Proposed Nonstructural Seismic Design Force Equations

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Abstract

This paper summarizes a proposal to make significant revisions to the nonstructural seismic design equations in the 2020 edition of the *NEHRP Recommended Provisions* and *ASCE 7-22*. The proposal is based on research performed in the Applied Technology Council ATC-120 project and subsequent work by the Building Seismic Safety Council (BSSC) Issue Team 5. The ATC-120 project resulted in two reports: *Seismic Analysis, Design, and Installation of Nonstructural Components and Systems – Background and Recommendations for Future Work* (NIST, 2017) and *Recommendations for Improved Seismic Performance of Nonstructural Component* (NIST, 2018). A goal of the ATC-120 effort was to develop equations that have a more rigorous scientific basis and that capture the key parameters that influence nonstructural component response, and yet remain appropriate for use in design by practicing engineers.

The ATC-120 project reviewed the literature, identified key parameters, assessed the influence of these parameters individually on component response, focused on parameters shown to strongly affect response, and then tested a set of equations combining all the selected parameters of interest using an extensive set of nonlinear analyses of archetype buildings and components as well as analysis of strong motion records from instrumented buildings. Parameters selected for inclusion in the final set of equations include ground shaking intensity, seismic force-resisting system of the building, building modal period, building ductility, vertical location of the component within the building, component period, component and/or anchorage ductility, inherent component damping, and component overstrength.

In addition to providing an improved force equation, the proposal simplifies the design coefficients for nonstructural components. Components are now classified based on 1) their likelihood of experiencing a resonance condition during strong shaking, and 2) their ductility (low, moderate or high). Provisions for the design of equipment supports such as platforms and frames have been expanded and coordinated with Chapter 15 of ASCE 7. Support platforms and frames will be designed using coefficients based on their structural properties, rather than the properties of the components they support.

Introduction

The ATC-120 project had two phases. NIST (2017) summarized the results of the first phase, which collected and documented available knowledge related to the seismic performance of nonstructural components, based on past earthquake observations, code development efforts, analytical research, experimental testing, and a project workshop.

Based on the recommendations from the first phase, the second phase focused on the following main subject areas, summarized in NIST (2018).

- Development of performance objectives for nonstructural components that can serve as the basis of code requirements.
- Development of improved nonstructural design equations based on a comprehensive review of all factors contributing to seismic performance of nonstructural components and systems, using the latest information from instrumented buildings, laboratory tests, and analytical studies.
- Detailed review of code requirements, with a focus on improving clarity, consistency, and enforceability.

This paper describes the current ASCE 7-16 nonstructural design equations, the studies done in the ATC-120 project on the influence individual parameters on nonstructural response, the proposed equations developed in the ATC-120 project and evaluation through analytical studies, revisions made as the proposed equations were refined by the BSSC Issue Team 5 (IT5) Nonstructural Committee, current status on the code development chronology, and conclusions.

Note that much of the text in this paper is taken verbatim, condensed, or paraphrased from Chapter 4 of NIST (2018) or from the 6/20/19 version of the IT5 code change proposal (Gillengerten, 2019).

ASCE 7-16 Nonstructural Design Equations

The current *primary* nonstructural design force equation in ASCE 7-16, Equation 13.3-1, can be written in the following form:

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

where:

F_p = horizontal design force for nonstructural component
W_p = component operating weight
S_{DS} = 5% damped design spectral response acceleration at short periods (0.2 seconds) per ASCE 7-16 and USGS at the project site
a_p = component amplification factor
R_p = component response modification factor
I_p = component Importance Factor
z = height in structure of point of attachment of component with respect to the base
h = average roof height of structure with respect to the base

As summarized in NIST (2018), this equation is a function of several variables: S_{DS}, a_p, R_p, I_p, and z/h. The following variables in the equation are surrogates for other parameters: a_p is an amplification factor of the peak floor acceleration (PFA) that is a function of component period and inherent component damping; R_p is a reduction factor based on the component ductility, inherent component damping, and component overstrength. The floor acceleration is estimated in Equation 13.3-1 as the ground acceleration, approximated with $0.4S_{DS}$, taken as linearly increasing from the ground to three times the ground acceleration at the roof. Additionally, the I_p term serves to reduce the effect of R_p, which in turn increases the design force, for critical components, to provide a higher reliability of not failing.

ASCE 7-16 also includes Equation 13.3-2 to set a maximum design force or "cap" of $F_p/W_p = 1.6\ S_{DS}\ I_p$, and Equation 13.3-3 to set a minimum design force or "floor" of $F_p/W_p = 0.3\ S_{DS}\ I_p$.

In addition, ASCE 7-16 includes Equation 13.3-4 that allows the use of dynamic analysis to identify project specific PFAs and incorporate them into the design force used for nonstructural components. New in the ASCE 7-16 edition is an alternative approach presented in ASCE 7-16 Sections 13.3.1.4.1 and 13.3.1.4.2 that also permits replacing a_p with a project specific floor spectra and peak component acceleration (PCA).

ATC-120 Findings on the Influence of Individual Parameters on Nonstructural Response

As noted in Section 4.1 of NIST (2018), ASCE 7-16 Equation 13.3-1 is a simplification, as there are many different parameters that can affect the force a nonstructural component experiences during response to seismic shaking. The parameters include the following.

- Ground shaking intensity, expressed in terms of peak ground acceleration (PGA)
- Seismic force-resisting system of the building (SFRS)
- Building modal periods, T_{nbldg}
- Building ductility, μ_{bldg}
- Inherent building damping, β_{bldg}
- Building configuration (such as plan and vertical irregularities), *IRR*
- Floor diaphragm rigidity, *DIA*
- Vertical location of component within the building, z/h
- Component period, T_{comp}
- Component and/or anchorage ductility, μ_{comp}
- Inherent component damping, β_{comp}
- Component overstrength, Ω_{0comp}

Section 4.2 of NIST (2018) provides the following summary of review of available research and studies conducted on the influence of these parameters on the response of nonstructural components to seismic forces.

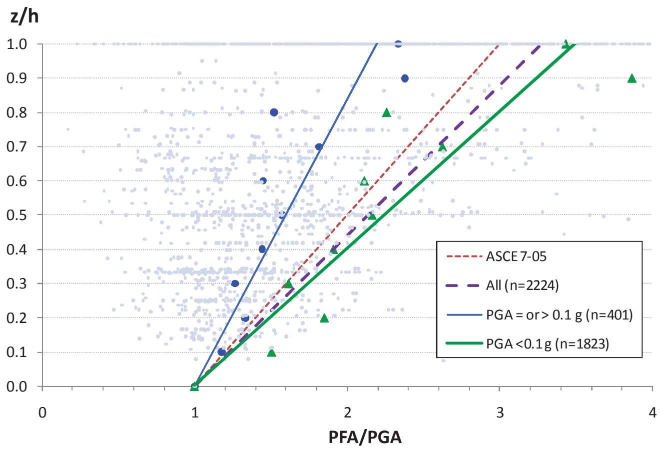
Peak Ground Acceleration

Shaking intensity has a direct effect on the level of excitation of the building and the components within it. Stronger shaking intensity leads to higher component response throughout the building. More subtly, shaking intensity can have an effect on the component response versus building height. Fathali and Lizundia (2011) showed how PFA/PGA is larger at higher diaphragm levels (higher z/h) with lower levels of PGA, as shown in Figure 1.

In these studies of instrumental recordings, it is likely that the large majority of records, even those with comparatively higher PGA, had essentially or largely elastic response. The vast majority of records available have relatively low PGA values. For example, at the time of the Fathali and Lizundia study, of the 2,224 data points used in Figure 1, 401 had PGA ≥ 0.1g, 136 had PGA ≥ 0.2g, 53 had PGA ≥ 0.3g, and only 19 had PGA ≥ 0.4g.

With elastic response of the building, from principles of structural dynamics, the ratio of PFA/PGA should not vary with changing PGA. Height, building period, SFRS, and ground motion frequency content, however, can affect the ratio of PFA/PGA.

For example, the frequency content of the smaller PGA records may be different than that of the larger PGA records, possibly because of the relationship between PGA at the building and the distance from the source. Thus, one explanation of the results in Figure 1 is that these other parameters are the source of the reduction in PFA/PGA with increasing PGA. This has not been explored in those datasets. Taghavi and Miranda (2006) showed that record-to-record variability (changes in PFA/PGA produced exclusively by changes in frequency content) has a coefficient of variation of approximately 0.25.



Line Name	Type	Formula
ASCE7-05	Building Code	$\frac{PFA}{PGA} = 1 + 2.00\,(z/h)$
All	Linear regression	$\frac{PFA}{PGA} = 1 + 2.27\,(z/h)$
PGA ≥ 0.1 g	Linear regression	$\frac{PFA}{PGA} = 1 + 1.19\,(z/h)$
PGA < 0.1 g	Linear regression	$\frac{PFA}{PGA} = 1 + 2.49\,(z/h)$

Figure 1: The effect of PGA on PFA/PGA from Fathali and Lizundia (2011). The regression lines are for the mean plus one standard deviation values of 2,224 data points taken from 151 fixed-base CSMIP building stations.

With nonlinear response of the building, the floor accelerations that the structure can deliver to the nonstructural component are effectively limited. PFA and PCA demands increase at a smaller rate with increasing PGA, and the PFA spectral shapes change with wider peaks at higher levels of PGA. The reduction in PFA/PGA with nonlinear building response has been shown in several studies including Sullivan et al. (2013) and Vukobratović and Fajfar (2016).

Given the significant effect of ground shaking on nonstructural component response, it was decided to continue to include the intensity of ground shaking in the proposed nonstructural design equation. The relatively limited amount of recorded data with large shaking levels underscores the importance of using analytical studies where shaking levels at the design earthquake level and beyond can be investigated.

Seismic Force-Resisting System

Different seismic force-resisting systems can have different dynamic characteristics including building modal periods and change in stiffness with height. This has been shown to affect the PCA/PFA response by Fathali and Lizundia (2011) and others. One approach is to categorize buildings into "stiff" (such as shear walls) and "flexible" (such as moment frames) groups and compare their response. Figure 2 shows the differences in the PCA/PGA spectral shape between stiff buildings and flexible buildings. Another approach is simply to use the building period as a surrogate for SFRS. The effect of SFRS was evaluated further in archetype analytical studies, and it is incorporated in building ductility assumptions.

Fundamental Period of the Building

Building modal periods have been shown to affect both PFA/PGA and PCA/PGA. Buildings with longer fundamental periods tend to have less amplification of PFA up the height of the building. See Miranda and Taghavi (2009) and Fathali and Lizundia (2011a).

Figure 3 and Figure 4 from Miranda and Taghavi (2009) are based on mean values from linear elastic models subjected to 80 recorded ground motions and show how a longer fundamental period of the building resulted in a lower ratio of PFA/PGA for both shear wall buildings and moment-resisting frame buildings and for buildings with lateral system stiffness somewhere in between. Figure 3 plots the impact of the fundamental period of the building on PFA/PGA for three different lateral stiffness ratios. Figure 4 conversely plots the impact for four different periods.

Component response tends to be amplified when T_{comp} and T_{bldg} approach resonance. This effect typically occurs at the first, second, and even third building modes in low-rise to mid-rise buildings. For taller structures, large amplifications can occur at modes higher than the third mode as well.

Although higher modes can be important, for simplicity, it was decided to use the fundamental translational period in the direction of interest in the nonstructural design equation in the derivation of PFA/PGA, as well as in aspects of PCA/PFA.

Building Ductility

Building ductility has been shown to affect component response. Typically, PCA/PGA is larger when the building is elastic and lower when there is nonlinearity of the building.

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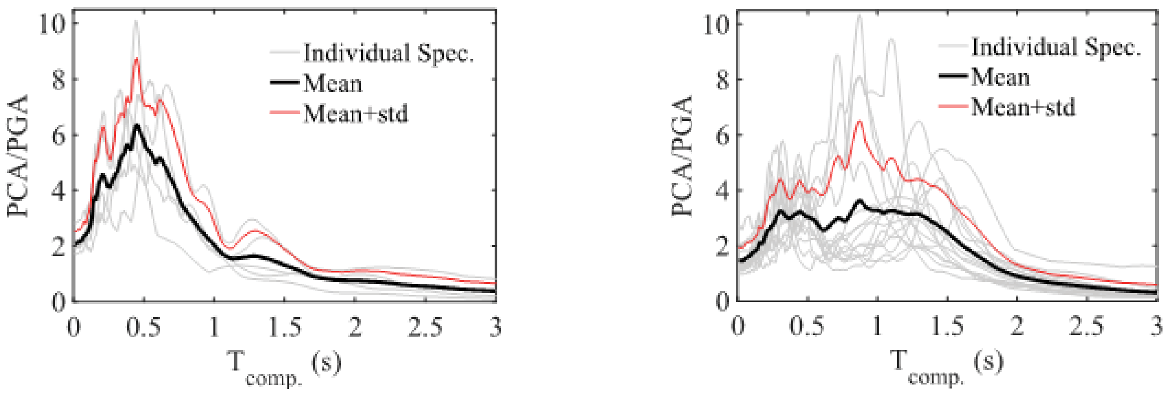


(a) normalized roof spectra for relatively stiff five-to -seven story buildings (a sample size of seven)

(b) normalized roof spectra for relatively flexible five-to-seven story buildings (a sample size of 24)

Figure 2: The effect of building stiffness on PCA/PGA for instrumental recordings. An elastic component is assumed with β_{comp} = 5%. The dataset includes 19 recordings with PGA > 0.15g.

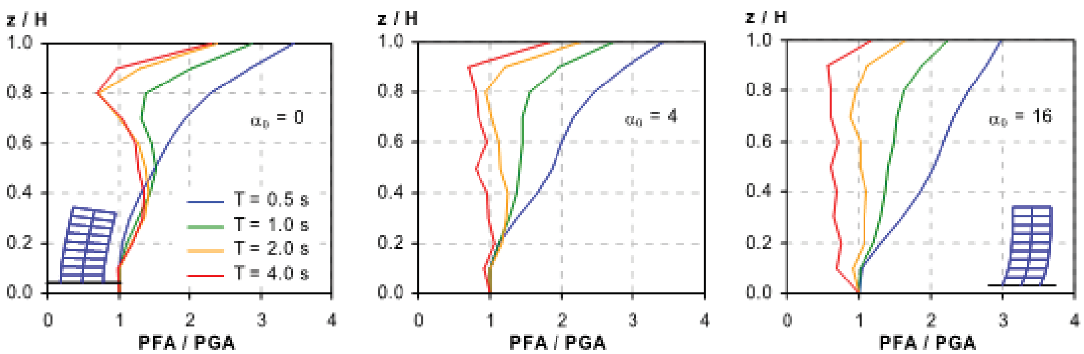


Figure 3: The effect of period of vibration and lateral system stiffness on PFA/PGA (from Miranda and Taghavi, 2009). The parameter α is the lateral stiffness ratio defined as $\alpha_0 = H(GA/EI)^{0.5}$, where *H* is height, *GA* is the shear rigidity of a shear beam and *EI* is the flexural stiffness. A value of α_0 = 0 represents a pure flexural model; a value of α approaching infinity represents a pure shear beam.

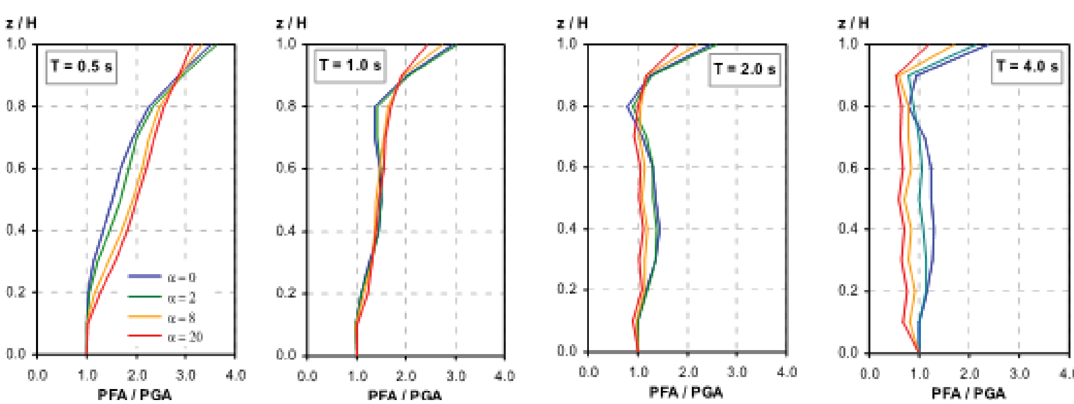


Figure 4: The effect of period of vibration and lateral system stiffness on PFA (from Miranda and Taghavi, 2009). The parameter α_0 is defined in Figure 3.

Studies showed significant reduction in PCA/PFA with increasing levels of shaking, though PCA itself still increases with increasing levels of shaking. It can be difficult to separate building ductility from ground shaking intensity.

The effect of building ductility is not only to reduce the PFA/PGA and PCA/PFA ratios. If an earthquake ground motion induces inelastic response in the supporting structure, the floor spectral accelerations will saturate over a wider non-structural period range because of the lengthening of the effective period of the supporting structure. Sullivan et al. (2013) showed the change in floor acceleration response spectral shapes resulting from the inelastic response of supporting structures.

Figure 5 shows results for a six-story steel SMRF archetype where building ductility is presented. The parameter R_{cb} is equal to the ratio of the floor response spectrum ordinate of the *elastic* version of the archetype model to the floor response spectrum ordinate of the *baseline* archetype model. Thus, a value of $R_{cb} > 1$ implies that building inelasticity reduces PCA responses. Component damping ratios are varied from 1% to 5%, and floor spectra are presented at the roof level and at the second floor level for an elastic nonstructural component. Figure 5 shows that for both damping levels and for both the roof and second floor, the mean R_{cb} of 20 spectrum-compatible (SC) records is above 1.0, except for a few periods where it is just a bit below 1.0. There are some individual records with values of $R_{cb} < 1$, typically at lower component periods, meaning in those cases building nonlinearity increased component response. The effect of building nonlinearity is more significant at the primary building modal periods. For example, at the building fundamental mode, for 5% damping at the roof, a nonlinear building reduces PCA by a factor of about 2 compared to an elastic version of the same SMRF archetype. NIST (2018) provides additional comparisons for other archetype structures showing the influence of building ductility on component response.

Given the importance of building ductility, a parameter to capture the effect of building ductility on reducing nonstructural response is included in the proposed primary nonstructural design equation. The system response modification factor, R, defined in the building code for all SFRSs was selected for simplicity.

Inherent Building Damping

Damping in a building can be categorized into inherent (or viscous) and hysteretic. Hysteretic damping occurs when there is yielding of the structural elements and subsequent post-yield ductility. Hysteretic building damping is thus covered by the previous section on building ductility.

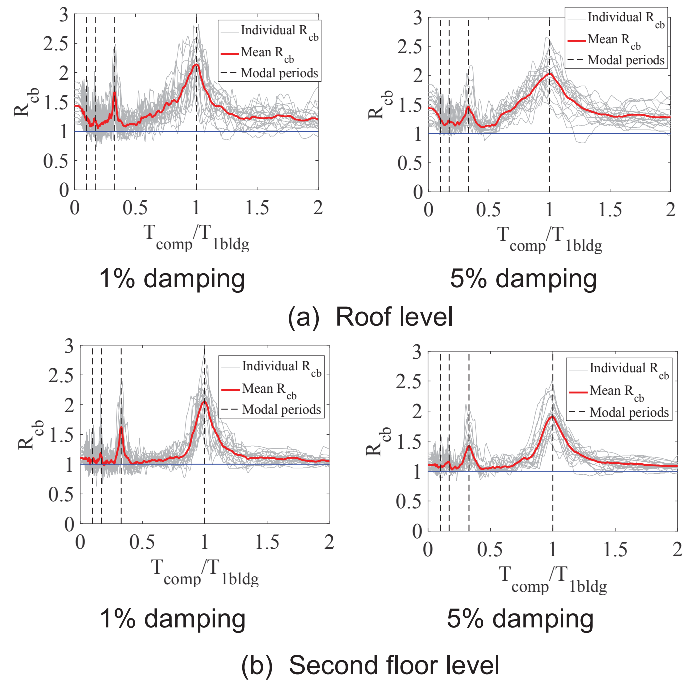


Figure 5: Effect of building nonlinearity on PCA; baseline archetype six-story steel SMRF.

Inherent building damping has a relatively small effect on component response. As an example, archetype studies conducted on a six-story steel SMRF building showed that as inherent damping was increased from 2.5% to 5%, the ratio of PCA/PGA in the first mode region decreased by only 5%. For an eight-story RCSW building, the ratio decreased by only 3%. In the second mode region, the effects were larger with decreases of 19% for the steel SMRF and 15% for the RCSW building.

Given the relatively small effect of inherent building damping on component response when compared to the effect of other parameters evaluated, inherent building damping was not included in the proposed nonstructural design equation.

Building Configuration

Building plan and vertical irregularities can affect the PFA. For example, a penthouse or a setback on a story can concentrate and amplify PFA. An "L"-shaped building with a reentrant corner can have increased amplification of PFA at the tips of the building wings. Although there are design requirements to address building structural system irregularities, the influence of building configuration is not included in the ASCE 7-16 nonstructural design equation. Studies were conducted to carefully scrutinize the building configuration of instrumented buildings, identify buildings with irregularities, and observe whether they can lead to increased PFA and PCA. These studies concluded that, in many instrumented buildings, the shape and magnitude of the floor response spectra can significantly depart from those obtained from two-dimensional numerical models. One of the primary reasons for these inconsistencies is the building's mass and/or stiffness irregularity. Detailed results from the project studies are presented in Appendix B of NIST (2018). See also Anajafi and Medina (2018).

Given the complexity of the issue, however, effects of building configuration were not included in the proposed nonstructural design equation.

Floor and Roof Diaphragm Flexibility

When project-specific dynamic analysis is used to determine nonstructural seismic design forces in ASCE 7-16 Section 13.3.1.4, ASCE 7-16 Equation 13.3-4 includes a torsional amplification factor, A_x, that accounts for torsional increases at the diaphragm perimeter. This approach is rarely employed in practice. Typically, ASCE 7-16 Equation 13.3-1 is used. It assigns the same force to nonstructural component design regardless of where the component is located on the diaphragm and regardless of whether the diaphragm is rigid or flexible. With a rigid diaphragm, there will be both actual and accidental torsional response that will increase PFA at the perimeter of the diaphragm with respect to PFA at the center of rigidity. With a flexible diaphragm, there can be substantial amplification of PFA at the center of the diaphragm as compared to the ends of the diaphragm over walls.

Torsional PFA increases in rigid diaphragms are dependent in part on how recordings are processed. For example, some studies simply take the mean of all the recordings at any floor in a particular direction. Others take the worst recording in the direction. See Fathali and Lizundia (2011) for a detailed summary of various potential approaches. The extent of PFA increase at the perimeter depends on the building floor geometry, SFRS layout and nonlinearity, and location of the perimeter point of interest with respect to the center of rigidity of the floor. For flexible diaphragms, the increase in midspan PFA needs to be evaluated carefully to account for interaction of component period. It is also dependent on the diaphragm aspect ratio and SFRS layout and nonlinearity.

Studies conducted as part of this project demonstrate that the in-plane diaphragm flexibility and torsional responses of the supporting buildings can significantly alter the shape and magnitude of the floor spectra with respect to those obtained from equivalent numerical building models that incorporate rigid diaphragms and symmetry in strength and stiffness of the SFRS elements. Amplifications in PFA and PCA responses in single-story buildings with plywood diaphragms can be as large as 5.0, whereas for the studied multistory buildings the amplifications are as large as 2.0. Torsional responses of the building can also increase the floor acceleration responses as well as acceleration demands on nonstructural components located in the floor periphery. This amplification is bounded to approximately 1.5 for the studied instrumented buildings. Detailed results from the project studies are presented in NIST (2018). See also Anajafi and Medina (2018).

Given the complexity of the issue, the effects of diaphragms are not included in the proposed primary nonstructural design equation. Response is based primarily on mean results from rigid diaphragm buildings. Response at the perimeter of buildings with significant torsion or at the midspan of flexible diaphragms may be worse.

Vertical Location of Component within the Building

PFA and PCA are known to increase with the height within the building. The ASCE 7-16 equation uses a linear increase from PGA at the ground to three times PGA at the roof. Fathali and Lizundia (2011) showed that this is generally conservative. Buildings with larger periods have less amplification. As noted above, they also found that amplification is lower with higher levels of PGA. Studies for the ATC-120 project found similar results. Fathali and Lizundia (2011) developed equations for different levels of shaking and for PFA/PGA for $T_{abldg} < 0.5$ seconds, $0.5 < T_{abldg} < 1.5$ seconds, and $T_{abldg} > 1.5$ seconds. The drawback is that there is a step in the PFA/PGA value between each group. Details are in NIST (2018).

Additional project studies were conducted using instrumental data from buildings PCA > 0.9g to develop a nonlinear equation for PFA/PGA that depends only on z/h and T_{abldg} but does not have steps at different periods. The equation is shown below.

$$\left(\frac{\text{PFA}}{\text{PGA}}\right) = 1 + a_1\left(\frac{z}{h}\right) + a_2\left(\frac{z}{h}\right)^{10}$$

where:

$a_1 = \dfrac{1}{T_{aBldg}} \leq 2.5$

$a_2 = [1 - (0.4/T_{abldg})^2] > 0$

T_{abldg} = ASCE 7-16 Equation 12.8-7

The equation is based on both the recorded variation in PFA normalized by PGA in instrumented buildings and the mean (average) variation computed in simplified continuous models consisting on a flexural beam laterally coupled with a shear beam as adapted from Taghavi and Miranda (2006) and from Alonso-Rodríguez and Miranda (2016). Figure 6 shows results for different building periods.

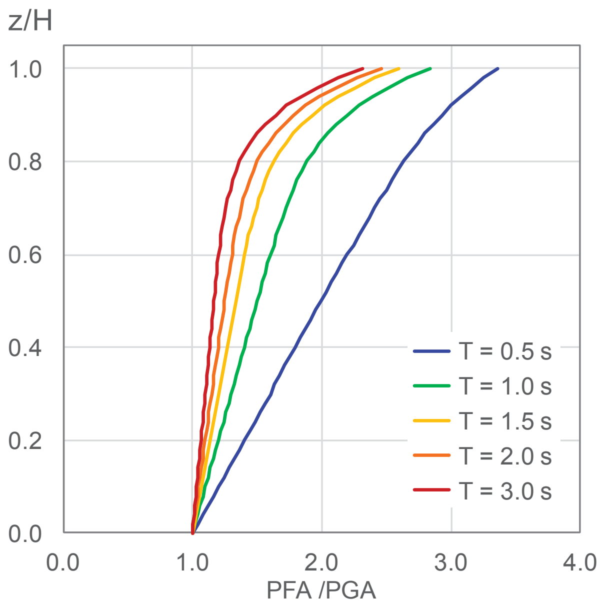


Figure 6: Sample equation for PFA/PGA for PCA > 0.9g.

For instrumented buildings, processed data were used. Earlier studies by others have often used uncorrected data. In some buildings, there are large differences between PFAs in uncorrected data and in processed data. Whenever possible, the PFA/PGA normalization was done using an instrument located at ground level directly below the upper sensors. The model was validated by comparing recorded vs. computed floor acceleration time histories and floor spectra in more than 100 instrumented buildings in California. The mean results were slightly modified to take into account (1) small variations in PFA/PGA between uniform and non-uniform buildings and (2) the difference between actual periods of vibration in the building and approximate equations computed by the code. It is well known that code equations typically underestimate building fundamental periods in order not to underestimate force demands (Goel and Chopra, 1997 and Goel and Chopra, 1998). Since the ASCE 7-16 formula for approximate fundamental period was selected for use in the proposed design equations, modifications were made in the PFA/PGA relationship developed using measured or calculated periods to correlate the relationship with the code formula.

The PCA/PFA relationship developed as part of this project was selected for use in the proposed nonstructural design equation.

Component Period

Component period has been shown to have a significant effect on PCA/PFA response. Floor spectra are characterized by ordinates equal or approximately equal to the peak floor acceleration for frequencies larger than about 30 Hz and with large amplifications at frequencies near or equal to modal frequencies of the building. Nonstructural components with components periods longer than about two times the fundamental period of the building (although this rarely occurs) typically undergo accelerations smaller than the peak floor acceleration. Figure 2 shows some example spectra that illustrate the general shape and some of the variations of instrumentally recorded floor spectra. The current ASCE 7-16 Equation 13.3-1 simplifies this effect with the a_p factor, such that PCA/PFA is equal to 1 for up to T_{comp} = 0.06 seconds and then jumps to PCA/PFA = 2.5 for T_{comp} > 0.06 seconds. Figure 7 compares the ASCE 7-16 formula with the maximum, mean plus one standard deviation, and mean for 3,743 floor acceleration histories (for an elastic component with 5% inherent component damping).

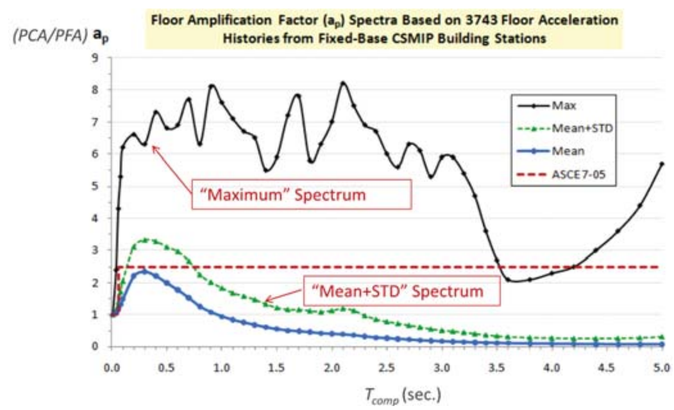


Figure 7: PCA/PFA versus T_{comp} in Fathali and Lizundia (2011). An elastic component is assumed with β_{comp} = 5%.

It has also been shown that PCA/PFA rises significantly as T_{comp} and T_{bldg} approach resonance. Figure 8 illustrates the PCA/PFA amplification factor with the spectral ordinates of the average of eight different recorded motions based on T_{comp} and the same motions with the x-axis normalized to T_{comp}/T_{bldg}. These records come from eight different buildings and five different earthquakes and were selected from a suite of 86 records with 5% PCA values over 0.9g. The significant amplification of demand when the component period matches one of the building periods, typically referred to as resonance, was an important subject of investigation since the peak component accelerations can greatly exceed those typically used for design.

Although ASCE 7-16 Equation 13.3-1 does not explicitly include a factor for T_{comp}/T_{bldg}, the commentary in ASCE 7-16 Section C13.3.1 does show Figure C13.3-1 for determining a_p when T_{comp}/T_{bldg} is known, based on research from the National Center for Earthquake Engineering Research. There are other modern building codes that do explicitly include the T_{comp}/T_{bldg}

ratio in the code design formulas, such as the Eurocode 8 (CEN, 2004).

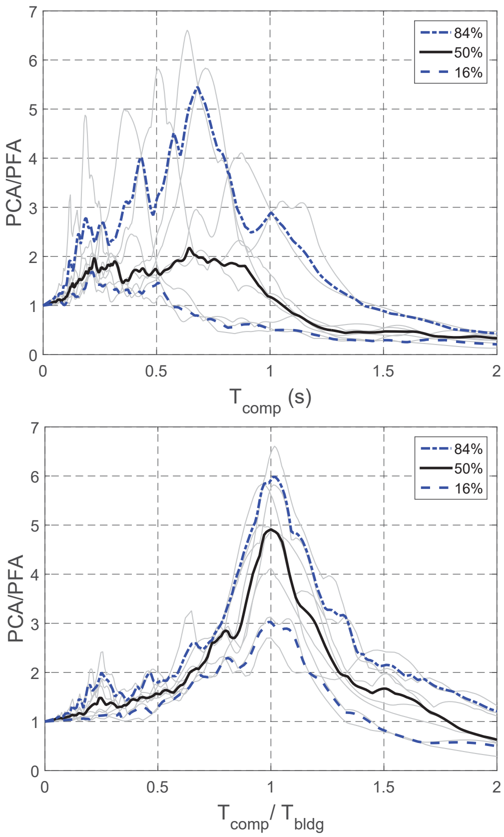


Figure 8: Relationship between PCA/PFA comparing spectra without (top) and with (bottom) normalization by T_{bldg}. An elastic component is assumed with βcomp = 5%. The dataset includes eight recordings with PCA > 0.9g. From Kazantzi et al. (2018).

It was decided to include the effect of resonance and the presentation approach of normalizing response against T_{comp} / T_{bldg} in the proposed nonstructural design equation.

Inherent Component Damping

Similar to damping for buildings, component damping can also be categorized into inherent (viscous) and hysteretic. Also like buildings, hysteretic component damping occurs when there is component yielding and thus component ductility which is addressed in the next section.

Analytical studies of buildings and nonstructural components have typically used 5% inherent damping for both. Actual inherent damping in buildings is known to be substantially more complicated, depending on the building characteristics and the level of shaking. For nonstructural components, various literature sources were investigated, and it was found that inherent damping in nonstructural components is typically less than 5%, with some as low as 2%.

Figure 9 shows the effect of inherent component damping on PCA. At T_{comp}/T_{bldg} resonance, PCA for 2% inherent component damping is 1.6 times that with 5% inherent component damping. This ratio drops to about 1.2 for T_{comp} /T_{bldg} < 0.7 and T_{comp} /T_{bldg} > 1.7.

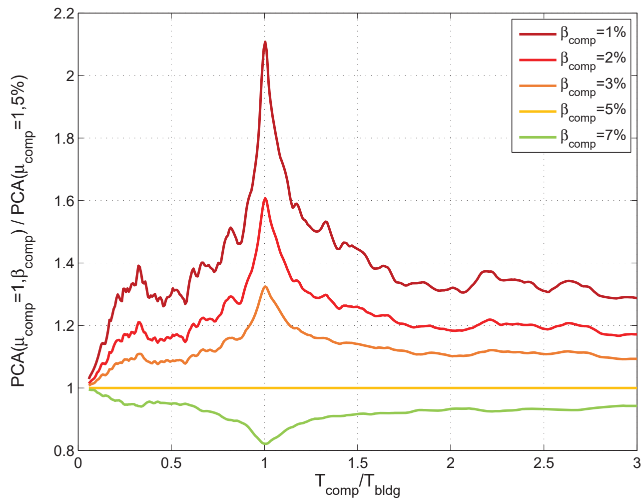


Figure 9: Mean elastic PCA ratios between inherent component damping, T_{comp}, of 5% and other damping levels. The dataset includes 86 recordings with PCA > 0.9g.

Initially, 2% inherent component damping was tentatively selected as the underlying basis of the nonstructural design equation. However, there is concern that there is insufficient research on damping for many components, particularly at high shaking levels of interest where it is assumed that damping will be larger; that distributed system components will likely have higher levels of inherent damping; and that use of 2% is inconsistent with the underlying 5% assumption of the equations in ASCE 7-16 Chapter 15 for nonbuilding structures and would lead to inconsistencies. As a result, it was decided to continue with the traditional assumption of 5% inherent component damping, highlight the need for additional research, and account for potentially lower levels of damping in some components like rigid mechanical equipment by potentially assigning them to lower ductility categories.

Component and/or Anchorage Ductility

ASCE 7-16 Equation 13.3-1 includes the R_p factor which indirectly accounts for the reductions in response that component ductility can provide, but there is not an explicit link in the code between the R_p and μ_{comp}. The New Zealand nonstructural design code (NZS, 2004), on the other hand, does explicitly permit incorporation of component ductility and provides design values to use for different levels of component ductility, and the NZS (2004) commentary provides component ductility values for a set of common components.

There can be ductility in the component, the attachment of the component to the anchor, the anchor itself, or a combination of any of the items. Anchorage ductility is often difficult or impossible to achieve given practical considerations of available substrate depth for anchor. Project studies lumped these three potential sources of component and anchorage ductility together into one simple model where the single-degree-of-freedom (SDOF) oscillator representing a component yields and then continues to deform, providing a measure of component ductility. Component ductility was found to have a significant effect.

For β_{comp} = 2% at T_{comp}/T_{bldg} = 1 resonance, PCA/PFA drops from 7.5 for an elastic component, to about 3.8 for a component with a ductility, μ_{comp}, of 1.25, to about 2.4 for μ_{comp} = 1.5, and then to about 1.5 for μ_{comp} = 2. Figure 10 overlays the mean response for each component ductility level for β_{comp} = 5%. For β_{comp} = 5% at T_{comp}/T_{bldg} = 1 resonance, PCA/PFA drops from about 4.6 for an elastic component, to about 2.8 for a component with a ductility, μ_{comp}, of 1.25, to about 2.0 for μ_{comp} = 1.5, then to about 1.4 for μ_{comp} = 2. Note that for these "constant component ductility" PCA/PFA spectra, the strength of the component is different for each level of μ_{comp} at each value of T_{comp}.

Given the significant effect of component and/or anchorage ductility on component response, it was decided to explicitly incorporate this effect in the proposed nonstructural component design equation.

Component Reserve Strength Margin

For *building* design, there is an inherent reserve strength margin between the design value and the eventual peak strength. This comes in part from capacity reduction factors, ϕ, but also as a result of other design factors, design simplifications, redundancy, and design decisions. This inherent reserve strength margin is a substantial part of the response modification coefficient, *R*, that is used to reduce elastic response levels down to design levels. See the discussion on R_o in the *SEAOC Recommended Lateral Force Requirements and Commentary* (SEAOC, 1999).

It is assumed that *components* also have some inherent reserve strength margin that occurs as part of the design process. This inherent reserve strength margin has traditionally been considered by code writers in the development of nonstructural design equations. While a component's inherent reserve strength margin factor has not been explicitly identified, effects have been considered as part of the R_p factor.

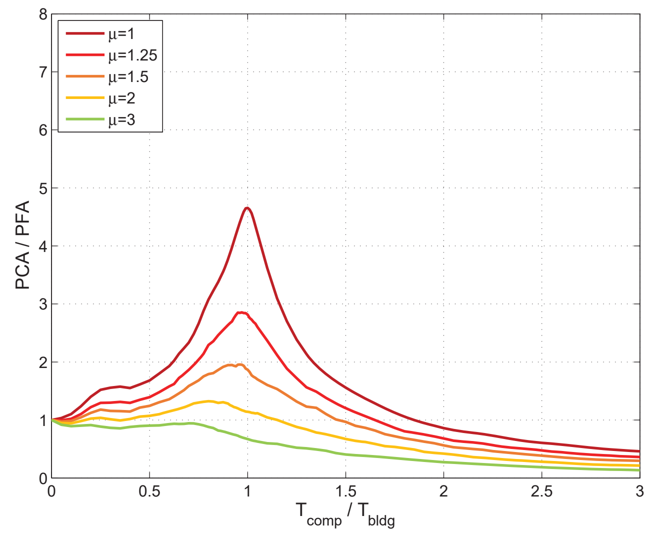


Figure 10: Comparison of mean response of PCA/PFA versus T_{comp}/T_{bldg} for different levels of component ductility for β_{comp} = 5%. The dataset includes 86 recordings with PCA > 0.9g.

The component reserve strength margin factor is related to the component itself and should not be confused with the component *anchorage* overstrength design force increase factor, Ω_0, provided in Tables 13.5-1 and 13.5-2 of ASCE 7-16. The background on the development of the component anchorage overstrength design force is given in NIST (2018).

It was decided to explicitly incorporate a placeholder value for the effect of component reserve strength in the proposed nonstructural component design equation. The ATC-120 project team decided to assume a placeholder value of 1.3 for the inherent component reserve strength margin. This is termed R_{pocomp}.

ATC-120 Proposed Nonstructural Design Equations

The ATC-120 proposed primary nonstructural design equation is summarized in NIST (2018) as follows. It incorporates the parameters found to have significant influence on component response, including ground shaking intensity, seismic force-resisting system of the building, building modal period, building ductility, inherent building damping, vertical location of the component within the building, component period, component and/or anchorage ductility, inherent component damping, and component overstrength.

Ground shaking intensity is directly captured by the PGA term in the above equation. Modification of ground shaking intensity from soil-structure interaction is not included in the formulation for simplicity.

The proposed equation framework makes the assumption that PFA/PGA and PCA/PFA can be effectively separated. This provides a distinction where PFA/PGA primarily relates to what the building does, and PCA/PFA relates to how the components respond to what the building is doing. It is recognized that the terms and underlying parameters are not independent. A few project studies were conducted that focused on PCA/PGA and thus effectively combining the PFA/PGA and PCA/PFA terms, and this has also been done by some researchers in the past. However, it was decided to keep the PFA/PGA and PCA/PFA terms separated for simplicity.

$$\frac{F_p}{W_p} = \mathrm{PGA} \times \left[\frac{\left(\frac{\mathrm{PFA}}{\mathrm{PGA}} \right)}{R_{\mu bldg}} \right] \times \left[\frac{\left(\frac{\mathrm{PCA}}{\mathrm{PFA}} \right)}{R_{pocomp}} \right] \times I_p$$

where:

PGA = peak ground acceleration. Use $0.4S_{DS}$ until this is directly provided in future ASCE 7 editions.

$$\left(\frac{\mathrm{PFA}}{\mathrm{PGA}} \right) = 1 + a_1 \left(\frac{z}{h} \right) + a_2 \left(\frac{z}{h} \right)^{10}$$

where:

$a_1 = \dfrac{1}{T_{aBldg}} \leq 2.5$

$a_2 = [1-(0.4/T_{abldg})^2] > 0$

T_{abldg} = ASCE 7-16 Equation 12.8-7

$R_{\mu bldg}$ = reduction factor to account for building global ductility

where:
$R_{\mu bldg} = (R_D)^{1/2} = (1.1R / \Omega_0)^{1/2}$
where R and Ω_0 are the response modification coefficient and the overstrength factor from ASCE 7-16 Table 12.2-1. $R_{\mu bldg}$ need not be taken as less than 1.0.

(PFA/PGA) = factor to account for component amplification, inherent component damping, and component ductility per Table 1.

R_{pocomp} = inherent component reserve strength margin factor. A value of 1.3 is used as a placeholder

PGA

The PGA parameter can be obtained directly from the forthcoming proposed USGS multi-period spectra plots or approximated as $0.4S_{DS}$.

Table 1: PCA/PFA Values

Location of Component	Possibility of Being in Resonance with Building	Component Ductility: *Category*(1)	Component Ductility: *Assumed Ductility*	$\left(\frac{\mathrm{PCA}}{\mathrm{PFA}}\right)$ (2)
Ground	More Likely	Elastic	$\mu_{comp} = 1$	2.5
		Low	$\mu_{comp} = 1.25$	2.0
		Moderate	$\mu_{comp} = 1.5$	1.8
		High	$\mu_{comp} \geq 2$	1.4
	Less Likely	Any	--	1.0
Roof or Elevated Floor	More Likely	Elastic	$\mu_{comp} = 1$	4.0
		Low	$\mu_{comp} = 1.25$	2.8
		Moderate	$\mu_{comp} = 1.5$	2.2
		High	$\mu_{comp} \geq 2$	1.4
	Less Likely	Any	--	1.0

(1) Categories are assigned to components similar to ASCE 7-16 Table 13.5-1.
(2) Inherent component damping of 5% is assumed.

PFA/PGA

The equation for PFA/PGA is based on the mean response of all recordings. Thus, it does not directly capture increases in PFA at the perimeter due to torsion, but since many sensors are located at the building perimeter, values are higher than at the center of rigidity. This is judged to be adequate for the proposed nonstructural design equation.

$R_{\mu bldg}$

Based on project studies, building ductility typically reduces component response. Section 105.3.2 of SEAOC (1999) defined the seismic response modification factor, R, as:

$$R = R_D \times R_o$$

where:
R_D = global ductility coefficient (commonly identified as μ and termed μ_{bldg} in this report) at design earthquake level
R_o = global overstrength coefficient

where:
$\Omega_0 = 1.1R_o$
R_D then equals $1.1R/\Omega_0$

Initially, it was assumed that the reduction in PFA is proportional to the square root of R_D. The factor for reduction is termed $R_{\mu bldg}$, and it is then defined as follows.

$$R_{\mu bldg} = (R_D)^{1/2} = (1.1R/\Omega_0)^{1/2}$$

Studies focused on the exponent in the $R_{\mu bldg} = (R_D)^x$ equation were conducted to test the effectiveness of assuming $x = 1/2$. Figure 13 shows the ratio of $F_{p,actual}/F_{p,proposed}$ for a six-story steel SMRF archetype at the roof level.

$F_{p,actual}$ is the mean of 20 spectrum-matched ground motions, and $F_{p,proposed}$ is the proposed equation for the more likely in resonance case, with μ_{comp} = 1.5, and with 5% inherent component damping. Note that when calculating $F_{p,actual}$ / $F_{p,proposed}$, the term R_{pocomp} (component reserve margin factor) is not included. Essentially, the designer uses a design force reduced by R_{pocomp}, but then the component inherently has the same value of overstrength, so the final or actual strength should not be reduced by R_{pocomp}.

Similar studies were run for eight-story RCSW archetypes and two-story steel SMRF and RCSW buildings. A value of 0.5 for the exponent envelopes the entire response (i.e., $F_{p,actual}$ / $F_{p,proposed}$ is at or below 1.0) for both the six-story steel SMRF and the 8-story RCSW baseline cases. A value of 0.5 for the exponent also captures the majority of the response for the overdesign cases.

PCA/PFA

The PCA/PFA term has a number of features embedded within it. These include narrow band amplification due to resonance with building periods, component ductility effects, the concept of flexible versus rigid components, and ground versus superstructure response.

Narrow Band Amplification

As previously discussed, the effect of amplification of the floor acceleration based on the period of the component occurs when the component is in resonance with one of the building periods.

This narrow band amplification is significant, but it may not be likely. A study was conducted to look at two groups of common building equipment. The first set was for equipment that was rigidly attached to the floor or roof. The second set was equipment that had flexible supports, such as vibration isolators. To study the likelihood of resonance, the probability of being within a region with narrow band acceleration was investigated by comparing the probabilities of the ratio of the component period to the building period being within the narrow band where amplifications occurs. Details are in NIST (2018).

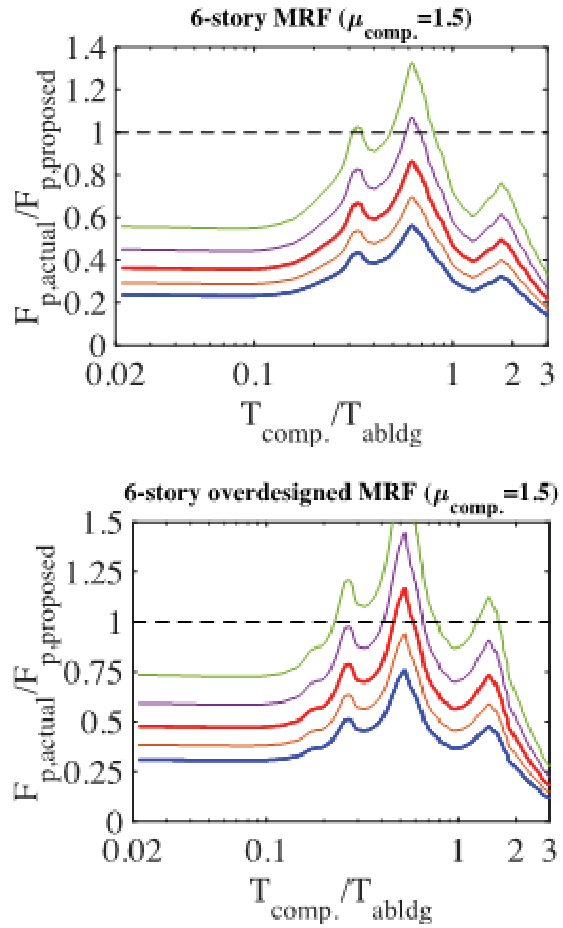


Figure 13: Ratio of $F_{p,actual}$ / $F_{p,proposed}$ at the roof for a 6-story steel moment frame archetype T_{abldg} = 0.93 s, assuming μ_{comp} = 1.5 and β_{comp} = 5% inherent component damping, where $F_{p,actual}$ is the mean of 20 spectrum-compatible ground motions. The plots are for different values of "x" in the $R_{\mu bldg} = (R_D)^X$, with the dark blue at the bottom being 0.1, the dark red in middle being 0.5, and the light green at the top being 0.9. The "overdesigned" archetype on the bottom has an additional strength of 1.5 compared to the baseline archetype on the top.

When $0.75 < T_{comp}/T_{nbldg} < 1.25$ is associated with a 10% probability band in the cumulative distribution functions, and if it is acceptable to have a 10% probability of forces exceeding those predicted by the equation, then PCA/PFA could be capped at some level. Using the 84th percentile curve from Figure 14, the capped value for PCA/PFA would be a value of 3 with $0.75 < T_{comp}/T_{nbldg} < 1.25$. When $0.85 < T_{comp}/T_{nbldg} < 1.15$ is associated with the 10% probability band in the cumulative distribution function, then the capped value would increase to a value of 4 in the 84th percentile curve from Figure 14.

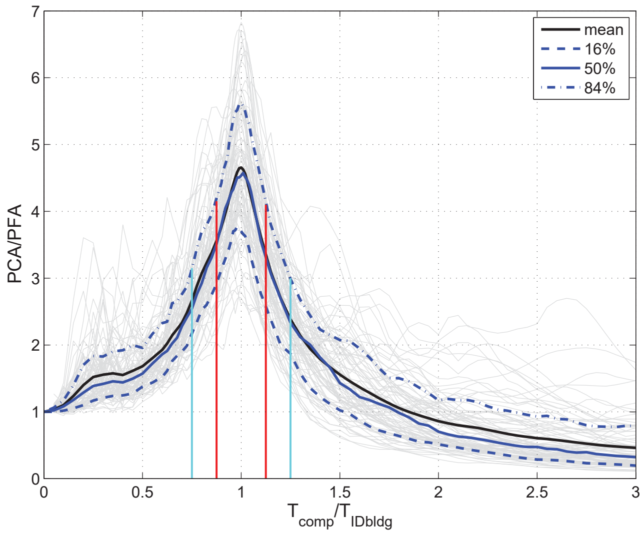


Figure 14: Component amplification and capping of PCA/PFA. Values outside the red band capture all cases except those with 0.85 < T_{comp}/T_{nbldg} < 1.15. Similarly, values outside the cyan band capture all cases except those with 0.75 < T_{comp}/T_{nbldg} < 1.25. β_{comp} = 5% inherent component damping is assumed. The dataset includes 86 recordings with PCA > 0.9g.

Component Ductility

Additional archetype studies were performed for four archetypes (two-story steel SMRF, six-story steel SMRF, two-story RCSW, and eight-story RCSW) using the proposed equation and different levels of component ductility. A peak-oriented (stiffness degrading) model with 3% strain hardening was used to account for the hysteretic behavior of the component. Figure 15 shows results for a six-story steel SMRF. Additional results and details are shown in NIST (2018). The proposed equation uses the values for components likely to be in resonance. Thus, the comparison is most relevant when T_{comp}/T_{abldg} is not small and not large. All of these archetypes include an overdesign factor that has been shown to be conservative. Results for μ_{comp} = 1.0, 1.5, 2.0, 3.0, and 4.0 are shown. The focus here is on the 1.0, 1.5, and 2.0 values. In general, the proposed equation envelopes the majority of response, though μ_{comp} = 1.0 and 2.0 are less conservative than the other values. For the μ_{comp} = 3.0 and 4.0 runs, the PCA/PFA values for μ_{comp} = 2.0 were assumed.

Flexible versus Rigid Components

ASCE 7-16, like many previous codes, defines "rigid" components as those with $T_{comp} \leq 0.06$ seconds, and "flexible" components as those with $T_{comp} > 0.06$ seconds. Rigid components are assigned an a_p (effectively PCA/PFA) of 1. Few components actually have $T_{comp} \leq 0.06$ seconds, particularly if base and anchorage flexibility are included. Nonetheless, ASCE 7-16 Tables 13.5-1 and 13.6-2 have many common components with an a_p = 1, including partitions, ceilings, cabinets, and the body of cladding.

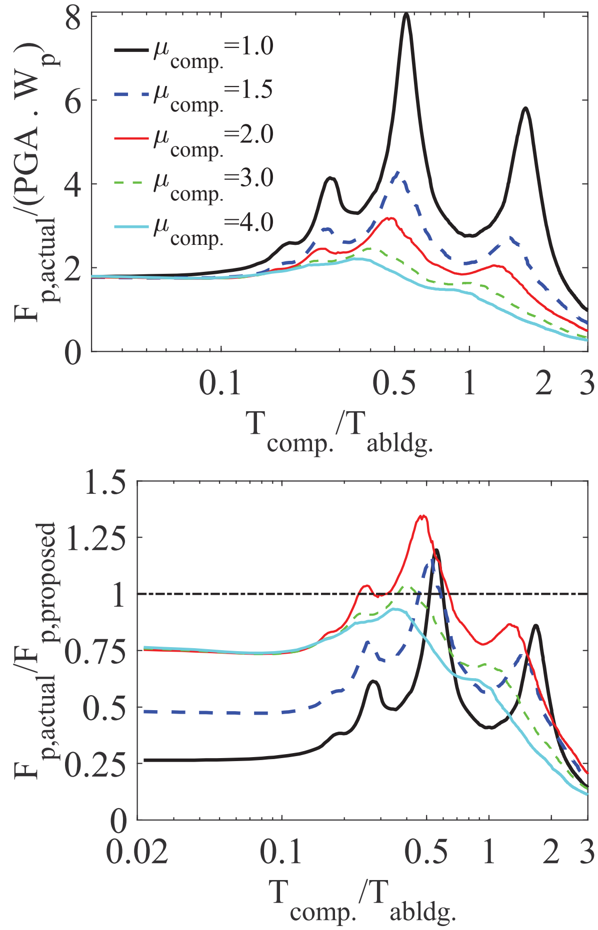


Figure 15: (top) Mean of the simulated $F_{p,actual}$ normalized by PGA and (bottom) normalized by $F_{p,proposed}$ at the roof for a 6-story steel moment-resisting frame archetype assuming an overdesign factor of 1.5, μ_{comp} = 1.0-4.0, and β_{comp} = 5% inherent component damping, where $F_{p,actual}$ is the mean of 20 spectrum-compatible ground motions. $F_{p,proposed}$ is for components *likely* to be in resonance.

Part of the original thinking of code writers behind assigning these components a value of a_p =1 was not that the components could rigorously be shown to have $T_{comp} \leq 0.06$ seconds, but rather they were more likely to be out-of-phase with typical building periods, and thus PCA/PFA ratios were comparatively small. As an approximate placeholder, this could be assumed to effectively mean widening the bands in Figure 14 to say $T_{comp}/T_{IDbldg} < 0.5$ or $T_{comp}/T_{IDbldg} > 1.5$. As such, the project team has chosen to replace the terms "flexible" and "rigid" with "more likely to be in resonance with the building" and "less likely to be in resonance with the building." For simplicity, those that are less likely to be in resonance are assigned a value of PCA/PFA = 1.0. The resulting effect is similar to the ASCE 7-16 approach, but it is more descriptively precise.

Figures 15 compared the proposed equation against results for a six-story steel SMRF archetype buildings where the proposed equation represented the values for components *likely* to be in resonance. Figure 16 applies the same approach except that $F_{p,proposed}$ represents the values for components *unlikely* to be in resonance and should thus be compared against low and high values of T_{comp}/T_{abldg}. As Figure 16 shows, as T_{comp} approaches zero, the proposed equation matches the archetype results.

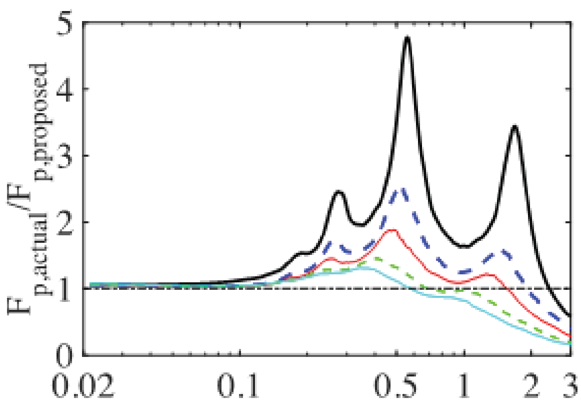


Figure 16: Mean of the simulated $F_{p,actual}$ normalized by $F_{p,proposed}$ at the roof for a six-story steel moment frame archetype assuming an overdesign factor of 1.5, μ_{comp} = 1.0-4.0, and β_{comp} = 5% inherent component damping, where $F_{p,actual}$ is the mean of 20 spectrum-compatible ground motions. $F_{p,proposed}$ is for components *unlikely* to be in resonance.

IT 5 Proposed Nonstructural Design Equations

As Issue Team 5 addressed moving the ATC-120 equations into a code update, key objectives for the proposal were to incorporate the influence of the building and supporting structure on nonstructural response based on detailed analytical work. The properties of the nonstructural components in the ASCE 7-16 Chapter 13 tables were examined, and new design coefficients were based chiefly on component properties as given in existing tables, modified when judged appropriate. Refinements of the component coefficients can be made as additional data becomes available. Several revisions were made, as Issue Team 5 refined the equations in NIST (2018). These include the following.

- Terminology revision: The terminology used for the variables in NIST (2018) was shortened.
- R_{pocomp} (now R_{po}) refinement: NIST (2018) has a placeholder value of R_{po} = 1.3 for all components. The consensus of code writers was that a value of R_{po} = 1.5 was reasonable for most components. However, components deemed to have somewhat higher than typical overstrength were assigned higher R_{po} values.
- R_{ubldg} (now R_u) clarification: For ground supported components, there is no building superstructure involved to modify response, so R_u was set to unity.
- Equipment support conditions: Equipment support conditions can have a significant impact on the response of the component itself. Provisions were added to address different types of support.
- Assignment of components to ductility and likelihood of resonance categories: Engineering judgment was used to assign components to whether they are likely or unlikely to be in resonance and if in resonance the appropriate ductility category.
- Maximum (cap) value: In the NIST (2018) report, a default component strength factor of R_{po} =1.3 was used together with a cap of $2.0S_{DS}I_pW$. With the typical value of the component strength factor increased to R_{po} =1.5 noted above, the ASCE 7-16 cap of $1.6S_{DS}I_pW_p$ will be generally compatible with the analytical results, and it was thus the consensus of code writers to retain the cap of $1.6S_{DS}I_pW_p$.
- The overstrength factors for anchors in concrete were slightly revised for compatibility with the proposed equation.

Excerpts from the 6/20/19 IT5 code change proposal are given below; the full proposal is too long for inclusion here.

13.3.1 Seismic Design Forces

The horizontal seismic design force, F_p, shall be applied at the component's center of gravity and distributed relative to the component's mass distribution, and be applied independently in at least two orthogonal horizontal directions in combination with service or operating loads associated with the component, as appropriate. The redundancy factor, ρ, is permitted to be taken as equal to 1.

The horizontal seismic design force shall be determined in accordance with Eq. (13.3-1):

$$F_p = 0.4S_{DS}I_pW_p\,[H_f/R_\mu]\,[C_{AR}/R_{po}] \quad (13.3\text{-}1)$$

F_p is not required to be taken as greater than

$$F_p = 1.6S_{DS}I_pW_p \quad (13.3\text{-}2)$$

and F_p shall not be taken as less than

$$F_p = 0.3S_{DS}I_pW_p \quad (13.3\text{-}3)$$

where

F_p = seismic design force;

S_{DS} = spectral acceleration, short period as determined in accordance with Section 11.4.5;

I_p = component Importance Factor as determined in accordance with Section 13.1.3;

W_p = component operating weight;

H_f = factor for force amplification as a function of height in the structure as determined in Section 13.3.1.1;

R_μ = structure ductility reduction factor as determined in Section 13.3.1.2;

C_{AR} = component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration as determined in Section 13.3.1.3;

R_{po} = component strength factor as determined in Section 13.3.1.4.

13.3.1.1 Amplification with Height, H_f

For components supported by a building or nonbuilding structure, the factor for force amplification with height, H_f, is determined by Eq. (13.3-4):

$$H_f = 1 + a_1\left(\frac{z}{h}\right) + a_2\left(\frac{z}{h}\right)^{10} \qquad (13.3\text{-}4)$$

where

$$a_1 = \frac{1}{T_a} \le 2.5 \qquad a_2 = [1 - (0.4/T_a)^2] > 0$$

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of $\frac{z}{h}$ need not exceed 1.0;

h = average roof height of structure with respect to the base; and

T_a = the approximate fundamental period of the supporting building or nonbuilding structure. For structures with combinations of seismic force-resisting systems, the lowest value of T_a shall be used.

For the purposes of computing H_f, T_a is determined using Eq. (12.8-7) for buildings. Where the seismic force-resisting system is unknown, T_a may be determined by Eq. (12.8-7) using the approximate period parameters for "All other structures." For nonbuilding structures, T_a may be taken as:

a. The period of the nonbuilding structure, T, determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 12.8.2, or
b. The period of the nonbuilding structure, T, determined using Eq. (15.4-6), or
c. The period T_a determined by Eq. (12.8-7), using the approximate period parameters for "All other structures."

For nonstructural components supported at or below grade, $H_f = 1.0$. Where the approximate fundamental period of the supporting building or nonbuilding structure is unknown, H_f may be determined by Eq. (13.3-5):

$$H_f = 1 + 2.5\left(\frac{z}{h}\right) \qquad (13.3\text{-}5)$$

13.3.1.2 Structure Ductility Reduction Factor, R_μ

For components supported by a building or nonbuilding structure, the reduction factor for ductility of the supporting structure, R_μ, is determined by Eq. (13.3-6):

$$R_\mu = (1.1R/\Omega_0)^{1/2} \qquad (13.3\text{-}6)$$

where

R = the response modification factor for the building or nonbuilding structure supporting the component, from Table 12.2-1, Table 15.4-1, or Table 15.4-2; and

Ω_0 = the overstrength factor for the building or nonbuilding structure supporting the component, from Table 12.2-1, Table 15.4-1, or Table 15.4-2.

The value of R_μ determined by Eq. (13.3-6) need not be taken as less than 1.3. For components supported at or below grade, the value of R_μ shall be taken as 1.0. When the seismic force-resisting system of the building or nonbuilding structure is not specified or is not listed in Table 12.2-1, Table 15.4-1, or Table 15.4-2, the value of R_μ shall be taken as 1.3.

If the building or nonbuilding structure supporting the component contains combinations of seismic force-resisting systems in different directions or vertical combinations of seismic force-resisting systems, the structure ductility reduction factor shall be based on the seismic force resisting system that produces the lowest value of R_μ. Where a nonbuilding structure type listed in Table 15.4-1 has multiple entries based on permissible height increases, the value of R_μ may be determined using values of R and Ω_0 for "With permitted height increase" entry.

13.3.1.3 Component Resonance Ductility Factor, C_{AR}

Components are assigned a component resonance ductility factor, C_{AR}, based on whether the component is supported at or below grade, or is supported above grade by a building or nonbuilding structure. Components that are in or supported by a building or nonbuilding structure and are at or below grade plane are considered supported at or below grade. All other components in or supported by a building or nonbuilding structure are considered supported above grade.

Architectural components are assigned a component resonance ductility factor in Table 13.5-1.

Table 13.5-1

Example		ARCHITECTURAL COMPONENTS	~~a_p~~ [a]	~~R_p~~	~~Ω_0~~ [b]	C_{AR} Supported at or below grade	C_{AR} Supported above grade by a structure	R_{po}	Ω_{op} [a]
		Interior nonstructural walls and partition[eb]							
5		~~Plain (unreinforced) masonry walls~~	~~1~~	~~1½~~	~~1½~~				
3		Light frame ≤ 9 ft (2.74 m) in height				1	1	1.5	1.5
4		Light frame >9 ft (2.74 m) in height				1.4	1.4	1.5	2
3		Reinforced Masonry				1	1	1.5	1.5
5		All other walls and partitions	~~1~~	~~2½~~	~~2~~	2	2.8	1.5	1.5
		Cantilever elements (unbraced or braced to structural frame below its center of mass)							
13		Parapets and cantilever interior nonstructural walls	~~2½~~	~~2½~~	~~2~~	1.8	2.2	1.5	1.75
13		Chimneys where laterally braced or supported by the structural frame	~~2½~~	~~2½~~	~~2~~	1.8	2.2	1.5	1.75
		Cantilever elements (braced to structural frame above its center of mass)							
3		Parapets	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
3		Chimneys	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
3		Exterior nonstructural walls	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
		Exterior nonstructural wall elements and connectionsb							
3		Wall element	~~1~~	~~2½~~	~~NA~~	1	1	1.5	1.5
3		Body of wall panel connections	~~1~~	~~2½~~	~~NA~~	1	1	1.5	1.5
9		Fasteners of the connecting system	1¼	~~1~~	~~1~~	2.2	2.8	1.5	1
		Veneer							
3		Limited deformability elements and attachments	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
1		Low-deformability elements and attachments	~~1~~	~~1½~~	~~2~~	1	1	1.5	1.5
		Penthouses (except where framed by an extension of the building frame)	~~2½~~	~~3½~~	~~2~~				
17		Seismic Force-Resisting Systems with R ≥6				NA	1.4	2	2
18		Seismic Force-Resisting Systems with 6>R ≥ 4				NA	2.2	2	1.75
19		Seismic Force-Resisting Systems with R < 4				NA	2.8	2	1.5
20		Other Systems				NA	2.8	1.5	1.5
		Ceilings							
3		All	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
		Cabinets							
3		Permanent floor-supported storage cabinets more than 6 ft (1,829 mm) tall, including contents	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
3		Permanent floor-supported library shelving, book stacks, and bookshelves more than 6 ft (1,829 mm) tall, including contents	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
3		Laboratory equipment	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
		Access floors							
3		Special access floors (designed in accordance with Section 13.5.7.2)	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
2		All other	~~1~~	~~1½~~	~~1½~~	2.2	2.8	1.5	1.5
13		Appendages and ornamentations	~~2½~~	~~2½~~	~~2~~	1.8	2.2	1.5	1.75
16		Signs and Billboards	2½	~~3~~	~~2~~	1.8	2.2	1.5	1.75
		Other rigid components				1	1	1.5	1.5
8		~~High-deformability elements and attachments~~	~~1~~	~~3½~~	~~2~~				
3		~~Limited-deformability elements and attachments~~	~~1~~	~~2½~~	~~2~~				
1		~~Low-deformability materials and attachments~~	~~1~~	~~1½~~	~~1½~~				
		Other flexible components							
21		High-deformability elements and attachments	~~2½~~	~~3½~~	~~2½~~	1.4	1.4	1.5	2
14		Limited-deformability elements and attachments	~~2½~~	~~2½~~	~~2½~~	1.8	2.2	1.5	1.75
10		Low-deformability materials and attachments	~~2½~~	~~1½~~	~~1½~~	2.2	2.8	1.5	1.5
3		Egress stairways not part of the building seismic force-resisting system	~~1~~	~~2½~~	~~2~~	1	1	1.5	1.5
14		Egress stairs and ramp fasteners and attachments	~~2½~~	~~2½~~	~~2½~~	1.8	2.2	1.5	1.75

Mechanical and electrical equipment are assigned a component resonance ductility factor in Table 13.6-1 [not provided here due to space limitations]. The component resonance ductility factor for mechanical and electrical equipment mounted on equipment support structures or platforms shall not be less than the component resonance ductility factor used for the equipment support structure or platform itself.

The component resonance ductility factor for equipment support structures or platforms shall be determined in accordance with Section 13.6.4.6 [not provided here due to space limitations]. The weight of supported mechanical and electrical components shall be included when calculating the component operating weight, W_p, of equipment platform or support structure.

Distribution systems are assigned a component resonance ductility factors in Table 13.6-1, to be used for the design of the distribution system itself (e.g. the piping, ducts, and raceways). The component resonance ductility factor for distribution system supports shall be determined in accordance with Section 13.6.4.7 [not provided here due to space limitations].

13.3.1.4 Component Strength R_{po}

The component strength factor, R_{po}, for nonstructural components is given in Tables 13.5-1 or 13.6-1.

13.3.1.5 Nonlinear Response History Analysis

Nonlinear response history analyses procedures of Chapter 16. 17, and 18 may be used to determine the lateral force nonstructural components in accordance with Eq. (13.3-7):

$$F_p = I_p W_p \, a_i \, [C_{AR}/R_{po}] \qquad (13.3\text{-}7)$$

Where a_i is the maximum acceleration at level i obtained from the nonlinear response history analysis. The upper and lower limits of F_p determined by Eqs. (13.3-2) and (13.3-3) shall apply.

Discussion of the Proposed Equations

The commentary to the 6/20/19 code change proposal (Gillengerten, 2019) provides a detailed discussion of key features of the proposed nonstructural design equations. Text below is taken from the proposal.

Key Features

Using the above selected parameters, the proposed equations in NIST (2018) and in Section 13.3.1 include a set of key features that differ from those in ASCE 7-16. These include:

- Ratio of PFA to PGA: Based on a detailed review of instrumented building strong motion records, a more refined equation was developed to relate PFA to PGA at different heights in the building. The equation incorporates building period. This is accounted for in the variable H_f of Equation 13.3-1.
- Building ductility: Increased building ductility has been shown to generally reduce nonstructural component response. This is captured by the variable R_u. The equation for determining R_u is based on a series of archetype case studies using different seismic force-resisting systems, numbers of stories, and overstrength assumptions.
- Ratio of PCA to PFA: The relationship between PCA and PFA, defined as C_{AR} in Equation 13.3-1, is affected by several parameters including the ratio of component period to building period, and component ductility. When component and building periods are close, component response is increased due to resonance; when component ductility is larger, component response decreases. These effects are captured by two concepts in the proposed equation framework. The first is whether component response is likely or unlikely to be in resonance with the building response. When the ratio of component period to building period is relatively small or relatively large, resonance is unlikely, and C_{AR} is set to 1.0. When the ratio is closer to unity, resonance is likely, and C_{AR} is amplified to account for resonance. The second concept is to create low, moderate, or high component ductility categories for situations with likely resonance. C_{AR} values for low ductility are higher than those for high ductility the values. The selected C_{AR} values are based on archetypes studies and account for some level of reduction from the theoretical peak value to address the probability of overlap between component and building periods. Although quantitative studies to determine the statistical reliability that the equations envelope archetype results were not performed, the number of studies was performed was substantial, and engineering judgment was used in parameter studies and final equation setting to target a general design level such that the proposed equations are approximately mean plus one standard deviation above archetype results. Due to the number of parameters involved and their potential for variation, in some cases, there will be a higher level of reliability; in other cases, there will be a lower level of reliability.
- Component Strength: Like building structural systems, the component and its attachments to the structure typically have some inherent overstrength. This is captured by the variable R_{po}. It serves to reduce the design force needed.
- Ground vs. Superstructure: The amplification of PCA/PFA as the ratio of component to building period

approaches unity comes from narrow band filtering of response by the dynamic properties of the building. Components that are ground supported can see dynamic amplification due to component flexibility, based on structural dynamics, but this amplification is typically less than what occurs in the building. Given that there are both theoretical and numerical differences between the ground and superstructure cases, it was decided to distinguish the two.

Building Ductility

Determination of the Structure Ductility Reduction Factor, R_u, relies on the R and Ω_o values in Table 12.2-1, Table 15.4-1, and Table 15.4-2. The code change proposal covers how to establish values for existing buildings or when limited information is available regarding the building SFRS.

Increasing building ductility generally reduces nonstructural component seismic demands, and reduced ductility generally increases component demands. Buildings with higher design forces (such as those using a Seismic Importance Factor $I_e > 1.0$) and/or higher levels of overstrength will have less ductility demand for the same level of seismic shaking than those designed to code minimums and with limited overstrength. To address this issue, the ATC-120 project analyses included building archetypes with both limited overstrength and with substantial overstrength. Calibration studies were done to reasonably bound results from the archetypes with substantial overstrength.

Determination of Likelihood of Being in Resonance

Nonstructural components have been categorized as likely or unlikely to be in resonance by engineering judgment. An underlying concept is that if the component period, T_{comp}, is less than 0.06 seconds, then resonance is unlikely regardless of building period, since the building period will typically be well above that level. In previous editions of the *Provisions* components with $T_{comp} \leq 0.06$ seconds were termed "rigid" and did not receive any amplification of PFA (while those with $T_{comp} > 0.06$ were termed "flexible" and received an increase of 2.5 times PFA). A second underlying concept is that, when the ratio of component period to building period is relatively low or relatively large, then resonance is also unlikely. A criterion of $T_{comp} / T_{abldg} < 0.5$ or $T_{comp} / T_{abldg} > 1.5$ can be used, as suggested by NIST (2018) as well as extrapolation of results from Hadjian and Ellison (1986). Distribution systems may experience resonance, but its effect is judged to be minimized due to reduced mass participation caused by multiple points of support.

Component Ductility Categories

Nonstructural components have been assigned to one of three categories of component ductility. In the ATC-120 studies, the following underlying relationships were used.

Ductility Category	Assumed Component Ductility, μ_{comp}	PCA/PFA (C_{AR})	
		Supported at or Below Grade	Supported Above Grade by a Structure
Elastic	1	2.5	4.0
Low	1.25	2.0	2.8
Moderate	1.5	1.8	2.2
High	2.0	1.4	1.4

As discussed in NIST (2018), the elastic category is used for reference only. It is assumed that typical nonstructural components and their attachments to the structure systems used in practice have at least the low levels of component ductility.

Equipment Support Conditions

Design coefficients are assigned to mechanical and electrical equipment based on the properties of the equipment item, but they may be supported on a platform or support structure that has structural properties that are substantially different from the component itself. In some cases, this can be beneficial, such as when a moderate or low ductility component is mounted on a platform or support structure with high ductility. In this case, the platform or support structure will limit the shaking demands on supported components, by providing a structure with a ductile behavior in the load path. To account for this, when determining the design lateral force, mechanical and electrical equipment mounted on platforms or support structures the component resonance ductility factor for the equipment itself may be used, or the component resonance ductility factor for the equipment support may be used.

Determination of Ground vs. Superstructure Category

Nonstructural components supported by slabs or foundation elements at grade that are not part not part of a building use the ground category. Similarly, nonstructural components supported by slabs or foundations, or other elements of the superstructure located at or below grade use the ground category.

Design Forces for Elements in the Load Path with Limited Ductility

Anchors in concrete or masonry that cannot develop a ductile yield mechanism are required to use design forces increased by the Ω_{op} factor. Designers should consider amplifying design forces by an overstrength factor for elements in the load path between the component and the anchor that have limited ductility.

Example Comparisons Between ASCE 7-16 and Proposed Equations

In order to review the impact of the code change proposal on the forces used for nonstructural component and anchorage design, extensive numeric comparisons were made. Figure 17 shows one example comparing a set of architectural components in ASCE 7-16 with the same a_p, R_p, and Ω_o values with results from the proposed design equations. Similar comparisons were done for 30 architectural, mechanical and electrical component categories supported by building structures covered by ASCE 7-16 Chapter 13 building structures and 26 component categories covered by ASCE 7-16 Chapter 16 nonbuilding structures, leading to a comparison document of over 200 pages of tables that accompanies the code change proposal. In some situations, such as Figure 17, the proposed equations result in lower forces; in others, they result in higher forces.

An example comparison between ASCE 7-16 and the proposed equations is for interior wall and partitions. ASCE 7-16 Table 13.5-1 has two interior wall and partition categories: plain (unreinforced) masonry and all other partition types. Unreinforced masonry walls and partitions are now prohibited in the code change proposal in areas with seismic shaking of significance. As such, the focus here is on the remaining wall and partition types, which in ASCE 7-16 Table 13.5-1 have $a_p = 1$ and $R_p = 2.5$. Thus, there is an implied presumption that dynamic amplification of floor acceleration is minimal, and the partitions have moderate ductility. However, per structural dynamic theory, component ductility does not reduce response for components as T_{comp} approaches zero.

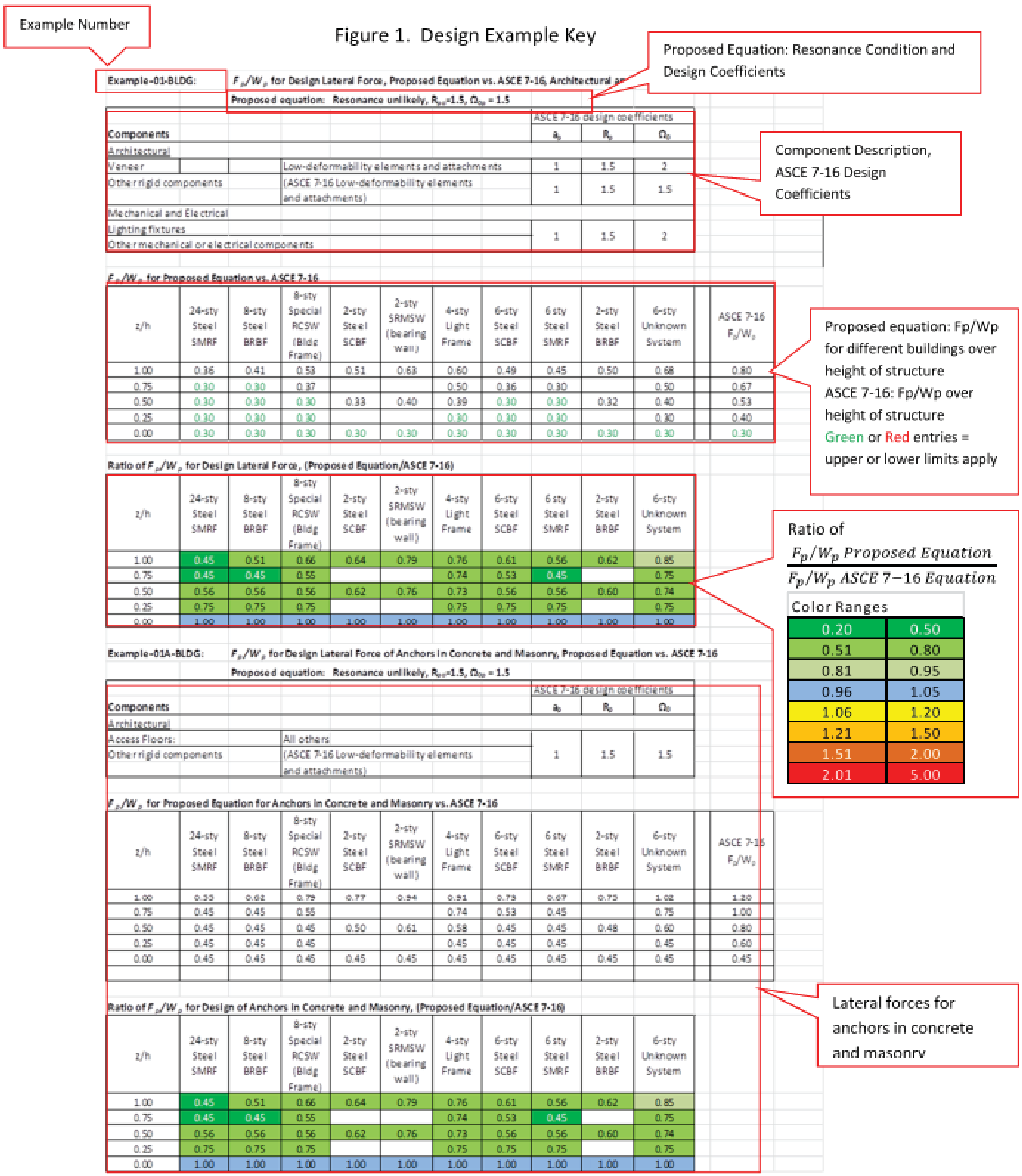
There are several issues to consider with partitions.

- Partitions typically span out-of-plane between diaphragms. Thus, their displacements and accelerations are affected by two diaphragms that will have different dynamic response. Out-of-phase behavior between the two diaphragms is likely to reduce partition response.
- The behavior of reinforced concrete masonry unit (CMU) partitions is likely to be different from wood and metal stud partitions with gypboard finishes. CMU partitions are likely to have to have less flexibility and ductility than wood and metal stud partitions, and CMU partitions will weigh more. Out-of-plane cracking of gypboard may mean that wood and metal stud walls and partitions have higher damping which could reduce response.
- For wood and metal stud walls and partitions, height may impact dynamic amplification. As the walls and partitions get taller, the out-of-plane period lengthens, and there is more likelihood that the component could become more in resonance with the building.

Given these issues, four wall and partition categories are proposed.

- Short, light frame: These are assumed to be wood and metal stud partitions of 9 feet high or less. These walls and partitions are assumed to have relatively short periods and a comparatively low T_{comp}/T_{abldg} ratio, and thus are unlikely to be in resonance with the building.
- Tall, light frame: These are assumed to be wood and metal stud partitions of over 9 feet in height. They are assumed to have longer periods and a T_{comp}/T_{abldg} ratio moving toward a central value, and thus they have the potential to be in resonance with the building. They also are assumed to have desirable damping and a high level of ductility.
- Reinforced masonry: Reinforced masonry partitions are assumed to have short periods and a low T_{comp}/T_{abldg} ratio, and thus they are unlikely to be in resonance with the building.
- All other walls and partitions: These are conservatively assumed to be likely in resonance with low ductility to cover any other type of situation.

Per ASCE 7-16, for a site with $S_{DS} = 1.0$g, $I_p = 1.0$, a partition at midheight of the building ($z/h = 0.5$), and $a_p = 1$ and $R_p = 2.5$, F_p/W_p is 0.32g, which is just above the 0.30g minimum. With the proposed equations, for a six-story steel SMRF, a short, metal stud partition or a reinforced masonry partition with the assumption of "unlikely to be in resonance," $H_f = 1.54$, $R_\mu = 1.71$, $C_{AR} = 1$, $R_{po} = 1.5$, and $F_p/W_p = 0.24$g, so it is governed by the minimum value of 0.30g. For a tall, metal stud partition, with the assumption of "likely to be in resonance" and "high ductility," $H_f = 1.54$ and $R_\mu = 1.71$ as before, $C_{AR} = 1.4$ for high ductility, $R_{po} = 2.0$, and $F_p/W_p = 0.25$g, so F_p/W_p is also governed by the 0.30g minimum.



Example-01-BLDG: F_p/W_p for Design Lateral Force, Proposed Equation vs. ASCE 7-16, Architectural a

Proposed equation: Resonance unlikely, R_{po}=1.5, Ω_{0p} = 1.5

Components		ASCE 7-16 design coefficients a_p	R_p	Ω_0
Architectural				
Veneer	Low-deformability elements and attachments	1	1.5	2
Other rigid components	(ASCE 7-16 Low-deformability elements and attachments)	1	1.5	1.5
Mechanical and Electrical				
Lighting fixtures Other mechanical or electrical components		1	1.5	2

F_p/W_p for Proposed Equation vs. ASCE 7-16

z/h	24-sty Steel SMRF	8-sty Steel BRBF	8-sty Special RCSW (Bldg Frame)	2-sty Steel SCBF	2-sty SRMSW (bearing wall)	4-sty Light Frame	6-sty Steel SCBF	6 sty Steel SMRF	2-sty Steel BRBF	6-sty Unknown System	ASCE 7-16 F_p/W_p
1.00	0.36	0.41	0.53	0.51	0.63	0.60	0.49	0.45	0.50	0.68	0.80
0.75	0.30	0.30	0.37			0.50	0.36	0.30		0.50	0.67
0.50	0.30	0.30	0.30	0.33	0.40	0.39	0.30	0.30	0.32	0.40	0.53
0.25	0.30	0.30	0.30			0.30	0.30	0.30		0.30	0.40
0.00	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30

Ratio of F_p/W_p for Design Lateral Force, (Proposed Equation/ASCE 7-16)

z/h	24-sty Steel SMRF	8-sty Steel BRBF	8-sty Special RCSW (Bldg Frame)	2-sty Steel SCBF	2-sty SRMSW (bearing wall)	4-sty Light Frame	6-sty Steel SCBF	6 sty Steel SMRF	2-sty Steel BRBF	6-sty Unknown System
1.00	0.45	0.51	0.66	0.64	0.79	0.76	0.61	0.56	0.62	0.85
0.75	0.45	0.45	0.55			0.74	0.53	0.45		0.75
0.50	0.56	0.56	0.56	0.62	0.76	0.73	0.56	0.56	0.60	0.74
0.25	0.75	0.75	0.75			0.75	0.75	0.75		0.75
0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Example-01A-BLDG: F_p/W_p for Design Lateral Force of Anchors in Concrete and Masonry, Proposed Equation vs. ASCE 7-16

Proposed equation: Resonance unlikely, R_{po}=1.5, Ω_{0p} = 1.5

Components		ASCE 7-16 design coefficients a_p	R_p	Ω_0
Architectural				
Access Floors: Other rigid components	All others (ASCE 7-16 Low-deformability elements and attachments)	1	1.5	1.5

F_p/W_p for Proposed Equation for Anchors in Concrete and Masonry vs. ASCE 7-16

z/h	24-sty Steel SMRF	8-sty Steel BRBF	8-sty Special RCSW (Bldg Frame)	2-sty Steel SCBF	2-sty SRMSW (bearing wall)	4-sty Light Frame	6-sty Steel SCBF	6 sty Steel SMRF	2-sty Steel BRBF	6-sty Unknown System	ASCE 7-16 F_p/W_p
1.00	0.53	0.62	0.79	0.77	0.94	0.91	0.73	0.67	0.73	1.02	1.20
0.75	0.45	0.45	0.55			0.74	0.53	0.45		0.75	1.00
0.50	0.45	0.45	0.45	0.50	0.61	0.58	0.45	0.45	0.48	0.60	0.80
0.25	0.45	0.45	0.45			0.45	0.45	0.45		0.45	0.60
0.00	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45

Ratio of F_p/W_p for Design of Anchors in Concrete and Masonry, (Proposed Equation/ASCE 7-16)

z/h	24-sty Steel SMRF	8-sty Steel BRBF	8-sty Special RCSW (Bldg Frame)	2-sty Steel SCBF	2-sty SRMSW (bearing wall)	4-sty Light Frame	6-sty Steel SCBF	6 sty Steel SMRF	2-sty Steel BRBF	6-sty Unknown System
1.00	0.45	0.51	0.66	0.64	0.79	0.76	0.61	0.56	0.62	0.85
0.75	0.45	0.45	0.55			0.74	0.53	0.45		0.75
0.50	0.56	0.56	0.56	0.62	0.76	0.73	0.56	0.56	0.60	0.74
0.25	0.75	0.75	0.75			0.75	0.75	0.75		0.75
0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 17: Comparison between ASCE 7-16 and Proposed Equations for Select Architectural Components

Code Development Chronology

A summary of key milestones in the code development process for the proposed nonstructural equations is as follows.

- 1997 UBC: Current form of the nonstructural design provisions first issued.
- 2015 *NEHRP Provisions* (FEMA, 2015): Most recent version of *NEHRP Provisions*. It includes current form of the nonstructural equations.
- ASCE 7-16: Includes current form of nonstructural design equations. Same as 2015 NEHRP provisions.
- ATC-120 project
  - NIST (2017) publication.
  - NIST (2018) publication.
- 2017-2019: Periodic presentations to the BSSC Provisions Update Committee (PUC) on the ATC-120 project and draft proposal development.
- 5/21/19: Issue Team 5 (IT5) workshop to refine proposal and specific properties for individual nonstructural components.
- 6/17/19: IT5 passed updated proposal resulting from 5/21/19 workshop. Updated 6/20/19 version prepared (Gillengerten, 2019).
- 6/26/19: IT5 proposal for revision to nonstructural design equations submitted together with a series of other proposals to PUC.
- 7/23/19: PUC balloting closed. Nonstructural design proposal received majority support, but there were comments to resolve. This paper was prepared following this milestone.
- 8/13/19 and 8/14/19: PUC meetings to resolve outstanding issues.
- Fall 2019: If approved at the 8/13/19 and 8/14/19 PUC meetings, proposal moves to BSSC member organizations for review and balloting.
- 12/4/19 and 12/5/19: PUC meetings to review of member organization comments. If member organizations approve and comments are resolved, proposal becomes part of the 2020 *NEHRP Provisions* and moves for consideration by ASCE 7 committee for potential inclusion in ASCE 7-22.

Conclusions

Although the outcome of the proposal to revise the nonstructural design equations in the *NEHRP Provisions* and ASCE 7 is not known time of writing of paper, the ATC-120 research, as summarized in NIST (2018), is a seminal contribution to the development of a more rigorous understanding of the influence of different parameters on the seismic response of nonstructural components, and the associated code development process represents a comprehensive effort to improve to the code provisions for nonstructural seismic design.

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- BSSC Issue Team 5
  - Voting Members: John Gillengerten (Chair), Pete Carrato, Travis Chrupalo, William T. Holmes, Bret Lizundia, John Silva, Greg Soules, Chris Tokas.
  - Corresponding Members: Hussain Bhatia, Phillip Caldwell, Meaghan Halligan, Matthew Hoehler, Robert Simmons, Siavash Soroushian.

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