Evaluation of Equivalent Static and Dynamic Analysis Method for Seismic Design of Non-Structural Elements

Su-Chan Jun[1], Cheol-Ho Lee[2*], Sung-Yong Kim[3] and Seung-Ho Lee[4]

[1] Dept. of Architecture and Architectural Engineering, Seoul National University, Seoul, Korea. *corrora90@snu.ac.kr*
[2*] Dept. of Architecture and Architectural Engineering, Seoul National University, Seoul, Korea. *ceholee@snu.ac.kr* (corresponding author)
[3]School of Architecture, Changwon National University, Changwon, Korea. *sungyong.kim7@gmail.com*
[4]Dept. of Architecture and Architectural Engineering, Seoul National University, Seoul, Korea. *shl136@snu.ac.kr*

Abstract
Seismic design of nonstructural elements has recently become one of the major topics in earthquake engineering because of increased concerns about additional economic losses from functionality and business interruption. The equivalent static approach is the most practical method in design of nonstructural elements due to its simplicity. Although straightforward in its applications, the equivalent static approach could result in considerable errors due to its negligence of key influential parameters and the assumption of no interaction between the structure and nonstructural elements. In this study, the equivalent static approach and dynamic method for seismic design of nonstructural elements are evaluated in order to show their pros and cons. Based on the elementary structural dynamics and 3-dimensional analytical building models, the effects of structural parameters nonlinearity and torsion on the response of floor accelerations are evaluated. Preliminary evaluation based on the up-to-date database of instrumented buildings is also presented. It is shown that the magnitude and profile of the maximum floor acceleration can significantly be affected by the dynamic characteristics of the supporting structure. Finally, based on the results of dynamic analysis methods proposed in design codes, design recommendations are suggested.

Keywords: Non-structural elements, Equivalent static method, Dynamic analysis, Floor acceleration

1. Introduction

It is now widely recognized that earthquakes may cause extensive damages to non-structural elements and the damages in non-structural elements can outweigh the damages in structural systems. However, there have been less research efforts in the field of seismic design of non-structural elements and the current design equations basically remain the same as the one suggested in the early 1990's.

In this study, to suggests further improvements and design recommendations, the design methods in major building codes were evaluated using elementary structural dynamics, an up-to-date database of instrumented buildings and 3-dimensional analytical models.

2. Design Methods in Building Codes

In determining the seismic design force, the major building codes suggest both the equivalent static method and the dynamic method. In this section, the design methods by ASCE 7-16 will be briefly reviewed.

The seismic design force per ASCE 7-16 can be determined as follows:

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

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 to 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 based. S_{DS} = short period spectral acceleration for 5% damping; a_p = component amplification factor 1.0 or 2.5; and R_p = component modification factor that varies 1.0 to 12.

In ASCE 7-16, in lieu of the equivalent static method, 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 spectral approach. Once the floor acceleration demand is obtained through dynamic analysis, the seismic design force for NSEs can be determined as follows:

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

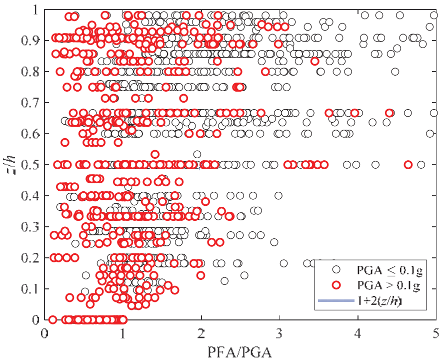
where a_i = maximum acceleration at level i obtained from the dynamic analysis; A_x = torsional amplification factor.

3. Evaluation of Peak Floor Acceleration

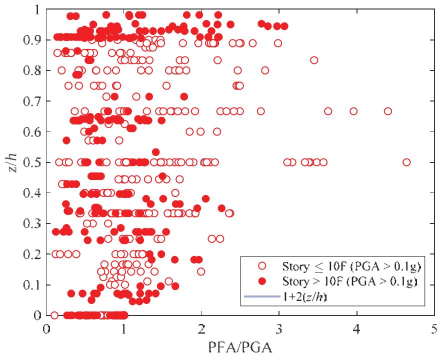
The equivalent static method in ASCE 7-16 was proposed based on the instrumented building floor acceleration data obtained from California earthquakes (Drake and Bachman, 1995). To retain practical simplicity and consistency with the measured data, the assumption of linear distribution of floor acceleration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

Figure 1 shows a preliminary analysis result of up-to-date database of instrumented building using the CESMD (Center for Engineering Strong Motion). The database was built based on 63 earthquakes covering 1978 Santa Barbara (5.1 M_w) to 2018 Thousand Palms (3.8 M_w).

As shown in Figure 1 (a), based on the up-to-data database the distribution of peak floor acceleration does



(a) All data



(b) PGA>0.1g

Figure 1. An up-to-data database of instrumented building

not well corroborate the magnitude and profile of the maximum floor acceleration specified by the current equivalent static approach. Also, as shown in Figure 1 (b), in the case of data obtained from structures having more than 10 stories, distribution of mean peak floor acceleration is almost constant along the height except at the roof. This is mainly due to the higher mode effect of supporting structure which will be discussed in detail in following theoretical and numerical analysis.

From the elementary structural dynamics (Chopra, 2007), the absolute peak floor acceleration and summation of each mode per SRSS can be approximately derived as follows:

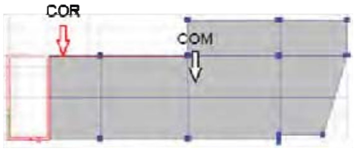
$$f_{n,\max} / m_j = \sqrt{\sum_{n=1}^{N} [\Gamma_n \phi_n \omega_n^2 S_D(T_n, \zeta_n)]^2} \qquad (3)$$

Compared with the equation (1), it 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 period. Also, the distribution of peak floor acceleration should follow structural mode shape (ϕ) rather than linear distribution. Considering that the contribution of each mode is proportional to the squared frequency, the contribution of higher modes can be significant in the case of long period structures which explains the distribution shown in Figure 1 (b).

3.1 Numerical analysis

In this section, 3-dimensional numerical analyses were conducted to evaluate the effect of supporting structure characteristics on the response of floor acceleration. A total of four three-dimensional models were analyzed using ETABS 2016. The first three models were modeled


(a) Building model



(b) Floor plan and the location of center of rigidity and mass

Figure 2. Four-story builidng with torsional irregularity

based on SAC building models (Gupta and Krawinkler, 1999) which are 3-, 9-, 20-story steel perimeter moment resisting frames. The last model is 4-story RC moment frame selected from the telecommunication center facilities in Korea (refer to Figure 2). The 4-story RC model was chosen to evaluate the influence of building torsional irregularity on the floor acceleration. Response spectrum analysis and linear time history analysis were conducted depending upon investigation purposes. The fundamental periods obtained from eigenvalue analysis of numerical models are summarized in Table 1.

From Figure 3, it is observed that the equivalent static method produced results comparable only to the 3-story building. As the fundamental period of building increases, the floor acceleration is highly overestimated by the equivalent static method. In the same context, due to the higher mode effects, the floor accelerations are almost constant over the height except at the top and the bottom floor.

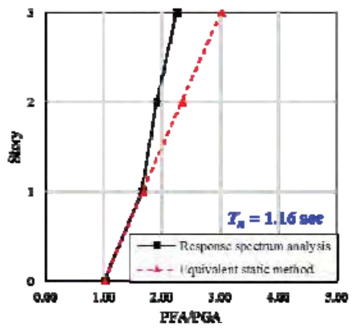
In Figure 4, the result of response spectrum analysis for torsional irregular structure is presented. The result was plotted using the maximum floor acceleration. Due to the torsional amplification caused by supporting structure, the peak floor acceleration is underestimated by the equivalent static method. It should be noted that unlike the design equation in dynamic method [equation (2)], the equivalent static method does not consider the torsional amplification caused by torsional irregularity.

4. Evaluation of Peak Component Acceleration

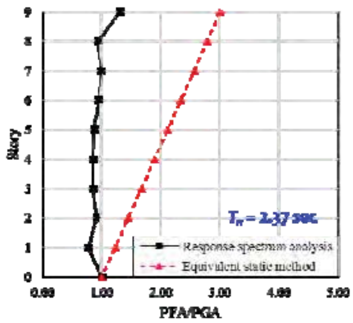
In this section, the peak component acceleration (PCA) was evaluated using the design methods suggested in ASCE 7-16.

Table 1. Summary of fundamental periods of structure models

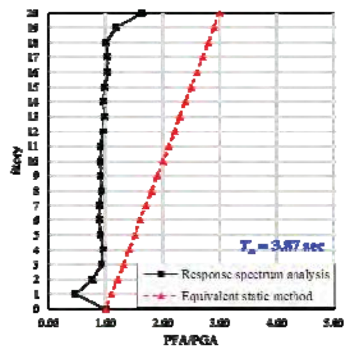
No.	Story	T_n	Structure type
1	3	1.16	Steel moment resisting frame
2	9	2.37	Steel moment resisting frame
3	20	3.87	Steel moment resisting frame
4	4	0.54	Reinforced concrete moment resisting frame (with irregular walls and corner staircase)



(a) 3-story



(b) 9-story



(c) 20-story

Figure 3. Effect of structural period on floor acceleration amplification and profile

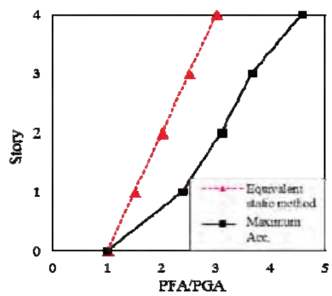


Figure 4. Effect of torsional behavior of supporting structure

The peak component acceleration was determined using the equivalent static method, the dynamic method using response spectrum analysis, the floor response spectrum method and the alternate floor response spectrum method.

Table 2. Ground motion record list

Earthquake	Year	Station	Mag. (Mw)	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

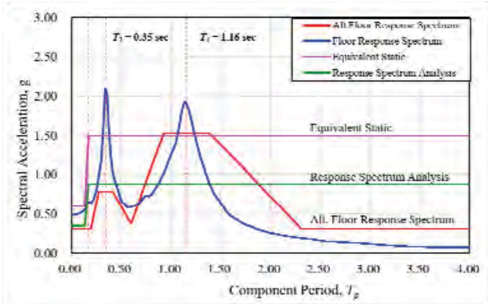
In evaluating PCA, assuming very light NSEs, the results obtained from the floor response spectrum method would be the numerical exact value because all the other methods determine PCA with an approximate component amplification factor (a_p). Also, it should be noted that in the case of determining the floor acceleration directly from the dynamic analysis (in this study, response spectrum analysis method), the torsional amplification (A_x) should not be applied. As shown in Figure 4, if the maximum floor acceleration is obtained from the analysis, the torsional amplification effect is already included. Therefore, applying A_x would result in double amplification of torsional effect.

In obtaining the floor response spectrum, at least seven ground motion should be analyzed. Therefore, a suite of seven far field ground motions were selected from PEER-NGA database and the list of ground motions are summarized in Table 2.

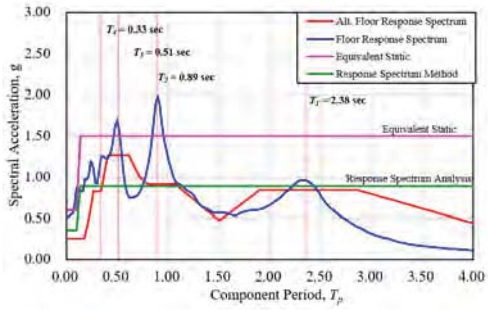
In Figure 5 and 6, the results obtained from each analysis method are summarized. In the case of torsional irregular structure, the results are summarized in each individual axis which has significant torsional effect as in Figure 6 (a) and relatively minor torsional effect as in Figure 6 (b) (refer to Figure 2 (b)).

From the results, it is observed that the floor spectral ordinates are significantly affected by the ratio of period of NSEs to the period of the supporting structure. In the case of SMRF as in Figure 5, the contribution of higher modes is evident in the spectra and the non-structural elements attract the largest acceleration when tuned to the second mode of 3- and 9-story buildings and to the third mode of 20-story building. Also, the amplification of component acceleration near the vicinity of one of the fundamental periods of supporting structure is generally higher for relatively low-rise structure as shown in Figure 5 (a) and it gradually reduces as the building gets higher.

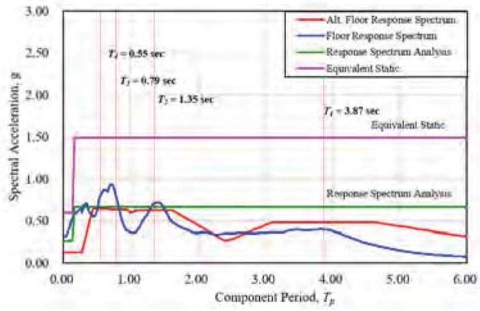
In Figure 6, the floor response spectrum for structure having torsional irregularity is summarized. First, from the Figure 6 (b) which has only minor torsional effect, it is shown that only the spike in the vicinity of fundamental period is observed and it is a general characteristic of the



(a) 3-story

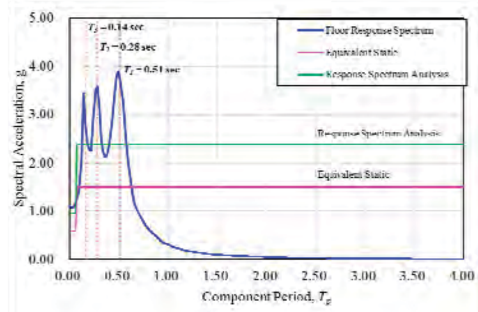


(b) 9-story

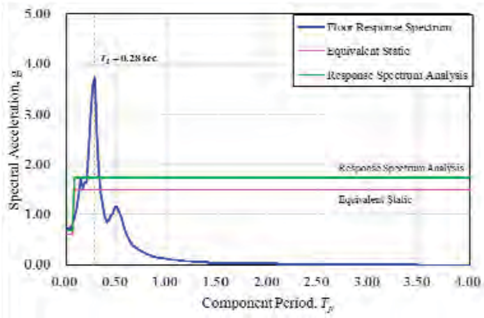


(c) 20-story

Figure 5. Floor response spectrum for SMRF buildings



(a) Floor response spectrum with significant torsional effect



(b) Floor response spectrum with minor torsional effect

Figure 6. Floor response spectrum for a torsional irregular building

floor response spectrum for relatively rigid shear wall systems (Anajafi and Medina, 2018). However, as can be seen in the Figure 6 (a), due to the closely spaced modes of vibration in a torsion building the additional significant second and third spikes are observed which make the range of tuning effects wider.

5. Conclusions

In this study, several key influential parameters affecting the acceleration demand on NSEs were investigated. The results can be summarized as follows.

i) Analyses of instrumented buildings with up-to-date database clearly indicate that the measured floor acceleration data does not well corroborate the current equivalent static approach.

ii) Both elementary dynamic theory and numerical results indicated that the peak floor acceleration should be based on the structural period-dependent spectral acceleration in order to avoid overdesign in 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.

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 inclusion of tuning effect seems inevitable for 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

6. 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.