EVALUATION OF EQUIVALENT STATIC AND DYNAMIC ANALYSIS METHOD FOR SEISMIC DESIGN OF NON-STRUCTURAL ELEMENTS

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ABSTRACT

Seismic design of non-structural elements (NSEs) has recently become one of the major topics in earthquake engineering because of costly and widespread damage reported in recent earthquakes. In major building codes, the seismic design forces are determined based on either the equivalent static method or dynamic analysis methods. However, simple and empirical equivalent static approach neglect several key influential parameters which could significantly affect the acceleration demand on NSEs. Alternatively, the dynamic analysis methods are expected to provide more reasonable results since they consider dynamic properties of primary structural systems in determining the seismic demand. In this study, in order to appraise the accuracy and the reliability of each method, a comparative case study was conducted using realistic 3-dimensional building models. It is shown that the magnitude and profile of the peak floor acceleration can significantly be affected by the characteristics of supporting structures such as modal periods, type of lateral resisting system, and inherent torsion. The floor response spectrum of a torsionally irregular building was unique in that the first three modes were collectively tuned within a narrow band because of the closely spaced nature of the first three modes. The code prescribed component amplification factor was unconservative around the tuning periods and varied depending on the location of floor. Dynamic analysis approach including tuning effect appears inevitable for more accurate and rational seismic design of NSEs.

Keywords: non-structural elements, equivalent static method, dynamic method, floor acceleration, floor response spectrum

INTRODUCTION

It is now widely recognized that earthquakes can cause extensive damages to non-structural elements (NSEs) and the damages in non-structural elements can outweigh structural system damages. Damage to NSEs can be triggered by ground shaking intensities much lower than those required to initiate structural damage. One of the main reasons for seismic vulnerability of NSEs in buildings is that NSEs are usually attached to the elevated portion of a building, thus subjected to amplified ground motion generated by the dynamic response of the building. Further, the material and details used for NSEs are often far from being ideal for seismic resistance.

There has been a lot of research efforts to develop rational seismic analysis methods of NSEs during the last four decades. Since rigorous analysis methods of a combined supporting structure and NSE system involve mathematical complexities almost impractical to handle, major building codes suggest a simple equivalent static method. However, due to its empirical and simplistic nature, the design equations for NSEs based on the equivalent static approach neglect several key influential parameters which could significantly affect the acceleration demand on NSEs. Alternatively, the dynamic analysis methods are also suggested with different degrees of approximation in calculating dynamic response of NSEs.

However, the accuracy and the reliability provided by the equivalent static and dynamic methods are not well-known yet and through comparative study needs to be performed using realistic 3-dimensional building models. In this study, several key influential parameters affecting the acceleration demand on

NSEs are investigated in order to evaluate the accuracy of the equivalent static and dynamic methods recommended in ASCE 7-16. Preliminary analysis results on floor acceleration obtained from an up-to-date database of instrumented buildings are also briefly presented.

DESIGN METHODS IN BUILDING CODES

In this section, design methods proposed by ASCE 7-16 will be briefly reviewed. As per ASCE 7-16, the seismic design force F_p can be determined as follow:

$$0.3S_{DS}I_pW_p \le \frac{0.4S_{DS}a_pW_p}{(R_p / I_p)}(1+2\frac{z}{h}) \le 1.6S_{DS}I_pW_p \tag{1}$$

where F_p = seismic design force to be applied at the center of gravity and distributed relative to component's mass distribution; I_p =component importance factor 1.0 or 1.5; W_p = component operating weight; z = height in structure of the highest point of attachment of component with respect to the base; and h = average roof elevation of structure above the base. S_{DS} = short period spectral acceleration for 5% damping; a_p = component amplification factor 1.0 or 2.5; and R_p = component response modification factor that varies from 1.0 to 12.

In Equation (1), the ground acceleration at the base $0.4S_{DS}$ is intended to be the same acceleration as design ground acceleration for the structure itself including site effects. This value is multiplied by a factor that varies linearly with height. The roof acceleration is limited to three times the design ground acceleration based on the review of recorded in-structure acceleration data for short and moderate-height structures under California earthquakes as will be discussed in the below. Note that the component amplification factor, a_p, is assigned as 1. 0 for rigid component and 2.5 for flexible components with somewhat arbitrary definition of 0.06sec as the borderline period to separate them.

In ASCE 7-16, in lieu of the equivalent static method above, one of the following dynamic analysis methods can be used to determine the acceleration demand on NSEs: linear dynamic analysis, nonlinear response history procedure, floor response spectra method and alternate floor response spectra approach. However, the linear/nonlinear time history analysis procedures require input selection, scaling and time-consuming numerical integration. In practice, the most feasible method is the linear dynamic analysis based on the response spectrum method.
In all the above mentioned dynamic analysis methods, the seismic design force F_p is determined as follows:

$$F_p = \frac{a_i a_p W_p}{(R_p / I_p)} A_x \tag{2}$$

where a_i = maximum acceleration at level i obtained from the dynamic analysis; A_x = torsional amplification factor.

The maximum acceleration (a_i) can also be determined using floor response spectra method or alternate floor response spectra method given in ASCE 7-16. It should be noted that the component amplification factor (a_p) should be taken as 1.0 in this case since the resonance effect between the main and non-structural system is already taken into account in the both methods. Alternate floor response spectra method is a direct floor response spectrum generating method since it does not require time history analysis which should be done in conventional floor response spectrum method. It accounts for component dynamic amplification in approximate manner as a function of the ratio between the component period to the building modal period. However, it is important to note that this method is only applicable when the dynamic characteristics of both the main and non-structural system are already known in advance.

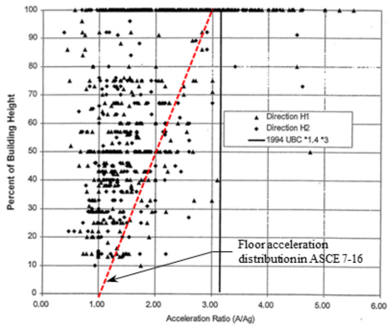
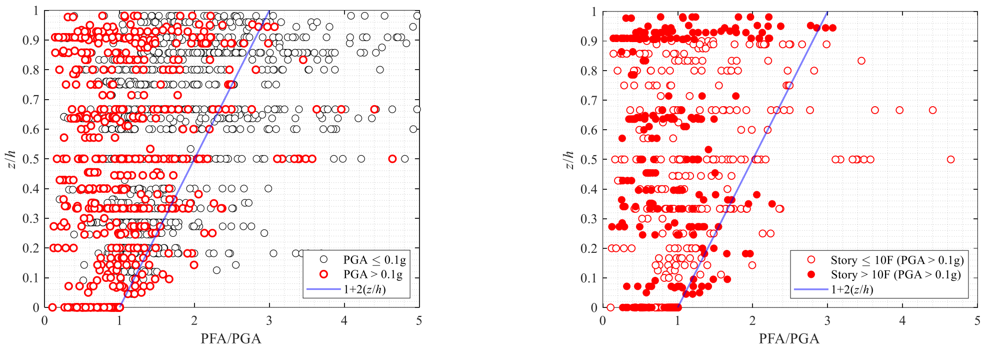


Figure 1. Instrumented building acceleration data (Drake and Bachman, 1995)



(a) All data (b) PGA>0.1g

Figure 2. An up-to-data database of instrumented building

EFFECTS OF SUPPORTING STRUCTURAL CHARACTERISTICS ON PEAK FLOOR ACCELERATION

The equivalent static method in ASCE 7-16 was proposed based on the instrumented building floor acceleration data obtained from California earthquakes (Drake and Bachman, 1995). To retain practical simplicity and consistency with the measured data, the assumption of linear distribution of floor acceleration up to three times amplification of peak ground acceleration at the top of the structure was adopted as in equation (1) (see Figure.1).

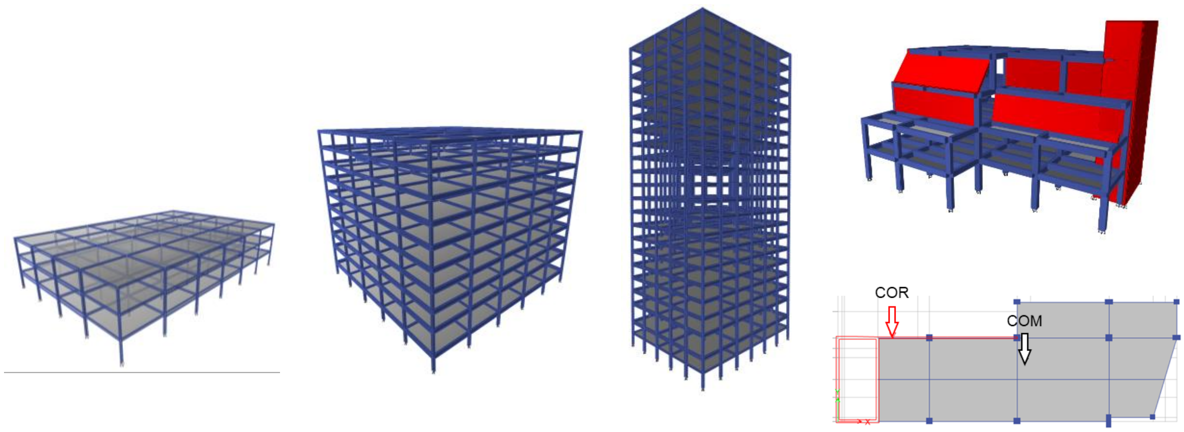
Figure 2 shows a preliminary analysis result of up-to-date database of instrumented buildings using the CESMD (Center for Engineering Strong Motion). The database was built based on 63 earthquakes covering 1978 Santa Barbara (5.1 M_w) earthquake to 2018 Thousand Palms (3.8 M_w) earthquake.

As shown in Figure 2 (a), based on the up-to-date database, the distribution of peak floor acceleration does not well corroborate the magnitude and profile of the maximum floor acceleration specified by the current equivalent static approach. Also, as shown in Figure 2 (b), based on the data from instrumented buildings having more than 10 stories, the distribution of mean peak floor acceleration is almost constant along the building height except at the roof. This is mainly due to the higher mode effect of supporting structure which will be discussed in following theoretical and numerical analysis.

From the elementary structural dynamics (Chopra, 2001), the absolute peak floor acceleration from summing vibrational modes per SRSS rule can be approximately derived as follows (equation 3) in which subscripts n and j indicate n^{th} mode shape and j^{th} story.

$$f_{j,\max} / m_j = \sqrt{\sum_{n=1}^{N}[\Gamma_n \phi_{nj} \omega_n^2 S_D(T_n, \zeta_n)]^2} \tag{3}$$

Compared with the equation (1), equation (3) clearly shows that the peak floor acceleration in principle should be based on the structural period-dependent spectral acceleration (S_a), rather than S_{DS} which is determined independent of structural periods. Also, the distribution of peak floor acceleration should



(a) 3-story SMRF (b) 9-story SMRF (c) 20-story SMRF (d) 4-story RC frame with torsional irregularity

Figure 3. Three dimensional structure models

follow structural mode shape (ϕ) rather than linear distribution. Considering that the contribution of each mode is proportional to the squared modal frequencies, the contribution of higher modes can be significant when structural periods become longer.

Three-dimensional Numerical Analysis

In this section, the effects of several supporting structural characteristics on the response of floor acceleration were evaluated using a total of four three-dimensional building models (Figure 3). ETABS 2016 were used for the analysis. The first three models [Figure 3. (a) ~ (c)] were perimeter moment resisting frames often referred to as SAC building models designed according to the UBC 1994 for the Los Angeles area (Gupta and Krawinkler, 1999). These three steel buildings may be classified as regular. The last model was selected from the telecommunication center facilities existing in Korea in order to investigate the effect of torsional behavior. The distance between center of rigidity(CoR) and the center of mass (CoM) are shown on the floor plans in Figure 3 (b). The eccentricity in the longitudinal direction is 40.5%. The fundamental periods obtained from eigenvalue analysis of numerical models are summarized in Table 1.

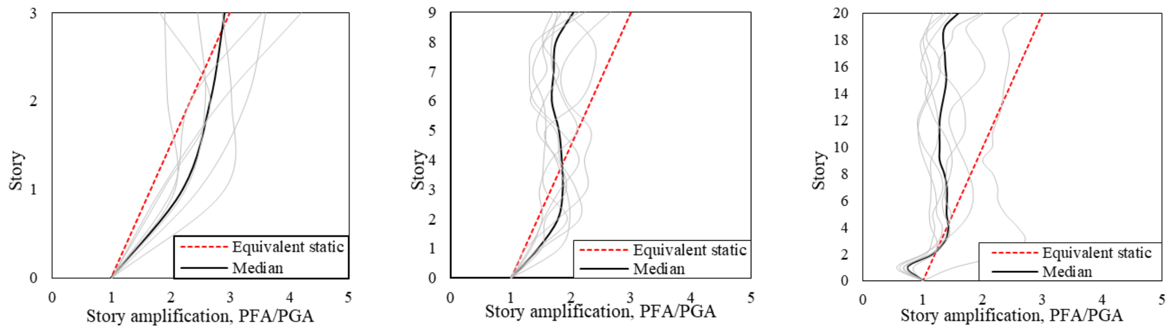
In this study, linear and nonlinear time history analyses were conducted using a suite of seven far field ground motions selected from PEER-NGA database. The lists of the ground motions are summarized in Table 2. The amplitude of input motions was scaled to be compatible with the design spectrum following the scaling method suggested in ASCE 7-16.

Figure 4 shows the results of linear time history analysis. The ordinate was normalized to the peak ground acceleration (PGA) to evaluate the in-structure amplification of acceleration along the height. First, the floor acceleration response is somewhat scattered because of random nature of input motions. As already pointed out above, the floor acceleration is highly overestimated by the equivalent static method as the fundamental period of building is increased [see Figure 4. (b), (c)], whereas, the floor acceleration of the 3-story building is underestimated by the equivalent static method [see Figure 4. (a)].

In the same context, due to the higher mode effects, the distribution of floor acceleration is almost

Table 1. Summary of fundamental periods of structure models

No.	Story	T_n	Structure type
1	3	1.16	Steel moment resisting frame
2	9	2.37	Steel moment resisting frame
3	20	3.87	Steel moment resisting frame
4	4	0.54	Reinforced concrete moment resisting frame (with irregular walls and corner staircase)



(a) 3-story (T = 1.16 sec) (b) 9-story (T = 2.37 sec) (c) 20-story (T = 3.87 sec)

Figure 4. Effect of structural period on floor acceleration amplification and profile

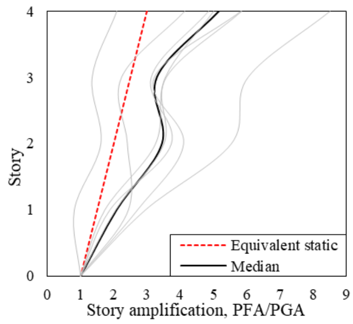
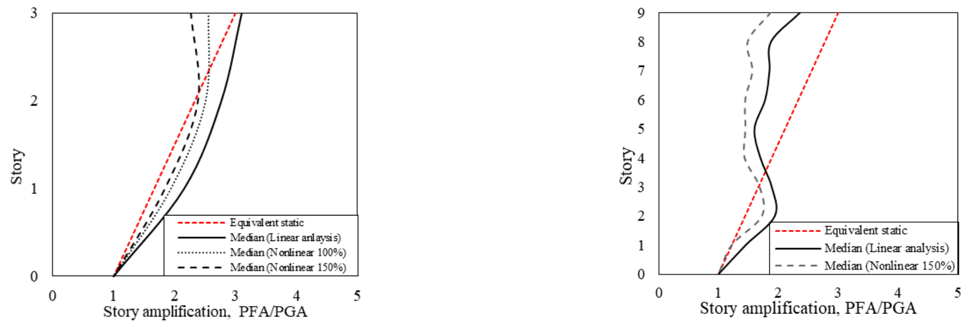
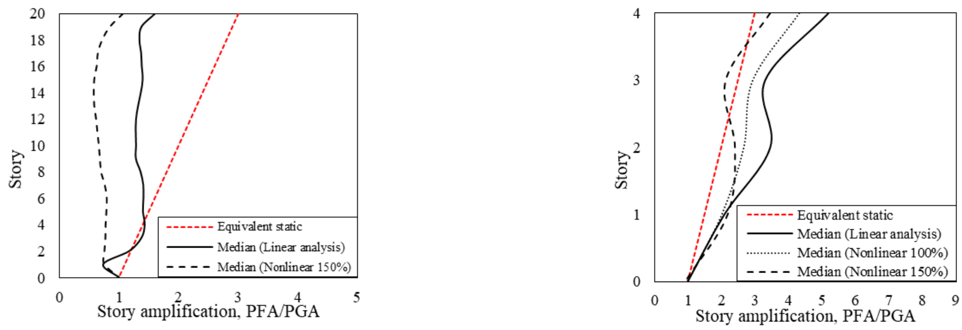


Figure 5. Effect of torsional behaviour of supporting structure



(a) 3-story (b) 9-story



(c) 20-story (d) 4-story (torsionally irregular building)

Figure 6. Effects of structural nonlinearity on the amplification of PFA

constant over the height except at the top and the bottom stories. Except for the short period building (3-story) which respond primarily in the linear-shape fundamental mode, the higher modes of long period buildings tend to produce the floor acceleration profile which is almost constant in vertical.

Table 2. Ground motion record list

Earthquake	Year	Station	Mag. (M_w)	Epicentral distance (km)
Chuetsu-oki, Japan	2007	Sanjo	6.8	21.4
Chuetsu-oki, Japan	2007	Niigata Nishi Kaba District	6.8	27.83
Imperial Valley	1979	El Centro Array #11	6.5	12.56
Loma Prieta	1989	Fremont - Mission San Jose	6.5	39.32
Iwata, Japan	2008	Misato Miyagi Kitaura - A	6.9	38.04
Chi-Chi, Taiwan	1999	TCU070	6.2	45.94
Kobe, Japan	1995	Tadoka	6.9	31.69

In Figure 5, the result of linear time history analysis for the torsionally irregular structure [Figure 3.(d)] is presented. Due to the torsional amplification caused by the torsional mode of the supporting structure, the peak floor acceleration is significantly underestimated by the current equivalent static method and the vertical profile is neither linear nor constant. It should be noted that unlike the design equation in dynamic method [equation (2)], the equivalent static method does not require any considerations for the torsional amplification that could be caused by torsional irregularity.

In Figure 6, the results of incremental nonlinear time history analysis for the four buildings are presented. The intensity of ground motions was arbitrarily increased to induce different degrees of nonlinearity in the structure. The ground intensity up to the 100% design earthquake did not cause significant nonlinear behaviour in the case of 9- and 20-story structures. It is observed in Figure 5 that as the intensity of ground motions increased, the floor acceleration is also reduced because of period lengthening due to yielding. It is interesting to note that the PFA of the 20-story model becomes even smaller than PGA when subject to 150% design earthquake.

In the case of the 4-story torsionally irregular building, the equivalent static method still underestimate the peak floor acceleration even though the structure experiences significant nonlinear behaviour.

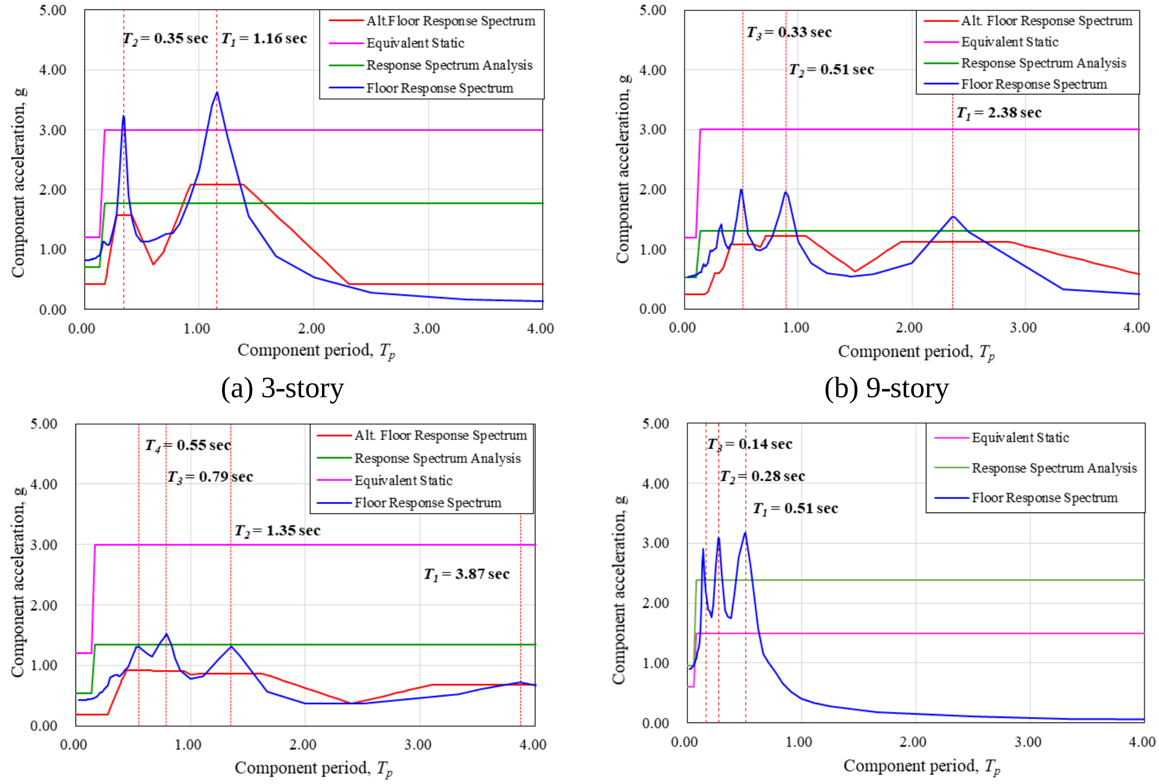
EFFECTS OF SUPPORTING STRUCTURAL CHARACTERISTICS ON PEAK COMPONENT ACCELERATION

In this section, the peak component acceleration (PCA) was evaluated using the equivalent static and dynamic design methods suggested in ASCE 7-16. The PCA was determined by using four different methods; the equivalent static method, the dynamic method using response spectrum analysis, the floor response spectrum method and lastly the alternate floor response spectrum method.

Assuming very light NSEs in evaluating PCA, the results obtained from the floor response spectrum method may be taken as the most accurate value because all the other methods determine PCA by using an approximate component amplification factor (a_p). Also, it should be noted that in the case of determining the floor acceleration directly from the dynamic analysis (in this study, response spectrum analysis method) there is no need to multiply the torsional amplification factor (A_x) to the maximum response since this is double-amplification.

Figure 7 summarizes the results obtained from each analysis method. It can be observed from results that the PCA is significantly affected by the ratio of period of NSEs to the period of the supporting structure. At the roof level of regular steel MRFs [Figure, 7 (a)-(c)], the component amplification factor recommended by ASCE 7 for flexible components (a_p = 2.5) is too conservative especially when the component's period becomes longer.

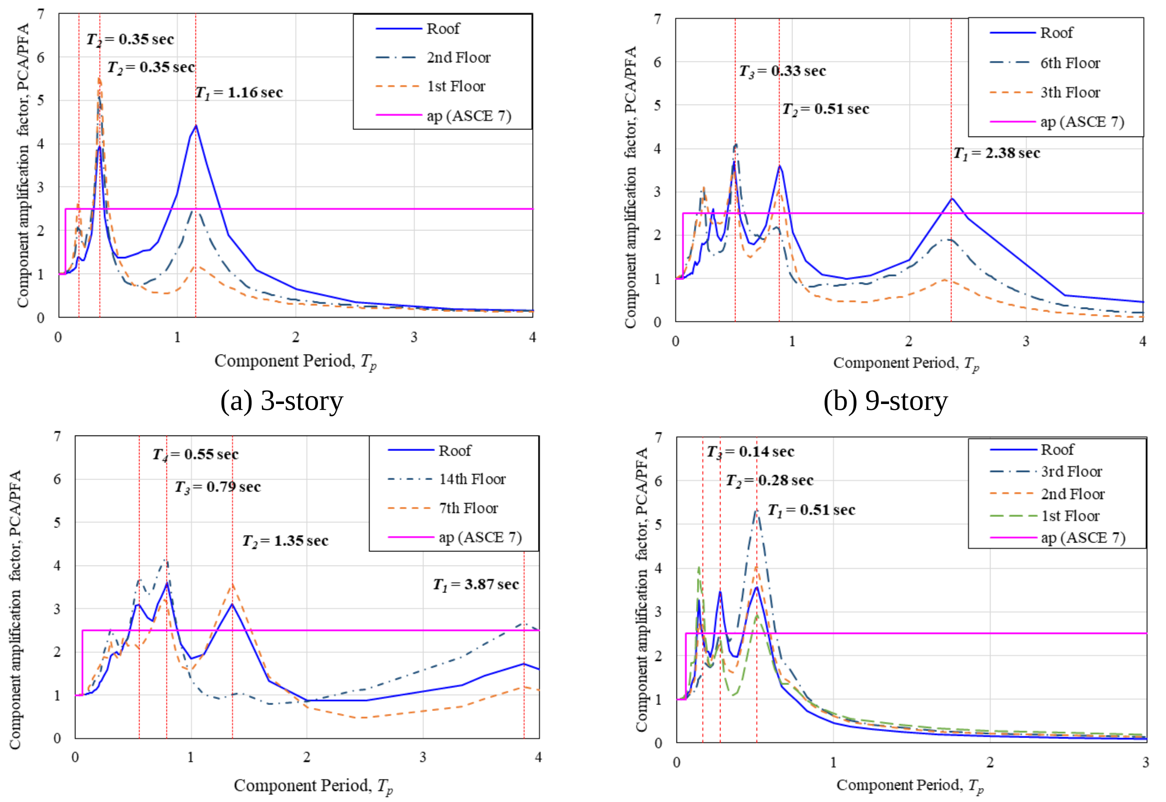
In the case of the torsionally irregular building, unlike the reported characteristics of relatively rigid shear wall systems (Anajafi and Medina, 2018) where the spike in the fundamental period of structure governs the peak floor acceleration, the additional significant second and third spikes are observed in a band due to the closely spaced modes of vibration in the torsion building. Therefore, for structures having



(a) 3-story (b) 9-story

(c) 20-story (d) 4-story (torsionally irregular building)

Figure 7. Comparison of PCA at roof level depending on analysis methods



(a) 3-story (b) 9-story

(c) 20-story (d) 4-story (torsionally irregular building)

Figure 8. Variation of component amplification factor (a_p) depending on floor location and tuning ratio

severe torsional irregularity, the NSEs are expected to subjected to larger acceleration owing to not only the effect of torsional amplification of floor acceleration but also the effect of closely spaced modes of torsional vibrations.

Figure 8 shows the effects of tuning ratio between the NSEs and the structure on the component amplification factor (a_p). The component amplification factor (a_p) suggested by ASCE 7-16 shows generally conservative results when the NSEs have fundamental periods not closely located with the one of the natural periods of supporting structures. But the factor gets unconservative when the fundamental periods of NSEs are in-tuned.

One more important thing that was observed in this study is that the component amplification factor (a_p) varies along the height of structure rather than a constant value as suggested by ASCE 7-16. As already reported in the previous study (Mirada and Taghavi, 2009), the component amplification factor (a_p)varies along the height and the contribution of higher modes is significant in the lower part of structures. Similar tendencies are observed in the torsionally irregular building despite its short fundamental period [refer to Figure 8. (d)]. Note that this tendency is not observed for the symmetric short-period structures.

Lastly, by comparing the results of peak component acceleration obtained by each of the four design methods suggested in ASCE 7-16, it is observed that the results of the equivalent static method and the dynamic method are generally conservative if the fundamental period of NSEs is not closely located with the one of the natural period of supporting structures because the component amplification factor (a_p) is generally less than the code prescribed 2.5 in this range. The alternate floor response spectrum provides similar trend with the floor response spectrum but becomes unconservative at the tuning structural periods.

CONCLUSIONS

In this study, several key influential parameters affecting the acceleration demand on NSEs were investigated in order to evaluate the accuracy of the equivalent static method and the dynamic methods used for seismic design of NSEs. The results can be summarized as follows.

i. Analyses of instrumented buildings with up-to-date database clearly indicated that the measured floor acceleration data does not well corroborate the current equivalent static approach in terms of both profile and magnitude.

ii. Both elementary dynamic theory and numerical results showed that the peak floor acceleration should be based on the structural period-dependent spectral acceleration in order to avoid overdesign in the velocity or displacement-sensitive spectral region.

iii. For multi-story buildings with long periods, due to the higher mode effect, the assumption of linear distribution along the building height is significantly violated.

iv. The torsional amplification of floor acceleration was shown to be very significant when torsional irregularity is large, indicating the necessity of including the torsional effect to the equivalent static approach as well.

v. The results of floor response spectrum analysis demonstrated that the acceleration demands on NSEs are strongly dependent on the primary building modal periods and the types of lateral-load resisting system. The code prescribed component amplification factor (a_p) is generally unconservative at the structural tuning periods where significant component amplification is observed. Especially, for the torsionally irregular building, the additional significant second and third spikes were observed within a narrow band due to the closely spaced modes of vibration. The inclusion of tuning effect seems inevitable for more accurate and rational seismic design of NSEs.

ACKNOWLEDGMENTS

This research was supported by a grant (18AUDP- C146352-01) from Architecture & Urban Development Research Program funded by Ministry of Land, Infrastructure and Transport of Korean government

REFERENCES

ASCE/SEI, 7-16 (2017). Minimum design loads and associated criteria for buildings and other structures. American Society of Civil Engineering, Reston, VA, USA.

Anajafi, H., Medina, R.A (2018). "Effects of supporting building characteristics on non-structural components acceleration demands," *Eleventh U.S. National Conference on Earthquake Engineering*, California, USA.

Chopra, A.K., (2001). *Dynamic of Structures: theory and applications to earthquake engineering*. Prentice-Hall.

Drake R.M, Bachman R.E., (1995). "Interpretation of instrumented building seismic data and implications for building codes," *Proceedings of the 64th Annual Convention, Structural Engineers Association of California Structural Engineers Association of California*; pp. 333-344, California, USA.

Miranda, E., and Taghavi, S., (2009). "A comprehensive study of floor acceleration demands in multi-story buildings," *Proceedings, ATC&SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, December 9-11, San Francisco California.

Gupta, A., Krawinkler, H., (1999). "Seismic demands for performance evaluation of steel moment resisting frame structures," *Report No. 132*, Department of Civil and Environmental Engineering, Stanford University, USA.