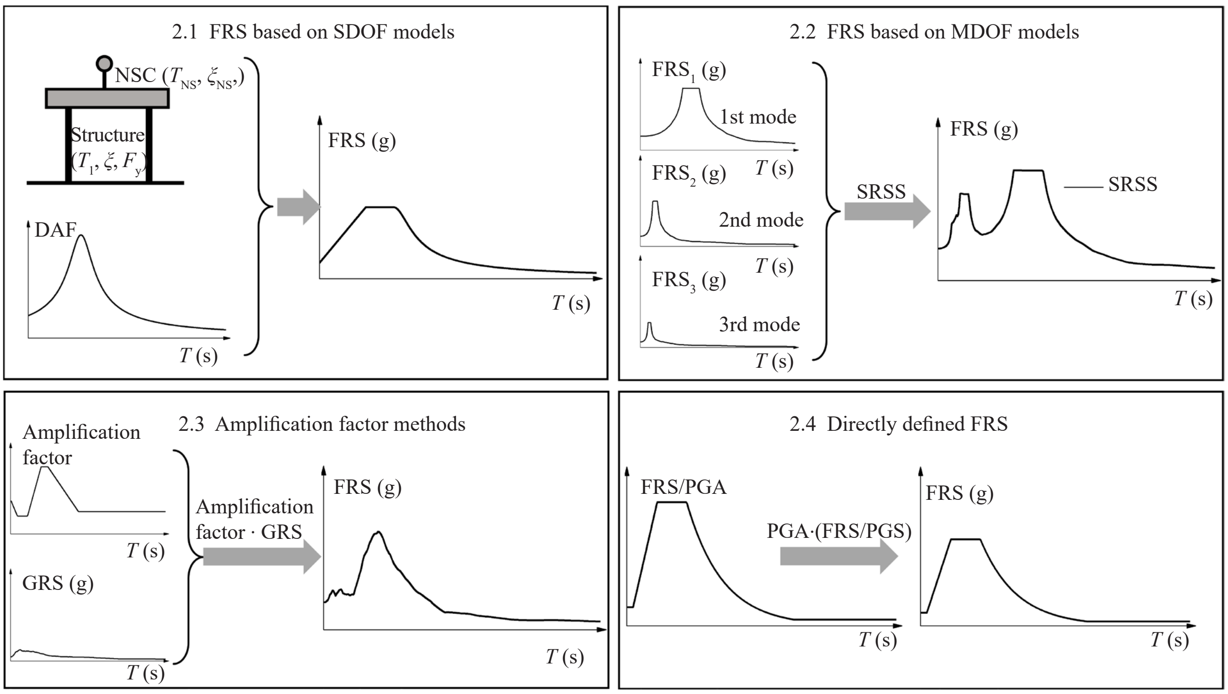


Fig. 3 Definitions of the four methods to develop FRS

The resonance region represents the portion of the floor spectrum that includes the peak and the surrounding high spectral values. Note that in the MDOF structures, the floor spectrum usually has more than one resonance region corresponding to several natural modes. The analytical results reported by Vukobratović and Fajfar (2012, 2013, 2015) show that, outside of the resonance region, the FRS obtained by the method of Yasui *et al*. (1993) match very well with the FRS obtained by THA. In the resonance region, however, a substantial difference was observed. Therefore, Vukobratović and Fajfar (2015) suggested using Eq. (2) to calculate the FRS in the resonance region, where AMP is an empirical amplification factor in the resonance region for the considered structure, and $\frac{S_a(T_\mathrm{S},\xi_\mathrm{S})}{R_\mu}$ represents the value in the inelastic acceleration spectrum that can be obtained by reducing the elastic acceleration spectrum using the strength factor R_μ (Vukobratović and Fajfar, 2015, 2016).

$$\mathrm{FRS}(T_\mathrm{NS},\xi_\mathrm{NS})=\mathrm{AMP}\cdot\frac{S_a(T_\mathrm{S},\xi_\mathrm{S})}{R_\mu} \tag{2}$$

Similarly, for the tuning (resonance) case when an NSC is tuned to the supporting structure, the concept of the tuning response spectra (tRS) was introduced by Jiang *et al*. (2015) and Li *et al*. (2015) for the accurate and efficient generation of the FRS. The statistical relationship between the tRS and GRS was developed based on the results of the THA.

Sullivan *et al*. (2013a, 2013b) used a dynamic amplification factor (DAF) to calculate the FRS in SDOF structures using Eq. (3). The DAF is the ratio of the maximum acceleration of the NSCs to the maximum acceleration of the floor on which the NSCs are mounted. An empirical expression of the DAF was provided by Sullivan *et al*. (2013b).

$$\mathrm{FRS}(T_\mathrm{NS})=\begin{cases}\frac{T_\mathrm{NS}}{T_\mathrm{S}}\left[a_\mathrm{max}\left(\mathrm{DAF}_\mathrm{max}-1\right)\right]+a_\mathrm{max}, & T_\mathrm{NS}<T_\mathrm{S}\\ a_\mathrm{max}\mathrm{DAF}_\mathrm{max}, & T_\mathrm{S}\le T_\mathrm{NS}\le T_\mathrm{e}\\ a_\mathrm{max}\mathrm{DAF}, & T_\mathrm{NS}>T_\mathrm{e}\end{cases} \tag{3}$$

where $\mathrm{FRS}(T_\mathrm{NS})$ is the spectral acceleration demand for a supported component with period T_NS, a_max is the maximum acceleration of the supporting structure (obtained for an SDOF system by dividing the structure's lateral resistance by the seismic mass).

2.2 FRS based on MDOF models

SDOF structure models cannot represent the response of multistory buildings accurately. Different methods for generating the FRS based on MDOF structural models were developed recently using a modal superposition method. Calvi and Sullivan (2014a, 2014b) extended the procedure of Sullivan *et al*. (2013a, 2013b) to MDOF structures as follows:

(1) Determine the acceleration demand $a_{\mathrm{max},m}$ on each floor for each mode m based on elastic modal analysis and the design response spectrum.

(2) Apply Eq. (3) for each vibration mode by replacing a_max by $a_{\mathrm{max},m}$ to obtain $\mathrm{FRS}_m(T)$ for mode m.

(3) For the upper floors, the FRS are calculated as the square-root-sum-of-squares (SRSS) of the modal spectra computed in Step (2).

(4) For the lower floors, the FRS is the maximum between the GRS and the spectral acceleration obtained from the SRSS of the modal spectra computed in Step (2).

The above procedure distinguishes between the upper and lower floors of the building based on the hypothesis of limited higher-mode filtering that occurs for the ground motion on the lower floors of a building (Calvi and Sullivan, 2014b; Filiatrault and Sullivan, 2014). Note that the above procedure has only been tested for elastic structures and was further improved (Welch, 2016; Aragaw, 2017; Aragaw and Calvi, 2018a, 2018b) to include the nonlinear behavior of the supporting structures by introducing modal reduction factors of the floor spectra. A modal reduction factor is defined as the ratio of the FRS from the linear response to that of the nonlinear response, which is dependent on the ductility of the supporting structure. Nonlinear regression was performed by Aragaw and Calvi (2018a, 2018b) to obtain the relationship between the modal reduction factor and the ductility demand of the supporting structure. Recently, the methodology proposed by Calvi and Sullivan (2014b) was updated by Merino *et al*. (2020) to account for the nonlinear behavior of the supporting structure. They developed a code-oriented methodology, which can provide an absolute acceleration FRS and a consistent relative displacement FRS.

The method proposed by Yasui *et al*. (1993) can also be used for MDOF structures. For the considered number of modes, the FRS of an individual mode is obtained using Eq. (4-1) for the off-resonance region and Eq. (4-2) for the resonance region (Vukobratović and Fajfar, 2017). In the resonance region, the FRS are defined as a product of the PFA_{ij} and AMP_i for mode i. PFA_{ij} is calculated by Eq. (4-3) and represents a special case of Eq. (4-1) for T=0. The FRS obtained for each mode should be combined to calculate the final FRS. SRSS or CQC rules were recommended for periods between zero and the end of the resonance region of the fundamental structural mode. In the post-resonance region of the fundamental structural mode, the algebraic sum (ALGSUM) should be used.

$$\mathrm{FRS}\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)_i=\frac{\varGamma_i\phi_{ij}}{\left|\left(T_{\mathrm{NS}}/T_i\right)^2-1\right|}\cdot\sqrt{\left[\frac{S_a\left(T_i,\xi_i\right)}{R_\mu}\right]^2+\left\{\left(T_{\mathrm{NS}}/T_i\right)^2 S_a\left(T_{\mathrm{NS}},\xi_{\mathrm{NS}}\right)\right\}^2} \tag{4-1}$$

$$\left|\mathrm{FRS}\left(T_i,\xi_{\mathrm{NS}}\right)_j\right|\le \mathrm{AMP}_i\cdot\left|\mathrm{PFA}_{ij}\right| \tag{4-2}$$

$$\mathrm{PFA}_{ij}=\varGamma_i\phi_{ij}\frac{S_a\left(T_i,\xi\right)}{R_\mu} \tag{4-3}$$

where T_i, ξ_i, and $\varGamma_i$ are the modal period, damping ration, and participation factor of mode i, ϕ_{ij} is the mode shape for floor j of mode i, PFA_{ij} is the peak floor acceleration on floor j for mode i, AMP_i is the empirical amplification factor for the considered mode i, as defined in Vukobratović and Fajfar (2017).

Pan *et al*. (2017b, 2018) proposed a method using equivalent SDOF (ESDOF) systems to estimate the FRS of an MDOF system based on modal pushover analysis (MPA). The FRS of each mode are obtained through the nonlinear dynamic analysis of each ESDOF system, and the total FRS is calculated for the considered modes using the SRSS combination rule.

2.3 Amplification factor methods

The amplification factor is defined as the ratio of the FRS to GRS and is usually used for generating the FRS directly from the GRS or the design response spectrum (Shooshtari *et al*., 2010). Different amplification factor functions have been proposed in recent years. Wieser *et al*. (2013) developed an empirical multi-linear envelope spectral acceleration amplification function, as illustrated in Fig. 4. The amplification function was developed based on the nonlinear THA (NTHA) results of four ductile flexible special moment resisting frames (SMRF). The function considers the influence of multiple factors, such as the period ratio of the NSCs to the first period of the supporting structure, the relative height, and the higher mode effect. The peaks located at the period ratios of 0, 0.3, and 1.0 correspond to the NSCs that are rigid, tuned to the second mode, and tuned to the first mode, respectively. The effective period of the structure is used to replace the elastic period (T_1) to consider the effect of structural yielding.

Five designs of reinforced concrete (RC) frames based on the Indian design codes were used by Surana *et al*. (2018a) to calculate the FRS while considering the inelastic behavior of the frames. The authors proposed floor amplification functions (Fig. 5) to estimate the FRS at any given height directly from the GRS if the supporting structure's first two mode shapes, the building modal periods, and the strength factor of the supporting structure are known. For short-period buildings, the amplification factor in the impact zones (i.e., the zones of the NSCs with periods between 0 and $0.5T_1$) of the second and higher modes can be considered a constant (Fig. 5(a)). For long-period buildings, the peaks of the first two modes are considered (Fig. 5(b)). These peaks have been represented by separate parabolic functions in the impact zone of each mode. Both elastic and inelastic structure responses were considered by Surana *et al*. (2018a).

The amplification factor method proposed by Surana *et al*. (2018a) was extended by Surana *et al*. (2018b) to

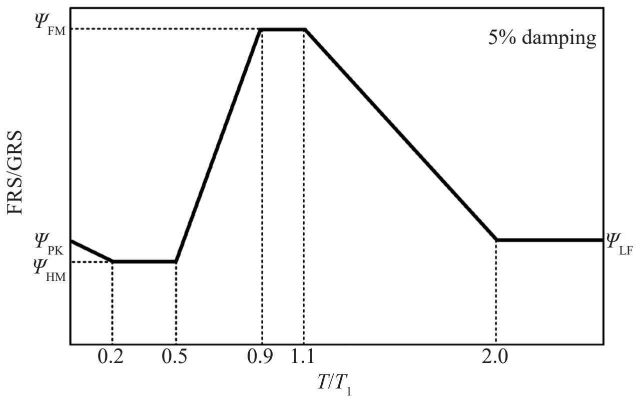
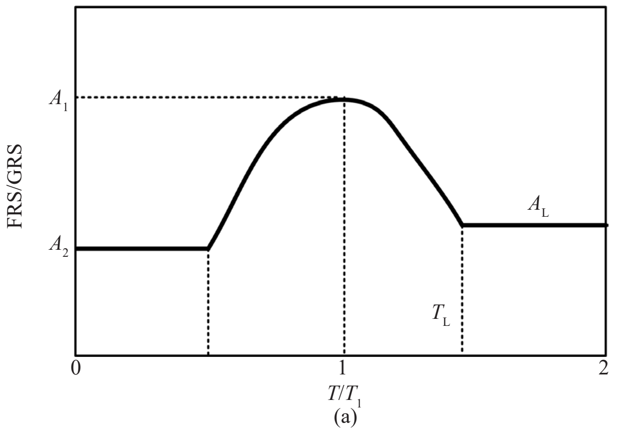


Fig. 4 Spectral acceleration amplification function by Wieser *et al*. (2013)



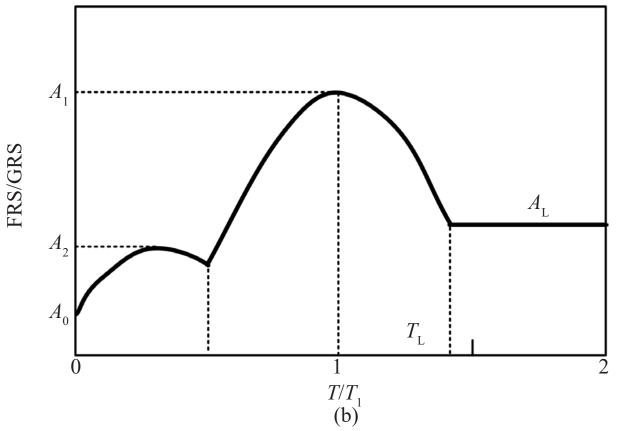


Fig. 5 Spectral acceleration amplification function by Surana *et al*. (2018a) (a) short-period buildings and (b) long-period buildings

predict the floor response of hill-side RC frame buildings. The building plan and elevation irregularities were considered when determining the spectral amplification functions. However, the proposed model only considered elastic structures.

2.4 Directly defined FRS

Different from the amplification methods, directly defined FRS are based on a directly defined function of the component acceleration factor (FRS/PGA). FRS can be directly calculated using the component acceleration factor for a given PGA level. Singh *et al.* (2006b) proposed equations for the FRS/PGA that considered the effect of possible resonance with higher modes, as shown in Fig. 6. For a building with an unknown period, the period of the NSCs is also assumed to be unknown. A conservative estimate of the FRS/PGA is provided in Eq. (5) for flexible NSCs with a damping ratio of 5%. For a 2% damping ratio, Eq. (5) must be multiplied by 1.5. A detailed discussion of the effects of the NSC damping ratio is provided in Section 3.5.

$$\frac{\text{FRS}}{\text{PGA}} = 6\left(1 + 2\frac{z}{h}\right) \tag{5}$$

The component amplification factor (a_p) is usually used for deriving the FRS from the PFA. a_p is defined as FRS/PFA with respect to the relative height and reflects the dynamic amplification effect of a specific NSC on a specific floor. A component acceleration factor was used by Medina (2013) to quantify the maximum component acceleration demand, which is equivalent to the product of the component amplification factor (a_p) and the in-structure amplification factor (PFA/PGA). The suggested component acceleration factor for linear NSCs with a 5% damping ratio is given based on the statistical results of the NTHA. For flexible NSCs mounted near the bottom or top of the structure, a value of 3 or 12 is recommended, whereas, for rigid NSCs, a value of 2 or 4 is recommended (Medina, 2013).

A simplified model for quantifying a_p was developed by Hou *et al.* (2018) using shaking table test results. A modification factor with a value of 1.35 was used to consider the tuning effects between the NSCs and supporting structures and prevent underestimation. However, the critical parameters used to define the simplified model were all fitted using the test results of a low-rise steel CBF building and cannot be applied to other structures.

Petrone *et al.* (2015) proposed a novel formulation, as shown in Fig. 7. The parameters a, b, and a_p were determined based on the building's fundamental period T_1, as shown in Table 1. This formulation considers the higher mode effects, and the predicted FRS are conservative for a wide range of periods, especially for periods close to T_1. The overestimation covers the uncertainties in the calculation of structural and NSC periods. The proposed formulations are related and limited to a set of RC frames designed according to Eurocode 8 (CEN, 2004). Modifications are needed when using these formulations in other structure types.

Fathali and Lizundia (2011) proposed a three-segment a_p spectrum to generate the FRS from the PFA values. The proposed spectrum was developed from recorded data in instrumented buildings and ranged from 1.0 to 2.5; it included a flat segment with a maximum value of 2.5 for medium-range periods and a nonlinear decaying segment for longer periods.

Table 1 Values of the parameters suggested by Petrone *et al.* (2015)

Fundamental period	a	b	a_p
$T_1 < 0.5$ s	0.8	1.4	5.0
$0.5 \text{ s} < T_1 < 1.0$ s	0.3	1.2	4.0
$T_1 > 1.0$ s	0.3	1.0	2.5

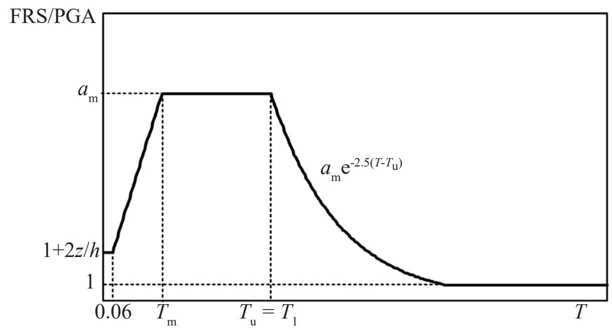


Fig. 6 Component amplification factor proposed by Singh *et al.* (2006b)

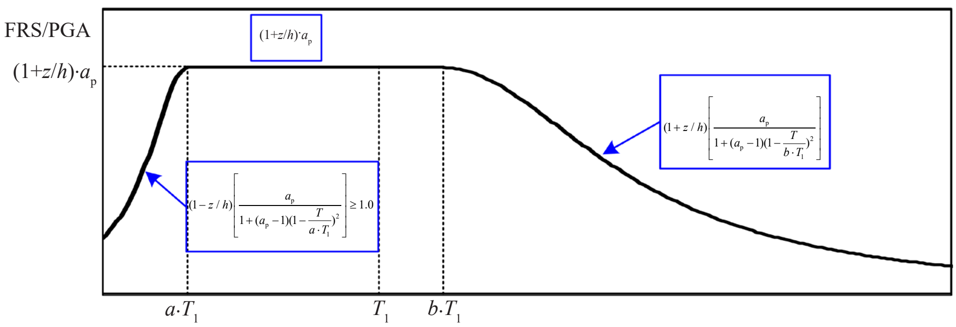


Fig. 7 Shape of the floor spectra proposed by Petrone *et al.* (2015)

In another study, Anajafi (2018) developed expressions for the generic floor response spectrum (GFRS) based on a statistical analysis of the FRS of instrumented buildings; the expressions did not require information on the supporting building type, NSC tuning ratio, and vertical location in the building. The objective of this GFRS was to generate an FRS that is neither building- nor component-specific. Anajafi (2018) suggested that the GFRS could be used for seismic tests of NSCs. Anajafi (2018) stated that in many cases when testing an NSC, no information is available on the detail of a component's support and/or its attachment to the supporting building, the dynamic characteristics of the building, and/or the NSC and the location (i.e., floor level) of the attachment of the NSCs. Therefore, Anajafi (2018) concluded that it might be justifiable to advocate the use of a GFRS for testing NSCs that is neither building- nor component-specific.

2.5 Acceleration demands of NSCs defined in seismic design codes

The seismic design of NSCs was first included in the Applied Technology Council report (ATC, 1978). Subsequently, more seismic design codes have proposed the use of seismic design methods for NSCs. The definitions of PFA/PGA in several current design codes are compared in Fig. 8. Currently, the ASCE 7-16 (2016), Eurocode 8 (CEN 2004), and Chinese GB 50011-2010 (2010) assume a linear distribution of the PFA demand along the height of the building, and the values of PFA/PGA at the roof are equal to 3.0, 2.5, and 2.0, respectively. In the New Zealand NZS 1170.5 code (2004), the PFA/PGA along the height of the building is calculated using the floor height coefficient, C_{Hi}, which is based on the building height, h. If h is less than 12 m, the PFA/PGA is linear with a value of 1.0 at the ground and a value of $1+h/6$ at the roof. For buildings with a total height larger than 12 m, the PFA/PGA is bilinear with a constant value of 3.0 for floors higher than 12 m (or higher than $0.2h$ when h>60 m). The definition of the PFA/PGA in the NEHRP code (FEMA P-750, 2009) is the same as that in the ASCE 7-16 code (2016).

The dynamic component amplification factor, a_p (Section 2.4), ranges from 1.0 to 2.5 for rigid NSCs (with a period shorter than 0.06 s) and flexible NSCs (with a period larger than 0.06 s) in ASCE 7-16 (2016), whereas the value for flexible NSCs is 2.0 in GB 50011-2010 (2010). The a_p factor in NZS 1170.5 (2004) is related to the NSC period without considering the structure period, as shown in Fig. 8(b), whereas the factors in the Eurocode 8 (CEN, 2004) and NEHRP (FEMA P-750, 2009) are related to both the NSCs and the supporting structure. However, the available codes and guidelines, such as ASCE 7-16 (2016) and GB 50011-2010 (2010) are based on past experiences and engineering judgment instead of test results or numerical analysis.

Note that the above definitions can be used to calculate the equivalent static force for the seismic design of NSCs. The peak component acceleration (PCA) demand in ASCE 7-16 (2016) is calculated as the product of a_p and PFA. Many studies have evaluated the definition of the PFA and a_p and suggested some modifications should be made to obtain a more accurate result (e.g., Anajafi, 2018; Anajafi *et al.*, 2018c; Kazantzi *et al.*, 2018). Dynamic analyses, including the linear dynamic analysis procedure (Section 12.9 of ASCE 7-16, 2016), nonlinear response history procedure (Chapters 16, 17, and 18 of ASCE 7-16, 2016), FRS methods (Section 13.3.1.4.1 of ASCE 7-16, 2016), and alternate FRS methods (Section 13.3.1.4.2 of ASCE 7-16, 2016) are also permitted by ASCE 7-16 (2016) to determine the design forces for NSCs. In the FRS method, the FRS is calculated for the design earthquake at each floor level based on a seismic response history analysis for each ground motion. In the alternate FRS method, the a_p factor is used. The peak acceleration response for the i-th mode is calculated as the product of the modal participation factor, the spectral acceleration, and the a_p factor for the i-th mode. The FRS takes the maximum value of the peak acceleration response at each modal period of the building (at least the first three modes) but is not less than the GRS values.

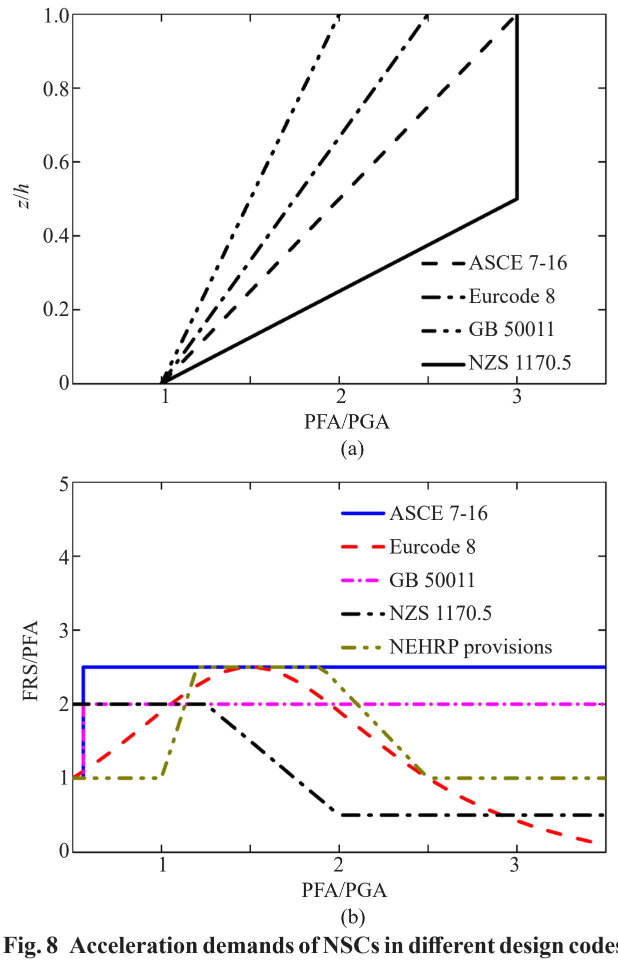


Fig. 8 Acceleration demands of NSCs in different design codes. (a) The PFA/PGA distribution (h = 24 m is used in the NZS 1170.5 (2004)) and (b) the component dynamic amplification factor (T_1 = 1.0 s is used in the Eurocode 8 (CEN, 2004))

3 Critical factors influencing the floor response spectra

3.1 Effects of nonlinear behavior in supporting structures

The nonlinear behavior of the supporting structures has a significant influence on the floor acceleration responses. This effect has to be considered in the seismic design of NSCs in most cases, considering that the acceleration or force demands on the NSCs are usually smaller than that induced in linear supporting structures when subjected to an earthquake of the same intensity. However, FRS values may be increased sometimes, especially for NSCs, with a period between the building's modal periods (Lin and Mahin, 1985; Toro *et al*., 1989; Chaudhuri and Villaverde, 2008; Sankaranarayanan and Medina, 2008; Anajafi, 2018). Several studies have investigated the nonlinear behavior effects of supporting structures on the acceleration demands and the FRS used in the NSC design (e.g., Igusa, 1990; Politopoulos and Feau, 2007; Vukobratović and Fajfar, 2015). Taghavi and Miranda (2012) demonstrated that a linear structure might amplify the ground motion, whereas floor accelerations may be smaller than the ground acceleration in nonlinear structures. Taller and flexible structures with longer periods are more affected by nonlinear behavior. Lin and Mahin (1985) defined an amplification factor as the ratio of the FRS of an inelastic structure to the FRS of an elastic structure to quantify the effect of inelastic responses of the supporting structure. The amplification factor was defined at four points, i.e., *A*, *B*, *C*, and *D*, as depicted in Fig. 9. The amplification factor values are constant before point *A* and after point *D*. Point *C* represents the building's fundamental period, and point *B* represents the maximum value. The recommended amplification factor can be used to generate the FRS considering the effects of the building's nonlinearity without conducting a nonlinear analysis.

Oropeza *et al*. (2010) found that the definition by Lin and Mahin (1985) may overestimate the effects of the inelastic behavior for structures with a

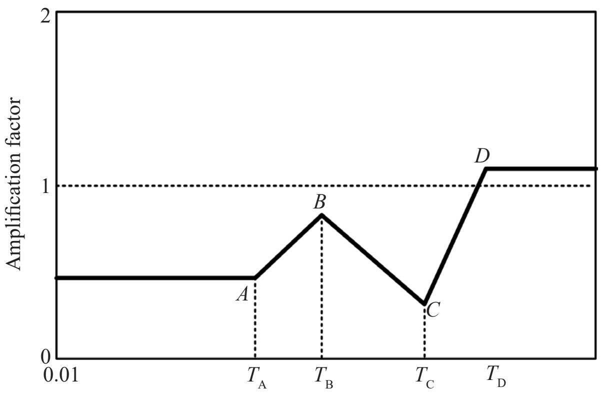


Fig. 9 Amplification factors defined by Lin and Mahin (1985)

high fundamental period and proposed an improved amplification factor based on the NTHA. Similarly, Flores *et al*. (2015a, 2015b, 2015c) conducted NTHA to evaluate the FRS demand and amplification factor in steel moment frames. Zhai *et al*. (2016) developed a predictive model to estimate the amplification factor and quantified the nonlinearity effect of the supporting structures while considering the period of the NSCs, the period of the supporting structures, and the primary structural ductility. The predictive model of Zhai *et al*. (2016) was then improved by Pan *et al*. (2017a), and a modified Park-Ang damage index was used to evaluate the damage of the supporting structures instead of the ductility factor. However, both predictive models were based on the NTHA results of specific structures and cannot be used for other types of structures.

The acceleration response modification factor (R_{acc}) was proposed by Sankaranarayanan (2007) and Medina (2007, 2009) to quantify the effects of the supporting structure's nonlinear behavior. The R_{acc} factor is the reciprocal of the amplification factor defined by Lin and Mahin (1985). This factor was also used to generate the FRS while considering the effects of building nonlinearity. Sankaranarayanan (2007) also pointed out that a reduction in the FRS values occurred as a result of the structure's nonlinearity when the NSC period was close to the building's modal periods, and a greater reduction was observed near the fundamental period than in the higher modal period. In addition, an increase in the FRS values was observed when the NSC period was located between two modal periods of the building. Note that linear NSCs were considered in these studies.

3.2 Story amplification factor

The story amplification factor or in-structure amplification factor, PFA/PGA, is used for the seismic design of rigid NSCs. The seismic design force of rigid NSCs can be quite different at different locations in a building. The code provisions in ASCE 7-16 (2016) use $1+2z/h$ to consider the height effect. However, the linear distribution of the PFA demand in the vertical direction of the building is building-independent and overly conservative, especially for tall buildings. Moreover, the nonlinear behavior of the supporting structures is not considered in this approach (Villaverde, 2006; Fathali and Lizundia, 2011; Wieser *et al*., 2013; Anajafi and Medina, 2018c; Anajafi *et al*., 2020). Singh *et al*. (2006a) proposed a new formula that considered the building's fundamental period. The formula is $1+2z/h$ for buildings with unknown periods. When the building's fundamental period T_1 is given, the peak acceleration differs for buildings of different heights and is calculated based on the number of stories in the building.

Based on the NTHA results of five 2D steel moment-resisting frames, Akhlaghi and Moghadam (2008) demonstrated that the PFA distribution depends on the behavior of the structures, the rigidity and flexibility of the buildings, and the fundamental period. A simplified distribution of the PFA was proposed, and the building's fundamental period was considered. Although this was a simplified approach, the suggested distribution showed better results for estimating the PFA distribution than the widely used profile 1+2*z*/*h*.

Surana *et al*. (2017) found that the PFA demand was primarily dependent on the relative height, as well as the fundamental period and the strength ratio of the supporting structure. The PFA demand on the building's roof decreased with an increase in the period and the strength ratio. A model was proposed to calculate the PFA by considering the strength ratio; this model reflects the nonlinear behavior of structures. The effect of the strength ratio was greater for buildings with fundamental periods less than 2.5 s. For buildings with periods larger than 2.5 s, it was assumed that the PFA demand was constant (i.e., PFA/PGA=1), regardless of the floor level.

Wieser *et al*. (2013) proposed a model that related the PFA demand to the fundamental period of the supporting structure. The proposed model for linear structures is given in Eq. (6-1) and that for nonlinear structures is given in Eq. (6-2).

$$\frac{\mathrm{PFA}}{\mathrm{PGA}} = 1 + \frac{2.5 - T_1}{T_1}\frac{z}{h} \tag{6-1}$$

$$\frac{\mathrm{PFA}}{\mathrm{PGA}} = 1 + \frac{2.5 - T_\mathrm{e}}{T_\mathrm{e}}\frac{z}{h} \tag{6-2}$$

$$T_\mathrm{e} = T_1\sqrt{\frac{\mu}{1+\alpha(\mu-1)}} \tag{6-3}$$

Using recorded acceleration data in buildings, Fathali and Lizundia (2011 and 2012) found that the PFA/PGA relationship in Eq. (13.3-1) of the ASCE 7 was suitable for buildings with periods smaller than 0.5 s but was conservative for buildings with larger periods. A new PFA/PGA relationship was defined based on the statistical results. The effects of the building's fundamental period and earthquake intensity were explicitly considered. It is important to note that most of the instrumented buildings in the United States exhibit responses in the elastic range (Fathali and Lizundia, 2011; Anajafi and Medina, 2019a). Therefore, equations derived based on the response of instrumented buildings might not be directly applicable to code-based designed buildings that are expected to exhibit an inelastic response when suffering a design earthquake ground motion (Anajafi and Medina, 2019a).

A simplified method was developed to rapidly estimate the floor acceleration demand in building structures that respond linearly to earthquake ground motions (Taghavi and Miranda, 2004; Taghavi and Miranda, 2005; Miranda and Taghavi, 2005; Reinoso and Miranda, 2005). The floor acceleration demands were approximated using the first three vibration modes of the building. The structure model was represented by a simplified continuum model consisting of a cantilever flexural beam connected laterally to a cantilever shear beam by axially rigid links that transmitted the horizontal forces. A comparison of the floor acceleration demand obtained from the approximation method and recorded data in six instrumented high-rise buildings indicated that the proposed method produced relatively good results with a very small computational effort and required only a small amount of information on the building. However, the method is limited to structures that remain elastic or practically elastic. The U.S. National Institute of Standards and Technology (NIST) proposed a new formula to estimate the PFA/PGA values based on recorded data and numerical analysis results (NIST, 2018). The method used the PFA normalized by the PGA recorded in 44 instrumented buildings, which had experienced earthquakes with PGA>0.15 g. Average PFA/PGA values that were computed using simplified continuous models adapted from Miranda and Taghavi (2005) were also used. The period of the supporting structure and the whiplashing effect of the higher modes were considered. NIST (2018) then suggested using the response modification factor (*R*) to capture the effect of the building ductility on the reduction in the acceleration demand of the NSCs. Later, Anajafi (2018) used the responses of many code-based designed buildings and proposed modifications to the expressions presented in NIST (2018) for the estimation of the *R* factor. Anajafi (2018) showed that equations in NIST (2018) might underestimate PFA/PGA responses for mid-height floor levels.

3.3 Interaction between structural components and infill walls

It is widely recognized that the presence of infill walls modifies the global structural response of buildings subjected to seismic loads and also affects PFA values, as well as the shape and maximum spectral accelerations in all stories. The effect of structural nonlinearity on the FRS is much more pronounced when infill walls are considered (Mollaioli *et al*., 2010; Lucchini *et al*., 2013, 2014; De *et al*., 2015). Mollaioli *et al*. (2011) considered 4-, 6-, and 8-story RC frames to estimate the changes in the PFA demand at different heights of different RC frames with or without infill walls. The results indicated that the influence of the infill walls decreased as the number of stories increased. Asgarian and McClure (2014) calculated the FRS and obtained similar results. Blasi *et al*. (2018) observed a noticeable amplification of the FRS due to the effects of infill walls; it was also found that the infill walls generally reduced the irregularity effect in elevation, resulting in a more uniform distribution of the PFA at different heights. Surana *et al*. (2018c) modified their previously proposed floor spectral amplification functions (Surana *et al*., 2018b, 2018c) for uniformly infilled and open ground story RC frames. The presence of infill walls and their mechanical properties should be considered when evaluating the seismic demand on NSCs. A set of mid-rise bare and uniformly infilled RC frame buildings were analyzed by Surana (2019) under different earthquake intensities to determine the FRS. The results verified that the effects of the infill walls were significant and should be considered, whereas the effect has not been considered in design codes. Although some studies have been conducted to investigate the effects of infill walls on the FRS, more studies are required to obtain more accurate FRS.

3.4 Interaction between NSCs and the supporting structures

For NSCs with mass ratios no larger than 1%, the dynamic interaction effect is relatively small (less than 10%) and can be neglected (Taghavi and Miranda, 2008). For NSCs with a larger mass, the interaction effect may be very large for NSCs tuned to the supporting structure. However, in many cases, the supporting structure and the NSCs are decoupled and analyzed individually (i.e., decoupling analysis). Decoupling analysis sometimes provides conservative results, especially when the natural period of an NSC is close to that of the supporting structure (Chen and Soong, 1988). Several studies have demonstrated that the interaction between the supporting structure and the NSCs may have a significant influence on the FRS (Adam and Furtmüller, 2008a, 2008b; Taghavi and Miranda, 2008; Adam *et al*., 2013; Lim and Chouw, 2014, 2018; Vela *et al*., 2018, 2019). The effect of the dynamic interaction is smaller for NSCs tuned to higher modes than NSCs tuned to the fundamental mode.

The NSC and the supporting structure were considered as a combined system by Sackman and Kelly (1979) to analyze the NSC response. Similarly, Igusa and Kiureghian (1985a, 1985b, 1985c) presented a new method for generating the FRS in the frequency domain. The method was derived from the fundamental principles of structural dynamics, random vibrations, and the perturbation theory. Therefore, structural nonlinearity and NSCs with a large mass cannot be considered. Asfura and Kiureghian (1986) developed a cross-oscillator cross-floor response spectrum (CCFS) method to consider the interaction effect to produce more realistic design criteria for NSCs. Suarez and Singh (1987a, 1987b) presented a mode synthesis-based direct approach to calculate the seismic response of NSCs. Unlike the perturbation technique, the modal synthesis approach does not assume small variations. Hence, it can be used for light and heavy NSCs, regardless of the mass. Similar conclusions, as mentioned above, were drawn from these studies.

3.5 Damping ratio of NSCs

The viscous damping ratios of NSCs were found to range from 1% to 30% (Aragaw and Calvi, 2018b). Similar to the effects on the ground motion acceleration response spectra, a low NSC damping ratio will result in a large FRS, especially for NSCs with a period that is similar to that of the supporting structure, and vice versa. The effect of NSC damping on the seismic demand of NSCs needs to be properly understood. The calculated FRS should account for the most likely damping level. However, most FRS studies considered a 5% NSC damping ratio. Only a few studies (e.g., Medina *et al*., 2006; Sankaranarayanan and Medina, 2007; Sadeghzadeh-Nazari and Ghafory-Ashtiany, 2011; Calvi and Sullivan, 2014b; Vukobratović and Fajfar, 2017) have investigated the effects of NSC damping on the FRS. Aragaw and Calvi (2018b) pointed out that the effects of NSC damping on the FRS can be neglected if the NSC periods are very small or very large relative to the supporting structure. Anajafi and Medina (2019b) conducted a comprehensive study to determine the effects of NSC damping on the FRS. A damping modification factor (DMF) was used, which is defined as the FRS with a given damping ratio relative to the FRS for a 5% damping ratio. Numerical analyses were conducted on structures following design codes to investigate the influence of the DMF. The results indicated that NSC damping ratio and tuning period ratio (period ratio between the NSC period and the building's fundamental period) significantly affected the DMF values. Based on the numerical results, empirical equations were proposed to predict the DMF given the NSC damping ratio and tuning period ratio. Kazantzi *et al*. (2020) proposed a probabilistic model that incorporated the mean and lognormal standard deviation of the DMF using recorded floor acceleration data.

3.6 Nonlinear behavior effects of NSCs

Component response modification (reduction) factors are usually used in seismic design codes of NSCs to account for nonlinear behavior effects of NSCs. For instance, ASCE 7-16 (2016) uses a component response modification factor (R_p) that ranges from 1.0 to 12.0 to account for the characteristics of an NSC (including viscous damping, nonlinear behavior, inherent over-strength). A similar behavior factor of the NSCs (q_a) was adopted in the Eurocode 8 (CEN, 2004). However, the values of R_p or q_a for different NSCs were established based on engineering expertise rather than experimental tests or numerical analysis. A few studies have investigated the nonlinear behavior effects of NSCs on FRS (e.g., Toro *et al*., 1989; Igusa, 1990; Adam and Fotiu, 2000; Villaverde, 2006; Vukobratović and Fajfar, 2017; Anajafi, 2018; NIST, 2018). Igusa (1990) derived an analytical solution for the response of a 2-DOF primary-secondary system with small nonlinearities using random vibration analysis and equivalent linearization techniques. Adam and Fotiu (2000) analyzed the response of SDOF oscillators attached to a four-story frame building. The results indicated an influence on the primary response in a large frequency range when an inelastic SDOF oscillator with a low yield level was considered. Strength reduction factors were used by Villaverde (2006) to account for the nonlinear behavior of NSCs and their supporting structure. Chaudhuri and Villaverde (2008) evaluated the seismic responses of inelastic NSCs supported by moment-resisting frames. Vukobratović and Fajfar (2017) suggested the consideration of the inelastic behavior of NSCs by increasing the damping of the NSCs when generating the FRS for ductile NSCs. Filiatrault *et al*. (2018) used the concept of the nonstructural equivalent damping ratio to account for the nonlinear behavior of NSCs (i.e., suspended piping systems in their study). The equivalent damping ratio was derived from pseudo-static cyclic test. They found that nonlinear behavior of NSCs reduced their displacement demand. Obando and Lopez-Garcia (2018) found that NSC inelasticity significantly decreased the displacement demands on tuned NSCs, especially those tuned to the fundamental mode. Kazantzi *et al*. (2018) achieved similar results and found that the inelastic behavior of NSCs reduced their force and displacement demands by constant-ductility floor spectra. Anajafi *et al*. (2020) generated the nonlinear FRS of several code-designed buildings considering different levels of NSC inelasticity. The results indicated that the NSC inelasticity significantly reduced the peak values of the FRS for elastic buildings in the vicinity of the building's modal periods and significantly de-emphasized the effects of the tuning period ratio, damping ratio, and the characteristics of the supporting building and ground excitation. Although previous studies have provided valuable insight into the nonlinear behavior effects of NSCs on FRS, further research is still required to incorporate this information in the seismic design of NSCs.

3.7 Other parameters that influence the acceleration demands on NSCs

It is well-known that the SSI can affect the seismic response of a building (Lou *et al*., 2011; Anand and Kumar, 2018) and influence the FRS. However, only a few studies investigated the effects of the SSI on the floor acceleration demands and the FRS. Kennedy *et al*. (1981) evaluated the FRS in an MDOF pressurized water reactor auxiliary building and considered the SSI effects. Chaudhuri and Gupta (2003) included the SSI by using the sub-structure approach for determining the FRS in a 15-story shear building. The results indicated that the SSI should not be neglected when generating the FRS unless the soil is quite stiff relative to the supporting structure. Raychowdhury and Ray-Chaudhuri (2015) found that a nonlinear SSI reduced the FRS values in a steel frame. It is important to note that the NSCs were modeled as linear SDOF systems, and the dynamic interaction between the supporting structure and the NSCs was not considered in this study. Shaking table test results from Zhang and Jiang (2017a and 2017b) also verified that the SSI could reduce the floor acceleration response.

Torsional response and diaphragm flexibility of supporting structures have been investigated by a few researchers (e.g., Çelebi *et al*., 1991; Tena-Colunga and Abrams, 1996); these factors have a significant influence on the shape and peak values of the FRS according to Anajafi and Medina (2019a). Qu *et al*. (2014) suggested that the torsional response can amplify the acceleration demands on NSCs located around the edge of the floor plan. Anajafi and Medina (2019a) achieved the same results using recorded acceleration data. Qu *et al*. (2014) and Anajafi and Medina (2019a) found that in-plane diaphragm flexibility can produce larger floor acceleration around the middle of a floor plan. These studies showed that typical buildings (even regular buildings with well-adopted floor systems) exhibited torsional behavior and diaphragm flexibility that may increase the force demands on NSCs by factors up to 2.0. Kollerathu and Menon (2017) conducted a THA of elastic and inelastic masonry structures. The results indicated that the PFA response increased with increasing diaphragm flexibility.

4 Effects of input ground motion

4.1 Vertical ground motions

It is commonly assumed that buildings are flexible in the horizontal direction and rigid in the vertical direction; as a result, the vertical acceleration demands of NSCs are usually not considered. In other cases, the vertical design spectra are determined based on the horizontal spectra. A scaling factor of 0.7 or 2/3 is usually applied to the horizontal FRS to generate the vertical FRS. However, recent research on the vertical FRS showed that ignoring the effect of the vertical component of the ground motion may underestimate the vertical demand on the NSCs (Swanson *et al*., 2012; Moschen *et al*., 2015, 2016; Gremer *et al*., 2018, 2019). Qu *et al*. (2014) investigated the distributions of the vertical and horizontal seismic acceleration demands on NSCs using data from instrumented buildings during earthquakes. It was found that the vertical peak floor acceleration (PFA_v) was not constant in the vertical direction, which is unlike the constant distribution assumed in the ASCE 7-16 (2016). Moschen *et al*. (2015, 2016) statistically assessed the PFA_v demands on column lines of elastic multistory steel frames. The results indicated that the vertical ground acceleration was amplified along the column line. Gremer *et al*. (2018, 2019) found that the PFA_v amplification was significantly larger than the horizontal peak floor acceleration. The location of the beam nodes has a significant influence on the PFA_v values. Therefore, it is suggested to use more than one DOF per story to evaluate the PFA_v. The largest amplification in a floor occurs at the mid-span of the beam, whereas the smallest amplification occurs at the interior column. The same results were also found by Francis *et al*. (2017), who reported that the amplification factor might be larger than six. Guzman *et al*. (2017) found that the slab accelerations were generally amplified by a factor of 2.5 to 6.5 relative to the vertical ground acceleration, and a slab amplification factor of 4 or 5 was suggested for the vertical direction. The PFA_v amplification factor ranged from 3 to 6 in a shaking table test of a full-scale steel moment frame (Ryan *et al*., 2016; Soroushian *et al*., 2016). Current design codes for NSCs neglect the amplification and are thus non-conservative.

4.2 Near-fault ground motions

Several researchers have studied the effects of NFGMs on NSCs, beginning with the study of Kennedy *et al*. (1981). Sankaranarayanan and Medina (2006) statistically analyzed the acceleration demands of NSCs mounted on moment-resisting frames under the excitation of NFGMs. Kanee *et al*. (2013) analyzed the FRS of a nonlinear frame using 49 NFGM records. Alonso-Rodríguez and Miranda (2015) investigated the floor acceleration demands in buildings subjected to NFGMs using the simplified building model developed by Miranda and Taghavi (2005). Acceleration demands were found to be sensitive to the high-frequency component of the NFGMs. The pulse duration is the most critical parameter and influences the floor acceleration demand because it induces large variations in the PFA. Zhai *et al*. (2016) and Pan *et al*. (2017a) compared the FRS for 81 NFGMs and 573 ordinary ground motions. The results showed that NFGMs significantly increased the FRS of NSCs that were tuned to the supporting structure.

Most current studies on the influence of input ground motions on the FRS have primarily focused on vertical ground motions and near-fault earthquakes. More research is required on the FRS of building structures exposed to long-period earthquakes, long-duration earthquakes, and mainshock-aftershock earthquake sequences.

5 Experimental studies and field observations

Lepage *et al*. (2012) investigated the effects of floor acceleration amplification using shaking table test data of 30 multistory RC structures. The results indicated that the amplification effects were more pronounced in the upper floors and decreased with increasing earthquake intensity. Astroza *et al*. (2015) found that the story amplification factors of the roof of a full-scale five-story RC building were lower than the value of 3.0 suggested by ASCE 7-10. The component amplification factor a_p significantly exceeded the 2.5 limit value in ASCE 7-10 for periods near the vibration period of the supporting building. An accurate FRS of the supporting building is critical to achieving consistent and high-quality test results in shaking table tests of NSCs. Different control methods have been proposed for deriving the FRS, including the work by Maddaloni *et al*. (2011) and Zhou *et al*. (2019). Floor acceleration responses are usually reproduced based on these control methods and a loading frame. Experimental tests have been conducted on different types of NSCs, including suspended ceilings (e.g., Badillo-Almaraz *et al*., 2007; Gilani *et al*., 2010; Magliulo *et al*., 2012; Ryan *et al*., 2016), piping systems (e.g., Zaghi *et al*., 2012; Soroushian *et al*., 2014), and other components.

Strong motion seismographs installed in structures have recorded earthquake data, which provide important references for experiments and the calculation of FRS. Naeim *et al*. (1998) compared the FRS calculated using data from six instrumented buildings in the 1994 Northridge earthquake with the calculated seismic demands of NSCs based on codes such as UBC-97, NEHRP-97, and FEMA-273. The results indicated that the acceleration demands on the roof exceeded the values recommended by these codes and guidelines. Acceleration data from eleven instrumented buildings in Taiwan were compared to modern code provisions by Assi *et al*. (2005). It was demonstrated that the code provisions ($1+2z/h$) might provide under- or over-estimation on the horizontal PFA of different buildings. Similar results of horizontal PFA estimations were reported by Qu *et al*. (2014). Assi *et al*. (2017) found that the vertical PFA/PGA values increased as the number of building stories increased but converged to 2.0. Wang *et al*. (2014) suggested that the code provisions ($1+2z/h$) overestimated the PFA demands, especially when the building was in a strongly nonlinear state. Floor acceleration data measured in seven instrumented buildings were used by Lepage *et al*. (2012) to investigate the effects of floor acceleration amplification. Similar results to those obtained from shaking table tests were found. A similar evaluation of the design codes was conducted by Fathali and Lizundia (2011) using data recorded in 151 fixed-base buildings. Acceleration data from 59 buildings were used by Anajafi and Medina (2018c, 2019a) to evaluate the ASCE 7 equations for the seismic design of acceleration-sensitive NSCs. It was found that, unlike the approach in ASCE 7-16 (2016), the component amplification factor, a_p, is a function of the ratio of the NSC period to the modal periods of the supporting building, the ground motion intensity level (i.e., building inelastic behavior), and the relative height of the point of attachment of NSC to the supporting building. It was also shown that the expression provided by ASCE 7-16 for the story amplification factor ($PFA/PGA = 1+2z/h$) in many cases overestimates the floor acceleration responses of inelastic structures. Wang *et al*. (2014) and Kazantzi *et al*. (2018) also indicated that the a_p values in ASCE 7-16 (2016) do not necessarily provide an adequate estimation of the NSC seismic demand using recorded floor motion data.

6 Case study

A series of 4-, 8-, and 12-story strong column-weak beam RC frames described in Wang *et al*. (2020) are used as prototypes in this study to evaluate and compare several methods mentioned above. A THA was conducted for a set of 20 earthquake records to obtain the FRS. The motion set consisted of 15 recorded ground motions selected from the PEER-NGA Database and five artificial ground motions. The acceleration response spectra of the selected ground motions are presented in Fig. 10. The acceleration at a period of 0 s is 0.2 g, which corresponds to the design-basis earthquake (DBE) in GB 50011-2010 (2010). The median spectrum for this set of ground motions is comparable in shape to the GB 50011-2010 (2010) design response spectrum for a type III site (the characteristic period T_g is 0.45 s) in the Intensity VIII region in China. The PGA of the ground motions was scaled to 0.07 g to conduct a THA for the elastic responses and 0.20 g for the inelastic responses.

6.1 Comparison of different design codes

Considering the fact that the current design codes for NSCs usually do not consider the nonlinear responses of supporting structures explicitly, the elastic structure responses are discussed in this subsection. The PFA/PGA values of the results from the THA (mean values, mean values plus or minus one standard deviation) were compared with the values of different design codes, as shown in Figs. 11(a)–11(c). The prediction accuracy of the current design codes are considerably different for the three structures. For the 4-story structure, the predicted results from GB 50011-2010 (2010), Eurocode 8 (CEN, 2004), and ASCE 7-16 (2016) are less than the mean values of THA, whereas the prediction from NZS 1170.5 (2004) has higher accuracy. However, the predictions from NZS 1170.5 (2004) for the 8-story and 12-story structures are relatively conservative.

The prediction results for the top-floor FRS are compared with the THA results in Figs. 12(a)–12(c). It is found that the predictions from the current design codes are markedly different from the THA results. The difference may be the result of the inaccurate definition of the dynamic component amplification factor and the story amplification factor, as well as the non-consideration of the higher mode effects.

6.2 Comparison of different methods

The PFA/PGA values calculated using different methods were compared with the THA results of elastic

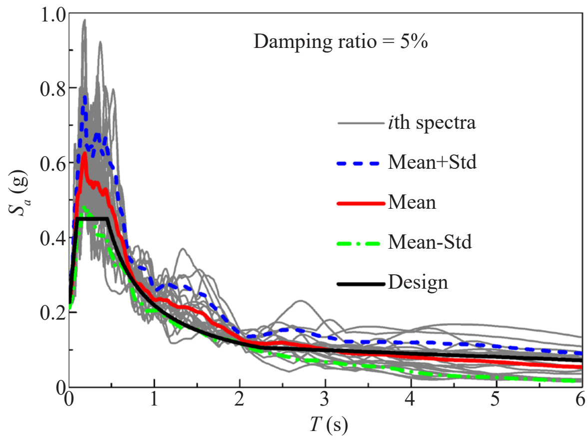


Fig. 10 Response spectra of the selected ground motion records

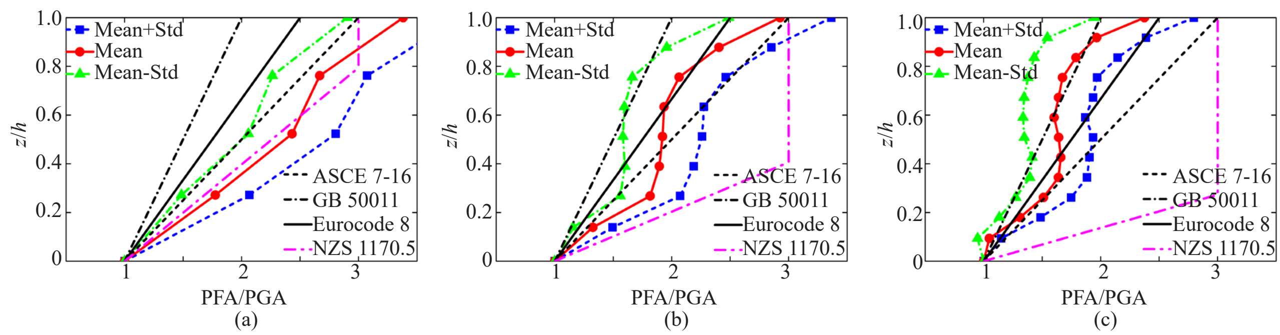


Fig. 11 Comparison of different definitions of the PFA/PGA values in different design codes; (a) 4-story, (b) 8-story, (c) 12-story

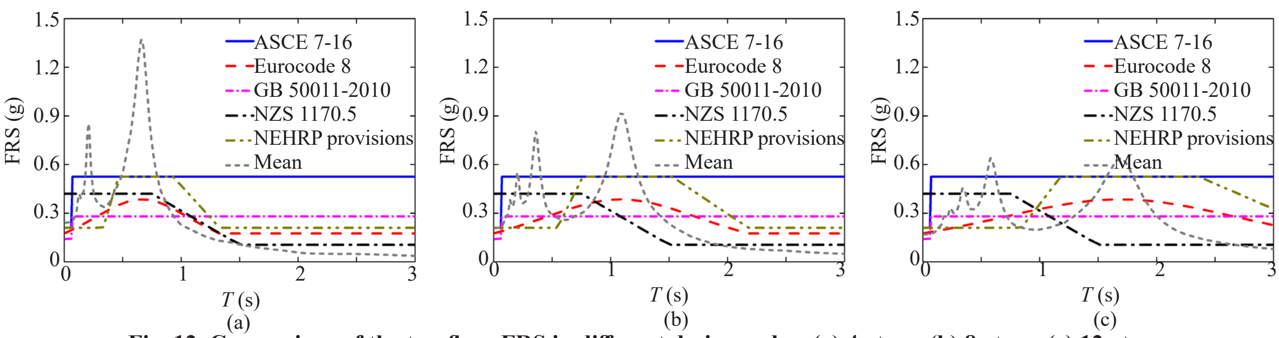


Fig. 12 Comparison of the top-floor FRS in different design codes; (a) 4-story, (b) 8-story, (c) 12-story

structure responses (Fig. 13(a)–13(c)) and inelastic structure responses (Fig. 13(d)–13(f)). The calculated values exhibit considerable differences. For the elastic structure responses, the method of Vukobratović and Fajfar (2017) predicts the PFA/PGA values more accurately than the other methods as the predicted values and shapes of the PFA/PGA at different heights accurately capture the mean values of THA results. The primary reason is that the method by Vukobratović and Fajfar (2017) considers the contribution of different vibration modes, whereas the other methods usually use a linear distribution (e.g., Akhlaghi and Moghadam, 2008) or directly define the shape of the PFA/PGA distribution (e.g., Fathali and Lizundia, 2012). The PFA/PGA responses for the inelastic structure are significantly lower than that of the elastic structure. The predicted results from various methods are quite different. Moreover, the accuracy is not consistent if the same method is applied to different buildings. For instance, the method proposed by Wieser *et al*. (2013) provides relatively accurate predictions for the 8-story and 12-story inelastic structures, but the predictions for the 4-story structures are too conservative.

The top-floor FRS are calculated for linear NSCs with a 5% damping ratio using the elastic and inelastic structure responses (mean values of THA, as depicted in Fig. 14). Large differences in the top-floor FRS are observed between these methods. For the elastic structure responses, the directly defined FRS by Singh *et al*. (2006b) captures the peak values of the FRS, while the rest underestimated them. However, the method by Singh *et al*. (2006b) results in a conservative prediction of FRS with a period between two adjacent vibration mode periods, where, on the contrary, the methods based on modal superposition (e.g., Calvi and Sullivan, 2014b and Vukobratović and Fajfar, 2017) provided more accurate predictions of FRS. Note that it is the difference in the design spectra and the mean spectra results in the underestimation of most direct methods. For the inelastic structure responses, the FRS generated using different methods also differ considerably. The directly defined FRS usually results in a more conservative estimate. However, the methods of Wieser *et al*. (2013), Vukobratović and Fajfar (2017), and Surana *et al*. (2018a) provide relatively acceptable predictions.

7 Summary

Past earthquake reconnaissance shows that damage to NSCs may disrupt the functionality of buildings and result in significant economic losses, injuries, and casualties. It is crucial to clarify the mechanical behavior, the damage development, and the resulting response of NSCs to determine the seismic performance of NSCs and develop appropriate designs. As a first step, many attempts have been made to investigate the NSC inputs, specifically, the FRS, which is often used to determine seismic force demands on NSCs. It is well known that the two most recent reviews on FRS were conducted about 20 years ago by Chen and Soong (1988) and Villaverde (1997). Since then, many theories and methods have been developed, and updates to seismic design codes were made. Therefore, it is necessary to summarize the progress made in the past 20 years and identify major knowledge gaps for better understanding, research, and application of FRS.

Many critical factors influencing FRS have been

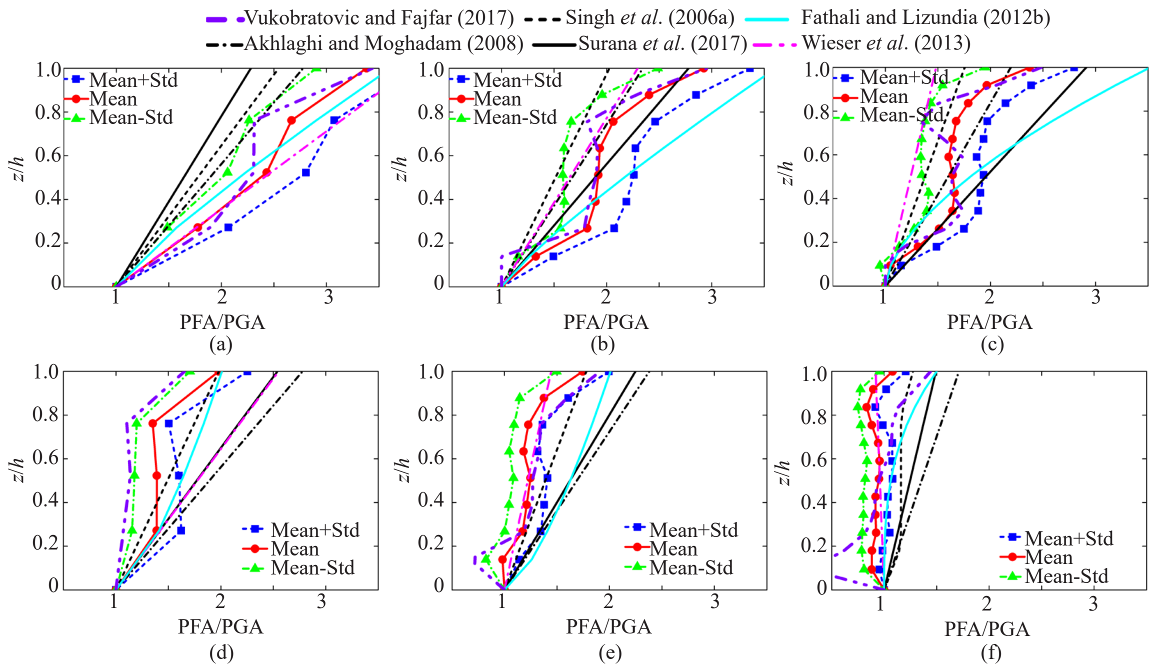


Fig. 13 Comparison of the results of different definitions of the PFA/PGA values; (a) 4-story elastic, (b) 8-story elastic, (c) 12-story elastic, (d) 4-story inelastic, (e) 8-story inelastic, (f) 12-story inelastic

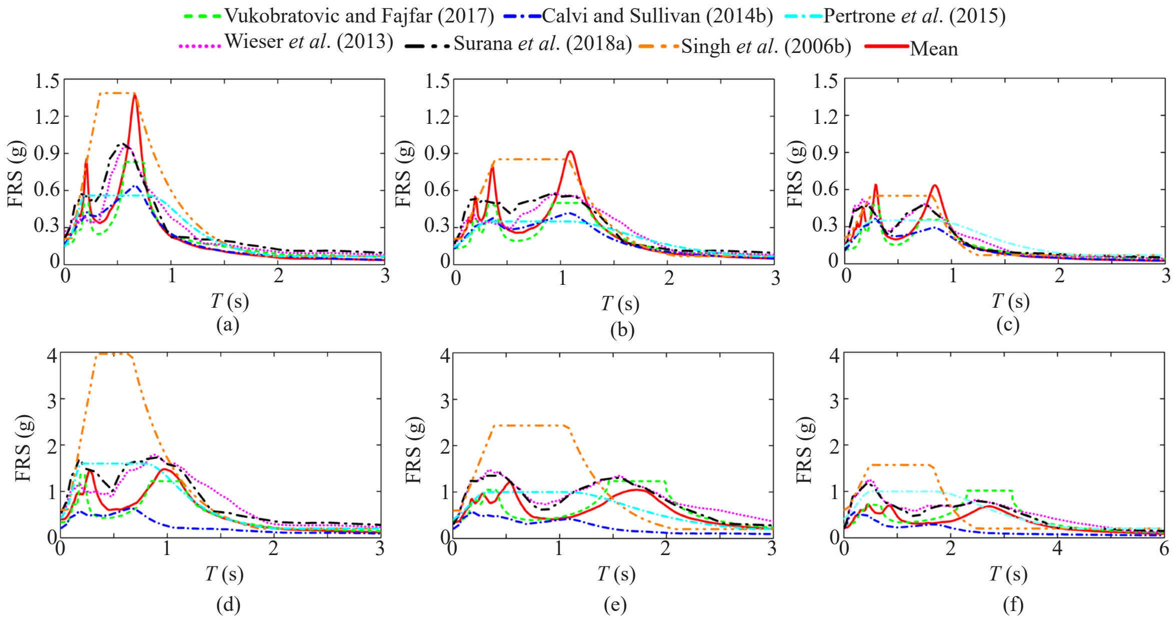


Fig. 14 Comparison of the top-floor FRS using different methods; (a) 4-story elastic, (b) 8-story elastic, (c) 12-story elastic, (d) 4-story inelastic, (e) 8-story inelastic, (f) 12-story inelastic

investigated, such as the location of the NSCs, the damping ratio of the NSCs, the interaction between NSC and structural components, the nonlinear behavior of the supporting structures and NSCs, the SSI effects, and the floor flexibility in the vertical direction and in-plane direction. The effects of input ground motions (especially vertical ground motions and near-fault earthquakes) on the FRS were also extensively investigated. Note that these factors are usually not independent. For example, the dynamics of a structure are usually affected significantly by the damping, most of which is provided by the nonlinear behavior of structures. Methods that consider either the structural and nonstructural responses do not provide an accurate prediction of the FRS. However, a combined model that considers both is challenging to solve efficiently due to the convergence problem. Therefore, although a general and more accurate FRS generation method that incorporates these influence factors will facilitate the seismic design of NSCs, more research is required to develop such a method.

Although numerous procedures have been proposed over the last few years, current building codes still do not reflect this knowledge and have not yet incorporated these procedures. One of the reasons might be the uncertainties involved in the generation methods of the FRS, which will lead to large discrepancies in the FRS and PFA demands. Some researchers have noticed this issue and tried to solve the problem. Inspired by the work of Clayton and Medina (2012), Lucchini *et al*. (2016) proposed a probabilistic seismic demand model (PSDM) to determine inter-story drifts and the FRS. Based on the PSDM, a method for computing uniform hazard floor acceleration spectra for linear NSCs attached to linear buildings was proposed (Lucchini *et al*., 2017a). Epistemic uncertainties in the modal properties of the supporting structure and aleatory uncertainties caused by input ground motions were subsequently investigated by Lucchini *et al*. (2017b). Uncertainties caused by material properties, building geometry, and live loads were considered by Perrone *et al*. (2020) to evaluate the FRS. However, there is currently no single method that considers all or even most uncertainties. Therefore, it remains challenging for researchers to identify the uncertainties associated with determining the FRS. Potential uncertainties include the type of ground motions, the SSI effects, the material properties of the building, NSC deployment, and modeling related uncertainties. The aleatory and epistemic uncertainties should be considered with different exceedance probabilities in generating FRS so that the seismic design of NSCs can be incorporated into the current performance-based design procedures of buildings.

In addition, the FRS is structure-dependent, unlike the design acceleration spectra used for structures. That is, the FRS of buildings with different seismic resisting systems, e.g., long-span structures, passively controlled structures, base-isolated structures, and super high-rise buildings, have significantly different characteristics. Pavlou and Constantinou (2006) found that the implementation of linear and nonlinear viscous dampers significantly reduced the FRS. Chalarca *et al*. (2019) confirmed that in most cases, the results of Pavlou and Constantinou (2006) are appropriate for viscous damping structures. However, they also found that the acceleration demand varied when the damping ratio and the velocity coefficient of the viscous dampers changed. The acceleration demand can exceed that of pure frames in some cases. Anajafi and Medina (2019a) also suggested that the FRS strongly depend on the type of seismic resisting systems. Methods for determining the FRS need to consider these differences caused by the type of seismic resisting systems. More importantly, potential methods should also be able to discretize the degree of importance of the influential factors mentioned above for different types of buildings.

Another major research gap exists in the type of FRS. To date, most FRS have been referred to as floor acceleration response spectra. However, as pointed out by Filiatrault *et al*. (2018), many acceleration-sensitive NSCs, for instance, suspended piping systems, were damaged by excessive displacement relative to the point of support on the structure. The floor acceleration response spectra solely do not provide sufficient input for all NSCs. Moreover, the floor acceleration response spectra inherently reflect the intensity and frequency of the components used as input into an NSC but cannot predict the input energy demand. It has already been observed in recent earthquakes that NSC damage was caused by long-duration shaking at relatively small intensities. It is gratifying that some studies have already started to generate relative displacement FRS or displacement demand used in NSC design (e.g., Calvi, 2014; Filiatrault *et al*., 2018; Obando and Lopez-Garcia, 2018; Merino *et al*., 2020). Much future work is required to conduct comprehensive studies on the absolute acceleration, relative velocity, and relative displacement FRS.

Finally, FRS research based on data obtained during earthquakes is still limited. One reason is that the number of seismographs installed in building structures is still small, and another is a lack of different types of instrumented structures. However, floor response data obtained from instrumented buildings in real earthquakes will be highly useful for comparing the generated FRS with the recorded FRS and calibrating the generation method (a few sample studies were conducted by Fathali and Lizundia, 2011; Anajafi and Medina, 2018c, 2019a). Recently, the China Earthquake Administration has initiated a major project called the "China Seismic Experimental Site," in which a large number of buildings with distinct dynamic characteristics and situated in seismic-prone areas will be instrumented. The recorded strong ground motions are believed to provide insights into the FRS.

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