EVALUATION OF ASCE 7 EQUATIONS FOR DESIGNING ACCELERATION-SENSITIVE NONSTRUCTURAL COMPONENTS

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ABSTRACT

A wide variety of instrumented buildings and models of a code-based designed building are used to validate the results of previous studies that highlighted the need to revise the ASCE 7-16 $F_p$ equation for designing acceleration-sensitive nonstructural components (NSCs) through utilizing simplified elastic and inelastic numerical models. The conducted evaluation shows that, unlike the ASCE 7 approach, the component amplification factor is strongly dependent on the ratio of NSC period to the supporting building modal periods, the ground motion intensity, and the NSC location. Results illustrate that with increasing the ground motion intensity, the in-structure and component amplification factors tend to decrease. It is shown that the recorded ground motions at the base of most instrumented buildings were significantly lower than the design earthquake (DE). Because ASCE 7 is currently meant to provide demands at a DE level, for a more reliable evaluation of the $F_p$ equation, a representative code-based designed building is exposed to ground motions with various intensity levels. Results shows that at the DE level the ASCE 7 in-structure amplification factor, $[1 + 2(z/h)]$, tends to significantly overestimate demands at all floor levels, whereas the ASCE 7 component amplification factor, $a_p$, in many cases underestimate the computed values. The product of these two amplification factors, which represents the normalized peak component acceleration, in several floors exceeds the normalized $F_p$ equation.

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Evaluation of ASCE 7 Equations for Designing Acceleration-Sensitive Nonstructural Components

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ABSTRACT

A wide variety of instrumented buildings and models of a code-based designed building are used to validate the results of previous studies that highlighted the need to revise the ASCE 7-16 \( F_p \) equation for designing acceleration-sensitive nonstructural components (NSCs) through utilizing simplified elastic and inelastic numerical models. The conducted evaluation shows that, unlike the ASCE 7 approach, the component amplification factor is strongly dependent on the ratio of NSC period to the supporting building modal periods, the ground motion intensity, and the NSC location. Results illustrate that with increasing the ground motion intensity, the in-structure and component amplification factors tend to decrease. It is shown that the recorded ground motions at the base of most instrumented buildings were significantly lower than the design earthquake (DE). Because ASCE 7 is currently meant to provide demands at a DE level, for a more reliable evaluation of the \( F_p \) equation, a representative code-based designed building is exposed to ground motions with various intensity levels. Results shows that at the DE level the ASCE 7 in-structure amplification factor, \([1 + 2 (z/h)]\), tends to significantly overestimate demands at all floor levels, whereas the ASCE 7 component amplification factor, \( a_p \), in many cases underestimate the computed values. The product of these two amplification factors, which represents the normalized peak component acceleration, in several floors exceeds the normalized \( F_p \) equation.

Introduction

In accordance with the ASCE 7-16 Equations 13.3-1 to 13.3-3, the horizontal seismic design force (\( F_p \)) applied at the nonstructural component (NSC) center of gravity, can be calculated based on a simplified equivalent static force given by Eq. (1)

\[
F_p = \frac{0.4S_{DS} W_p}{R_p/I_p} a_p [1 + 2 (z/h)]
\]  

(1)

where \( S_{DS} \) is the short-period 0.05-damped, pseudo-spectral acceleration for the supporting building site. \( W_p \) is the NSC operating weight. \( a_p \) is the NSC amplification factor = 1.0 for rigid

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NSCs that are rigidly attached to a floor; $2\frac{1}{2}$ for all flexible or flexibly mounted NSCs; and $1\frac{1}{4}$ for the fasteners of the connecting system in exterior nonstructural wall elements and connections; as per Section 11.2 of ASCE 7-16, the period 0.06 s is the threshold for differentiating rigid and flexible NSCs. The remaining parameters in Eq. (1) are as follows: $I_p$ is the NSC importance factor that varies from 1.0 to 1.5; $R_p$ is the NSC response modification factor that varies from 1 to 12 for different types of NSCs; $z$ is the height in the supporting structure of the point of attachment of the NSC with respect to the seismic base; and $h$ is the average roof height of the supporting structure with respect to the seismic base. $F_p$ is not required to be taken as greater than $1.6S_{DS}W_pI_p$ and shall not be taken as less than $0.3S_{DS}W_pI_p$.

Three alternative dynamic analysis methods are also provided in ASCE 7-16 for estimating $F_p$. Because the vast majority of seismic designs in practice are based on the equivalent static method, the dynamic analysis approaches are not discussed herein (the term “ASCE 7 equation” in this paper refers to the equivalent static equation of ASCE 7-16 (Eq. 13.3-1) including its upper and lower limits, i.e., Eqs. 13.3-2 and 13.3-3, respectively). In Eq. (1) the term $0.4S_{DS}$ essentially represents the peak ground acceleration (PGA) at the base of the supporting building at the design earthquake (DE) level, denoted as the design PGA in this paper. The peak component acceleration (PCA) demand normalized to the design PGA, assuming an elastic NSC with an importance factor of $I_p = 1.0$, can be written in the form given by Eq. (2). This form of the ASCE 7 equation has been more frequently used in previous works than Eq. (1).

$$\frac{PCA}{PGA} = \frac{F_p/W_p}{0.4S_{DS}} = a_p [1 + 2(z/h)]$$  

(2)

In Eq. (2), $a_p$ is the ratio of the PCA to the peak floor acceleration (i.e., PCA/PFA) for an elastic NSC, denoted as component amplification factor. The term $[1 + 2(z/h)]$, which relates an upper-floor horizontal acceleration to the ground-level horizontal acceleration (i.e., PFA/PGA), can be interpreted as an “in-structure” amplification factor. Based on ASCE 7-16 Eqs. 13.3-2 and 13.3-3, the lower and upper limits for PCA/PGA are 0.75 and 4.0, respectively.

As seen in Eq. (2), the design PCA is simply a function of the NSC’s flexibility, the vertical location of the NSC within the supporting building, and the design PGA. Several previous research studies have reported potential shortcomings associated with the ASCE 7 approach. Nowadays, it is well understood that such design-oriented methods do not account for all relevant factors that can significantly affect the acceleration response of NSCs. The parameters that are not explicitly incorporated into the design equations, but are influential on the PCA demand, include the supporting building characteristics (i.e., modal periods, global ductility demand, and in-plane floor diaphragm flexibility), and NSC damping ratio. In addition, significant shortcomings exist in the parameters that are taken into account in the design equations: the lack of dependency of the component amplification factor on the NSC location along the building height; the assumption of a linear variation of the in-structure amplification factor over the building height regardless of the building system and lateral strength (i.e., the expected level of inelastic behavior at the DE level); and the somewhat arbitrary definition of 0.06 s as the period threshold to separate rigid and flexible NSCs.

Previous numerical studies, while providing valuable insight into understanding the influential parameters on NSCs seismic demands, in many cases cannot accurately represent
important characteristics present in the response of actual buildings. Some of these works (e.g., [1]) are based on the assumption that the supporting building responds elastically. This assumption seems to be adequate for the design of essential facilities such as emergency centers and hospitals, which are typically designed to remain elastic or nearly elastic during severe earthquakes. However, such an assumption is not directly applicable to most nonessential buildings that are designed to undergo inelastic deformations during the DE even when the presence of overstrength is accounted for. In this context, several numerical studies have highlighted the significant influence of the supporting building inelasticity on NSCs acceleration demands [2-5]. These studies have been mostly based on SDOF [e.g., 5] or two-dimensional generic building models [e.g., 4] that do not adequately represent relevant characteristics present in the response of actual multistory buildings. In the present paper, floor response spectra of a total of 118 instrumented building directions in the US are studied to evaluate the adequacy of the current ASCE 7-16 equivalent static equation for designing acceleration-sensitive NSCs. An evaluation of the responses of instrumented buildings facilitates the understanding and quantification of underlying phenomena and characteristics that can significantly affect the NSCs acceleration demands but are difficult to capture using simplified numerical models. These relevant phenomena and characteristics include, but are not limited to, the in-plane floor diaphragm flexibility; torsional responses of the supporting building; vertical mass and stiffness irregularities; the distribution of seismic damage; the contribution of infill and partition walls to the building lateral stiffness and strength; soil-foundation-structure interaction; as well as potential interactions between heavy NSCs and their supporting buildings; among others.

During the past years, a few studies have used the responses of instrumented buildings in California to evaluate the ASCE 7 equation [6-9]. These studies while comparing the normalized acceleration demands (i.e., PFA and PCA responses normalized to the recorded PGA at the building base) with the ASCE 7 equation in its normalized form (i.e., Eq. 2), did not consider the effect of the ground motion intensity. The only exception is [8], which was based on the responses of only two instrumented buildings. The present study illustrates that an adequate evaluation of the ASCE 7 equations in their normalized format should account for the effect of the ground motion intensity level (i.e., the supporting building inelasticity). It is shown that the recorded PGA at the base of most instrumented buildings was significantly smaller than the design PGA at the site. Thus, the authors hypothesize that these buildings primarily remained in their linear-elastic range resulting in relatively large normalized acceleration demands. ASCE 7 is meant to provide component seismic demands for the DE level. Typical buildings are designed to undergo inelastic deformation at a DE. This inelastic behavior can potentially decrease NSCs normalized acceleration demands, especially in tuning situations. Hence, an adequate evaluation of the ASCE 7 equation in its normalized form cannot be performed solely based on the response from instrumented buildings and/or linear-elastic models. To further investigate the effect of the building inelasticity on the evaluation of the ASCE 7 equations, a code-based designed (archetype) eight-story reinforced concrete shear wall building is exposed to ground motions scaled to different intensity levels. The effects of additional characteristics of the instrumented buildings (e.g., lateral-load resisting system, in-plane floor diaphragm flexibility, and torsional responses of the supporting building) on NSCs acceleration demands are not part of this study and were evaluated comprehensively in [10].
Instrumented Buildings and Methodology Adopted in this Study

For the evaluation conducted in this paper, 59 instrumented buildings are selected from the CESMD database (www.strongmotioncenter.org). Given that for each individual building dynamic characteristics and recorded ground motions in two orthogonal horizontal directions are different, a database with 118 building directions (i.e., 59×2) is compiled. All floor motions available for these buildings in the CESMD database (approximately 600 motions) are used in the conducted evaluation. The floor response spectrum (FRS) method is used to estimate seismic demands on NSCs. Consistent with many previous studies, floor spectra in this study are based on a 0.05 damping ratio. The objective herein is not to determine the most realistic value of NSC damping ratio but to use a value that will permit the direct evaluation of the ASCE 7 equation using the same basic assumptions. The NSC inelasticity has the effect of substantially reducing the NSC strength demands, and is addressed in the code provisions through the $R_p$ factor. The focus of the present study is elastic NSCs.

Evaluation of ASCE 7 Design Equations Using Floor Spectra of Instrumented Buildings

In several previous research works the normalized PCA responses (i.e., maximum spectral ordinate over the NSC period range normalized to PGA) were used to evaluate the adequacy of the ASCE 7 equation (Eq. 2), which is meant to provide NSC equivalent static demands at the DE level. The main purpose of this section is to illustrate the shortcomings associated with such an approach when the supporting building responds elastically. To this end, 0.05-damped floor spectra for all existing roof floor motions in the compiled database are generated. Figs. 1a and 1b illustrate roof floor pseudo-spectral acceleration responses normalized to the recorded PGA and to the roof PFA, respectively. The parameter illustrated in Fig. 1b is the component amplification factor. In these figures $T_{1bldg}$ is the supporting building fundamental period.

Figure 1. 0.05-damped roof floor spectra for instrumented blds. normalized to the: (a) PGA, (b) PFA.

An evaluation of Figs. 1a and 1b reveals that, unlike the adopted approach by ASCE 7, normalized spectral ordinates (FRS/PGA) and also component amplification factor (FRS/PFA) are a function of the ratio of the component period to the building modal periods (i.e., the tuning ratio). As seen in Fig. 1a, the computed PCA/PGA at the roof varies from 1.2 to 36.0 with a mean plus one standard deviation of 12.9, which exceeds the ASCE 7 upper limit of 4.0 by a factor of 3.2. An evaluation of Fig. 1a illustrates that PFA/PGA ranges from 0.4 to 5.6, with a mean plus one standard deviation of 3.1, which is comparable to the ASCE estimate of 3.0 for the roof level. From Fig. 1b it can be observed that the parameter PCA/PFA ranges between 1.5
and 8.0 with a mean plus one standard deviation of 4.6, which is 1.8 times the ASCE 7 value of $2\frac{1}{2}$. The lack of an explicit dependence of NSC design force values on the NSC tuning ratio is a significant shortcoming in the ASCE 7 approach. However, the large normalized acceleration responses observed in Figs. 1a and 1b do not necessarily mean that the ASCE 7 equations underestimate the absolute value of NSC demands at the DE level. Further investigations illustrate that many of the observed large normalized responses belong to instrumented buildings that experienced ground motion intensity levels well below the DE. To clarify this statement, Fig. 2 depicts the ratio of the recorded PGA to the design PGA (i.e., $\frac{PGA}{PGA_{design}}$) for all instrumented building directions. For each building, $S_{DS}$ is computed using the “USGS tool” based on the information provided in CESMD. As seen in Fig. 2, in many cases the recorded PGAs are significantly smaller than the design PGA (i.e., $PGA_{design} > 1.0$). An evaluation of the percentage of the data points smaller than a specific $PGA_{design}$, shown in Fig. 3, reveals that, for example, the PGA of 85% of these records is less than 0.50 $PGA_{design}$. Hence, it is reasonable to infer that most of these structures behaved in the linear-elastic range. This elastic behavior could significantly bias any evaluation of the ASCE 7 equation based on the response of instrumented buildings alone. To clarify his statement, normalized PCA responses for three different bins are shown in Figs. 4a-4c for instrumented building directions with $PGA_{design}$ larger than (a) 0.05, (b) 0.50, and (c) 0.75.

Figure 2. $PGA/PGA_{design}$ for instrumented bldgs.

Figure 3. Percentage of data points smaller than a specific $PGA/PGA_{design}$.

Figure 4. PCA/PGA vs. the relative height for instrumented bldg. directions with $PGA/PGA_{design}$ greater than (a) 0.05, (b) 0.50, (c) 0.75 (w/o limit: without applying the upper limit of 4.0).

An evaluation of the trend observed in Fig. 4a-4c illustrates that with increasing the ground motion intensity the relatively large normalized PCA responses are no longer observed. In Fig. 4a, where the responses of building directions with all $PGA/PGA_{design}$ ratios are considered, the PCA/PGA response in many cases significantly exceed the ASCE limit of 4.0. However, in Fig.
4c, where only the building directions with PGA/PGA_{design} ≥ 0.75 are present, PCA/PGA is limited to 7.8. If the upper limit of 4.0 is not applied to Eq. (2), this equation results in a normalized value of 7.5 for the roof level, which is close to the observed value of 7.8. Figs. 5a-5c present the NSC amplification factor (i.e., PCA/PFA) for different ground motion intensity scenarios. The trend observed in these figures suggests that with increasing the ground motion intensity level the NSC amplification factor tends to decrease. For example, in Fig. 5a, where responses for ground motions with any intensity levels are present, the maximum observed PCA/PFA at the roof level (i.e., z/h =1.0) is 7.7, whereas, in Fig. 5c, where only cases with PGA/PGA_{design} ≥ 0.75 are considered, this quantity is limited to 4.5. Despite the decreasing trends observed in Figs. 4a-4c and 5a-5c, even for high ground motion intensity levels, the ASCE 7 limits of 4.0 and 2\frac{1}{2} for the PCA/PGA and PCA/PFA are exceeded at different floor levels (e.g., by factors up to 1.8 for PCA/PGA and 2.0 for PCA/PFA). An evaluation of the in-structure amplification factor (i.e., PFA/PGA) for different ground motion intensity scenarios (graphs are not presented herein due to space limitation) illustrates that for low intensity ground motions, the recorded in-structure amplification factors can exceed the ASCE 7 estimate, 1 + 2(z/h). However, for higher ground motion intensities, the ASCE 7 equation significantly overestimates the magnitude of most recorded in-structure amplification factors [see 13].

![Figure 5. PCA/PFA vs. the relative height for instrumented bldg. directions with PGA/PGA_{design} greater than (a) 0.05, (b) 0.50, (c) 0.75.](image)

Presented evaluations in this section reveal significant shortcomings associated with the ASCE 7-16 \( F_p \) equation. However, a final conclusion about the adequacy of the ASCE 7 upper limits cannot be performed solely based on the responses of instrumented buildings that were mostly exposed to ground motions with relatively small intensities. In other words, given that at high intensity ground motions typical nonessential buildings are designed to experience inelastic actions, a linear extrapolation of PCA/PGA values from low to higher intensity ground motions is highly questionable. To further investigate the effect of the building inelasticity on the evaluation of the ASCE 7 equation in its normalized form, in the next section results of simulating an archetype building under ground motions with different intensities are discussed.

**An Archetype Building to Evaluate the ASCE 7 Equation for Designing NSCs**

Floor acceleration responses of a reinforced concrete shear wall (RCSW) archetype building are evaluated under ground motions with different intensities. This archetype building is consistent with the designed buildings used as part of the ATC-63 project, which was documented as FEMA P695 [11]. RCSW buildings were designed for two axial force levels and
two seismic design categories. An eight-story building from the group with the lower axial force and higher seismic design load is selected for the evaluation conducted in this section. The RCSW is modeled in SAP2000 [12] using nonlinear shell elements based on steel reinforcing and concrete material characteristics provided in FEMA P695. With respect to FEMA P695, the steel reinforcement is modified in some cases to improve the system global ductility [see, 13]. The lateral-load resisting system elements are located at the building perimeter in two principal directions. A two-dimensional numerical model of the archetype building is developed in SAP 2000. The global/structure P-Delta effects of gravity loads that are not tributary to the lateral-load resisting system are incorporated via a leaning P-Delta column with zero lateral stiffness attached to the model with axially rigid pin-ended members. Spectrum compatible (SC) records are used in nonlinear response history analyses. The target spectrum used to generate SC records is similar to the design spectrum used in FEMA P695 (\(S_{DS} = 1.0 \text{ g}\) and \(S_{DS} = 0.55 \text{ g}\)). As illustrated by [14], even with a tight match to the target spectrum, record-to-record variability in seismic responses, especially higher-mode dominated responses, under a set of SC records can be significant. Hence, in this study 20 SC records are used to account for this type of record-to-record variability. The archetype building is simulated under the SC records scaled to different intensity levels varying form 0.25 DE to 1.50 DE.

**Simulation Results for the Eight-Story RCSW Archetype Building**

Fig. 6a illustrates the dispersion in the 0.05-damped roof spectra due to record-to-record variability present in the set of SC records at the DE level. As seen, the dispersion in spectral ordinates, especially in the vicinity of the supporting building second mode (i.e., \(T_{NSC} = 0.13 \text{ s}\)), is significant. Fig. 6b presents mean values of roof spectra for different intensity levels.

![Figure 6](image)

**Figure 6.** (a) Dispersion in the normalized 0.05-damped elastic roof spectra due to the record-to-record variability of the SC record set, (b) mean normalized roof spectra of the eight-story RCSW building exposed to SC records scaled to different intensities.

Fig. 6b illustrates that with increasing the intensity, at most component periods, especially at tuning situations, the normalized roof spectral ordinates markedly decrease. At longer component periods (e.g., \(T_{NSC} > 1.2 \text{ s}\)) inelasticity may slightly increase the normalized demands. However, these periods are not in the practical range of typical acceleration-sensitive NSCs, and moreover, the normalized demands at these periods do not dominate the peak values of the response spectra. At the DE level when component periods are in the vicinity of the second mode, on average, the normalized demands exceed the ASCE limit of 4.0 by a factor of 1.85. In the rest of this section the mean value of responses under the 20 SC records is used for evaluation. As seen in Fig. 6b, the maximum value of the mean spectrum for all intensity levels,
except for 0.25 DE, occurs at component periods in the vicinity of the supporting building second modal period. For consistency, when evaluating responses with respect to the intensity level, for 0.25 DE, the parameter PCA is defined as the maximum ordinate in the higher-mode region. Figs. 7a-7c present an evaluation of the ASCE 7 design equations at different relative heights (i.e., $z/h$) for different ground motion intensity levels. The overall trend in these figures suggests that with increasing the ground motion intensity, the normalized acceleration responses decrease. Fig. 7a illustrates that at the DE level the ASCE 7 equation overestimates the PFA/PGA responses at all floor levels. For example, at the roof level, the PFA/PGA ratio is 1.8, which is smaller than the ASCE 7 estimate of $3.0$. Fig. 7b illustrates that at the DE level, the ASCE 7 value of $\alpha_p = 2.5$ in many floor levels tend to underestimate the computed PCA/PFA responses. The PCA/PGA profiles versus the relative height for different intensity levels are presented in Fig. 7c. An evaluation of Fig. 7c reveals that at the DE level, PCA/PGA at the roof and middle floor exceeds the ASCE 7 estimates by a factor of 1.9 and 1.8, respectively. The trends observed in these figures are consistent with the general trends obtained from the responses of instrumented buildings discussed in the previous section of this paper.

The results presented in Fig. 7c also confirm that the relatively large normalized acceleration responses are due to the elastic behavior of the supporting building. The maximum PCA/PGA at the 0.25 DE, which corresponds to the elastic behavior for the RCSW building, is 13.1. For the instrumented buildings, which were generally exposed to low intensity ground motions, significant PCA/PGA responses (e.g., 33.0 and 36.0) were also observed in Fig. 1a. In a different study, Anajafi and Medina [10] illustrated that these significant normalized acceleration responses occur because of special behaviors such as vertical building irregularities in mass and stiffness, torsional responses and in-plane floor diaphragm flexibility. These behaviors are not modeled in the simplified two-dimensional numerical model of the archetype building. If the responses of special cases are excluded, PCA/PGA ratios for the rest of the instrumented buildings are limited to 12.0 (refer to Figs. 7a-7c in [10]), which are commensurate with the PCA/PGA ratios observed in the elastic archetype building models.

The maximum relative PCA response defined as the normalized peak acceleration at roof level is $\alpha_p = 2.5$. In addition, the PFA/PGA ratio was computed by a factor of 1.8 at the roof level, which is smaller than the ASCE 7 estimate of $3.0$. The overall trend in these figures reveals that with increasing the ground motion intensity, the normalized acceleration responses decrease. For example, at the roof level, the PFA/PGA ratio is 1.8, which is smaller than the ASCE 7 estimate of $3.0$. Fig. 7b illustrates that at the DE level, the ASCE 7 value of $\alpha_p = 2.5$ in many floor levels tend to underestimate the computed PCA/PGA responses. The PCA/PGA profiles versus the relative height for different intensity levels are presented in Fig. 7c. An evaluation of Fig. 7c reveals that at the DE level, PCA/PGA at the roof and middle floor exceeds the ASCE 7 estimates by a factor of 1.9 and 1.8, respectively. The trends observed in these figures are consistent with the general trends obtained from the responses of instrumented buildings discussed in the previous section of this paper.

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![Figure 7](image.png)

Figure 7. Evaluation of the ASCE 7 design equations at different relative heights based on the mean floor response spectra of the eight-story RCSW building exposed to a set of SC records scaled to various intensity levels (a) PFA/PGA, (b) PCA/PFA, (c) PCA/PGA.

Conclusions

This study uses a wide variety of instrumented buildings in California and models of a code-based designed building to validate the results of previous studies that highlighted the need to revise the ASCE 7 equivalent static equation for designing acceleration-sensitive nonstructural
components (NSCs) when utilizing simplified generic elastic and inelastic numerical models. In this evaluation, 59 instrumented buildings (i.e., 118 building directions) and approximately 600 floor motions are used. Acceleration demands on NSCs are estimated using the floor response spectrum method assuming a 5% damping ratio and an elastic behavior for NSCs. It is shown that many of these buildings experienced relatively small ground motion intensities. Hence, it is reasonable to infer that most of them behaved in their linear-elastic range. A consequence of this elastic behavior is the presence of relatively large in-structure and component amplification factors (i.e., PFA/PGA and PCA/PFA, respectively), and consequently large normalized peak component acceleration (PCA/PGA) responses, which in many cases exceed the ASCE 7 estimates based on Eq. (2). For example, the computed PGA/PFA at the roof level of these buildings varies from 1.2 to 36.0 with a mean plus one standard deviation of 12.9, which exceeds the ASCE 7 upper limit of 4.0 by a factor of 3.2. The parameter PCA/PFA ranges between 1.5 and 8.0 with a mean plus one standard deviation of 4.6, which is 1.8 times the ASCE 7 value of $2^{\frac{3}{2}}$. PFA/PGA is between 0.4 and 5.6 with a mean plus one standard deviation of 3.1, which is comparable to the ASCE estimate of 3.0 for the roof level.

The effect of ground motion intensity on floor spectra of the instrumented buildings is investigated revealing that as this parameter increases, normalized acceleration demands decrease. This latter observation demonstrates the drawbacks of an evaluation of the ASCE 7 equation in its normalized form exclusively based on linear-elastic models or instrumented buildings that experienced relatively small recorded PGAs. However, the conducted evaluation of the responses of the instrumented buildings is useful to illustrate significant shortcomings associated with the two components of the ASCE 7 equation. It reveals that, unlike the current ASCE 7-16 approach, the component amplification factor (PCA/PFA) is a function of the ratio of NSC period to the modal periods of the supporting building; ground motion intensity level; and the NSC location along the building height. It also illustrates that the ASCE 7 period threshold of 0.06 s used to determine the NSC amplification factor warrants modification.

To further investigate the effect of the ground motion intensity level (or supporting building inelasticity) on the evaluation of the ASCE 7 $F_p$ equation, a representative code-compliant building is exposed to ground motions with various intensity levels (including those representative of the ones experienced by instrumented buildings and the design earthquake). Simulation results, consistent with previous numerical studies, illustrate that the parameters PCA/PGA and PFA/PGA at all floor levels and PCA/PFA at most floor levels tend to decrease with increasing ground motion intensity. At the design earthquake level the ASCE 7 estimate of in-structure amplification factor, $\text{PFA}/\text{PGA} = 1 + 2(\frac{z}{h})$, significantly overestimates demands at all floor levels, whereas, the ASCE 7 component amplification factor, $\text{PCA}/\text{PFA} = 2^{\frac{1}{2}}$, at most floors underestimates the computed NSC amplification factors. The product of these two amplification factors (that represents the normalized peak NSC acceleration) at roof and midheight levels exceeds the ASCE 7 upper limit by a factor up to 1.9.

This study provides information useful for the development of improved ASCE 7 equations for designing acceleration-sensitive NSCs. To develop such equations as part of parallel studies that are in progress by the authors, archetype buildings with different number of stories, modal periods and lateral load resisting systems are developed. The influence of parameters such as
the in-plane floor diaphragm flexibility, torsional responses of the supporting building, NCSs damping ratio and ductility capacity are also investigated.

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