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

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Abstract. In this study, the equivalent static approach, which is the most popular but largely empirical, for seismic design of non-structural elements (NSEs) is critically evaluated in order to seek the possibility of its improvement. To this end, the advanced analytical methods as well as the equivalent static methods in ASCE 7 and Eurocode 8 are first reviewed. The inaccuracy of the equivalent static approach resulting from the negligence of the fundamental period of supporting structures is clearly illustrated using elementary structural dynamics. Based on numerical dynamic analysis of 3-dimensional building models, it is also shown that the magnitude and distribution of the maximum floor acceleration can significantly be affected by the supporting structural characteristics such as fundamental period, higher modes, nonlinearity, and torsion. Up-to-date database of instrumented buildings does not well corroborate the magnitude and profile of the maximum floor acceleration specified by the current equivalent static approach. The current equivalent static approach needs to be improved such that some of the key influential structural parameters be selectively included within the limit of practicality.

Keywords: Seismic Design, Non-Structural Elements, Equivalent Static Method, Dynamic Method, Floor Acceleration, Torsional Amplification.

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1. INTRODUCTION

Historically, achieving life safety has been the prime performance objective in general building design and construction. Seismic design was not the exception until the end of 20^{th} century; the basic performance objective implied in traditional seismic design codes was fulfilling the goal of life safety through preventing severe damage or collapse of the main structural systems. Although extensive seismic damages on non-structural elements (NSEs) were frequently reported, they were often neglected and due design considerations were not given to NSEs. However, with the advent of more advanced seismic design practice like the performance based seismic design after the 1994 Northridge earthquake (Vision 2000, ASCE 41-13) and the surveys carried out in the aftermath of past major earthquakes, it is now widely recognized that major property and functional losses and serious injuries and death can result from the poor performance of NSEs even though they are not part of the main structural system.

Generally, damage to NSEs can be triggered by ground shaking intensities much lower than those required to initiate structural damage, leaving the building structurally intact but no longer functional as observed in the recent 2016 Gyeongju and 2017 Pohang earthquakes in South Korea (Figure 1). These two just moderate quakes (Mw = 5.4) caused a variety of damages mostly to non-structural elements. One of the main reasons for seismic vulnerability of NSEs in buildings is that NSEs are generally 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 [for example, Singh 1987, Villaverde, 2004; Anajafi and Medina, 2018a]. Since rigorous analysis methods of a combined supporting structure and NSE system involve mathematical complexities almost impractical to handle, major building codes suggest the equivalent static approach in favour of its practical simplicity. However, due to its empirical and simplistic nature, the design equations for NSEs based on the equivalent static approach fails to consider several key influential parameters which could significantly affect the acceleration demand on NSEs, thus leaving some room for further improvements.

In this study, the equivalent static methods suggested by major building codes are critically reviewed through theoretical and numerical analysis, and recommendations for possible improvements are suggested.

2. BRIEF SUMMARY OF ADVANCED ANALYTICAL METHODS

The development of advanced methods for seismic analysis of NSEs has mainly been led by the need from nuclear power plant engineering over the past four decades. The major difficulty in accurately predicting the seismic demand on NSEs comes from the filtered effect of a supporting structure and the interaction effect of a combined structure-nonstructural system. A direct modelling of the combined structure-nonstructural system results in a mathematical model with an excessive number of degrees of freedom and the large differences

(a) 2016 Gyeongju earthquake (Mw = 5.4) (b) 2017 Pohang earthquake (Mw = 5.4)

Figure 1. Non-structural elements failure in the recent earthquakes in South Korea

in the mass, stiffness and damping values, thus making such an analysis expensive, inaccurate, inefficient and impractical. In order to avoid the analysis of a combined system but to consider engineering practicality as well, somewhat compromised methods were proposed, and they are briefly summarized in the below based on the work of Villaverde [2004] in order to better understand the limitations of any approximate methods.

Floor response spectrum method is among the most straightforward one since it assumes that there is no interaction between the main structure and NSEs system. Although there are some problems associated with such decoupled analysis, it has been the most commonly used method in nuclear engineering practice. Floor response spectrum may be obtained by means of time history analysis of a set of several different ground motions or using artificial ground motions that are fitted to envelop a target ground design spectrum. Another alternative is to generate the floor response spectrum directly from a specified ground response spectrum without conducting time history analysis. Although straightforward in its applications, floor response spectrum method could result in considerable errors to conservative side because the response is obtained without due consideration of interaction between the supporting structure and NSEs. It may be noted in Figure 2 that neglecting the interaction is always conservative. The interaction effect becomes significant when the mass of NSEs is not too small relative to the supporting structure and when the natural frequency of the NSEs is close to one of the dominant natural frequencies of the supporting structure [Singh, 1987]. When the structural properties of the two systems are significantly different, the combined system does not possess classical modes of vibration. Large errors may be possible if the nonclassical damping effects are not properly accounted for [Singh, 1987].

Despite the limitations of floor response spectrum method, in recognition of its convenience and popularity in the nuclear power industry, several researchers [for example, Lee and Penzien, 1983] proposed approximate floor response spectrum method by introducing corrections which account for the interaction and nonclassical damping effects. Another alternative method was proposed based on random vibration analysis of the combined system through a modal synthesis [Schroeder and Backman, 1994; Adam and Fotiu, 2000 and others]. This method appears conceptually simple in that it uses individual component's properties in determining the response of NSEs from a combined system. However, its implementation is still far from being practical because of the need to conduct modal analysis and combine these modal responses using a modal combination rule. Since the natural frequencies and mode shapes of the combined system are complex valued, the combination of its modal responses requires an accurate rule to combine.

However, none of the methods mentioned above has become the industry standard even in nuclear power plant engineering. In conventional building engineering, the much simpler and empirical method such as equivalent static method is preferred in favor of practicality as will be discussed in the next section.

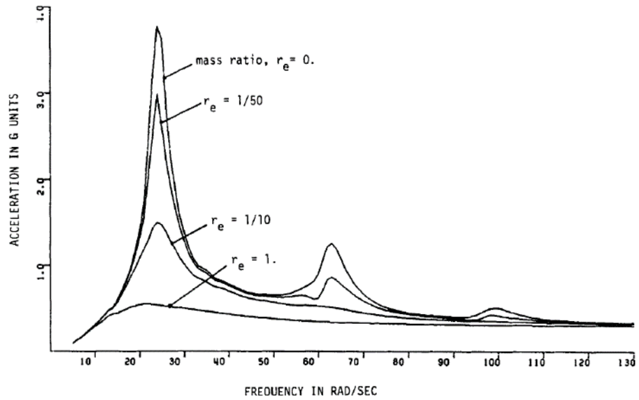


Figure 2. Floor response spectra corresponding different mass ratio [Singh, 1987]

3. DESIGN APPROACH IN BUILDING CODES

In this section, the design equations proposed by ASCE 7-16 and Eurocode 8 will briefly be reviewed. Since the main idea behind the design equations in either code is similar, only the major difference will be highlighted for Eurocode 8.

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 [Kehoe and Freeman, 1998]. 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 threshold 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 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.

The dynamic analysis method proposed in ASCE 7-16 seems a reasonable alternative to the equivalent static method because it can take into account of the effect of a supporting structure, using the maximum floor acceleration (a_i) through dynamic analysis of a supporting structure. The torsional amplification factor (A_x) is to consider additional amplified motion due to structure's torsional behaviour and it can be determined as follows:

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}}\right)^2 \tag{3}$$

where δ_{max} = maximum displacement at level x computed assuming A_x = 1; δ_{avg} = average of the displacements at the extreme point of the structure at level x computed assuming A_x = 1; A_x shall not be less than 1 and is no required to exceed 3.

The A_x factor, proposed on an empirical basis, was first introduced by the SEAOC Seismology Committee [SEAOC, 1988]. In general, structures having plan asymmetry would show the maximum amplified responses at the farthest location from the center of rigidity due to torsional behaviour. Therefore, it seems reasonable to understand the intention of the torsional amplification, A_x, as for obtaining the amplified motion at the farthest location (maximum response) by multiplying it to the floor acceleration at the center of rigidity (COR). If the maximum value of floor acceleration is directly taken from the dynamic analysis results (this value is usually available in commercial analysis software nowadays), there no need to multiply the A_x factor to the maximum response since this is double-amplification. However, because ASCE 7-16 defines a_i as "the maximum floor acceleration", there exists some confusion among practicing engineers in properly applying the factor A_x. This issue will be further discussed in the next section.

In Eurocode 8, very similar to ASCE 7-16, two methods are proposed, dynamic method (essentially floor response spectrum method) and equivalent static method. Except for the NSEs which are of critical importance, simple equivalent static method is allowed. The major difference with that of ASCE 7-16 is that Eurocode 8 takes into account of the component amplification as a function of the fundamental period ratio between the component and the building. However, its application is limited to the case where the dynamic characteristics of both the main and non-structural systems are known.

4. EVALUATION OF EQUIVALENT STATIC APPROACH

The main motivation behind the equivalent static method in ASCE 7-16 seems to retain practical simplicity and consistency with the instrumented building floor acceleration data obtained from California earthquakes [Kehoe and Freeman, 1998].

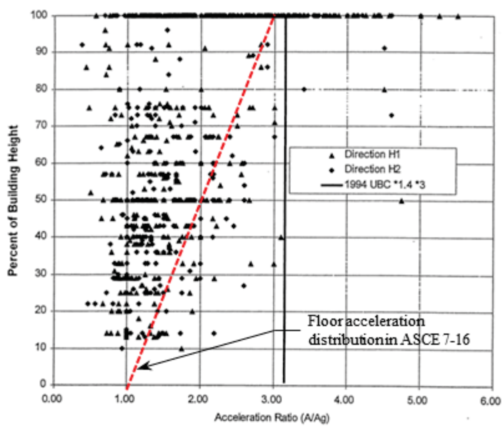


Figure 3. Instrumented building acceleration data [Drake and Bachman, 1995]

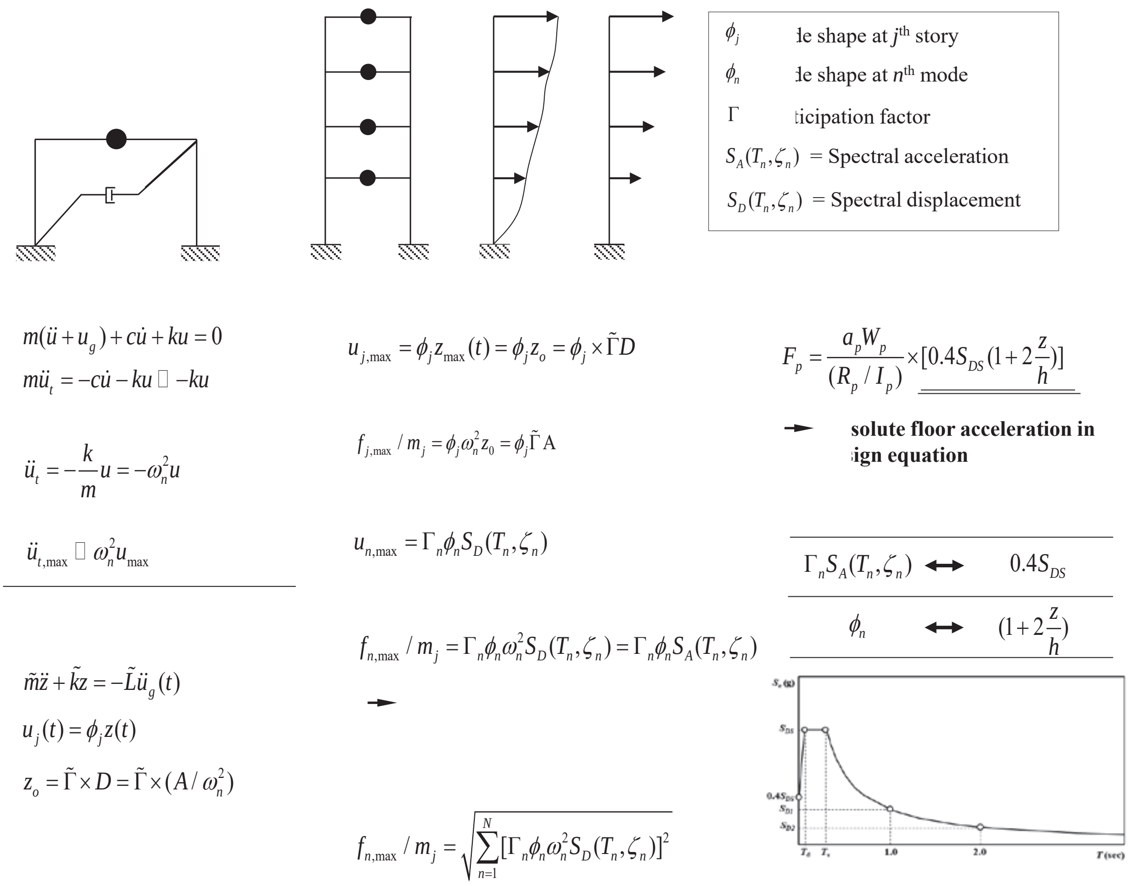


Figure 4. Summary of floor acceleration prediction based on elementary structural dynamics

Figure 3 shows the data analysed by Drake and Bachman [1995] to derive the assumption of linear distribution of floor acceleration and the maximum three times amplification of peak ground acceleration

(PGA). From the figure, it is observed that, although the maximum amplification factor of 3.0 seems reasonable ignoring the data at the roof level, the majority of data does not follow linear distribution along the building height. Also, as indicated by Villaverde [2004], the equivalent static approach in ASCE 7-16 neglects the effect of inelastic behaviour of the supporting structure in the erroneous belief that it is a conservative consideration. The validity of the current approach should be further evaluated based on up-to-date and augmented database of instrumented buildings.

In this section, the equivalent static method in ASCE 7-16 will be evaluated based on elementary structural dynamics and numerical case study of realistic 3-dimensional buildings.

4.1 FLOOR ACCELERATION PREDICTION FROM ELEMENTARY STRUCTURAL DYNAMICS

Before evaluating the equivalent static method with numerical case studies and measured floor acceleration data, the theoretical equation for predicting absolute floor acceleration is approximately derived and compared with the design equation in order to show some basic flaws existing in the current design equation. The derivation is summarized in Figure 4 to save space. Just elementary structural dynamics is sufficient for the derivation [Chopra, 2007].

The comparison in the right-middle of Figure 4 clearly shows that the maximum floor acceleration in principle should be based on the structural period-dependent spectral acceleration S_a, not S_{DS} which is the short-period constant spectral acceleration independent of structural period. It is well known from Newmark's spectrum theory that when structures belong to the velocity-sensitive region, S_a is inversely proportional to structural period (T_n). When T_n becomes longer, the floor acceleration would be much smaller than based on the F_p formula in ASCE 7-16 (Equation 1).

The equation $f_{n,max}/m_j = \Gamma_n \phi_n \omega_n^2 S_D(T_n,\xi_n)$ indicates that the floor acceleration is proportional to the squared frequency ω_n^2. Thus the contribution of higher modes can be significant even if the modal participation factor Γ_n becomes smaller for higher modes. There is a high degree of possibility that the linearly-varying floor acceleration profile assumed in the ASCE 7-16 may significantly be violated due to higher mode effect as will be discussed in the next section.

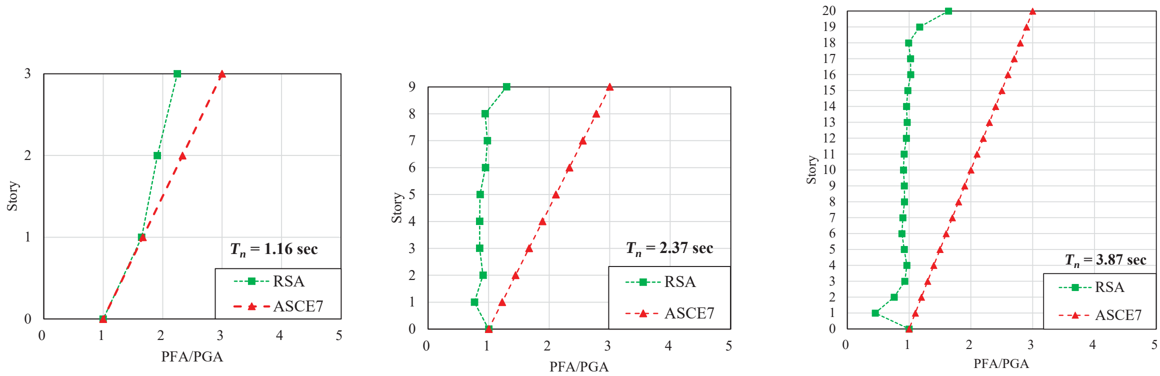
4.2 EFFECTS OF SUPPORTING STRUCTURAL CHARACTERISTICS

This section evaluates the effects of several supporting structural characteristics on the response of floor acceleration. A total of five three-dimensional models were analysed using ETABS 2016. In the first set of analysis, three SAC building models designed according to the UBC 1994 for the Los Angeles area [Gupta and Krawinkler, 1999] were adopted. The perimeter moment frames provided the seismic resistance needed for these regular buildings. These building models were used to investigate the effect of structural period, higher modes, and structural nonlinearity. In the second set of analysis, two 5- and 9-story buildings were selected from the telecommunication center facilities in Korea in order to investigate the effect of torsional behaviour.

The fundamental periods obtained from eigenvalue analysis of the three SAC buildings were 1.16 sec (3-story), 2.37 sec (9-story) and 3.87 sec (20-story), respectively. Response spectrum analysis with 100/30% bi-directional loading or linear/nonlinear time history analysis were conducted depending upon investigation purposes. The UBC 1994 design spectrum was used as the input spectrum for response spectrum analysis. Two sets of earthquake records (Sylmar, 1994 and Imperial Valley, 1940) were selected for time history analysis. Imperial Valley record (1940) is usually classified as the "standard" earthquake input with sufficient duration and moderate epicentral distance while Sylmar record (1994) is close to pulse-type excitation with short epicentral distance. The amplitude of input motions was scaled to be compatible with the design spectrum at the fundamental period of each building model. Plastic hinges were assigned at beam ends as a point hinge with a bilinear moment-rotation relationship with 3% strain-hardening. Modelling parameters for the plastic hinges were determined following ASCE 41-13.

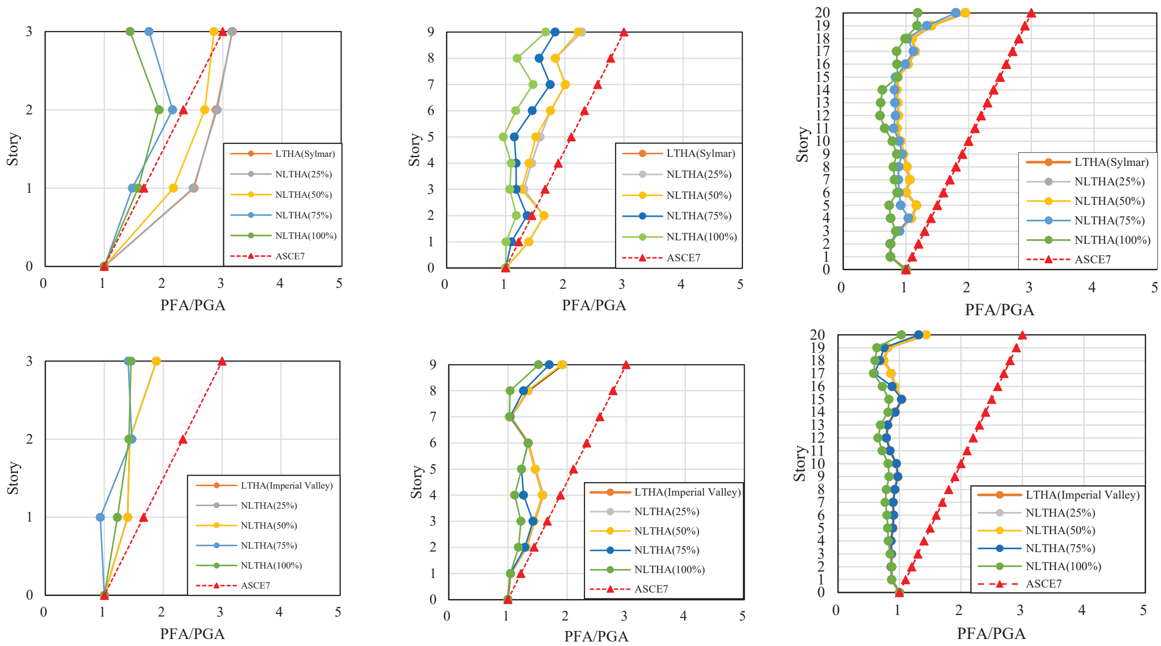
4.2.1 Effect of Structural Period

Figure 5 shows the results from response spectrum analysis. The ordinate was normalized to "the effective PGA" (= $0.4S_{DS}$). The equivalent static equation (Equation 1) produced results comparable only for the 3-story building.



(a) 3-Story **(b) 9-Story** **(c) 20-Story**

Figure 5. Effects of structural period on floor acceleration amplification and profile



(a) 3-Story **(b) 9-Story** **(c) 20-Story**

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

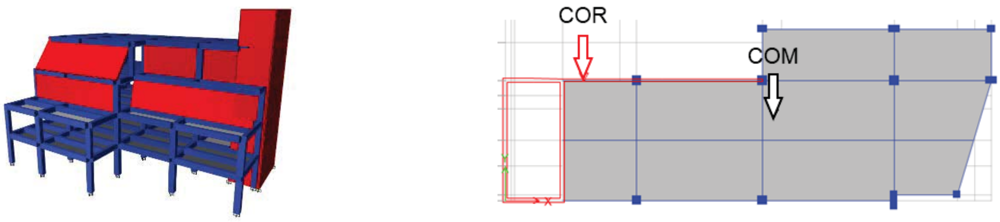
As the fundamental period of buildings is increased, the floor acceleration is highly overestimated by the equivalent static method. This trend was already expected in section 4.1 when it was emphasized that the floor acceleration should be predicted considering the structural period (T_n).

4.2.2 Higher Mode Effect

In the same context, as shown in the Figure 5 (b) and (c), due to the higher mode effects, the floor accelerations are almost constant over the height except at the top and the bottom stories. This is because higher mode effects overshadow the first mode accelerations; the floor acceleration is proportional to the square of modal frequencies (see the derivation in Figure 4). Similar trends were also observed in the study of two-dimensional moment frame buildings by Kehoe and Freeman [1988].

4.2.3 Structural Nonlinearity

Figure 6 summarizes the results of incremental nonlinear time history analysis. The intensity of ground motion was arbitrarily increased to induce different degrees of nonlinearity in the structure. The ground intensity up to 50% did not cause significant nonlinear behaviour in the considered buildings. As the intensity of ground motion is increased, the floor acceleration is also reduced as expected. However, it should be noted that, non-structural elements might be subjected to increased acceleration if the one of the dominant natural frequencies of the supporting structure shifts into that of NSEs as a result of yielding.



(a) Building model **(b) Floor plan and the location of center of rigidity and mass**

Figure 7. Five-story building with significant torsional irregularity

(a) Building model **(b) Floor plan and the location of center of rigidity and mass**

Figure 8. Nine-story building with minor torsional irregularity

4.2.4 Effect of Torsion

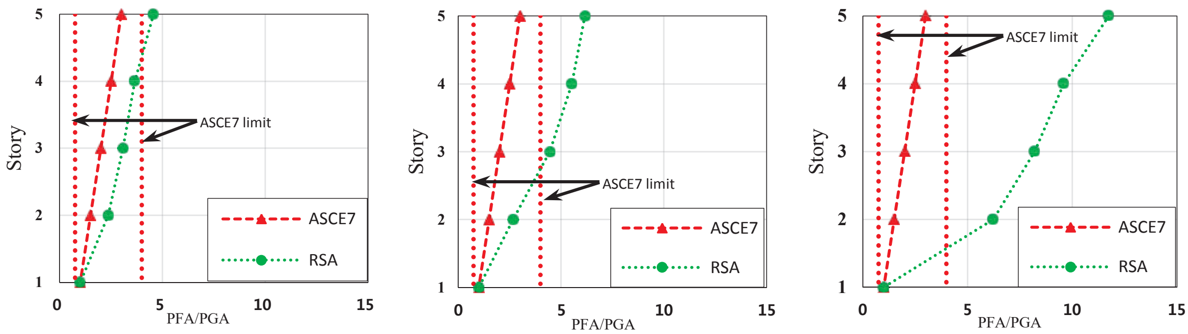
This section evaluates the influence of building torsional irregularity on the floor acceleration. Two 5- and 9-story buildings with asymmetry in plan were chosen from the telecommunication buildings as illustrated in Figure 7(a) and Figure 8(a). The fundamental period obtained from eigenvalue analysis was 0.54 sec for the 5-story building and 0.68 sec for the 9-story building, respectively. The center of mass (COM) and the center of rigidity (COR) are shown on the floor plans of the two buildings in Figure 7(b) and Figure 8(b). The distance between COM and COR are 40.5% and 6.8% of the building dimension perpendicular to the direction for which response analysis is conducted. The 5-story RC building with corner staircase wall has very severe torsional irregularity than the 9-story steel building with centered corewall.

Response spectrum analysis (bi-directional analysis with 100/30 rule) was performed to compute the floor acceleration taking the torsional mode of vibration into account. The torsional amplification factors according to ASCE 7-16 (Equation 3) for the two buildings are summarized in Table 1. As expected, the 5-story building has very high torsional amplification factors over 2.5.

Figure 9 and 10 shows a comparison of the maximum floor acceleration distributions plotted based on: (a) $a_{i,max}$, the maximum value directly taken from the ETABS output (the value at the point farthest from the COR, that is, the numerical exact value); (b) $a_{i,COR} \cdot A_x$, the maximum floor acceleration at the COR ($=a_{i,COR}$) amplified by A_x (correct application of Equation 2); (c) $a_{i,max} \cdot A_x$ (erroneous interpretation of Equation 2, or double multiplication). First, it is observed from these figures that the erroneous application of Equation 2, or the case (c), can lead to huge overestimation of the floor acceleration demand when torsional irregularity is very severe and should be avoided (see Figure 9c). The correct application of Equation 2 shows somewhat conservative results compared with the numerical exact value (compare Figure 9a and 10b). However, these trends tend to diminish in the 10-story building of minor torsional irregularity building (see Figure 10). It is noted that the upper bound of ASCE 7-16 limits the extent of overestimation caused by A_x in actual design especially when torsional effect is large. The equivalent static approach gives reasonable results, comparable to dynamic results, for the 9-story "regular" building with relatively short period of 0.67 sec. If possible, the value at the point farthest from the COR (or the numerical exact value), usually directly available from the software output, should be used irrespective of the degree of torsional irregularity. However, there remain some engineering judgements on which floor acceleration should be used for NCEs on the same floor $a_{i,max}$ vs. $a_{i,COR}$. For example, A_x shall be included in Equation 2 only when a_i is calculated at the COR and the NSE is located very far from the COR. When the NSE is around the COR, the use of $a_{i,COR}$ may be justified. Or to be conservative, $a_{i,max}$ may be used for all NSEs on a floor.

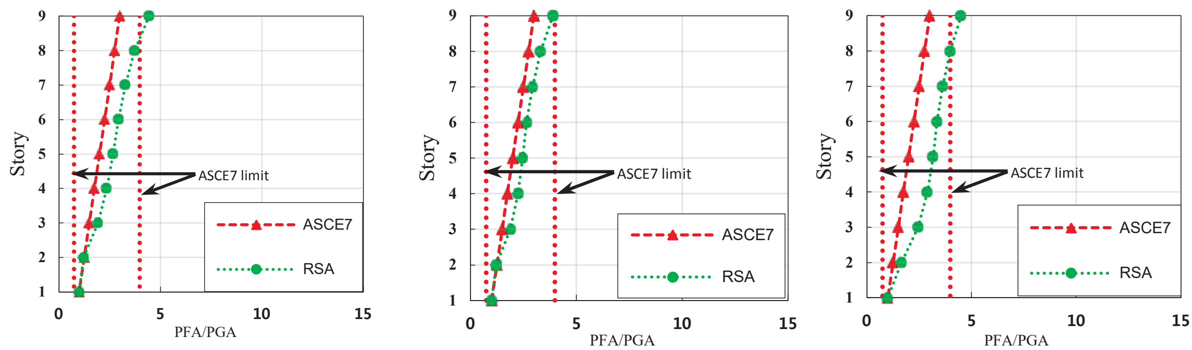
Table 1. Torsional amplification factor (A_x) calculated at each story

	1	2	3	4	5	6	7	8	9
5-story building	1.00	2.63	2.66	2.62	2.57	-	-	-	-
9-story building	1.00	1.35	1.27	1.22	1.18	1.14	1.10	1.07	1.01



(a) $a_{i,max}$/PGA **(b) $a_{i,COR}\cdot A_x$/PGA** **(c) $a_{i,max}\cdot A_x$/PGA**

Figure 9. Comparison of floor acceleration_ 5-story building



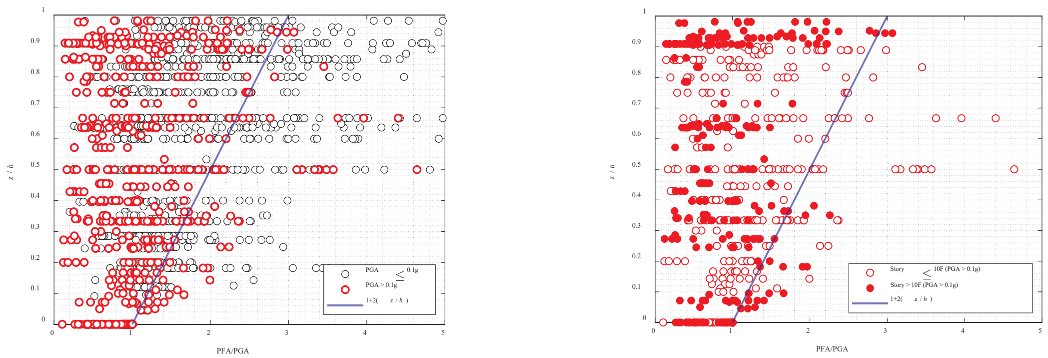
(a) $a_{i,max}$/PGA **(b) $a_{i,COR}\cdot A_x$/PGA** **(c) $a_{i,max}\cdot A_x$/PGA**

Figure 10. Comparison of floor acceleration_ 9-story building

The possibility of increasing the accuracy of the torsional amplification factor, which seems conservative, needs to be investigated. Finally, according to ASCE 7-16, every dynamic analysis should consider the effect of torsional amplification whereas the equivalent static approach requires nothing about torsional effect, thus yielding unconservative results compared to dynamic ones when torsional irregularity is severe as was shown in this case study. This seems technically unfair.

4.2.5 Preliminary Analysis of Instrumented Building Database

In many cases, simplified numerical building models cannot capture the response characteristics of real buildings related to: flexibility of floor diaphragm, torsional response (accidental/inherent), real distribution of damage, contribution of infill and partitions, soil-foundation-structure interactions, damping effect, interaction between NCEs and supporting structure [Anajafi and Medina, 2018b]. Analysis of up-to-date database of instrumented buildings using the CESMD (Center for Engineering Strong Motion: www.strongmotioncenter.org) is under way. Just preliminary analysis results for the peak floor acceleration profile are presented in the below because of space limitations. The up-to-date database of instrumented buildings (about 3, 000 records) was built based on 63 earthquakes including 1978 Santa Barbara (5.1 M_w), 1992 Landers (7.3 M_w), 1994 Northridge (6.4 M_w) and 2018 Thousand Palms (3.8 M_w) and 66 buildings ranging from a single- to a 54-story. Figure 11 compares the measured peak floor acceleration (PFA) normalized to the PGA with the linear acceleration profile of ASCE 7-16. First of all, Figure 11 (a) based on the up-to-date database shows the peak floor distribution very different from that of Figure 3 which was as the empirical basis of the equivalent static design equation of ASCE 7-16. It is also observed from Figure 11 (a) that for higher ground motion intensity (PGA > 0.1g), PFA response is reduced, implying the effect of supporting structure nonlinearity. In Figure 11 (b), it is noted that the PFA in the buildings more than 10-story high is smaller than those in the lower-story buildings, implying reduced spectral acceleration effect of longer period building noted in the section 4.1. This preliminary analysis results clearly indicate that the up-to-date database of instrumented buildings does not well corroborate the magnitude and profile of the maximum



(a) All (b) PGA>0.1g

Figure 11. Peak floor accelerations as affected by PGA magnitude and building period

floor acceleration specified by the current equivalent static approach. More detailed analysis considering specific supporting structural characteristics is on-going including the analysis of data points deviating far from equivalent lateral force profile.

5. SUMMARY AND CONCLUSIONS

In this paper, several key influential parameters affecting the maximum floor acceleration were investigated through theoretical and numerical analysis in order to evaluate the accuracy of the equivalent static approach for seismic design of NSEs. The results can be summarized as follows.

Both theoretical analysis and the instrumented building data showed that the maximum floor acceleration should be based on the structural period-dependent spectral acceleration (S_a), not S_{DS} which is independent of structural period. The floor acceleration demand calculated based on the equivalent static approach is expected to be too conservative for highrise (long period) buildings. Further, due to the higher mode effect, the assumption of linear distribution along the building height could be significantly violated.

Supporting structure's nonlinear behaviour was shown to have the effect of reducing the peak floor acceleration. However, final effect on NCEs would vary depending upon the tuning ratio after yielding.

The torsional amplification observed in the very irregular building with 40% torsion-eccentricity was as high as 2.7, indicating the necessity of considering torsional effect in design. According to ASCE 7-16, however, every dynamic analysis should consider the effect of torsional amplification whereas the equivalent static approach requires nothing about torsional effect, thus yielding unconservative results when torsional irregularity is severe as was shown in this study. This seems technically unfair. The inclusion of suitable torsional amplification to the equivalent static approach appears necessary.

The possibility of increasing the accuracy of the torsional amplification factor, which seems conservative, needs to be investigated. If possible, the value at the point farthest from the COR (or the numerical exact value), usually directly available from the software output, should be used irrespective of the degree of torsional irregularity. However, there remain some engineering judgements on which floor acceleration should be used in designing NCEs on the same floor as discussed in this paper.

Preliminary analysis 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 the magnitude and profile of the maximum floor acceleration. The current equivalent static approach needs to be improved such that some of the key influential structural parameters be selectively included in design within the limit of practicality.

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