16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 2715

Registration Code: S-L1462914560

DESIGNING NONSTRUCTURAL COMPONENT ANCHORAGE USING APPROXIMATE FLOOR RESPONSE SPECTRA

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Abstract

The seismic behavior of nonstructural components is affected by the interaction of the dynamic behavior of the nonstructural component and the building. Building code requirements for the design of nonstructural components provide approximate procedures for estimating the floor accelerations of a building based on assumptions regarding the dynamic behavior of the building. Since the dynamic behavior of many nonstructural components is not usually well known, building code design methodologies provide only limited consideration of the building's dynamic characteristics and their effect on the nonstructural component's dynamic behavior.

The interaction between nonstructural components and the building could be determined by the development of floor response spectra, but the effort required to produce floor response spectra are typically not cost effective. A procedure has been developed and recently implemented into the U.S. building codes that allows for an improved consideration of the interaction of nonstructural components and the building. This procedure produces approximate floor response spectra based on an elastic modal analysis of the building. This approximate floor response spectra procedure is described and results are compared to the building code design requirements and the results using floor response spectra generated from response history analyses using several types of nonstructural components for example buildings. Recommendations are made for conditions where the use of the floor response spectra procedures would be beneficial.

Keywords: nonstructural, floor response spectra

1. Introduction

The design of nonstructural components for seismic loading is required for buildings in areas of moderate to high seismicity. The calculated seismic design force prescribed in building codes provides a simplification of the complex interaction of the behavior of nonstructural components attached to a building. The actual response of a nonstructural component to earthquake shaking depends in large part on both the vibrational characteristics of the building and the nonstructural component.

2. Background

The design seismic forces for nonstructural components prescribed by building codes has traditionally been based on simplified procedures. There are a number of justifiable reasons for using simplified design procedures for seismic design of nonstructural components, as described by Kehoe (2014). These simplified design equations are generally considered to be conservative estimates of the seismic effects on nonstructural components and failures of nonstructural components are typically related to lack of seismic design or inadequate installation rather than deficiencies in the design procedures. Little effort however has been made to understand how well these procedures predict the actual response of nonstructural components.

2.1 Building Code Design

The design procedure for nonstructural components in ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures* (2010) and its predecessor documents prescribes a horizontal seismic force be calculated and applied to the component. Eq. (1) shows the equation used to calculate the horizontal seismic force for a nonstructural component.

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

Where:

F_p is the design lateral seismic force on the nonstructural component,
S_{DS} is the design spectral acceleration at short periods
I_p is the component importance factor
a_p is the component response amplification factor
R_p is the component response factor
z is the height of attachment of the nonstructural component to the building, and
h is the height of the building.

This equation makes a number of simplifying assumptions. The period of vibration of the nonstructural component is not explicitly required nor are the periods of vibration of the building structure. Thus, nonstructural components are prescribed as being flexible or rigid, as indicated by the assigned value of the component response amplification factor. The variation in the floor accelerations for the building are assumed to be linearly increasing over the height of the building, which is based on the dynamic behavior of the building response being predominantly due to the first mode. The nonstructural force also does not account for differences in building behavior in different orthogonal directions.

2.2 Floor Response Spectra

A common simplification for the evaluation of the seismic effects on nonstructural components mounted within a building is the characterization of the nonstructural components as secondary components that are decoupled from the overall dynamic behavior of the structure. Based on this assumption, the response of the structure can be determined and the resulting response used as input to determine the response of the nonstructural components within the building. This process typically involves subjecting a model of the building to a response history analysis and extracting from that analysis the acceleration responses at each floor level of the building, which can in turn be used to calculate response spectra at each floor. These acceleration response spectra are referred to as floor response spectra.

Although floor response spectra can provide an estimate of a floor's acceleration response, this type of analysis is rarely used for the design of nonstructural components due to the effort required. The floor response spectra are only valid for the specific ground motion response used as building input, thus a suite of time history responses would need to be performed and aggregated to develop an appropriate design floor response spectrum for each floor.

2.3 Approximate Floor Response Spectra Method

An alternative method for determining the seismic design force for nonstructural components has been introduced in the 2016 Edition of ASCE 7 *Minimum Design Loads for Buildings and Other Structures*. This procedure is based on the methodology described by Kehoe and Hachem (2003).

A modal analysis of the building is performed to determine the period and mode shapes for the significant modes of the building. The number of modes that need to be considered is dependent on a number of factors including the characteristics of the earthquake and the modal characteristics of the building. For short period buildings, a minimum of three modes in each principal orthogonal direction of the building is recommended. For buildings with longer fundamental periods, the fourth mode and possibly the fifth mode should be considered.

For each mode of the building, the maximum floor acceleration at each floor is calculated separately for each mode in each orthogonal direction as the product of the spectral acceleration for the mode times the participation factor for the mode at the floor, as described in Eq. (2).

$$PF_{in} = \phi_{in} L_n / M_n \tag{2}$$

Where:

PF_{in} is the Participation Factor at level i for mode n,

ϕ_{in} is the Eigenvector value at level i for mode n,

L_n is the dot product of the mass matrix and Eigenvector for mode n, and

M_n is the effective modal mass.

Some software programs, such as ETABS and SAP 2000 (CSI 2015), define the modal participation factor as L_n/M_n and normalize the mode shapes such that the effective modal mass, M_n, is unity.

For nonstructural components that are flexible or flexibly supported, the maximum floor acceleration is amplified by a magnification factor to account for the resonance between the nonstructural component and the building. The peak modal response for a flexible component is approximated to be 5.0 when the period of the nonstructural component equals one of the periods of the building based on an assumed damping ratio of 5 percent for the nonstructural component, which is consistent with the assumptions made in the ASCE 7 equation. No amplification is assumed when the period of vibration of the nonstructural component is more than a factor of 2 or less than 1/2 of one of the building periods. Because of the uncertainty in the periods of the building and the nonstructural component, the generalized magnification factor in Fig. 1 includes a broad plateau of plus or minus 20 percent where the period of the nonstructural component matches one of the building periods. This is similar to the recommendations for evaluating nonstructural components for the U.S nuclear industry, which suggests broadening the spectra by plus or minus 15 percent.

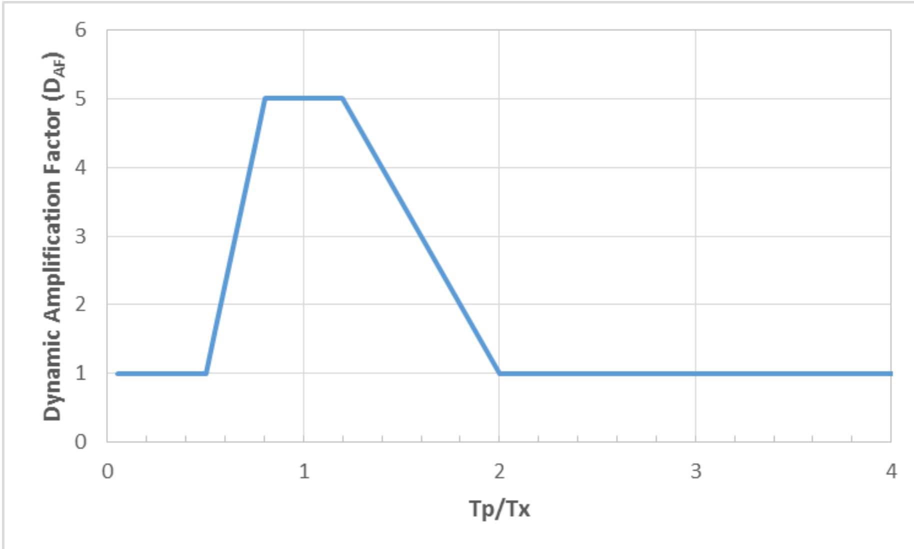

Fig. 1 – Nonstructural Dynamic Amplification Factor

For each building mode considered, the maximum floor acceleration is multiplied by the magnification factor to create a modal floor response spectrum, as shown in Eq. (3).

$$A_{in} = PF_{in} S_{an} D_{AF} \tag{3}$$

Where:

A_{in} is the component acceleration at level i for mode n,

PF_{in} is defined in Eq. (2),

S_{an} is the spectral acceleration for mode n, and

D_{AF} is the dynamic Amplification Factor from Fig. 1.

Since the generalized spectrum in Fig. 1 is normalized by the ratio of the period of the nonstructural component to the building period (T_P/T_X), the horizontal axis of each of these spectra is multiplied by the period of the mode to produce a set of spectra relating the nonstructural component period to the acceleration. The spectra are then combined and the design spectrum is taken as the envelope of the maximum acceleration value at each period from each of the modal spectra. The minimum design acceleration is spectral acceleration at the base of the building.

For the design of nonstructural components, the component acceleration, A_{in}, accounts for three of the terms in Eq. 1: the ground motion term (0.4 S_{DS}), the component response amplification factor (a_p), and the variation of response over the height of the building (1+2x/h). Thus, the nonstructural component design force using the approximate floor response spectra method is shown in Eq. (4), where the acceleration used is the maximum modal response acceleration for the level.

$$F_p = \text{Max}(A_{in})W_p\,(R_p/I_p) \tag{4}$$

3. Example Analyses

The approximate floor response spectra procedure has been performed for two sample buildings. Each building model was subjected to a series of earthquake ground motions and the floor response spectra were calculated. The calculated floor response spectra from the response history analysis is compared to the results of the approximate procedure.

3.1 Example Building Responses

Mathematical models of each of the two sample buildings were analyzed. One of the buildings was modeled using SAP 2000 from Computers and Structures Inc. (CSI) (2015) and one building was modeled using ETABS from CSI (2015).

3.1.1 25-Story Steel-Framed Building

An analysis was performed for a 25-story steel building. The building is roughly rectangular in plan and the plan shape changes configuration at two levels over the height of the building. The analysis model is shown in Fig. 2. The lateral force resisting system for the building includes steel moment-resisting frames and diagonal steel braced frames. The floors are constructed with steel beams supporting concrete-filled metal deck slabs. The building is supported on concrete piles. The exterior walls include precast concrete spandrel panels and insulated glass panels.

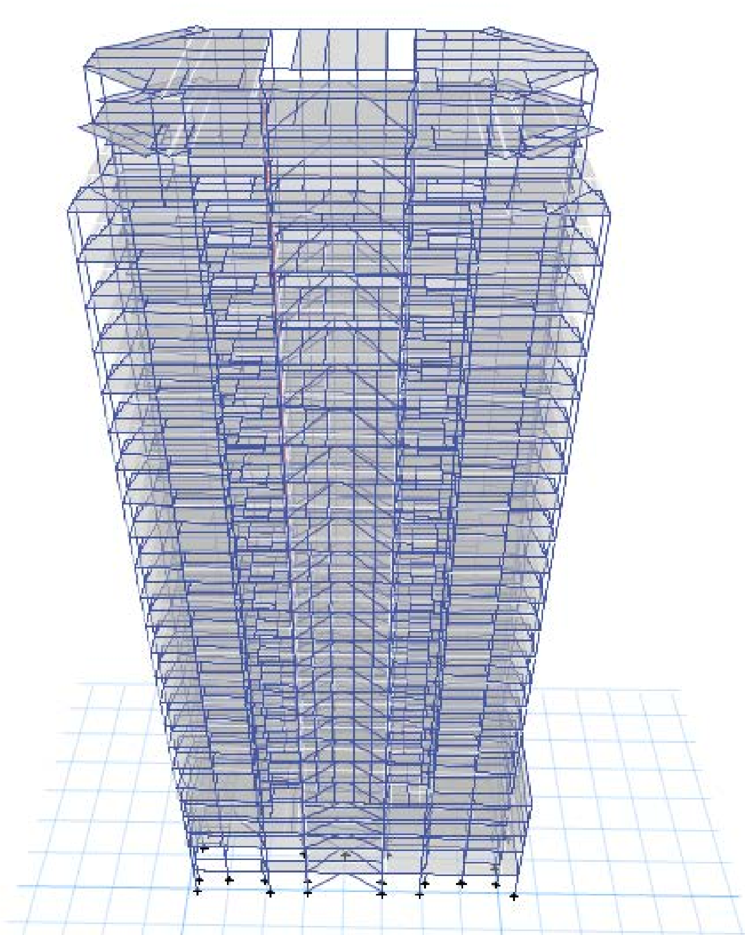

Fig. 2 – Model of 25-Story Building

The dynamic characteristics of the building are summarized in Table 1.

Table 1 – Dynamic Characteristics for 25-Story Sample Building

Mode	**Period (sec)**	**Direction**
1	3.169	Longitudinal
2	2.784	Transverse
3	2.249	Torsion

4	1.057	Longitudinal
5	0.837	Transverse
6	0.727	Longitudinal
7	0.594	Torsion
8	0.442	Transverse

The 25-story building was analyzed under eight ground motion acceleration time histories. These time history records represent the two horizontal components of four different recorded earthquake motions. The earthquake records used are from two California earthquakes. The earthquake records used and the peak ground acceleration (PGA) for each record in each direction are listed in Table 2.

Table 2 – Ground Motion Records Used

Earthquake / Recording Station	**PGA - direction 1**	**PGA - direction 2**
1994 Northridge /Corralitos - Eureka Canyon Rd	0.63 g	0.46 g
1989 Loma Prieta / Hollister	0.37 g	0.17 g
1994 Northridge / Newhall Fire Station	0.57 g	0.63 g
1994 Northridge / Sylmar	0.84 g	0.60 g

To apply the approximate floor response spectrum method, the response spectra for each of the ground motion records was used. Fig. 3 shows a plot of the response spectra.

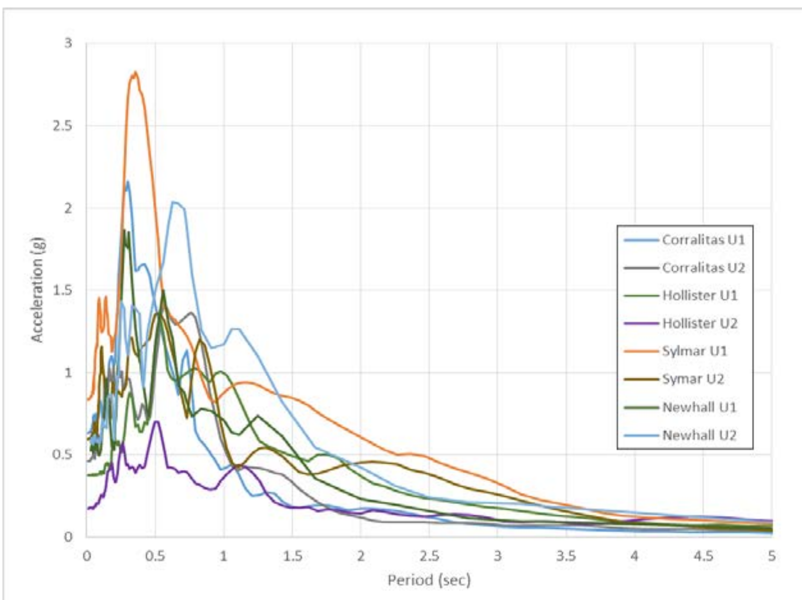

Fig. 3 – Ground Motion Response Spectra

3.1.2 4-Story Steel-Framed Building

An analysis was performed for a 4-story steel building. The structure consists of a full basement level, a one-story podium level, and three story tower floors. The analysis model is shown in Fig. 4. The lateral force resisting system for the building primarily consists of a space steel moment-resisting frame with moment connections to the column in two directions. The floors are constructed with steel beams supporting concrete-filled metal deck slabs.

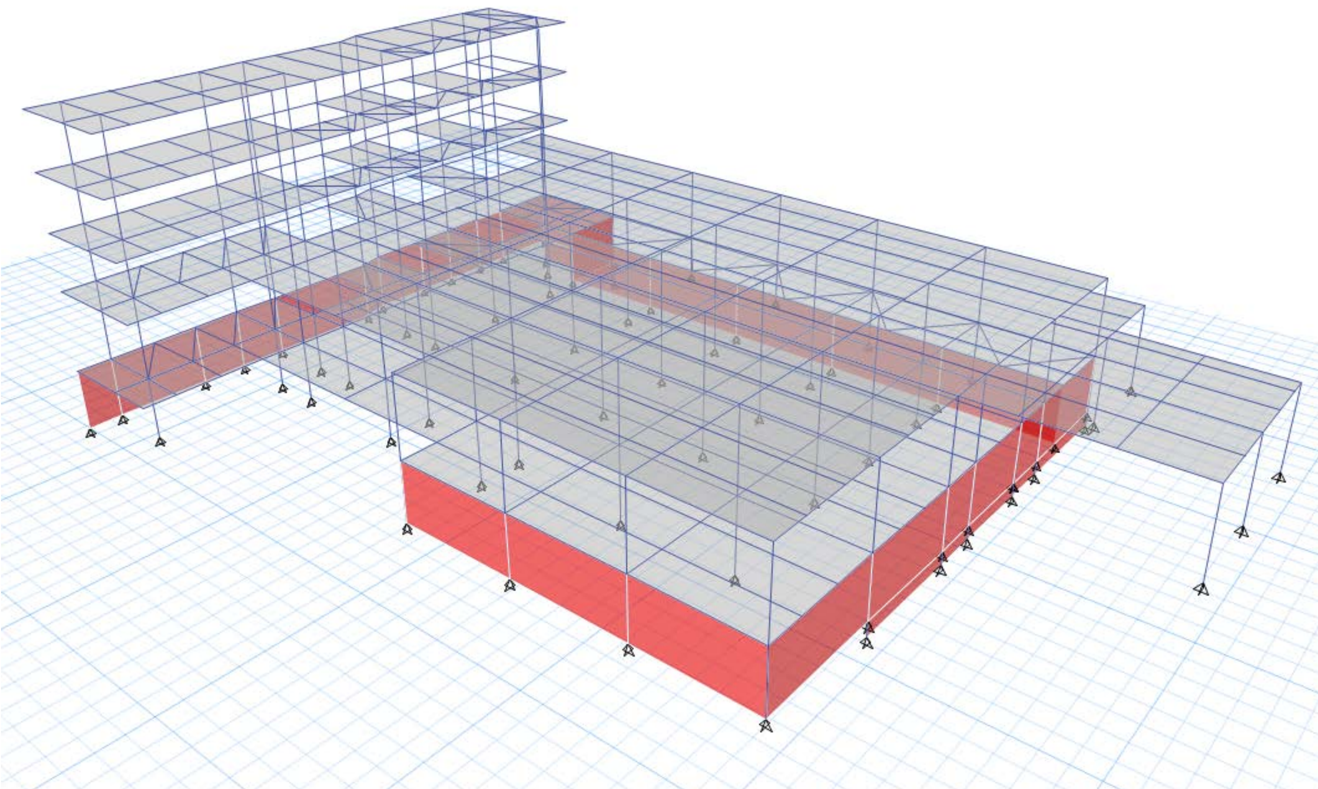

Fig. 4 – Model of 4-Story Building

The dynamic characteristics of the building are summarized in Table 3.

Table 3 – Dynamic Characteristics for 4-Story Sample Building

			Mass Ratios			Cumulative Mass Ratios		
Mode	**Period**	**Direction**	**UX**	**UY**	**RZ**	**Sum UX**	**Sum UY**	**Sum RZ**
1	0.718	Y+X+RZ	8%	24%	17%	8%	24%	17%
2	0.638	X+Y	24%	13%	9%	32%	37%	26%
3	0.513	X	7%	3%	3%	39%	40%	30%
4	0.351	X+RZ	15%	2%	18%	54%	42%	47%
5	0.327	Y	3%	17%	5%	57%	58%	52%
…								
13	0.029	Y	0%	27%	1%	59%	86%	65%
14	0.025	X	31%	1%	6%	90%	88%	71%
15	0.023	Y+X	8%	11%	3%	97%	99%	74%

The computed modes generally exhibit significant coupling between the modes. This is likely caused by the asymmetric configuration of the building and the coupling between torsional and longitudinal modes.

The 4-story building was analyzed under one ground motion acceleration time history that was recorded during the 2014 Napa earthquake, at CGS-SMIP Station 68294 in Vallejo, California. The record had peak ground accelerations (PGA) of 0.22g and 0.47g in direction 1 (90º component) and direction 2 (360 º component), respectively. A linear time history analysis was performed using the linear modal combination method, with a damping ratio of 2% in all modes.

To apply the approximate floor response spectrum method, the response spectra for each of the ground motion components was used. Figure 5 shows a plot of the response spectra under 2% and 5% damping.

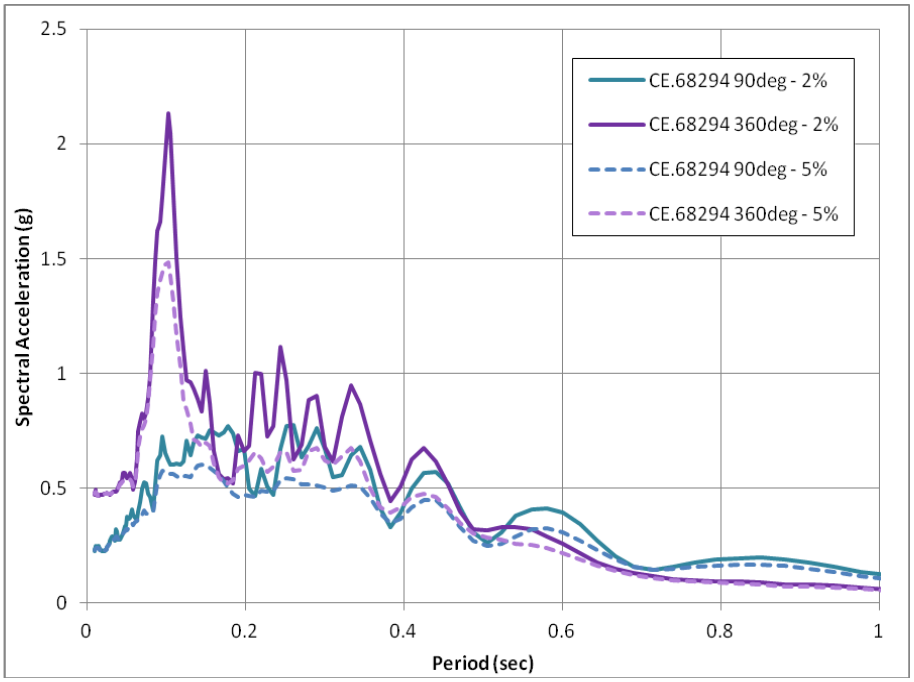

Fig. 5 – Ground Motion Response Spectra

3.3 Nonstructural Component Response

To evaluate the component responses for the sample ground motions, the peak ground accelerations are used in lieu of the $0.4S_{DS}$ term in Eq. (1). Table 4 summarizes the design accelerations for each of the sample components for each sample ground motion at the roof level using the ASCE 7 equation ($h_x = h_r$). The values in Table 4 account for the peak ground acceleration and the component amplification factor a_P at the roof level, which produces the maximum design acceleration. Note that ASCE 7-10 considers the wall cladding and the interior partitions as rigid components with a component amplification factor of 1.0 and the water pipe and appendage are considered flexible components with a component amplification factor of 2.5.

Table 4 –Component Design Accelerations (g) Using ASCE 7-10 for Sample Ground Motions

Ground Motion/Direction	Component			
	Wall Cladding	Interior Partition	Water Pipe	Appendage
Corralitas 1	1.43	1.43	3.59	3.59
Corralitas 2	1.89	1.89	4.72	4.72
Hollister 1	0.53	0.53	1.33	1.33
Hollister 2	1.11	1.11	2.77	2.77

Newhall 1	1.74	1.74	4.37	4.37
Newhall 2	1.76	1.76	4.42	4.42
Sylmar 1	1.81	1.81	4.53	4.53
Sylmar 2	2.53	2.53	6.32	6.32

The fundamental periods of vibration have been calculated for several typical nonstructural components as described below.

3.3.1 Exterior Wall Cladding

The period of vibration for a precast concrete wall panel was calculated assuming that the panel acts as a simply supported beam spanning between connections, which occur at the floor framing. The cladding is assumed to be reinforced concrete, 12.7 cm (5 in) thick, with a compressive strength of 27.6 MPa (4000 psi). The vertical span between supports is assumed to be 4 m (13 ft. 1 inch). The effective moment of inertia is assumed to be 0.35 times the gross moment of inertia, as permitted by ACI 318. Based on these assumptions, the fundamental period of the claddings is 0.077 seconds. The component is considered flexible since the period is slightly larger than 0.067 second, which is considered to be the upper bound limit for a component to be considered rigid.

3.3.2 Interior Partition

The period of vibration for an interior, non-loadbearing partition was calculated. The partition is assumed to consist of light-gage steel studs at a spacing of 40.c cm (16 inches). The partition is assumed to span vertically with anchorage to the floor system at the base and lateral bracing along the top of the partition. The height of the partition from the floor to the lateral bracing at the top is assumed to be 2.75 m (9 ft). The steel studs have an effective moment of inertia of 45.7 cm^4 (1.1 in^4). The gypsum board wall sheathing is assumed have a thickness of 134 Pa (2.8 psf). Based on these assumptions, the fundamental period of vibration of the partition is 0.067 seconds. The component is considered rigid since the period is considered to be equal to the upper bound limit for a component to be considered rigid.

3.3.3 Water Pipe

The period of vibration of a horizontally oriented steel pipe containing water was calculated assuming that the pipe acts as a simply-supported beam between lateral supports. The pipe is assumed to have a diameter of 10.2 cm (4 in) and a thickness of 0.57 cm (0.226 in). The distance between supports is assumed to be 3.5 m (11.5 ft) and the bracing is assumed to be rigid. The piping is assumed to be connected with threaded joints. Based on these characteristics, the period of vibration for the pipe is assumed to be 0.057 seconds. The component is considered rigid since the period is less than the upper bound limit for a component to be considered rigid.

3.3.4 Appendage

The period of vibration was calculated for a sign mounted on the side of a building with cantilever beams. The length of the cantilever is 1.5 m (4.9 feet). The cantilever beam is a square structural tube about 76 mm wide with a thickness of about 6.3 mm. The tributary weight of the sign on the cantilever is 678 kg (1500 pounds). Based on these characteristics, the period of vibration for the sign is 0.35 seconds. The component is considered flexible since the period is larger than 0.067 second and therefore should be considered flexible.

4. Comparison of Results

4.1 25-Story Sample Building

For each of the ground motions, the floor response spectra were computed for the roof level and the thirteenth floor in each direction. The approximate floor spectra were also computed based on the procedure described above. Fig. 6 shows a comparison of the spectra for the roof level in each orthogonal direction computed based on the Corralitas direction 2 ground motion histories applied in the longitudinal and transverse building directions to the approximate roof response spectrum. In the figure, the spectra for each of the predominant modes in the transverse direction are shown with dashed lines and the envelope of maximum of the responses is shown in the dark solid line.

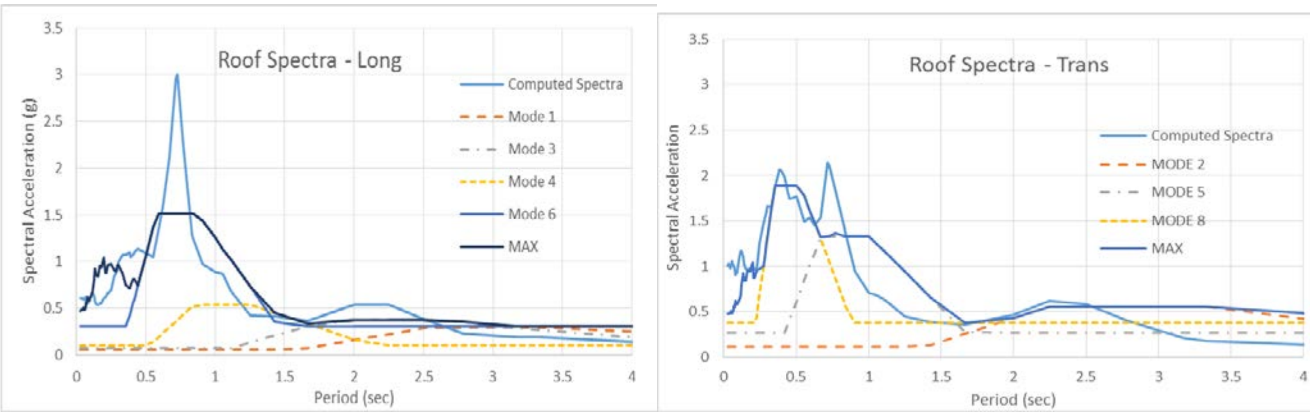

Fig. 6 – Roof Spectra for 25-Story Sample Building in Longitudinal and Transverse Directions

The results of the approximate floor response spectra procedure are compared with the results using the floor response spectra for each ground motion at three floor levels for the example nonstructural components described above. Table 5 summarizes the results for three of the ground motions applied in the longitudinal direction of the building.

Table 5 – Comparison of Component Accelerations (g) for 25-Story Sample Building

Component	**Floor Level**	**Corralitas2 Computed Accel (g)**	**Corralitas2 Approx. Accel (g)**	**Hollister2 Computed Accel (g)**	**Hollister2 Approx. Accel (g)**	**Newhall2 Computed Accel (g)**	**Newhall2 Approx. Accel (g)**
Wall Cladding	Roof	0.52	0.55	0.40	0.46	0.94	1.03
	16th	0.41	0.55	0.33	0.34	0.79	0.76
	8th	0.40	0.55	0.42	0.44	0.97	0.99
Interior Partition	Roof	0.50	0.53	0.39	0.46	0.92	1.03
	16th	0.38	0.53	0.32	0.34	0.75	0.76
	8th	0.40	0.53	0.46	0.42	0.95	0.99
Water Pipe	Roof	0.51	0.49	0.38	0.46	0.90	1.03
	16th	0.39	0.49	0.31	0.34	0.73	0.76
	8th	0.42	0.49	0.42	0.44	0.92	0.99
Appendage	Roof	0.89	0.75	0.52	0.45	1.18	1.39
	16th	0.87	0.75	0.46	0.34	1.31	0.92
	8th	1.20	0.75	0.64	0.44	1.33	1.39

4.2 4-Story Sample Building

For each direction of the building and corresponding record, the floor response spectra were computed for the fifth floor (roof) and the third floor. The approximate floor spectra were also computed based on the procedure described above. Fig. 7 and Fig. 8 show a comparison of the spectra for the roof and third floors in each orthogonal direction computed based on the Napa ground motion histories applied in both building directions simultaneously. In the figures, the spectra for each of the predominant modes are shown with dashed lines and the envelope of maximum of the responses is shown in the dark solid line. All floor spectra were computed using 5% damping.

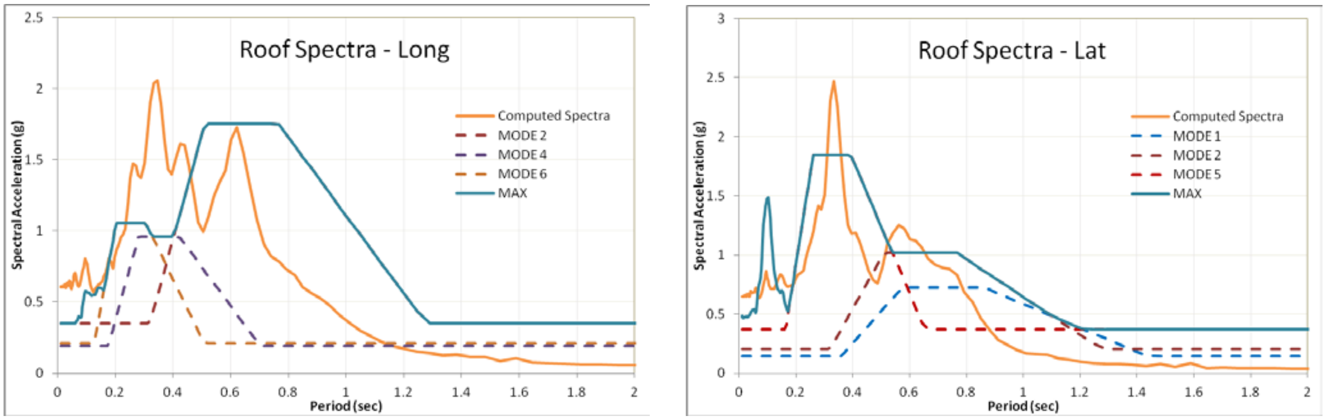

Fig. 7 – Roof Spectra for 4-Story Sample Building in Longitudinal and Transverse Directions

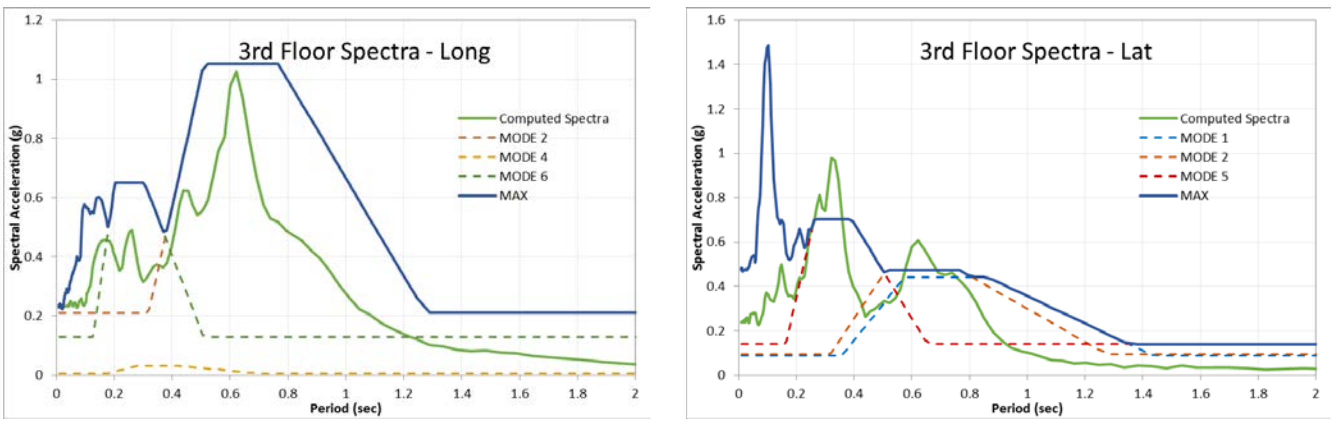

Fig. 8 – Third Floor Spectra for 4-Story Sample Building in Longitudinal and Transverse Directions

The proposed procedure appears to provide a good estimate of floor spectra over a wide range of periods, despite the somewhat irregular configuration of the building. At short periods, the procedure tends to underestimate the spectral accelerations, though the discrepancy is significantly reduced when using the ground level spectrum as a minimum, especially at lower floors. It is noted that lateral component of the ground motion had a sharp peak in the response spectrum at around 0.1 second, which was not observed in the computed spectra at the upper floors.

5. Conclusions

A floor response spectrum provides the expected design acceleration for non-structural components. The floor response spectra vary depending on the ground motion, the dynamic properties of the building, and the location within the building. A procedure for developing approximate floor response spectra has been introduced into the latest version of ASCE 7 that uses the design response spectra in lieu of response history records to generate the design spectra. The approximate floor response spectra has been shown to provide a reasonable estimate of a floor response spectra by accounting for the amplification effects of each of the significant modes of vibration of the building. The computed floor response spectra occasionally produces design accelerations that exceed the approximate floor response spectra. This variation is due to peaks in the response spectra that are smoothed out when developing response spectra for design. In this study, the base spectra tends to control the floor spectra at short periods and the approximate floor response spectra may underestimate the component acceleration for flexible components, but provides a good estimation of the component acceleration for rigid components.

The ASCE 7 component amplification factors may not accurately represent components that should be categorized as rigid or flexible based on the actual dynamic properties of the component. The floor response spectra computed using actual ground motions however, indicates that the upper bound period limit of 0.067 seconds for a component to be considered rigid may be increased since very little amplification of component accelerations were found for components with periods up to about 0.10 seconds.

The results also showed that the ASCE 7 equation for designing nonstructural components overestimates the design acceleration values across the range of component periods. These results also confirm previous findings that for long period buildings there is less amplification of the horizontal acceleration over the height of the building than prescribed by the ASCE 7 design equation.

6. References

[1] ASCE 4-98 (2000) *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, American Society of Civil Engineers, Virginia, USA.

[2] ASCE/SEI. ASCE/SEI 7-16 (2016 anticipated) *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Virginia, USA.

[3] Computers and Structures, Inc. (2015), *CSi Analysis Reference Manual*, Computers and Structures, Inc., Berkeley, California, USA

[4] Kehoe, BE (2014) "Defining Rigid vs. Flexible Nonstructural Components", *Proceedings of the 10 US National Conference on Earthquake Engineering*, Anchorage, Alaska, USA.

[5] Kehoe BE, Hachem M, Procedures for Estimating Floor Accelerations (2003), *ATC 29-2 Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities*, Applied Technology Council, California, USA.