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# Evaluation of seismic design provisions for acceleration-sensitive non-structural components

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**Abstract.** A set of mid-rise bare and uniformly infilled reinforced-concrete frame buildings are analyzed for two different seismic intensities of ground-motions (i.e., 'Design Basis Earthquake' and 'Maximum Considered Earthquake') to study their floor response. The crucial parameters affecting seismic design force for acceleration-sensitive non-structural components are studied and compared with the guidelines of the European and the United States standards, and also with the recently developed NIST provisions. It is observed that the provisions of both the European and the United States standards do not account for the effects of the period of vibration of the supporting structure and seismic intensity of ground-motions and thereby provides conservative estimates of the in-structure amplification. In case of bare frames, the herein derived component amplification factors for both the design basis earthquake and the maximum considered earthquake exceeds with their recommended values in the European and the United States standards for non-structural components having periods in vicinity of the higher modes of vibration, whereas, in case of infilled frames, component amplification factors exceeds with their recommended value in the European standard for non-structural components having periods in vicinity of the fundamental mode of vibration, and only for the design basis earthquake. As a consequence of these observations, as well as capping on the design force (in case of United states standard and NIST provisions), in case of the design basis earthquake, the combined amplification factor is underestimated for non-structural components having periods in vicinity of the higher modes of vibration of bare frames, and also for non-structural components having periods in vicinity of the fundamental mode of vibration of infilled frames. At the maximum considered earthquake demand, excepting non-structural components having periods in vicinity of the higher modes of vibration of bare frames, all provisions generally provide conservative estimates of the design floor accelerations.

**Keywords:** RC frame buildings; seismic design; non-structural components; component amplification; in-structure amplification

## 1. Introduction

Seismic design provisions for buildings and non-structural components (NSCs) are undergoing rapid development in the recent years. Post-earthquake reconnaissance mainly in the United States as well as in the other developed nations following some of the past earthquakes, i.e., Loma Prieta, 1989 (e.g., Rihal 1992), Northridge, 1994 (e.g., Reitherman and Sabol 1995), Chile, 2010 (Miranda *et al.* 2012), Darfield, 2010 (Dhakal 2010), Christchurch, 2011 (Baird *et al.* 2014), Emilia, 2012 (Magliulo *et al.* 2014) has revealed that the majority of losses (often expressed in terms of 3D's i.e., deaths, dollars, and downtime) in building structures are due to the either direct or indirect consequences of damage to NSCs. These post-earthquake assessments have shown that the supporting structures (the term "supporting structure" refers to "building superstructure" to which NSC is attached) designed following the modern seismic design codes and provisions are successfully able to limit the structural damage during severe earthquakes, but at the same time, the seismic damage to NSCs can be extensive, very costly and sometimes even life threatening (e.g., falling hazards, blockage of the emergency exits) to the building occupants (Sullivan *et al.* 2013). The another important factor contributing to the requirements for adequate safety of NSCs is associated with the cost of the NSC itself, which can be as high as 80-90% of the total supporting structure cost (Taghavi and Miranda 2003) in case of important buildings (e.g., office and hospital buildings). Furthermore, the damage to NSCs can also result in business interruptions and downtime losses. On the basis of the aforementioned potential consequences of a major earthquake, it is widely accepted and recognized world-over to develop seismic design provisions which reliably estimate the floor acceleration response of NSCs to prevent damage during earthquake shaking and thereby ensure the supporting structure's functionality in the aftermath.

Based on sensitivity of the seismic response of NSCs, they can be classified under three different categories: (i) acceleration-sensitive NSCs (e.g., parapets and suspended ceilings), (ii) drift-sensitive NSCs (e.g., windows and elevator cabins), and (iii) combined acceleration- and drift-sensitive NSCs (e.g., infill walls). Acceleration-sensitive NSCs are sometimes also referred as 'lumped systems' whose points of attachments are subjected to uniform excitation. In such cases, the dynamic excitation problem

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can be significantly simplified and simulated by considering the NSC as an equivalent single-degree-of-freedom (SDOF) system. Contrarily, the drift-sensitive NSCs are usually attached at multiple points along the height of the supporting structure. Therefore, the relative displacements occurring between successive attachment points generate the internal forces in this type of NSCs. Some NSCs which possess the characteristics of both acceleration- and drift-sensitive components fall under the category of combined acceleration- and drift-sensitive NSCs. Infill walls can be considered as a classical example of combined acceleration- and drift-sensitive NSC, which are sensitive to floor accelerations in the out-of-plane direction of excitation, whereas relative floor displacements in the in-plane direction of excitation.

This article evaluates the adequacy of the existing seismic provisions related to design of acceleration-sensitive NSCs of three different seismic design codes/standards i.e., EN 1998 (2004), ASCE 7-16 (2016) and NIST (2018). A set of mid-rise bare and uniformly infilled reinforced-concrete (RC) frame archetypes are developed and analyzed for a suite of far-field ground-motion records using nonlinear dynamic analyses and the effect of seismic intensities of the ground-motions (i.e., 10% probability of exceedance in 50 years, hereafter referred as 'Design Basis Earthquake', DBE, and 2% probability of exceedance in 50 years, hereafter referred as 'Maximum Considered Earthquake', MCE) on the floor acceleration response and design force on NSC is studied. A total of 352 peak floor acceleration (PFA) amplification profiles and 1,408 5%-damped floor spectra are obtained at the centre of mass of different floor levels of the considered bare and uniformly infilled RC frame supporting structures. The obtained floor acceleration response from the nonlinear dynamic analyses is compared with the recommendations of the considered design codes and standards for in-structure amplification factor, component amplification factor, and combined in-structure and component amplification factor. Major shortcomings of the existing provisions of the considered codes/standards for seismic design of NSCs are highlighted.

## 2. Background and past studies

A significant number of research attempts (e.g., Taghavi and Miranda 2005, Villaverde 2006, Singh *et al.* 2006a, b, Clayton and Medina 2012, Calvi and Sullivan 2014, Lucchini *et al.* 2014, Petrone *et al.* 2015, Petrone *et al.* 2016, Pan *et al.* 2017a, b, Pan *et al.* 2018) have been made in the past two decades to study floor acceleration demand on NSCs attached to regular buildings and also to assess the adequacy of the code provisions for seismic design of NSCs. A comprehensive state-of-the-art on issues related to seismic design of NSCs is also available in earlier studies (e.g., Villaverde 1997, Filiatrault and Sullivan 2014).

Past studies (Chaudhuri and Villaverde 2008, Chaudhuri and Hutchinson 2011, Weiser *et al.* 2013, Surana *et al.* 2017) identified that the fundamental period of the supporting structure plays a crucial role in estimating the 'in-structure amplification' (defined as the ratio of PFA to peak ground acceleration, PGA) and in general, it reduces with an increase in the fundamental period and inelasticity of the supporting structure. The in-structure amplification further showed a distinct characteristic in mid- and high-rise buildings, in the form of a sudden increase in its value at floors close to the roof level, due to the whiplashing effect of higher modes (Singh *et al.* 2006a).

A number of studies identified the role of dynamic characteristics of the supporting structure, and observed sharp peaks in the floor response spectra (FRS) corresponding to the modal periods of the supporting structure (e.g., Rodriguez *et al.* 2002, Medina *et al.* 2006). These peaks in FRS were further observed to be dependent on the damping ratio of NSCs, and a lower amount of damping in NSC resulted in relatively sharper peaks in FRS (Sankaranarayanan and Medina 2007). It has already been established in the literature (e.g., Lin and Mahin 1985, Vukobratović and Fajfar 2015, Vukobratović and Fajfar 2016) that the amplification of the peaks in FRS reduces with an increase in the inelasticity (ductility demand) of the supporting structure.

Earlier studies (Jiang *et al.* 2015, Surana *et al.* 2018a, b) also identified the influence of the ground-motion characteristics and highlighted that the FRS are amplified ground-motion spectra and therefore can be obtained directly from the ground response spectra, if the supporting structure's dynamic characteristics (periods and mode shapes) and the inelasticity levels (ductility demand) are known (Surana *et al.* 2018a, b). Further, the generation of FRS directly from GRS resulted in the least variability while compared with either the PFA or the PGA (Surana *et al.* 2018a, b). Some of the recent studies (e.g., Lucchini *et al.* 2016, Lucchini *et al.* 2017) on evaluation of the floor acceleration demands aimed at developing probabilistic seismic demand models for considering the record-to-record variability of the floor response and also the models for predicting the floor acceleration demands even for buildings with the plan and elevation irregularities (Aldeka *et al.* 2014, Aldeka *et al.* 2015, Surana *et al.* 2018c). Recently, Filiatrault *et al.* (2018) proposed the concept of the displacement-based seismic design of NSCs.

As most of the earlier studies are mainly based on either steel or RC bare frames which have a significantly different dynamic behavior as compared to the URM infilled frames, mainly due to complex interaction between the adjoining frame and infills, in both the elastic as well as inelastic ranges. Despite being a significant number of existing studies reported earlier in this article, there exist critical gaps in the literature as well as in building codes related to the seismic design of NSCs namely: (i) the scarcity of the studies related to design of NSCs mounted on un-reinforced masonry (URM) infilled frames (which is a most common building typology in Europe, India and also in some other parts of the world), and (ii) the adequacy of the recently developed NIST provisions for different structural systems and seismic intensities of ground-motions.

Accordingly, in the present study, a comparative assessment of seismic design provisions of the existing codes (EN 1998 2004, ASCE 7-16 2016) and standards (NIST 2018) for design of acceleration-sensitive NSCs attached to bare and uniformly infilled RC frames is presented.

## 3. Provisions for seismic design of NSCs in the considered codes/standards

Seismic design of NSCs in most of the building design codes (e.g., ASCE 7-16 and EN 1998) is based on the consideration of a linear variation of the PFA along the height of the supporting structure. It is principally based on the assumption that seismic response of the supporting structure is dominated by the fundamental mode of vibration, which is a reasonable assumption, in case of low- and mid-rise regular supporting structures. The peak values of in-structure amplification recommended in ASCE 7 and EN 1998 are 3.0 and 2.5, respectively. To consider the effect of the frequency tuning between the supporting structure and NSCs, building codes recommend a component amplification factor, $a_p$, defined as the ratio of the peak component acceleration (PCA) to the PFA. In case of ASCE 7-16, two different values of $a_p$ equal to 1.0 and 2.5 are recommended for rigid and flexible NSCs, respectively. Further, an NSC is classified as rigid NSC, if it has a period of vibration less than or equal to 0.06 s, else, it is classified as flexible NSC. Contrary to ASCE 7, a more convincing model of $a_p$ is defined in EN 1998, where a parabolic variation in terms of the normalized period, $T/T_1$, (where $T$ is the period of vibration of the NSC and $T_1$ is the fundamental period of the supporting structure) is considered. The EN 1998 model shows a peak corresponding to $T/T_1$ equal to unity, and the value of $a_p$ reduces parabolically for NSCs having periods different than the fundamental period of the supporting structure. EN 1998 recommends a peak value of $a_p$ equal to 2.2, at the roof level. For seismic design of NSCs, both the in-structure amplification and component amplification are multiplied together, which results in a peak combined amplification (at the roof level) of 5.5 times PGA in case of EN 1998, and 4 times PGA in case of ASCE 7 (as the provisions in ASCE 7 are based on the elastic response of the supporting structure, therefore, the maximum design force is capped, to account for the inelastic response of the supporting structure).

One of the crucial steps in the development of code provisions related to design of NSCs is the availability of the recorded floor motions under past earthquakes with varying ground-motion intensities. The earlier developed code provisions (e.g., ASCE 7-16) for in-structure amplification are mostly based on very limited recorded floor motions on supporting structures under frequent earthquakes, thereby, most of the supporting structures exhibited the elastic response. Furthermore, the component amplification factor recommended in design codes (e.g., ASCE 7-16) is also based on the peak value of FRS and do not account for the inherent damping of NSC, which is relatively lower (e.g., Medina *et al.* 2006) than the assumed viscous damping of 5% in seismic design of buildings and NSCs. Therefore, to upgrade the existing seismic design provisions for NSCs in United States building code, NIST (2018) conducted a detailed study based on the recorded floor motions in 44 instrumented buildings which experienced earthquakes with PGA>0.15g (Anajafi and Medina 2018, Anajafi and Medina 2019). NIST (2018) identified twelve different influential parameters (i.e., ground shaking intensity, seismic force resisting system of the supporting structure, supporting structure's modal periods, supporting structure ductility, inherent supporting structure damping, supporting structure configuration, floor diaphragm rigidity, vertical location of the component within the supporting structure, component period, inherent component damping, component ductility, and component over-strength) affecting the force demand on NSCs. Based on the above identified parameters, NIST (2018) proposed Eqs. (1)-(3) to estimate the in-structure amplification

$$\frac{PFA}{PGA} = 1 + a_1\left(\frac{Z}{H}\right) + a_2\left(\frac{Z}{H}\right)^{10} \quad (1)$$

$$a_1 = \frac{1}{T_{aBldg}} \le 2.5 \quad (2)$$

$$a_2 = \left[1 - \left(0.4\big/T_{aBldg}\right)^2\right] > 0 \quad (3)$$

where, $a_1$ and $a_2$ are the constants which are computed from $T_{aBldg}$ (which is an empirical estimate of the fundamental period of the supporting structure) based on Eqs. (12.8-7) of ASCE 7-16. It can be observed that the Eq. (1) account for the effects of the period of vibration of the supporting structure, and also for the whiplashing effect of higher modes, by including a higher order term. To also account for the inelastic response (global ductility of the supporting structure) of the supporting structure, NIST (2018) recommended a reduction in the in-structure amplification by a factor, $R_{\mu Bldg}$ as defined in Eq. (4).

$$R_{\mu Bldg} = \sqrt{1.1\frac{q}{\Omega}} \ge 1.0 \quad (4)$$

where, $q$ is the behavior factor (also known as response reduction factor), and $\Omega$ is the over-strength factor (e.g., ASCE 7). Over and above the in-structure amplification, NIST (2018) recommended $a_p$ values of 2.5 and 4.0 for the flexible NSCs, at the ground and at the roof levels, respectively. Furthermore, NIST (2018) increased the design force capping limit to 5 times of PGA, in contrast to 4 times of PGA, as recommended in ASCE 7-16 (2016).

## 4. Numerical study

For the present study, two different structural systems i.e., bare and uniformly infilled RC frame supporting structures are considered. The building plan was chosen from a field survey to consider the variety of characteristics of the building stock in the National Capital Region (NCR) of India (DEQ 2009). The details of the building plan are shown in Fig. 1. The heights of these buildings are considered as 4- and 8-storeys, representing the mid-rise building stock typical for the NCR of India. The storey height is taken as 3.3 m, consistent with the field observations (DEQ 2009). The thickness of URM infill walls is considered as 230 mm and 110 mm for exterior and interior walls, respectively. The compressive strength of masonry is taken as 4.1 MPa, considering the fair quality of

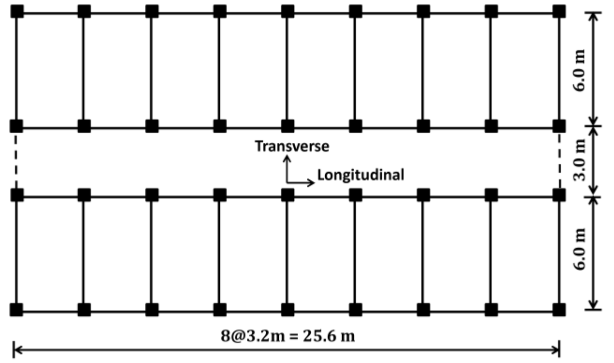


Fig. 1 Generic building plan chosen for the present study. The dashed lines in the floor plan represent the floor slab boundaries, which are assumed to be rigid in plane

Table 1 Periods of vibration of the considered supporting structures

| Building Model | Direction of Excitation | Period of vibration (s) | |
|---|---|---|---|
| | | $T_1$ | $T_2$ |
| 4I | Longitudinal | 0.40 | 0.14 |
| | Transverse | 0.54 | 0.19 |
| 8I | Longitudinal | 0.71 | 0.24 |
| | Transverse | 1.00 | 0.33 |
| 4B | Longitudinal | 1.21 | 0.39 |
| | Transverse | 1.77 | 0.52 |
| 8B | Longitudinal | 2.60 | 0.90 |
| | Transverse | 4.10 | 1.28 |

I-Infilled frame; B-Bare frame

masonry, also consistent with typical compressive strength values for solid clay brick masonry in Northern India (Kaushik *et al.* 2007, Haldar *et al.* 2013).

The buildings are modelled in the building analysis and design software ETABS (CSI 2016). Beams and columns are defined as 3D frame elements and slabs are considered as rigid diaphragms. The cracked section properties of beams and columns are derived following ASCE 41 (2013). Dead and live loads on the buildings are assigned according to IS 875 Part 1 (1987a) and IS 875 Part 2 (1987b), respectively. To model the URM infills, the eccentric strut model of ASCE 41 with modelling guidelines as per Burton and Deierlein (2014) are used. The initial (un-cracked) stiffness of the masonry infill wall is considered as twice of the stiffness obtained from the equivalent strut model of ASCE 41, as recommended by Burton and Deierlein (2014), based on experimental investigations on URM infill walls.

All the buildings are designed as Special Moment Resisting Frames (SMRF), following the Indian codes of practice (IS 1893 Part 1 2016, IS 13920 2016). The buildings are designed for sesimic actions corresponding to Indian seismic zone IV (Effective Peak Ground Acceleration=0.24 g), and assumed to be situated on soil type I (hard soil/rock). All the considered building models are designed for the strong-column weak-beam design criteria as per IS 13920 (2016). P-delta effects are also considered both in the analysis and design process. The periods of vibration corresponding to the first two contributing modes of vibration for the considered supporting structures, obtained from the modal analysis are reported in Table 1. It can be observed that the considered supporting structures have a wide range of the fundamental periods varying between 0.40-4.10 s.

To model inelastic behavior, a lumped-plasticity model is used for both beams and columns. Flexural (M3) hinges and interacting (P-M2-M3) hinges are assigned at both ends of beams and columns, respectively, and the corresponding force-deformation relationships are derived following ASCE 41 guidelines. These force-deformation parameters in ASCE 41 have been obtained from a cyclic envelope curve, and thereby include strength deterioration effects. To consider stiffness degradation under cyclic loading, an energy-based degrading hysteresis model has been used. The additional details about the chosen hysteretic model are available in earlier studies (Surana *et al.* 2018d). The shear failure of columns due to the strut action of the infills is modelled following ASCE 41 guidelines. Based on the experiments conducted on infill panels, Burton and Deierlein (2014) proposed a trilinear force-deformation curve for URM infill walls which also includes the post-peak behavior. In the present study, the backbone curve parameters for URM infill walls are adopted from Burton and Deierlein (2014). To compute the strength of infill walls, three different failure modes namely: (i) sliding shear, (ii) diagonal tension, and (iii) diagonal compression have been considered. In the present study, it is discovered that the strength of URM infill walls is minimum in sliding shear, and thereby the sliding shear failure mode governs the inelastic modelling of URM infills. This observation is in agreement with the earlier studies (e.g., Haldar *et al.* 2013) on infilled frames with similar aspect ratios and compressive strength of masonry.

## 5. Methodology

The estimation of the seismic response of NSCs based on the combined primary (supporting structure) and the secondary system (NSC) is termed as 'coupled' analysis, and it explicitly considers the full dynamic interaction between the supporting structure and the NSC. It has already been established in the earlier studies (e.g., Toro *et al.* 1989, Adam *et al.* 2013) that if the secondary system has negligible mass (e.g., by a factor of 1000 or more) compared to the primary system, in those cases, the dynamic interaction between the supporting structure and the NSC can be ignored. Therefore, the response of the supporting structure at any given floor is obtained independent of the NSC. As both the supporting structure and the NSC are treated independently, this approach is termed as 'de-coupled analysis' or 'Floor Response Spectrum' (FRS) method. However, it has been reported (e.g., Toro *et al.* 1989, Adam *et al.* 2013) that this approach may lead to overly conservative floor acceleration demands (particularly under tuned conditions) if the NSCs possess significant mass as compared to the supporting structure. In the present study, the FRS method has been used to estimate the floor acceleration demands and therefore, the

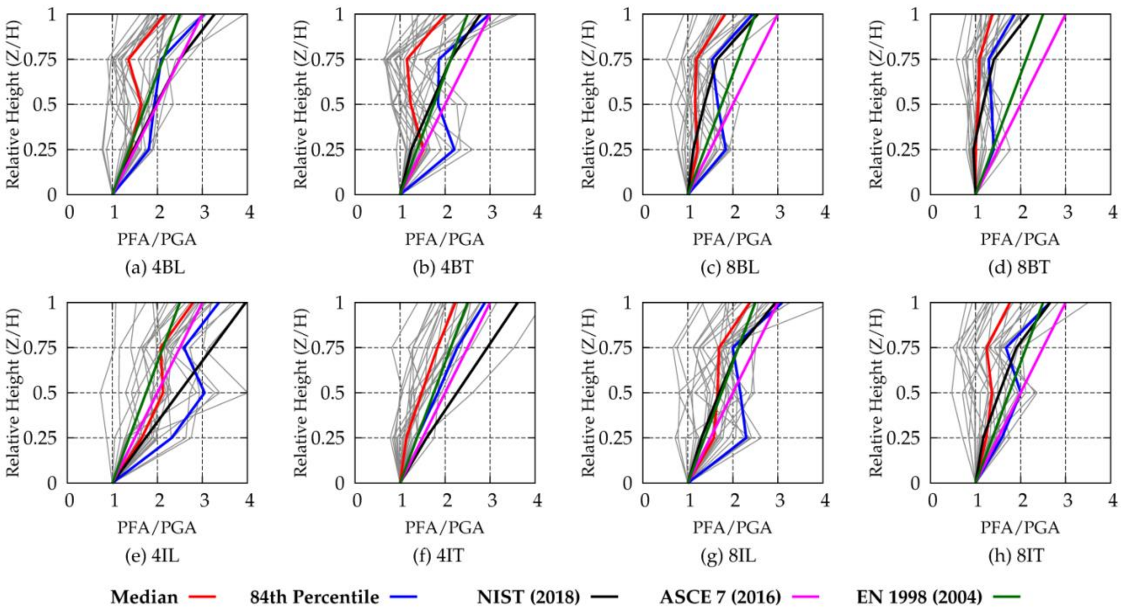


Fig. 2 Comparison of the in-structure amplification (PFA/PGA) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the design basis earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '*Z*' is the height of the floor level under consideration and '*H*' is the total height of the supporting structure measured above the base

observations and conclusions of the present study are strictly applicable to NSCs whose masses are negligible as compared to the mass of the supporting structure. However, it is a reasonable assumption in case of the most of NSCs.

To investigate the floor acceleration demand in the considered structural models, nonlinear dynamic analyses are conducted using a suite of 22 far-field ground-motion records as identified in FEMA P695 (2009). For each of the structural model, the major component of each ground-motion (i.e., the horizontal component with higher PGA) record is applied along both principal directions of the supporting structure, separately. The ground-motion is scaled to two different seismic intensities representative of DBE and MCE demands in seismic zone IV, based on the design response spectrum recommended in Indian Seismic Design Code (IS 1893 Part 1 2016). To conduct nonlinear dynamic analyses, a Rayleigh damping of 5% is assigned to the periods corresponding to the fundamental mode and the mode resulting in a cumulative mass participation of 95%, for the supporting structure, in the considered direction of excitation.

## 6. Results and discussions

### *6.1 Amplification of PFA*

In the present study, the nonlinear dynamic analyses have been conducted and the obtained results are presented for each of the ground-motion record and also summarized in the form of median and 84$^{th}$ percentile estimates. However, all the results have been discussed with respect to the median values obtained from the nonlinear dynamic analyses. Figs. 2 and 3 present a comparison of the in-structure amplification (also called as PFA amplification) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records with the recommended provisions of NIST, ASCE 7 and EN 1998 for the investigated bare and uniformly infilled RC frames, for the DBE and the MCE demands, respectively. In case of bare RC frames subjected to the DBE demand, the current code (ASCE 7 and EN 1998) provisions for in-structure amplification have been observed to be conservative (Fig. 2(a)-(d)). The observed conservatism of ASCE 7 and EN 1998 models for bare frames increases with an increase in the fundamental period (Fig. 2(a)-(d)) and ductility demand (Fig. 2(a)-(d) and Fig. 3(a)-(d)) of the supporting structure. Contrary to the current code models, NIST provisions resulted reasonable estimates of the in-structure amplification factor (Fig. 2), except at the floors in the lower quarter (Fig. 3(a)-(d)), particularly for the MCE demand.

In case of the infilled RC frames, provisions of current codes (ASCE 7 and EN 1998) for in-structure amplification have been observed to be reasonable, whereas NIST provisions have been observed to be conservative, for the DBE demand (Fig. 2(e)-(h)). On the other hand, for the MCE demand, the current codes provisions for in-structure amplification become conservative, whereas NIST provisions become more reasonable (Fig. 3(e)-(h)), except at the floors in the lower quarter. The observed conservatism of the current code models for in-structure amplification in case of bare frames (for both the DBE and the MCE demands) and also for infilled frames (for the

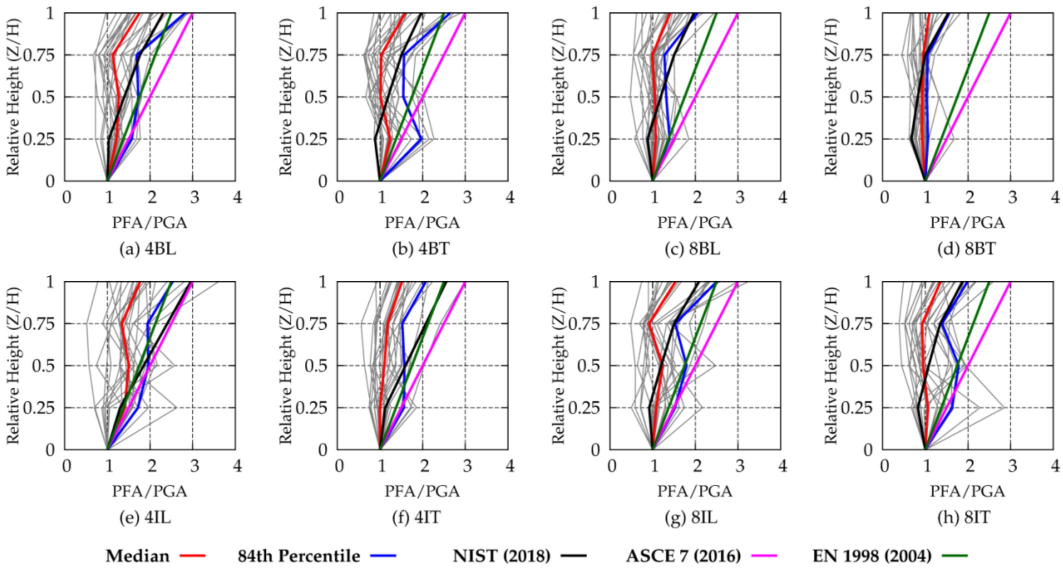


Fig. 3 Comparison of the in-structure amplification (PFA/PGA) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the maximum considered earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '*Z*' is the height of the floor level under consideration and '*H*' is the total height of the supporting structure measured above the base

MCE demand) can be attributed to the facts that both ASCE 7 and EN 1998 models for PFA amplification do not account for the effects of reduction in PFA amplification due to increased period of vibration and inelasticity of the supporting structure.

It is interesting to note that the variation of the median PFA amplification along the height of the supporting structures are significantly different for bare frames (Fig. 2(a)-(d)) as compared to infilled frames (Fig. 2(e)-(h)) for the DBE demand, whereas, very similar for the MCE demand (Fig. 3(a)-(d) and Fig. 3(e)-(h)). In case of the bare frames, the median PFA amplification is almost constant in the lower three quarter of the supporting structure, for both the DBE and the MCE demands, whereas in case of the infilled frames, the variation of the median PFA amplification is almost linearly increasing for the DBE demand, and almost constant in the lower three quarter for the MCE demand. The presented observations can be explained by the fact that at the MCE level of seismic demand, failure of most of the infill walls occurs, which result in a significant change in the dynamic characteristics of the infilled frame supporting structures, and the behavior of the infilled frames changed to bare frames. Another interesting observation related to variation in the PFA amplification is a sudden increase in PFA amplification at the roof level (Figs. 2 and 3). This observation can be attributed to the whiplashing effect of higher modes. As evidenced from Figs. 2 and 3, both the European and the United States codes do not account for this effect explicitly, however, to some extent this effect is included in NIST provisions, but it is mostly visible in case of bare frames and not visible in case of infilled frames. This observation can be attributed to the fact that the infilled supporting structures are relatively rigid, and in these cases the factor '$a_2$' (Eq. 1) becomes negative, and therefore, lower bound value of factor '$a_2$' (equal to zero) controls the PFA amplification, and diminishes the whiplashing effect of higher modes in estimation of PFA amplification. The presented observations highlight that the current code provisions of EN 1998 and ASCE 7 for in-structure amplification needs to be upgraded to account for the effects of the fundamental period of vibration of the supporting structure, seismic intensity of the ground-motions (or inelasticity of the supporting structure), and whiplashing effects of higher modes.

### *6.2 Component amplification factor*

Figs. 4 and 5 present a comparison of the component amplification factor ($a_p$) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records with the recommended provisions of NIST, ASCE 7 and EN 1998 for the investigated bare and uniformly infilled RC frames for the DBE and the MCE demands, respectively. It is to be noted here that NIST recommends $a_p$ values of 2.50 and 4.0 at the ground and at the roof level, respectively. Therefore, for computing $a_p$ value at the mid-height of the supporting structures, based on NIST provisions (Figs. 4 and 5), a linear interpolation has been used.

It can be observed that for both the DBE and the MCE demands, the recommended value of $a_p$ in both EN 1998 and ASCE 7 is non-conservative for periods in vicinity of the higher modes of vibration of bare frames (Fig 4(a)-(d), 4(i)-(l), and Fig 5(a)-(d), 5(i)-(l)). Further, the observed non-conservatism in $a_p$ value is significant for EN 1998 and slight for ASCE 7. For NSCs having periods in vicinity of

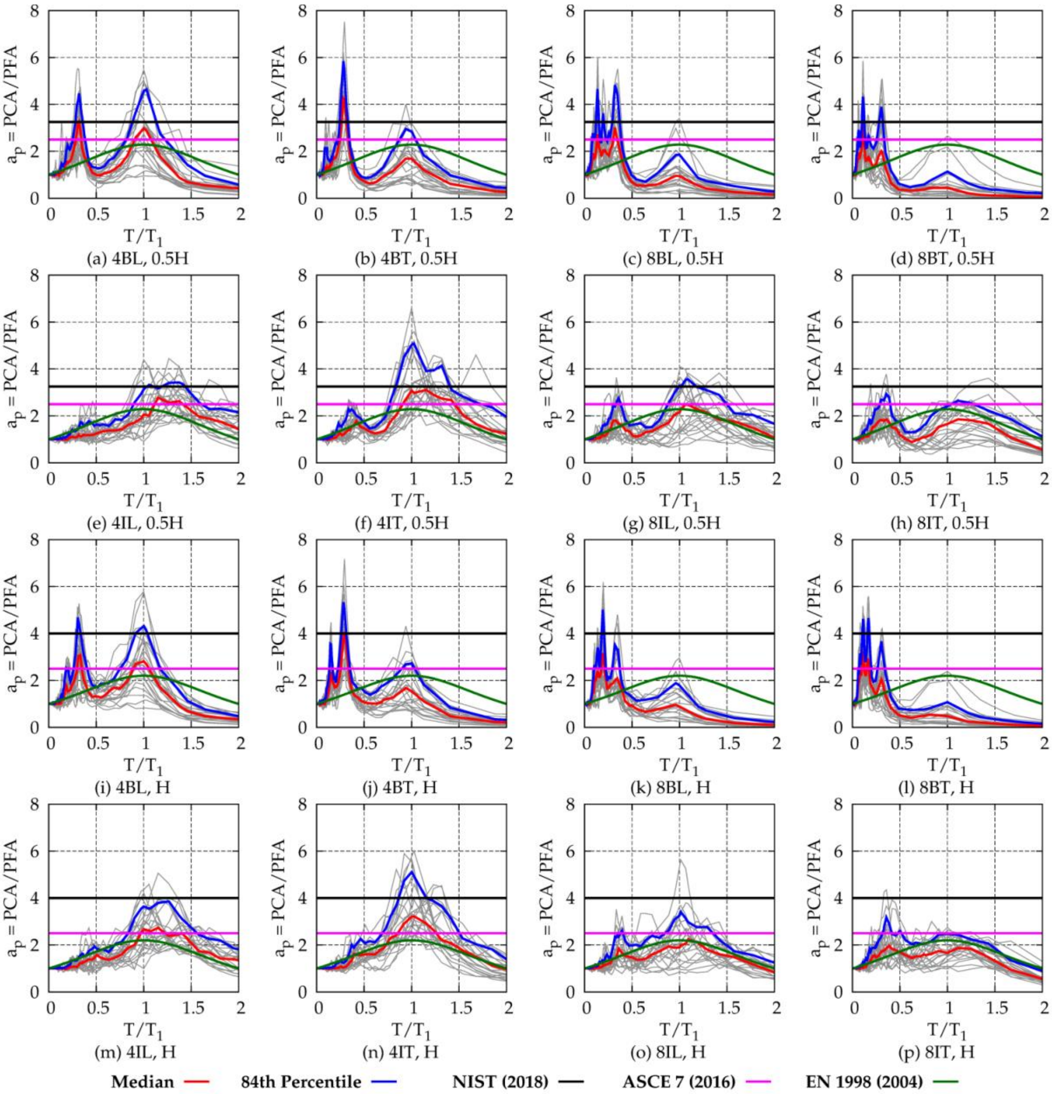


Fig. 4 Comparison of the component amplification factor ($a_p$) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the design basis earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '0.5*H*' and '*H*' corresponds that the derived results are for the mid-height and the roof level, respectively

the fundamental mode of vibration of the bare frames, the $a_p$ value recommended in both EN 1998 and ASCE 7 are significantly conservative (Fig. 4(b)-(d), 4(j)-(l), and Fig. 5(a)-(d), 5(i)-(l)) in cases of both the DBE and the MCE demands, excepting the case of 4-storey building (Fig. 4(a), 4(i)), subjected to the DBE demand. On the other hand, the $a_p$ value recommended in NIST has been observed to be satisfactory (when compared with the median values obtained from the numerical analyses) even for the periods in vicinity of the higher modes of vibration and for both the DBE and the MCE demands.

In case of the infilled frames, the $a_p$ value recommended in EN 1998 is reasonable for periods in vicinity of the higher modes of vibration, whereas slightly non-conservative for periods in vicinity of the fundamental mode of vibration (Fig. 4(e)-(h), 4(m)-(p), and Fig. 5(e)-(h), 5(m)-(p)) for both the DBE and the MCE demands. Further, for NSCs having periods in vicinity of higher modes of vibration of infilled frames, $a_p$ values recommended in both ASCE 7 and NIST are significantly conservative (Fig. 4(e)-(h), 4(m)-(p), and Fig. 5(e)-(h), 5(m)-(p)), and for NSCs having periods in vicinity of the fundamental mode of vibration $a_p$ value is reasonable in case of ASCE 7 and significantly conservative in case of NIST (Fig. 4(e)-(h), 4(m)-(p), and Fig. 5(e)-(h), 5(m)-(p)). Furthermore, for NSCs having periods sufficiently longer than the fundamental period of the supporting structure (Figs. 4 and 5), the provisions of both ASCE 7 and NIST are significantly conservative, for both the DBE and the MCE demands, whereas EN 1998 provides more reasonable estimates of the $a_p$ value. The reason for observed non-conservatism/conservatism in $a_p$ values recommended in

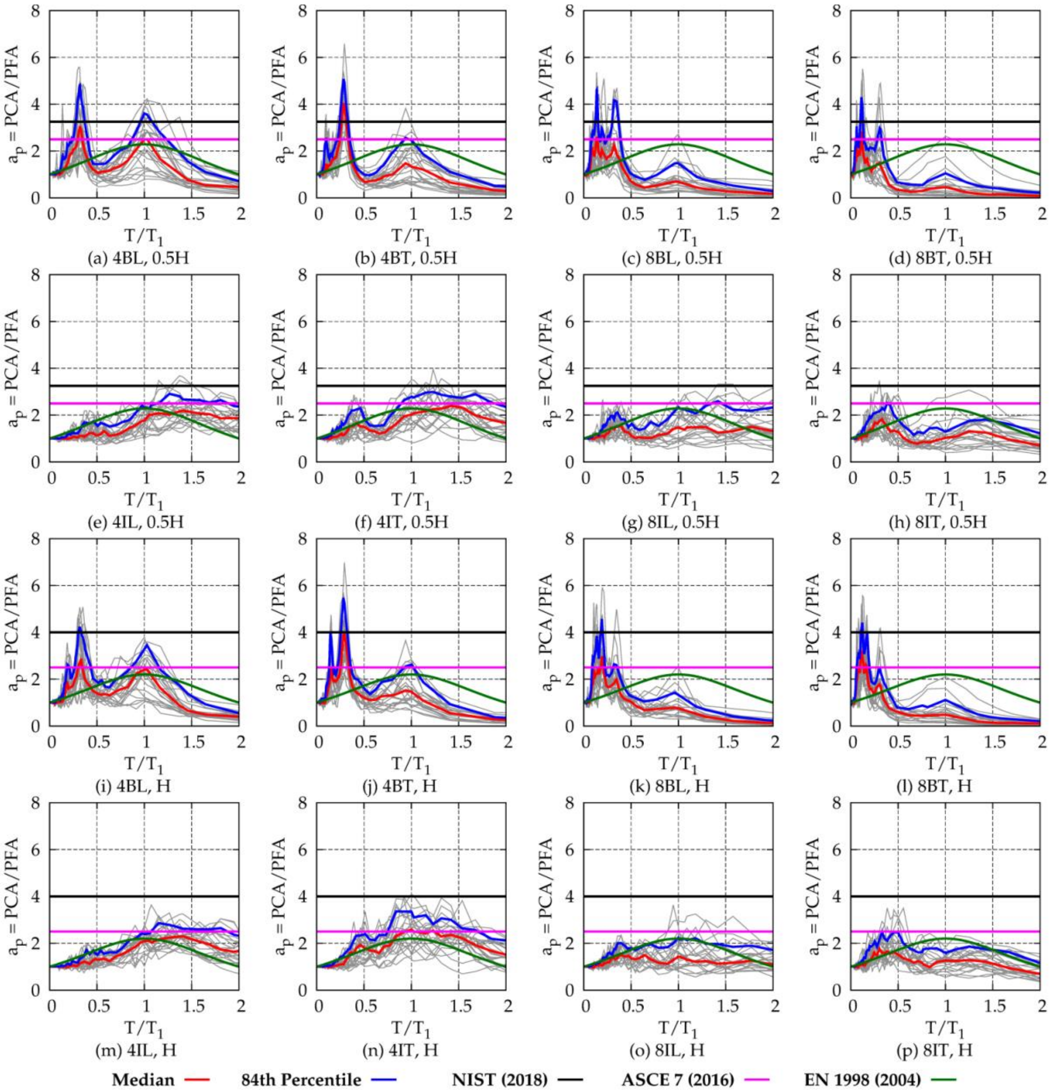


Fig. 5 Comparison of the component amplification factor ($a_p$) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the maximum considered earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '0.5*H*' and '*H*' corresponds that the derived results are for the mid-height and the roof level, respectively

current codes mainly lies in the spectral shape of the ground-motion records, in general, which have their spectral peaks in vicinity of higher modes of the supporting structure for bare frames, and in vicinity of the fundamental mode of the supporting structure, for infilled frames (Table 1).

### *6.3 Combined amplification factor*

Figs. 6 and 7 present a comparison of the combined effects of the in-structure and the component amplification obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records with the recommended provisions of NIST, ASCE 7 and EN 1998 for the investigated bare and uniformly infilled RC frames for the DBE and the MCE demands, respectively.

For bare frames subjected to the DBE demands, the combined effects of the in-structure and the component amplification obtained from provisions of both EN 1998 (at mid-height as well as roof level) and ASCE 7 (particularly at roof level) have been observed to be non-conservative (Fig. 6(a)-(d) and 6(i)-(l)), for NSCs having periods in vicinity of the higher modes of vibration. The observed non-conservatism in prediction of the combined amplification factor in case of ASCE 7 can be mainly attributed to the capping on the design force for NSCs, at 4 times of PGA. On the other hand, for NSCs having periods in vicinity of the fundamental mode of vibration, the combined effects of the in-structure and the component amplification obtained from all the considered provisions (EN 1998, ASCE 7 and NIST) in case of bare frames have been observed to be significantly conservative (Fig. 6(b)-(d) and 6(j)-(l)), for the DBE demand, excepting 4-storey

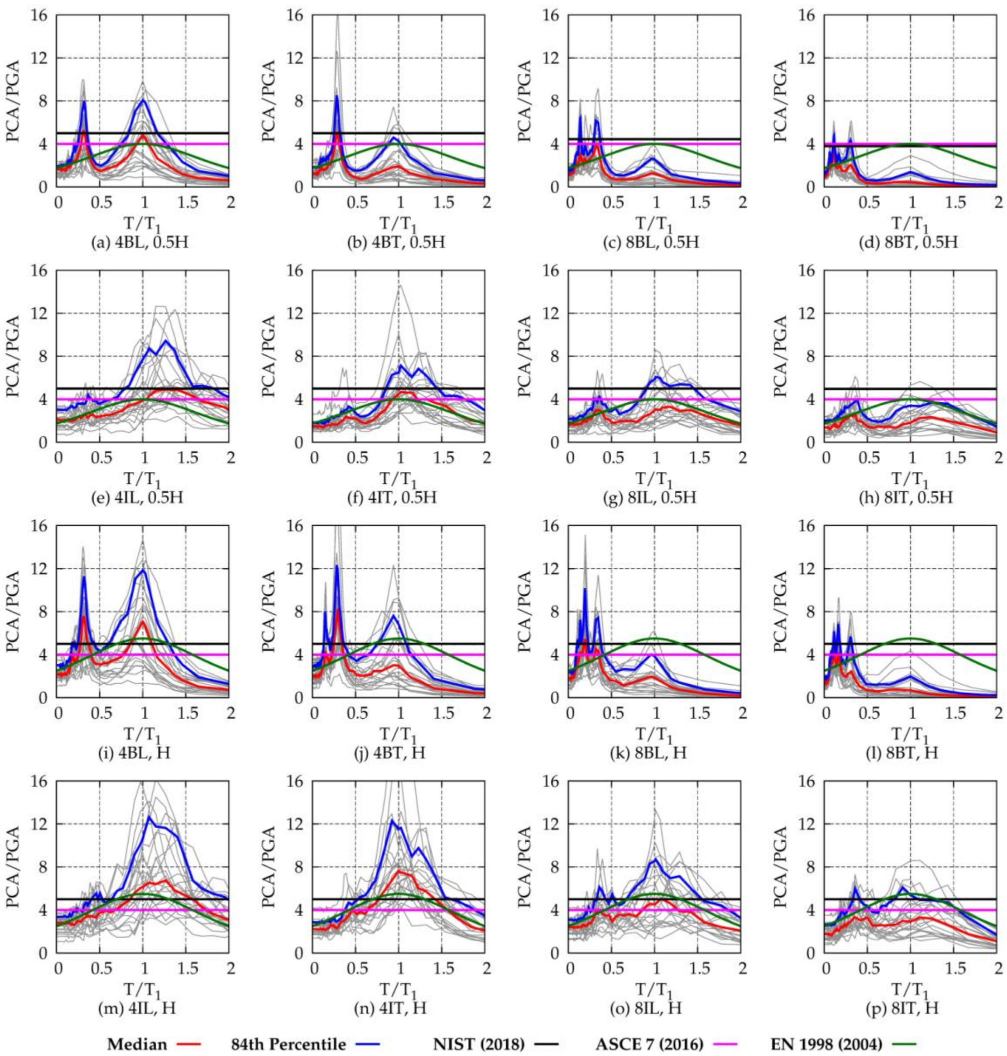


Fig. 6 Comparison of the combined amplification factor (PCA/PGA) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the design basis earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '0.5*H*' and '*H*' corresponds that the derived results are for the mid-height and the roof level, respectively

building (Fig. 6(a), 6(i)).

In case of infilled frames subjected to the DBE demand, NSCs having periods in vicinity of the fundamental mode of vibration, the combined effects of the in-structure and the component amplification exceeds their corresponding value recommended in EN 1998, (Fig. 6(e)-(f) and 6(m)-(n)). For bare frames subjected to the MCE demands, the combined effects of the in-structure and the component amplification reduce as compared to the DBE demand, and the non-conservatism of EN 1998 provisions also reduce to some extent (Fig. 7(a)-(d) and 7(i)-(l)), particularly, for NSCs having periods in vicinity of the higher modes of vibration, whereas conservatism of all the considered provisions (EN 1998, ASCE 7 and NIST) for NSCs having periods in vicinity of the fundamental mode of vibration further increases as compared to the DBE demand. In case of infilled frames subjected to the MCE demand, EN 1998 provisions result more reasonable estimates of the combined effects of in-structure and component amplification as compared to ASCE 7 and NIST provisions (Fig. 7(e)-(h) and 7(m)-(p)). Further, for NSCs with periods other than the modal periods of the supporting structures, both ASCE 7 and NIST provisions for the combined effects of the in-structure and component amplification have been observed to be significantly conservative (Fig. 7(e)-(h) and 7(m)-(p)), whereas EN 1998 has been observed to be reasonably conservative.

It is to be noted that all the results presented in this study are based on the assumption of 5% damping in NSCs (which .is also consistent with the assumed value of the damping ratio in the current codes for design of NSCs). In general, most of the NSCs have a lower damping ratio (e.g., of the order of 2%), therefore, the non-conservatism of the code provisions will further increase particularly for NSCs

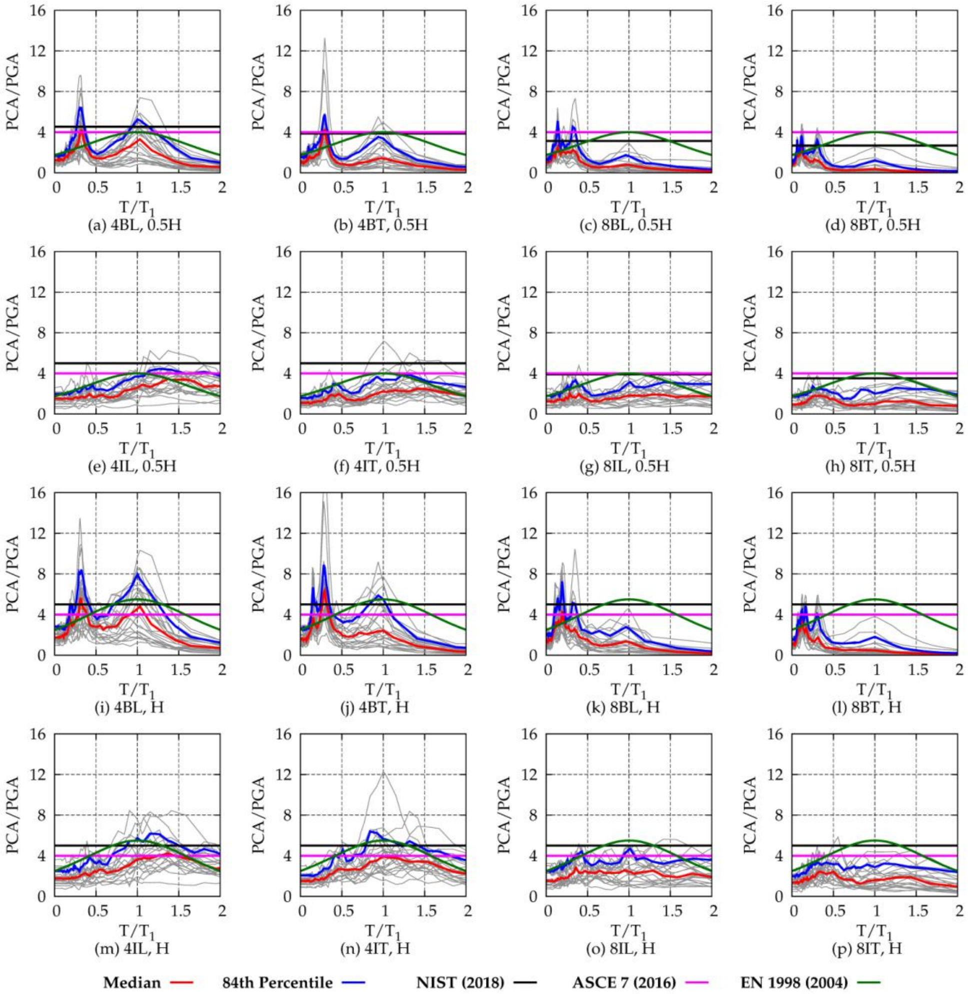


Fig. 7 Comparison of the combined amplification factor (PCA/PGA) obtained from the nonlinear dynamic analyses using a suite of 22 ground-motion records for the maximum considered earthquake demand with the recommendations of the European and the United States standards and NIST provisions. Here, in case of 4BL, 4BT, 8BL, 8BT, 4IL, 4IT, 8IL and 8IT, the numeric represents number of storeys, the first alphabet represents the framing system, where '*B*' stands for 'bare frame' and '*I*' stands for 'infilled frame' and the second alphabet represents the direction of excitation, where '*L*' stands for 'longitudinal' and '*T*' stands for 'transverse' direction. '0.5*H*' and '*H*' corresponds that the derived results are for the mid-height and the roof level, respectively

tuned with the higher modes of vibration, whereas, the conservatism of the code provisions will reduce for NSCs tuned with the fundamental mode of vibration of the supporting structure. The presented observations highlights a need to further improve the provisions for seismic design of NSCs. Therefore, it is recommended that future research should focus on the development of ductility demand specific capping provisions and floor amplification models accounting for the spectral shape of ground-motion records.

## 7. Conclusions

To assess the adequacy of the current code provisions for seismic design of acceleration sensitive NSCs, floor acceleration response of a set of bare and uniformly infilled RC frame supporting structures has been studied. Nonlinear dynamic analyses for seismic intensities corresponding to the DBE and the MCE were performed on the considered supporting structures and the derived floor acceleration response parameters, i.e., in-structure amplification, component amplification and combined amplification factors were compared with the provisions of the existing seismic design codes (i.e., ASCE 7 and EN 1998) as well as newly developed provisions by NIST based on recorded floor motions.

It has been observed that for the DBE demand, provisions of current codes for prediction of in-structure amplification are significantly conservative for bare frames and reasonable for infilled frames. With a further increase in the seismic intensity (i.e., the MCE demand), the provisions becomes further conservative for both bare and infilled frames. In contrast to code provisions, NIST provisions for in-structure amplification have been observed to be more reasonable, as these provisions account for the effects of the period of vibration, ductility demand in the supporting structure, and whiplashing effect of the higher modes to some extent.

The component amplification factors recommended in current codes (ASCE 7 and EN 1998) have been observed to be non-conservative for NSCs having periods in the vicinity of the higher modes of vibration of the bare frame supporting structures and also in the vicinity of the fundamental mode period of vibration of the infilled frame supporting structures, mainly due to spectral shape of ground-motion records. On the other hand, the component amplification factor recommended in NIST has been found to be in reasonable agreement with the peak value of component amplification factor obtained from the numerical analyses, on all the considered supporting structures.

The combined effects of in-structure and component amplification for the DBE demand has been observed to be non-conservative (at roof level) for all the considered provisions (NIST, ASCE 7 and EN 1998) particularly for NSCs having periods either in vicinity of the higher modes of vibration of bare frames, or in vicinity of the fundamental mode of vibration for infilled frames, mainly due to the capping on design force (in case of ASCE 7 and NIST). However, this non-conservatism reduces with an increase in seismic intensity of ground-motions from the DBE to the MCE, and in general, for the MCE demands, NIST provisions provide conservative estimate of the combined effects of the in-structure and component amplification for NSCs with different periods of vibration and attached to different structural systems (i.e., bare and infilled frames). The present study also highlights that the capping on design force suggested in NIST provisions is adequate at the MCE demand, but non-conservative at the DBE demand. Therefore, there exists a need to develop ductility demand specific capping limit for seismic design of NSCs.

The findings of the present study are limited to the mid-rise bare and infilled RC frame supporting structures. The ground-motions used in the present study consisted of a far-field record suite. Separate studies based on near-field records are recommended to generalize these findings, as those exhibits significantly different spectral shape as compared to the far-field ground-motions.

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