EVALUATION OF ASCE/SEI 7 EQUATIONS FOR SEISMIC DESIGN OF NONSTRUCTURAL COMPONENTS USING CSMIP RECORDS

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Abstract

A recently completed California Strong Motion Instrumentation Program (CSMIP) data interpretation project used recorded ground and floor motion data to evaluate a key ASCE/SEI 7-05 (and 7-10) equation for seismic design of acceleration-sensitive building nonstructural components. CSMIP motions from 73 earthquakes recorded in 151 fixed-base buildings were used in the evaluation. An improved equation was developed with two categories of revisions. First, the current code formula considers a linear relationship between the peak floor acceleration (PFA) and relative height of the component in the building with a roof PFA that is three times that of the peak ground acceleration. The analyses of the recorded motions showed that improved results could be obtained by using a nonlinear relationship and by considering both the building approximate period, T_a, and the level of ground motion. Second, the code formula considers a component amplification factor, a_p, that takes values between 1.0 and 2.5 depending on the flexibility of the nonstructural component. Analyses showed that component amplification factor can be better represented using a three-segment spectrum composed of a linear rise from 1.0 to maximum value of a_p at short periods, a flat segment with the maximum value of a_p at medium range periods, and a nonlinear decaying segment at longer periods. The shape and amplitude of the spectrum was found to vary depending on T_a.

Objectives

In a CSMIP-sponsored study, Fathali and Lizundia (2011a) compared the response data recorded from instrumented buildings with the equations in ASCE/SEI 7-05 used for seismic design of acceleration-sensitive nonstructural components and recommended modifications for improvement. These equations are unchanged in ASCE/SEI 7-10. The study focused on two primary tasks. The first was to compare the relationship $[1 + 2\ (z\ /\ h)]$ in Equation 13.3-1 that relates upper floor acceleration to ground level acceleration. The second primary task was to study the a_p parameter of Equation 13.3-1 that is essentially the ratio between the peak acceleration response of the elastic component to the peak floor acceleration. A large database was created from available recorded motions, and a proposed equation was developed as a result of the study that involves changes to both aspects of the equation.

The paper is organized with the following sections: a brief review of relevant literature, a description of the current code equations used in seismic design of nonstructural components, a summary of the earthquake records in our database, the methodology used to evaluate the code equations and recorded response, a summary of the revised equation proposed in Fathali and

Lizundia (2011a), and our conclusions. The majority of the work presented in this paper was previously published by Fathali and Lizundia (2011b).

Literature Review

Extensive research can be found in the literature on the history and development of various equations that have been used for seismic design of nonstructural components. A more detailed review is contained in Fathali and Lizundia (2011a). A brief summary of some key studies is provided here.

Uniform Building Code: The first Uniform Building Code (UBC) in 1927 (ICBO, 1927) makes reference to designing "parts and portions" of the building for seismic forces and provided force levels to use that were the same as the overall lateral force-resisting system. The next edition in 1935 (ICBO, 1935) provides explicit seismic design provisions for general nonstructural building components. Only architectural components are addressed. Noteworthy changes were made in the 1961, 1976, 1979 and 1997 editions of the UBC, including the addition of mechanical, electrical and plumbing (MEP) components in 1976.

ATC 3-06: One of earthquake engineering's seminal documents is ATC 3-06 (ATC, 1978 and 1984). It was a large effort by a multi-disciplinary team to develop new seismic design provisions. Chapter 8 contains provisions for seismic design of nonstructural components. Requirements for mechanical and electrical components were included and had a slightly different equation than the equation for architectural components. Per the ATC 3-06 commentary, the forces used in nonstructural seismic design were based in part on the UBC. The form of the equation, though, is substantially different than the UBC equation at the time and includes additional variables. In 1985, the BSSC published the first *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC, 1985). The ATC 3-06 provisions were used as a basis for NEHRP Provisions. For nonstructural seismic design, ATC 3-06 and the 1985 NEHRP Provisions are identical.

NCEER-93-0003: Another key early publication is the *NCEER-93-0003* (Soong, et al., 1993) report which reviewed the seismic design requirements for nonstructural components in the 1991 NEHRP Provisions (BSSC, 1992) and made recommended revisions. Many of the concepts proposed in the NCEER-93-0003 study were incorporated in the 1994 NEHRP Provisions (Bachman and Drake, 1994).

Bachman and Drake (1995): The most comprehensive early work involving strong motion records was done by Robert Bachman, Richard Drake and John Gillengerten. It is summarized in several related papers, including Drake and Gillengerten (1994), Bachman and Drake (1994), Drake and Bachman (1995), Bachman and Drake (1995), and Drake and Bachman (1996). Detailed information is contained in Bachman and Drake (1995), including tables listing the data sets. 405 data sets were compiled, taken from 16 California earthquakes, ranging from the 1971 San Fernando Earthquake to the 1994 Northridge Earthquake. A dataset was derived by taking the peak acceleration at a floor (PFA) in each direction and then averaging the values from each direction. This average peak floor acceleration was then divided by a similar average peak ground acceleration (PGA) to derive a relationship of PFA to PGA. A series of plots were made of the ratio of upper floor response to ground floor response, and the plots were compared with equations in the 1994 NEHRP Provisions.

Gillengerten and Bachman (2003): A major change in the NEHRP Provisions was made with the 1997 edition (BSSC, 1998). Equations are the same as three of those in ASCE/SEI 7-05. The background on the development of these provisions is provided in Gillengerten and Bachman (2003). They note that "while there is considerable scatter in the data, the amplification term of…" [1 + 2(z/h)] "…bounds the mean plus standard deviation of the peak accelerations well." Gillengerten and Bachman (2003) also note that "the simplifying assumption that the force increases linearly with height was necessary to keep the complexity of the method at a reasonable level."

They also discuss the code range of 1.0 to 2.5 for the component amplification factor, a_p. They acknowledged that "amplification factors greater than 2.5 may occur, depending upon the period of the component, the dynamic characteristics of the supporting structure, and the amount of damping present in the component or its supports." However, they point out that "the value of 2.5 for most flexible components appears reasonable, since in strong shaking, neither the period of the structure nor the period of the component is likely to remain constant. The shift in period is likely to drive the component response off of the peak."

The Influence of Period: Many papers have investigated the influence of building period on component response, including Schroeder and Bachman (1994), Horne and Burton (2003), Singh, et al. (2006a, b), and Miranda and Taghavi (2009). They all show the influence of the dynamic properties of the building, such as the fundamental period of vibration, on component response.

Component Amplification Factor: The component amplification factor, a_p, has evolved in the various code provisions and research studies. The split into rigid and flexible components using the 0.06 second fundamental component period has been in UBC provisions since the 1988 edition. Similar definitions were added to the NEHRP Provisions in the 1994 edition. The initial NEHRP Provisions had a component amplification factor a_c that varied depending on the ratio of the component to structural periods. This was eliminated in the 1994 edition. The commentary to the 2003 NEHRP Provisions (BSSC, 2004b) describes a study for NCEER by Bachman, Drake and Richter (1993) which recommended a spectral shape.

Current Code Equations

ASCE/SEI 7-05 (ASCE, 2006) was adopted by model codes such as the 2009 International Building Code (ICC, 2009) and the 2010 California Building Code (CBSC, 2010). The next edition of ASCE/SEI 7 is ASCE/SEI 7-10 (ASCE, 2010). It represents the current source document for the seismic design of nonstructural components in the United States. It is referenced in model codes such as the 2012 International Building Code (ICC, 2012). In Section 13.3.1 of ASCE/SEI 7-05, there are four equations which provide the forces for use in determining the seismic design demands for nonstructural components. They are provided below. Note that they are unchanged in ASCE/SEI 7-10; this paper, for consistency with Fathali and Lizundia (2011a), uses the ASCE/SEI 7-05 references.

$$F_p = \{0.4\, S_{DS}\, a_p\, (1 + 2\,(z/h))(I_p/R_p)\}W_p \qquad \text{(ASCE/SEI 7-05 Equation 13.3-1)}$$

$$F_p \leq 1.6\, S_{DS}\, I_p\, W_p \qquad \text{(ASCE/SEI 7-05 Equation 13.3-2)}$$

$$F_p \geq 0.3\, S_{DS}\, I_p\, W_p \qquad \text{(ASCE/SEI 7-05 Equation 13.3-3)}$$

In these equations, F_p is the lateral seismic force at the LRFD force design level, W_p is the weight of component, a_p is the component amplification factor that ranges from 1.0 to 2.5 and accounts for amplification due to component flexibility, S_{DS} is the site-specific short period spectral acceleration, z is the component elevation in structure relative to grade, h is the roof elevation in structure relative to grade, R_p is the component response modification factor which represents the ability of the component to absorb energy, and I_p is the component importance factor.

When a modal analysis is performed using $R = 1.0$, nonstructural seismic design forces can be determined from the following Equation 13.3-4 in lieu of Equation 13.3-1. The upper and lower limits of Equations 13.3-2 and 13.3-3 still apply.

$$F_p = ((a_i a_p)(I_p/R_p))\, A_x\, W_p \qquad \text{(ASCE/SEI 7-05 Equation 13.3-4)}$$

In this equation, a_i is the acceleration at level i obtained from modal analysis, A_x is the torsional amplification factor, and the remaining values are the same as those used in the previous equations.

The vast majority of seismic design efforts in practice use Equations 13.3-1, 13.3-2, and 13.3-3, rather than Equation 13.3-4 since Equation 13.3-4 requires a dynamic analysis to determine the component a_i. Equations 13.3-2 and 13.3-3 set maximum and minimum limits on the forces used, depending on the short period spectral acceleration, S_{DS}, assigned to the site.

Summary of Database Characteristics

The database developed for this study included entries from 169 CSMIP building stations (151 fixed base and 18 seismically isolated), and 73 earthquakes occurred in the period of 1978 to 2010 in California. The buildings of the database are in the range of one to 54 stories with a roof elevation of 10 to 716 ft, and approximate period of 0.11 to 5.22 sec. 11 different types of lateral-force-resisting systems are found in the buildings of the database.

The 73 earthquakes of the database had a PGA in the range of 0.01 to 0.86g. Figure 1 shows the distribution of fixed-base buildings of the database in terms of the experience PGA. Some of the building stations provided us with more than one set of records (at least one pair of ground and floor acceleration along the same direction); thus, the database included 541 sets of "building-earthquake records" from fixed-base buildings. Each set of building-earthquake records provided at least one point for the study of PFA/PGA profile over the building height, and at least one floor spectrum for the study of floor amplification factor.

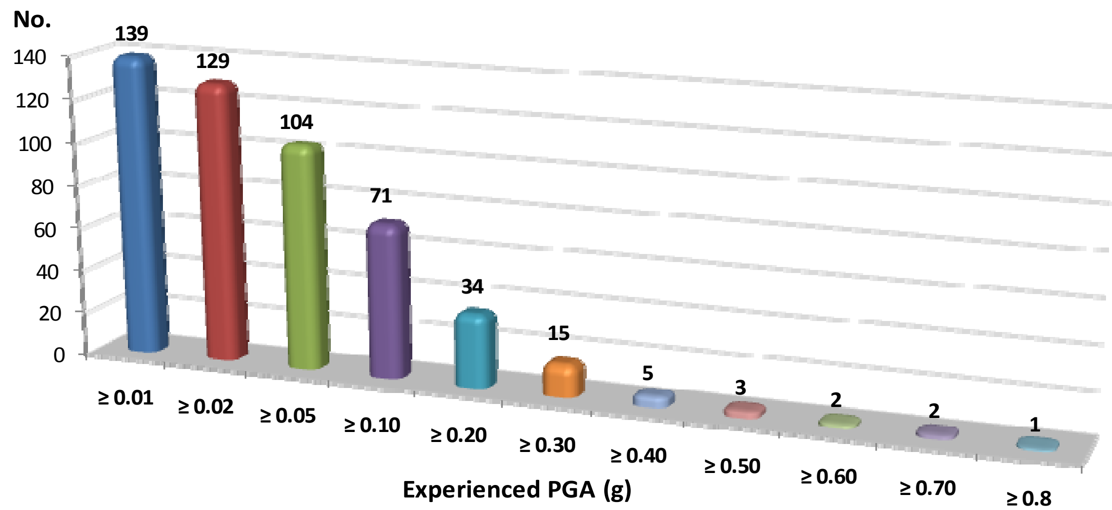


Figure 1. Distribution of Fixed-Base Buildings of Database in Terms of Experienced PGA

Methodology

The current code formula implicitly simplifies the variation of PFA over the building height to a linear relationship between two dimensionless ratios: the ratio of PFA to PGA and the ratio of the floor to roof elevations from the building grade (z/h). Therefore, each pair of ground and floor acceleration histories recorded in a CSMIP building station can provide a data point to validate the current code formula as long as the following four parameters are available: peak acceleration recorded by the floor accelerometer/channel (PFA), peak acceleration recorded by the ground-level accelerometer/channel (PGA), and elevations of the roof and floor (h and z) from the building grade.

The database of PFA/PGA versus z/h for this study was developed such that each data point has a series of attributes including the building height, number of stories, lateral force-resisting system type, approximate period, and the ground motions properties. None of these attributes is explicitly included in the code formula. The developed database permitted investigating whether these attributes influence the relationship between the PFA/PGA and z/h ratios.

The calculation of PFA/PGA ratio at any floor of a building under an earthquake ground motion was straightforward only if the building is instrumented only in one direction and there is only one accelerometer at each level. However, for the typical situation, when the building is instrumented along both directions and there are multiple recorded acceleration histories at the floor or ground level, the decision of how to calculate the PFA/PGA ratio can become rather complicated. For such cases, different answers to the following questions would result in different methodologies to calculate the PFA/PGA ratio:

(a) Should responses along the two orthogonal principal axes of the building be considered separately or collectively?

(b) Which recorded response should be used in the calculation of PFA/PGA ratio: the response at the vicinity of the center of rigidity of the floor plate or responses near the perimeter of the floors?

(c) If there is no recorded response available at the preferred locations (the answer to the previous question), is it permissible and possible to calculate virtual acceleration histories based on the available recorded acceleration histories at other locations of the floor?

(d) If the responses along the two orthogonal principal axes are not combined, should the minimum, mean, maximum (or some other statistical function) of the individual recorded peak acceleration values be used in the calculation of PFA/PGA ratio?

(e) If the responses along the two orthogonal principal axes are combined, should the resultant acceleration histories, or a combination of the PFA/PGA ratios calculated along the two orthogonal principal axes, be used in the calculation of PFA/PGA ratio?

(f) If the combined PFA/PGA ratio is calculated based on the PFA/PGA ratios along the two orthogonal principal axes, should the minimum, mean, maximum (or other statistical functions) of the two PFA/PGA ratios be used in the calculation of PFA/PGA ratio?

Based on the recommendation from the SMIP Building Subcommittee of the Strong Motion Instrumentation Advisory Committee (compromised of practicing engineers and academicians), it was decided to use the following two rules when calculating the PFA/PGA ratio:

I. Only actual/recorded acceleration histories along the same direction were used to calculate the PFA/PGA ratios (no virtual/calculated records, and no combination of response along the two orthogonal principal directions).

II. If at one of the building floors along a given direction there was more than one recorded acceleration history, the mean of the peak acceleration values of those records was used to calculate the PFA/PGA ratio.

These rules were selected for the following three reasons. First, several CSMIP building stations have different lateral force-resisting systems type, and fundamental period, and consequently different seismic behavior along their two orthogonal principal axes. Moreover, for a large portion of the database, the ground motions along the two principal axes of the building stations are considerably different in terms of the amplitude and frequency content. Therefore, it was decided that the methods that do not combine the response along the two orthogonal principal axes are preferable for this research study. Second, calculations of virtual acceleration histories need a fairly precise knowledge of the locations of the existing channels and the center of rigidity of the floor plate, and are possible only for the buildings with rigid diaphragms. Lastly, compared to the maximum value, the mean of the peak acceleration values recorded through different channels at a building floor is more representative of the peak response at that floor since it uses all of the recorded acceleration histories.

The study compiled 2224 data points from above ground level ($z/h > 0$) data, as shown in Figure 2. In this figure, the solid and dashed lines represent the 1997 UBC and ASCE/SEI 7-05 equations for the relationship between PFA/PGA and *z/h*. At any given *z/h* value, the PFA/PGA ratios established based on the CSMIP floor motions can be compared to the corresponding value predicted by the current and previous code formula.

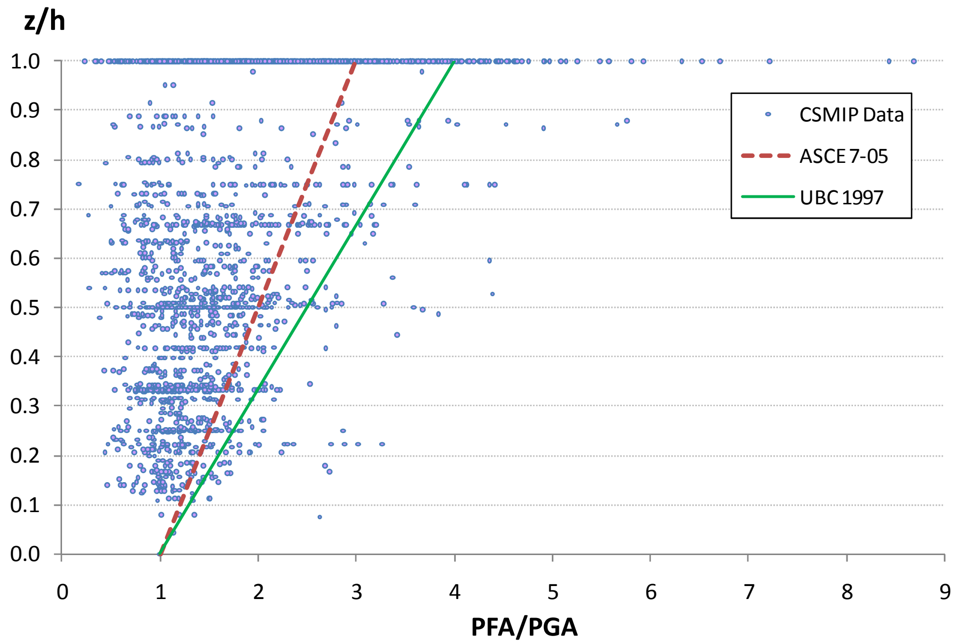


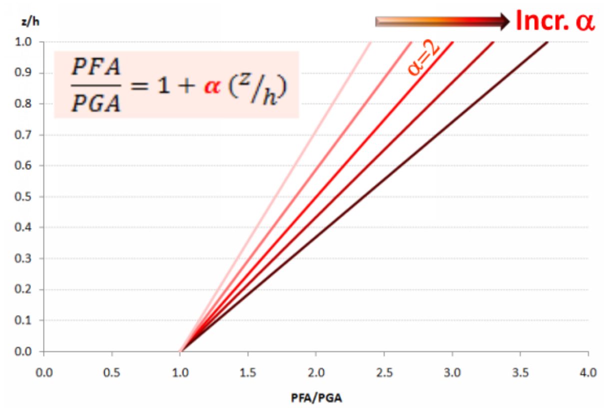
Figure 2. Data Points vs. Building Code Formula for Relationship between PFA/PGA and *z/h* (2224 above-Ground Level Data Points from 151 Fixed-Base Building Stations)

After development of the database of PGA/PFA versus *z/h* ratios, establishing the best-fit equation to between these two ratios could be undertaken by linear or nonlinear regression analyses. Equations established by linear regression analyses through the database would be in the general form of Equation 1. Figure 3(a) shows how variations of parameter α affect the shape of the profile of peak floor acceleration over the building. As indicated in this figure, larger values of parameter α correspond to larger amplifications of peak floor accelerations over the building height, and α equal to 2 corresponds to the ASCE/SEI 7-05 code formula.

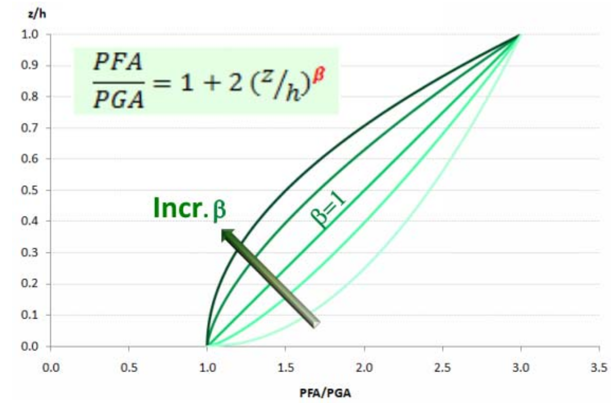
$$\mathrm{PFA/PGA} = 1 + \alpha\,(z/h) \tag{1}$$

However, reviewing the actual responses recorded in CSMIP buildings during past earthquakes shows that in several cases the profile of PFA/PGA over the building height is significantly different from a straight line and can be much better presented by Equation 2, which is a nonlinear equation that allows *z/h* ratio takes exponents smaller or larger than 1.

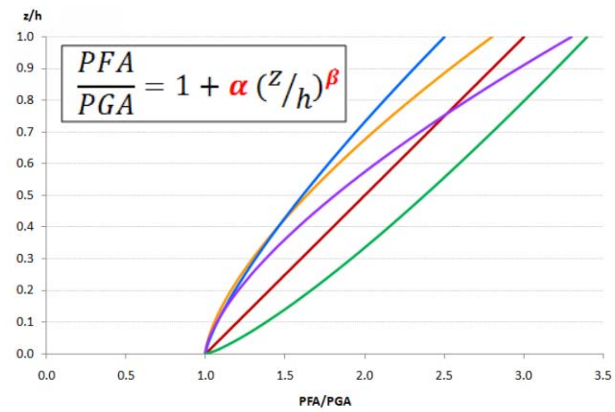
$$\mathrm{PFA/PGA} = 1 + \alpha\,(z/h)^{\beta} \tag{2}$$



(a) Effect of Parameter α



(a) Effect of Parameter β



(c) Combined Effect of Parameters α and β

Figure 3. General Nonlinear Relationship Between PFA/PGA and *z/h* Ratios

Figure 3(b) shows how variations of parameter β, the exponent of the *z/h* ratio, change the shape of the profile of the peak floor acceleration over the building height. β values larger than 1 suggest that the rate of increase in the amplification of floor acceleration is proportional to the floor height. β values smaller than 1, on the other hand, produce profiles such that the rate of increase in the amplification of floor acceleration is inversely proportional to the floor height. Therefore, very small values of parameter β can produce profiles with almost constant peak floor acceleration over the building height. Figure 3(c) shows how combined variations of parameters α and β of Equation 2 result in different shapes for the profile of PFA/PGA ratio over the building height. Since Equation 1 is a special case of Equation 2 ($\beta = 1$) the search for the best-fit equation through the database of PFA/PGA versus *z/h* ratios was pursued by nonlinear regression analyses to establish values of parameters α and β.

Since a large portion of the data points established based on the CSMIP records are located at the roof level ($z/h = 1$), the parameter α of Equation 2 will be mainly governed by the response at the roof level of the buildings. To reduce this effect, a weighted window averaging technique can be used. To do so, a series of imaginary windows are considered that cover the entire database without any overlaps. Each window has finite, arbitrary width in terms of the *z/h* ratio, but an unlimited width in terms of PFA/PGA ratio. Then, all of the data points within each window are presented by a single data point whose PFA/PGA ratio is equal to the mean plus standard deviation of PFA/PGA ratio of all the data points in that window. The z/h ratio of the two data points representing the lowest and highest windows are considered as 0 and 1, respectively. For the other windows, on the other hand, the z/h ratio of the representative point is equal to the z/h ratio at the center of the window.

Estimating the peak floor acceleration is only the first step in calculating the seismic demand of acceleration-sensitive components. A given floor acceleration history would induce different seismic forces in nonstructural components with different dynamic properties. The peak acceleration response of elastic, rigid nonstructural components that are rigidly anchored to the building floors is equal to the peak floor acceleration. For flexible nonstructural components (or any flexibly-mounted nonstructural components), on the other hand, the peak acceleration response could be smaller or larger than the peak floor acceleration.

The current code formula addresses this issue by using the parameter a_p, the component amplification factor, which is the ratio between the peak acceleration of elastic response of a component to the peak floor acceleration. Per Table 13.2-1 of ASCE/SEI 7-05 which lists the a_p values of different nonstructural components, a_p can take three different values: 1 for rigid nonstructural components that rigidly attached to the floor, 2.5 for all flexible or flexibly mounted nonstructural components, and 1.25 for the fasteners of the connecting system for exterior nonstructural wall elements and connections. For other components that are not listed in that table, the code requires the designer use the period of the mounted nonstructural component to decide whether it is rigid (a_p=1) or flexible (a_p=2.5). Per Section 11.2 of ASCE/SEI 7-05, rigid and flexible nonstructural components are separated at the component period of 0.06 sec. The code allows using dynamic analyses to find a_p of a component as long as it is not smaller than 1.

One of the major objectives of this study was to compare the current code values for the a_p parameters to the a_p of different nonstructural components under actual floor accelerations recorded by CSMIP. The a_p values of nonstructural components with different periods can be presented in a spectrum format. To calculate the a_p spectrum of a floor acceleration history, the absolute acceleration response spectrum is calculated first, and then it is normalized by the peak floor acceleration (which is equal to the spectral acceleration at the infinitely small period). This process was repeated for each spectra.

Figure 4 shows the mean, mean plus standard, and maximum of all of the 3742 5%-damped a_p spectra. The red dashed line shows values of 1 and 2.5 for component periods shorter and longer than 0.06 seconds, respectively, and it represents the a_p value per ASCE/SEI 7-05.

As it can be seen in Figure 4, a_p of flexible components under some of the floor accelerations recorded in the past has been much larger than 2.5 (the maximum is about 8.2). The maximum a_p spectrum shows that for a wide range of component periods (0.1 to 3 sec.), a_p could take values larger than 5. However, the intention of the building codes is usually to design for approximately the mean plus standard deviation of the demand.

The comparison of the code values for a_p and the mean plus standard deviation a_p spectrum shows that for a range of component period between 0.1 and 0.75 seconds the established a_p spectrum exceeds the code value of 2.5 and reaches a maximum value of 3.3 at the component period of 0.3 seconds. Outside this range of period, on the other hand, the code value is conservative. Contrary to what the code formula implies that any nonstructural component with a period of longer than 0.06 seconds experiences the maximum value of a_p (2.5), the results show that in the range of component period of 0.06 to 0.3 seconds, a_p can take values smaller than 2.5. As it is shown in Figure 4, over the entire range of periods for flexible components, the mean a_p spectrum is smaller than 2.5. The mean a_p spectrum reaches a value of about 2.5 at the component period of 0.3 seconds.

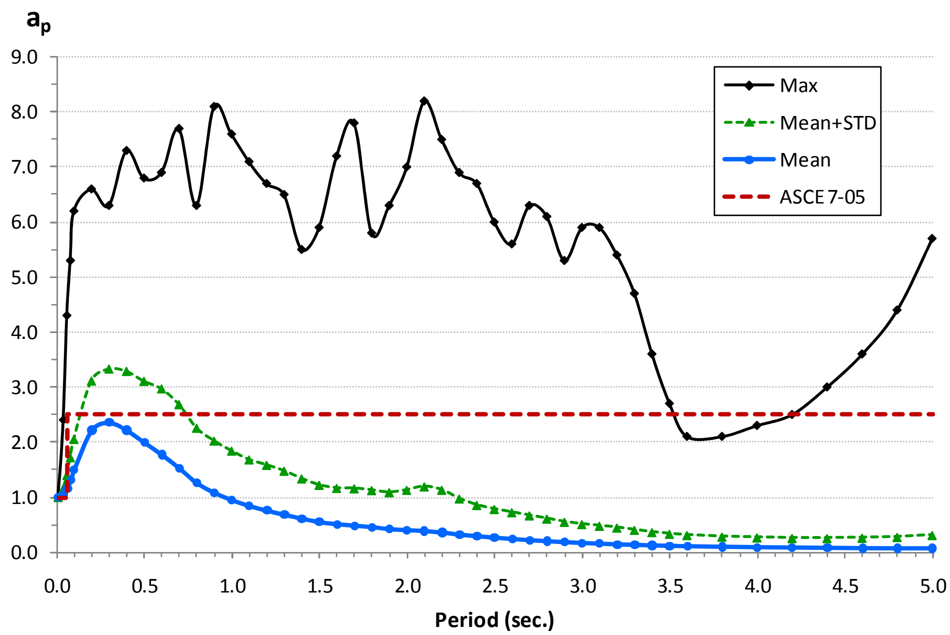


Figure 4. Comparison of Code Values for Component Amplification Factor, a_p, and Mean, Mean Plus Standard Deviation, and Maximum 5%-Damped a_p Spectra Calculated Based on 3742 CSMIP Floor Acceleration Histories from Fixed-Base Buildings

Reviewing the a_p spectra of several CSMIP recorded responses showed that the a_p spectrum typically consists of three segments: first, it rises from a_p of 1 for rigid components to a maximum value of a_p; then it remains relatively constant over a range of component periods; and finally, it begins to decay for long component periods. This pattern, which was consistently seen across the existing database, suggested that a simplified three-segment spectrum similar to the code response spectrum used for seismic design of buildings would be a good fit for the general floor a_p spectrum. Therefore, a search was undertaken in this study by searching for the parameters that govern the shape of a general three-segment floor a_p spectrum that is the best fit to the building responses recorded by CSMIP during the past earthquakes.

Modified Equations Proposed by Fathali and Lizundia (2011a)

In a CSMIP research study, Fathali and Lizundia (2011a) proposed Equation 3 as a modified version of Equation 13.3-1 of ASCE/SEI 7-05 to calculate F_p:

$$F_p = \{0.4\ S_{DS}\ a_p\ (1 + \alpha\ (z/h)^{\beta})(I_p/R_p)\}W_p \tag{3}$$

In this equation, α and β are coefficients that depend on building approximate period (T_a) and peak ground acceleration ($0.4S_{DS}$), and a_p is the component amplification factor that is defined based on the proposed floor a_p spectrum shown in Figure 5. Remaining parameters of Equation 3 have the same definition as the corresponding parameters in Equation 13.3-1 of ASCE/SEI 7-05.

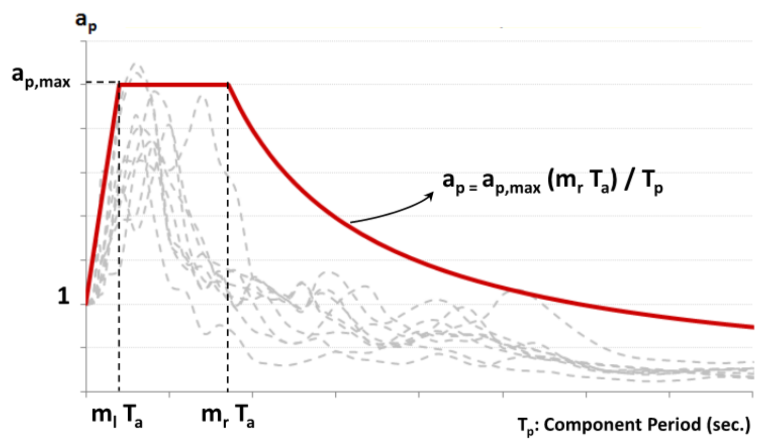


Figure 5. Floor a_p Spectrum Proposed by Fathali and Lizundia (2011a)

To establish values of parameters α and β and the spectrum of a_p used in Equation 3, Fathali and Lizundia (2011a) performed two series of nonlinear regression analyses through data points corresponding to the mean and to the mean plus one standard deviation of the response recorded in the past earthquakes. The results corresponding to the mean of the response recorded in the past are presented in Tables 1 through 3. These results are recommended for the seismic evaluation of acceleration-sensitive nonstructural components in existing buildings. Figure 6 shows how different values of parameters α and β established for different ranges of building approximate period and PGA change the shape of the profile of peak floor acceleration over the building height.

The results corresponding to the mean plus one standard deviation of the response recorded in the past are presented in Tables 4 through 6. These results are recommended for the seismic design of acceleration-sensitive nonstructural components in new construction.

The profiles of peak floor acceleration shown in Figure 7 demonstrate that compared to the code formula that assumes a threefold amplification at the roof level, the established equations for the short-, medium- and long-range period buildings under strong earthquakes with PGA ≥ 0.20g suggest about 10%, 70% and 100% less amplification, respectively. Note that at the design level, PGA equals 0.4 S_{DS}, and PGA ≥ 0.20g corresponds to Seismic Design Category D buildings of Occupancy Category I through III per Table ASCE/SEI 7-05).

Table 1. Values of Parameter α of Equation 1 Recommended for Seismic Evaluation of Acceleration-Sensitive Nonstructural Components in Existing Buildings

	$0.4\ S_{DS}$=PGA< 0.067 g	$0.067 \leq 0.4\ S_{DS}$=PGA < 0.20 g	$0.4\ S_{DS}$=PGA $\geq$ 0.20 g
$T_a < 0.5$ sec.	1.26	1.04	0.99
$0.5 \leq T_a < 1.5$ sec.	1.52	1.02	0.65
$T_a \geq 1.5$ sec.	0.90	0.72	0.00

Table 2. Values of Parameter β of Equation 1 Recommended for Seismic Evaluation of Acceleration-Sensitive Nonstructural Components in Existing Buildings

	$0.4\ S_{DS}$=PGA< 0.067 g	$0.067 \leq 0.4S_{DS}$=PGA < 0.20 g	$0.4\ S_{DS}$=PGA $\geq$ 0.20 g
$T_a < 0.5$ sec.	1.09	1.29	0.89
$0.5 \leq T_a < 1.5$ sec.	1.57	1.63	1.55
$T_a \geq 1.5$ sec.	1.69	3.00	1.00

Table 3. Parameters of General 5%-Damped Floor a_p Spectrum Recommended for Seismic Evaluation of Acceleration-Sensitive Nonstructural Components in Existing Buildings

	m_l	m_r	$a_{p,max}$
$T_a < 0.5$ sec.	0.9	1.2	2.5
$0.5 \leq T_a < 1.5$ sec.	0.3	0.8	2.1
$T_a \geq 1.5$ sec.	0.1	0.3	2.1

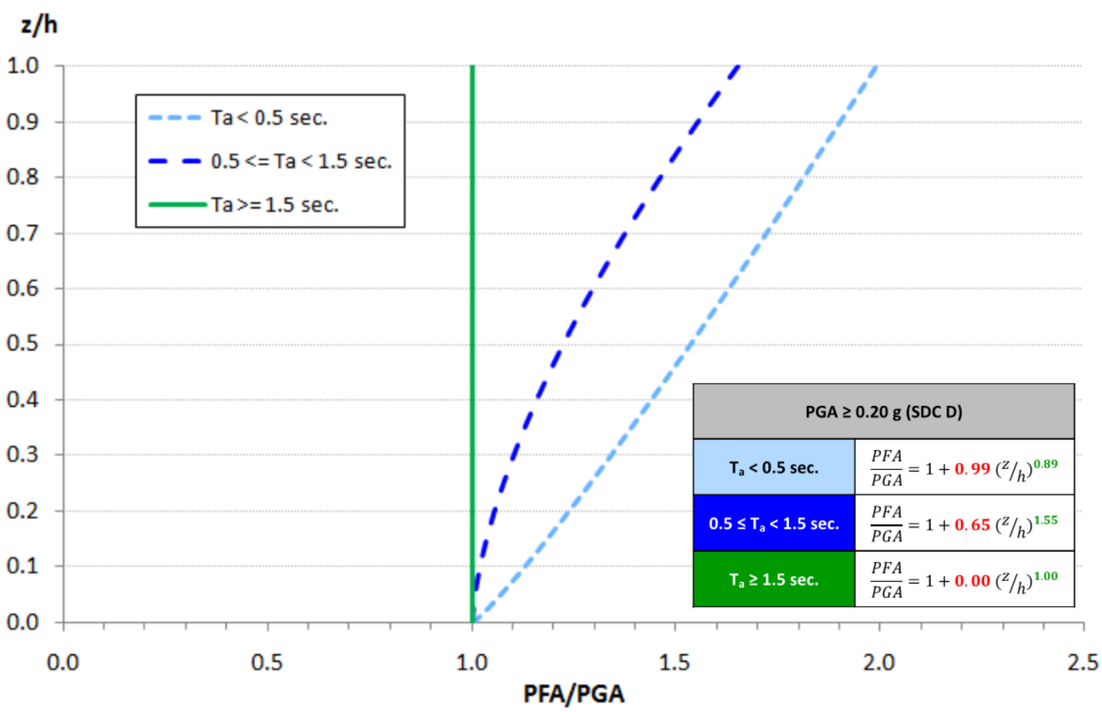


Figure 6. Profile of PFA/PGA over Building Height Recommended for Seismic Evaluation of Acceleration-Sensitive Nonstructural Components in Existing Buildings under Earthquakes with PGA $\geq$ 0.20 g (SDC D or Higher of Occupancy Category I through III)

Table 4. Values of Parameter α of Equation 1 Recommended for Seismic Design of Acceleration-Sensitive Nonstructural Components in New Constructions

	PGA=0.4 S_{DS} < 0.067 g	0.067 ≤ PGA=0.4 S_{DS} < 0.20 g	PGA=0.4 S_{DS} ≥ 0.20 g
T_a < 0.5 sec.	2.12	1.93	1.75
0.5 ≤ T_a < 1.5 sec.	2.61	1.55	1.01
T_a ≥ 1.5 sec.	2.52	1.53	0.50

Table 5. Values of Parameter β of Equation 1 Recommended for Seismic Design of Acceleration-Sensitive Nonstructural Components in New Constructions

	PGA=0.4 S_{DS} < 0.067 g	0.067 ≤ PGA=0.4 S_{DS} < 0.20 g	PGA=0.4 S_{DS} ≥ 0.20 g
T_a < 0.5 sec.	0.78	1.25	0.92
0.5 ≤ T_a < 1.5 sec.	1.16	0.75	0.69
T_a ≥ 1.5 sec.	1.64	1.65	3.00

Table 6. Parameters of General 5%-Damped Floor a_p Spectrum Recommended for Seismic Design of Acceleration-Sensitive Nonstructural Components in New Constructions

	m_l	m_r	$a_{p,max}$
T_a < 0.5 sec.	0.8	1.4	3.3
0.5 ≤ T_a < 1.5 sec.	0.3	1.0	2.9
T_a ≥ 1.5 sec.	0.1	0.3	2.5

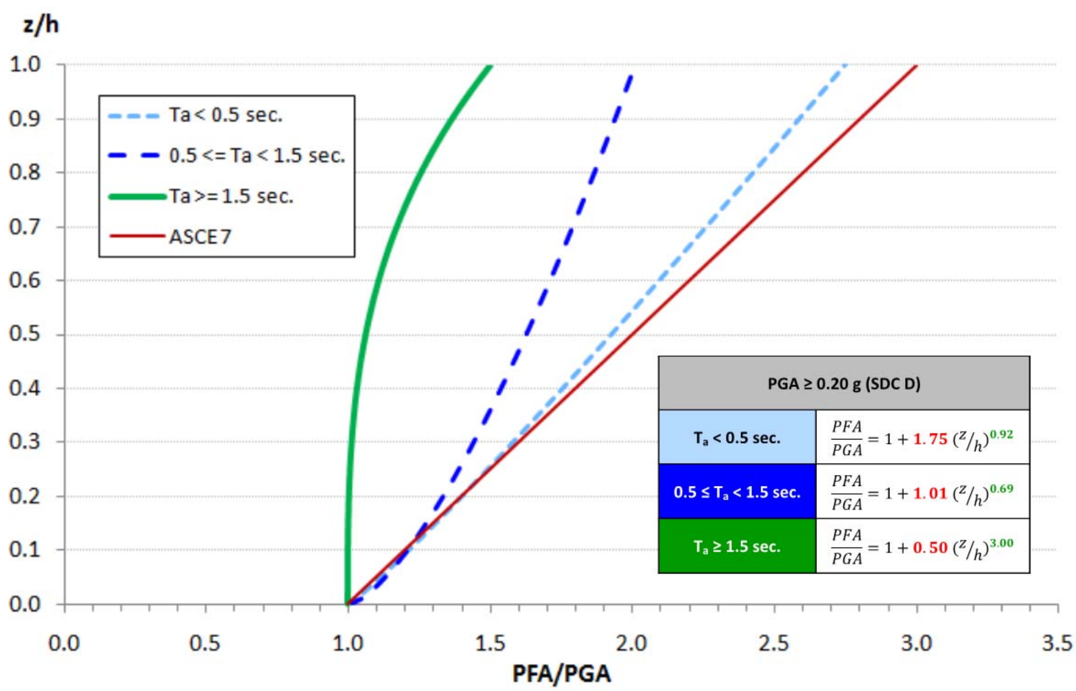


Figure 7. Profile of PFA/PGA over Building Height Recommended for Seismic Design of Acceleration-Sensitive Nonstructural Components in New Constructions under Earthquakes with PGA ≥ 0.20 g (SDC D or Higher of Occupancy Category I through III)

The profiles of peak floor acceleration over the building height for different ranges of PGA and building approximate period shown in Figures 6 and 7 demonstrate that the amplification of peak floor acceleration is inversely proportional to the building period and the effect of building period on the profile of peak floor acceleration over the building height is stronger under the strong ground motions than it is under moderate and minor earthquakes.

The results presented for parameters α and β in Tables 4 and 5 show that for some ranges of PGA and building period the proposed nonlinear profile for the PFA/PGA ratio over the building height is significantly different from the linear equation used by the code formula. To quantify how much this significant difference results in improvement in the goodness of the fit to the data points obtained from the CSMIP records, Coefficient of Determination or R^2 (R-squared) Error was used. It should be noted that contrary to what the name implies, the larger values of the R^2 Error correspond to better fits (the maximum value of the R^2 Error corresponding to a perfect fit equals one), and for poorer fits the index can take negative values. For different ranges of PGA and T_a, the R^2 Error of the proposed nonlinear equation and ASCE/SEI 7 formula are compared to each other in Table 7. Note that the "Best Fit" in Table 7 refers to the trendline through mean plus one standard deviation data points established by window averaging method that was previously explained in Methodology Section of this paper. As it can be seen in this table, for all different ranges of PGA and T_a, using the nonlinear equation instead of the linear equation of the code improved the goodness of the fit (results in larger values of R^2 Error). This improvement is particularly significant for longer building period and larger PGA values. Using the proposed nonlinear equation instead of the linear equation of the current code formula for those ranges of PGA increases the value of the R^2 error from negative values to values close to 0.6.

Table 7. Comparison of R^2 Error (Coefficient of Determination) of Proposed Equation and ASCE/SEI 7-05 equation for Profile of PFA/PGA Ratio over Building Height

	PGA < 0.067 g (SDC A)		0.067 ≤ PGA < 0.20 g (SDC B&C)		PGA ≥ 0.20 g (SDC D)	
	Best Fit	**ASCE 7**	**Best Fit**	**ASCE 7**	**Best Fit**	**ASCE 7**
T_a < 0.5 sec.	0.65	0.50	0.89	0.80	0.46	0.40
0.5 ≤ T_a < 1.5 sec.	0.80	0.65	0.58	0.38	0.54	-2.62
T_a ≥ 1.5 sec.	0.67	0.60	0.50	-0.08	0.59	-12.79

Conclusions and Summary

Fathali and Lizundia (2011a) provided a general evaluation of the key Equation 13.3-1 used by ASCE/SEI 7-05 for the seismic design of nonstructural components. This equation is unchanged in the current ASCE/SEI 7-10. A proposed revision to the equation was developed and was provided as Equation 3 above.

PFA/PGA Relationship: The code relationship between *z/h* and PFA/PGA is linear and amplifies PFA up to a value of three times that of the PGA. Our conclusions include the following.

- Equation 13.3-1 is a good fit for short-period buildings (fundamental period less than 0.5 seconds) in low-to-moderate seismicity areas such as those characterized by Seismic Design Category (SDC) B and C.

- Equation 13.3-1 was found to be significantly conservative (up to 100%) for medium-range period buildings (period between 0.5 seconds and 1.5 seconds) and long period buildings (period over 1.5 seconds).

- Equation 13.3-1 does not explicitly account for parameters found to be influential in this study, including building period, and PGA. Damping is likely to be influential as well, but was not directly investigated in this study.

- Simple improvements can be made to Equation 13.3-1 that will provide a better fit for the recorded data, using the code equation for fundamental period and USGS mapped values for site seismicity as reflected in the parameter S_{DS}.

Component Amplification Factor: The component amplification factor, a_p, used in Equation 13.3-1 accounts for the dynamic amplification of the component response as compared to the PFA. The code sets the value for a_p to be 1.0 for rigid components (defined as those with a fundamental period of 0.06 seconds or less) and 2.5 for flexible components (defined as those with a fundamental period greater than 0.06 seconds). Our conclusions include the following.

- The code values for a_p are conservative when the component fundamental period is longer than the building fundamental period.

- The code values for a_p are conservative when the component fundamental period is away from the range of periods that include the periods of the building modes that participate in the building response.

- The code values for a_p is less than the a_p value obtained from recorded values for flexible components located in short-period buildings (period less than 0.5 seconds) and medium-range period buildings (period between 0.5 seconds and 1.5 seconds). All our studies were done with 5% damped spectra. For components that have less damping, the difference would be even larger.

- The ASCE/SEI formulation for a_p does not consider the building fundamental period, which is expected from structural dynamics to have a significant influence and which was confirmed in our study.

- The ASCE/SEI formulation can be improved by use of a proposed a_p spectrum that is based on the building fundamental period as calculated using the standard code formula for building period.

- The improved formulation for a_p is relatively simple, inherently addresses the effect of the building lateral force-resisting system (through the use of building period), permits values of $a_p < 1$ where relevant, and permits use of values between the maximum and 1.0 for flexible components.

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