PEAK FLOOR ACCELERATIONS IN MULTISTORY BUILDINGS SUBJECTED TO EARTHQUAKES

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ABSTRACT:

In this paper, an approximate procedure to estimate peak floor accelerations in linearly elastic multistory building structures is investigated. The approximate procedure, applicable when either linear static analysis or modal response spectrum analysis is performed, consists of dividing the effective seismic forces by the corresponding floor masses. Estimates of peak floor accelerations given by the approximate procedure were compared with results obtained by time history analysis considering several 2D models of building structures having different structural and dynamic characteristics. Seismic excitations were modeled as nonstationary random processes having far-field and near-fault characteristics, and the response of the building models was obtained by Monte Carlo simulation. It is concluded that: (a) when effective seismic forces are calculated by linear static analysis, the approximate procedure gives consistently inaccurate results; and (b) when effective seismic forces are calculated by modal response spectrum analysis, the approximate procedure may provide the basis for a convenient alternative to the "default" linear height-wise variation of peak floor accelerations indicated in current seismic design specifications.

KEYWORDS: floor accelerations, nonstructural components, building structures, seismic response, Monte-Carlo simulation

1. INTRODUCTION

In the ASCE/SEI 7-05 standard *Minimum Design Loads for Buildings and Other Structures* (ASCE 2006), from now on referred to as ASCE 7-05, and in the 2003 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC 2004), from now on referred to as FEMA 450, the seismic design force on a given nonstructural component is equal to the acceleration response of the component times its mass (this design force is modified further by an importance factor and by a component force reduction factor, but this modification is not related to the topic of this paper). In turn, the acceleration response of the component is equal to the acceleration response at the floor where the component is located times an amplification factor that accounts for possible dynamic response amplification due to the flexibility of the component. Since the value of the amplification factor is equal to either 1.0 in the case of "rigid" components or 2.5 in the case of "flexible" components, the magnitude of the seismic design force on nonstructural components depends then mainly on the magnitude of the floor acceleration response, a quantity that will subsequently be referred to as "peak floor acceleration" or simply as "PFA". In both the ASCE 7-05 and the FEMA 450 specifications, PFAs are in principle assumed to vary linearly from a value equal to the peak ground acceleration (PGA) at the ground level to a value equal to three times the PGA at the roof level. This linear heightwise variation of PFAs is stipulated for any building type having any number of floors, and regardless of whether the building behaves linearly or nonlinearly. Alternatively, in both the ASCE 7-05 and the FEMA 450 specifications, PFAs might also be determined by modal analysis assuming that the value of the response modification factor R is equal to unity (i.e., assuming linear response).

In this paper, an alternative procedure to estimate PFAs in linearly elastic multistory building structures is presented, and its accuracy is examined by comparing the corresponding results with "exact" values obtained through time history analysis. The study presented in this paper is limited to 2D frame models, which in turn are representative of building structures not significantly affected by torsion effects.

2. APPROXIMATE PROCEDURE TO ESTIMATE PEAK FLOOR ACCELERATIONS

In order to derive an approximate expression to estimate PFAs, it is convenient to consider that multistory building structures can be adequately represented by 2D lumped-mass models in which the DOFs are the lateral displacements of each floor level. When the model is subjected to a seismic excitation, the equation of motion corresponding to the i-th floor level is given by (Chopra 2007):

$$m_i \; a_i(t) = V_i(t) - V_{i+1}(t) + D_i(t) \tag{2.1}$$

where m_i is the seismic mass of the i-th floor level, a_i is the absolute acceleration of the i-th floor level, V_i is the i-th story shear, and D_i is a force, acting at the i-th floor level, associated with damping effects. If the latter is neglected, the absolute acceleration response at the i-th floor level is then given by:

$$a_i(t) = \frac{V_i(t) - V_{i+1}(t)}{m_i} \tag{2.2}$$

and the corresponding peak value is then given by:

$$\mathrm{PFA}_i = \frac{\max\left|V_i(t) - V_{i+1}(t)\right|}{m_i} \tag{2.3}$$

If seismic effects are determined by: (a) linear static analysis, denominated "Equivalent Lateral Force Procedure" (from now on referred to as ELFP) in the ASCE/FEMA specifications; or (b) response spectrum analysis, denominated "Modal Response Spectrum Analysis" in ASCE 7-05 and "Response Spectrum Procedure" in FEMA 450 (from now on referred to simply as "Modal Analysis"), the term "max $|V_i(t) - V_{i+1}(t)|$" is nothing but the effective seismic force acting at the i-th floor level F_i. Hence, when building structures are analyzed either by the ELFP or by Modal Analysis, an estimate of the peak floor acceleration at the i-th floor level may then be obtained by:

$$\mathrm{PFA}_i = \frac{F_i}{m_i} \tag{2.4}$$

3. EVALUATION OF THE APPROXIMATE PROCEDURE

3.1. Description of the building structures

A total of 7 building structures were considered. The first three structures are the 3-story, 9-story and 20-story steel building models developed for the SAC Phase II Steel Project considering the seismic hazard corresponding to Los Angeles (U.S.A.). A detailed description of these building structures can be found in Ohtori et al. (2004). Aditionally, two RC wall structures having 10 and 20 stories were also considered. The characteristics of these structures, described in detail in Magna (2006), are typical of traditional RC wall structures built in Chile. Finally, two RC dual wall-frame structures having 10 and 20 stories were also considered. The characteristics of these structures, described in detail in Toro (2006), are representative of more recent (and current) RC structures built in Chile.

3.2. Description of modeling assumptions

In the case of the steel structures, only the north-south perimeter frames were considered. They are steel moment-resisting frames, and are the sole lateral-load resisting system of the corresponding building models. In the case of the RC structures, all the lateral-load resisting planes along one of the principal directions were considered. In all cases, each structure was modeled as a 2D frame structure made up of 6-DOFs frame elements. Rigid offsets were considered. In the case of the RC structures, effective (rather than gross cross-section) member stiffness was considered based on recommendations found in the literature (Paulay and Priestley 1992), and diaphragm constraints were incorporated into the models at each floor level. For the purposes of performing time history analysis, only the lateral floor displacements were considered as DOFs, and the corresponding condensed stiffness matrices were obtained through static condensation techniques. Mass matrices were then set equal to diagonal matrices whose elements are equal to the seismic masses associated to each floor level, and damping matrices were obtained following the Clough-Penzien procedure (Clough and Penzien 2003) considering that modal damping is equal to 5% for all modes.

3.3. Description of the seismic excitations

Seismic excitations were modeled as nonstationary random processes. The first process is representative of far-field seismic excitations, and was modeled considering a modified Kanai-Tajimi stationary power spectral density function and a modulating time function. The values of the parameters of the modified Kanai-Tajimi function were defined in such a way that the frequency content of the resulting excitation process is similar to typical frequency contents of actual seismic accelerations recorded on firm soil conditions. For illustration purposes, a sample realization and the mean pseudo-acceleration response spectrum are shown in Fig. 1.

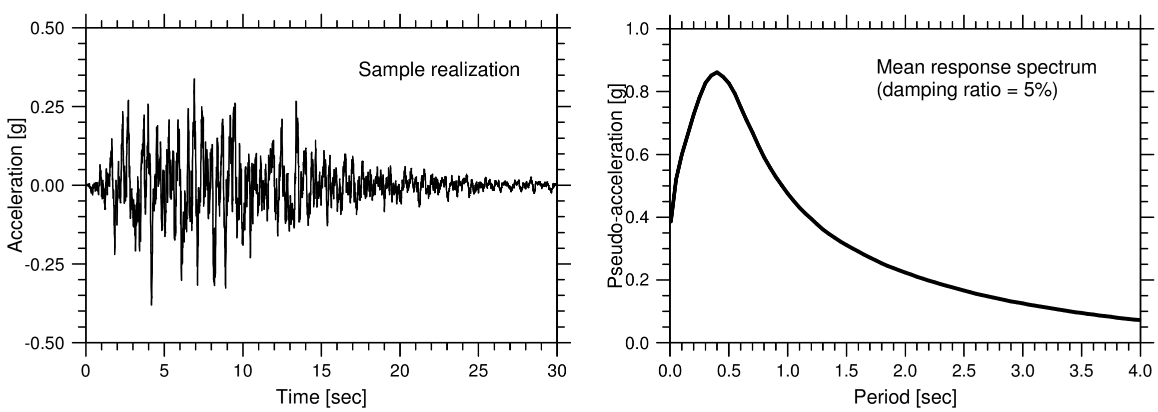

Figure 1 Far-field seismic excitation

The second process is representative of near-fault seismic excitations and was modeled following the procedure proposed by Mavroeidis and Papageorgiou (2003), according to which the incoherent (high frequency) and coherent (low frequency) components are combined in both the frequency and time domains. The characteristics of the incoherent (high frequency) component were assumed equal to those of the first excitation process described before. The coherent (low frequency) component was generated considering the model proposed by Mavroeidis and Papageorgiou (2003). The value of the amplitude of the velocity pulse was set equal to 100 cm/sec and the value of the pulse duration T_P was set equal to 2.0 sec. The time instant t_0 of the peak of the pulse's envelope was set equal to the time instant of the peak of the modulating function of the incoherent (high frequency) component (= 6 sec). The values of the remaining parameters were defined in such a way that the shape of the resulting ground velocity pulse is essentially equal to the shape of a half-sine curve. For illustration purposes, a sample realization and the mean pseudo-acceleration response spectrum are shown in Fig. 2. The relatively high values of the ordinates of the response spectrum at low frequencies are clearly associated with the near-fault characteristics of the excitation, although the low-frequency portion of the spectrum reaches its maximum value at $T = 1.1$ sec rather than at $T = T_P$ (= 2.0 sec in this case).

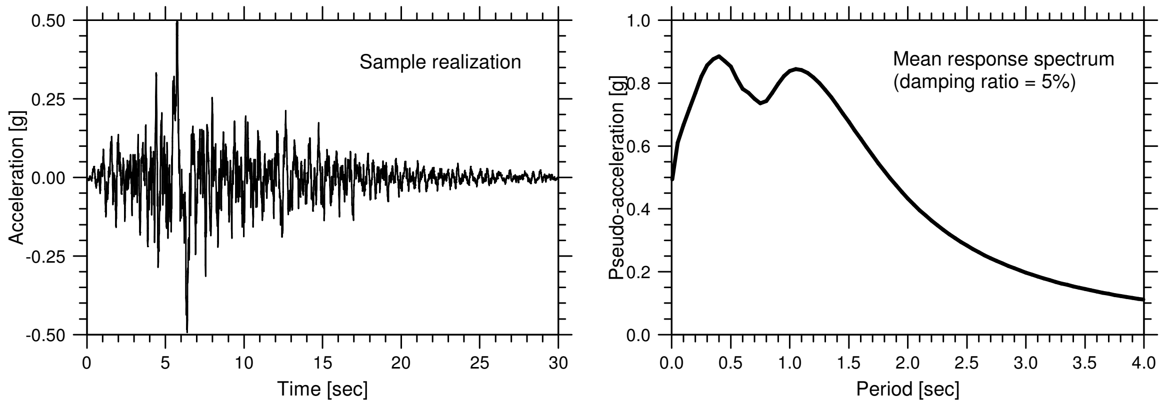

Figure 2 Near-fault seismic excitation

3.4. Description of the evaluation procedure

The response of the building models to the seismic excitations was obtained by Monte-Carlo simulations. Realizations of the far-field excitation process were generated in accordance with standard simulation techniques (Soong and Grigoriu 1993). Realizations of the near-fault excitation process were generated by combining, in both the frequency and time domains, realizations of the incoherent (high frequency) and coherent (low frequency) components (Mavroeidis and Papageorgiou 2003). Realizations of the incoherent (high frequency) component were again generated in accordance with standard simulation techniques, whereas only a single realization of the coherent (low frequency) component was considered (this realization of the coherent component was combined with each realization of the incoherent component). A total of 1000 realizations were generated for each excitation, and the duration of the realizations was set equal to 30 sec in all cases. The response of the building models to each realization of the excitations was obtained numerically by time history analysis (direct integration). Response quantities of interest were in all cases assumed equal to the mean response value (i.e., the mean of the response values obtained considering all of the realizations of each excitation processes). Estimates of PFAs were calculated using Eqn. 2.4 and considering: (a) effective seismic forces F_i calculated in accordance with AISC/FEMA's ELFP; and (b) effective seismic forces F_i obtained by Modal Analysis. In the latter case, the number of modes considered was gradually incremented up to the point where, at any floor level, the response obtained considering j modes was less than 5% different from the response obtained considering j-1 modes. Both the SRSS and CQC modal combination rules were considered. In both the ELFP and Modal Analysis, pseudo-acceleration spectral values were obtained from the mean pseudo-acceleration response spectrum of the excitation (damping ratio = 5%), and the values of the response modification factor R and the occupancy importance factor I were set equal to unity. Estimates of PFAs were then compared with PFAs obtained directly from the Monte-Carlo simulations (i.e., the "exact" values).

3.5. Analysis of results

Results are summarized in Figs. 3, 4 and 5, where PFAs are normalized by the PGA. In all cases, the left-side plots show results corresponding to the far-field seismic excitation, and the right-side plots show results corresponding to the near-fault seismic excitation. In addition to the estimates of PFAs calculated as described in the former section, estimates obtained by a single-mode modal analysis are also shown. For comparison purposes, the "default" ASCE/FEMA linear height-wise variation of PFAs is shown as well.

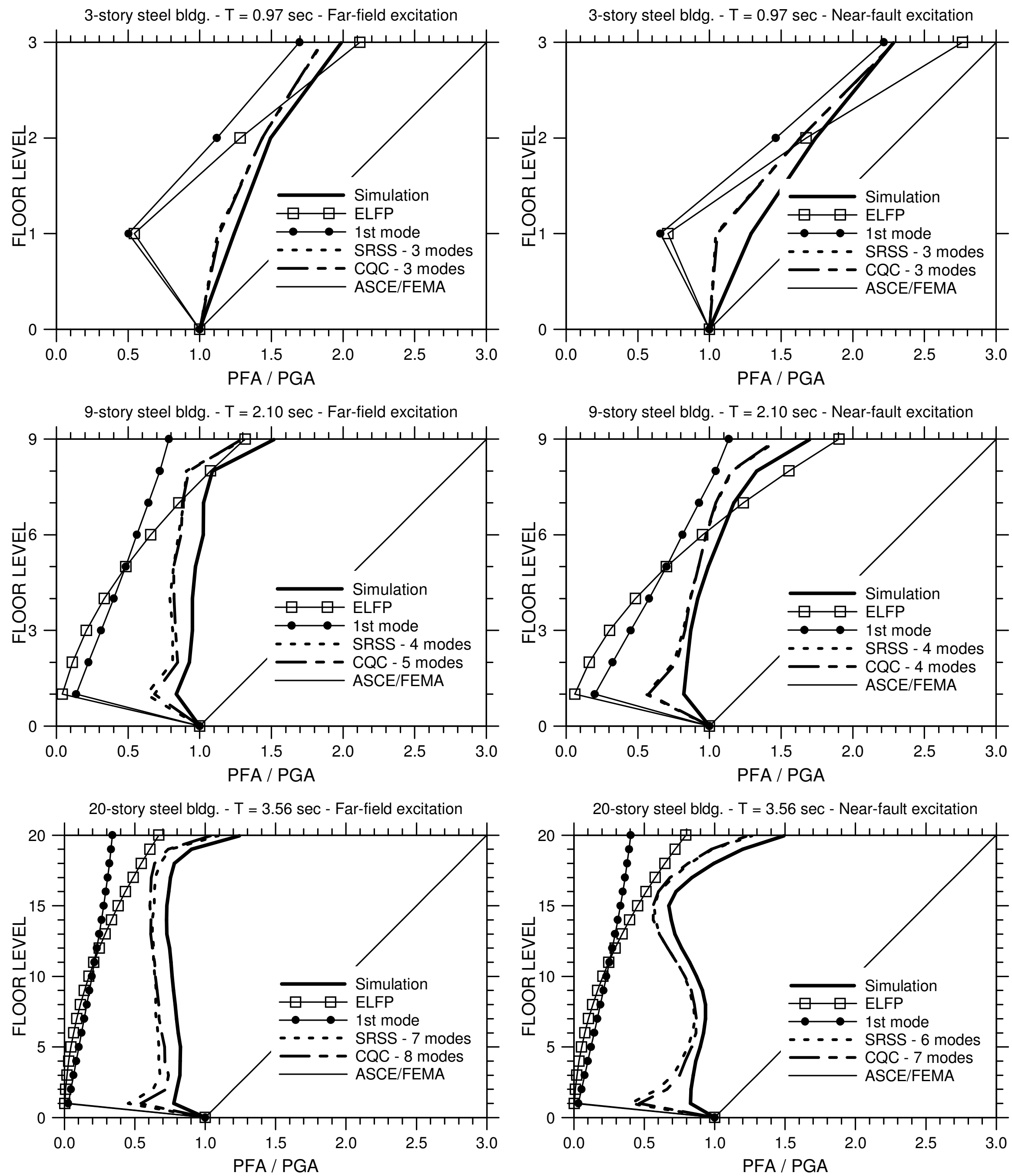

Figure 3 Peak floor accelerations: steel building models

Interestingly, results show a number of common characteristics regardless of the type of structural system and the kind of excitation. Firstly, estimates of PFAs given by the ELFP are always very inaccurate. They are consistently unconservative, generally by a wide margin, at the lower floor levels, and are generally (but not always) conservative at the upper floor levels. Secondly, estimates of PFAs given by a single-mode modal analysis are consistently unconservative, and generally by a wide margin. This observation clearly indicates that the contribution of higher modes to the absolute acceleration response cannot be neglected, not even in the case of low-rise buildings. Thirdly, the accuracy of estimates of PFAs given by Modal Analysis depends on the floor location. At the mid-height and upper floor levels, estimates of PFAs given by Modal Analysis are consistently unconservative but only by a narrow margin (20% at most). Perhaps more important, the *pattern* of the height-wise variation of PFAs is very accurately predicted. At the lower floor levels, on the other hand, estimates of PFAs given by Modal Analysis are consistently unconservative by wide margins, by more than 100% in some cases at the first floor level. Regardless of the floor location, estimates of PFAs obtained considering the SRSS and CQC modal combination rules are essentially identical to each other, and the number of modes needed to adequately estimate the absolute acceleration response turned out to be definitely greater than the number of modes usually needed to adequately estimate the displacement response. Finally, the "default" ASCE/FEMA linear height-wise variation of PFAs turned out to be very conservative, except in a few cases and only at the top floor level.

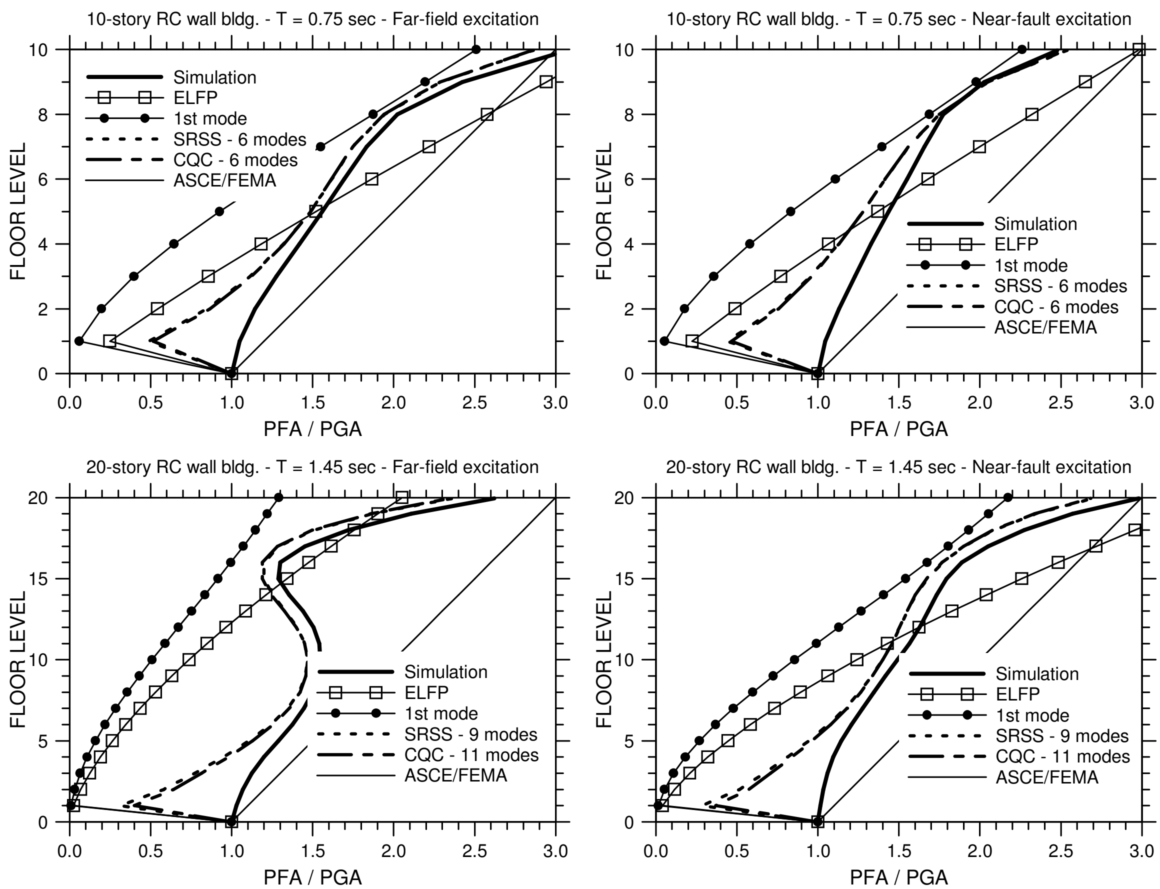

Figure 4 Peak floor accelerations: RC wall building models

4. CONCLUDING DISCUSSION

Results shown in this paper suggest that, when modal response spectrum analysis is performed to assess the linear response of building structures, peak floor accelerations might be adequately estimated by dividing the effective seismic force at a given floor level by the corresponding floor mass, provided that some sort of "correction factor" is introduced. A convenient definition of this "correction factor", intended to be used in practice, is currently being investigated. Upon success, resulting estimates of peak floor accelerations will then be much more accurate than the "default" linear height-wise variation of peak floor accelerations indicated in the ASCE/FEMA specifications, which was found to be very conservative. Results shown in this paper also suggest that, when effective seismic forces are calculated in accordance with the ASCE/FEMA's Equivalent Lateral Force Procedure, resulting estimates of peak floor accelerations are consistently inaccurate. These observations were found to be independent of whether the seismic excitation has far-field or near fault characteristics, provided that the response spectrum of the excitation is known (this is not always the case at locations where near-fault effects are expected).

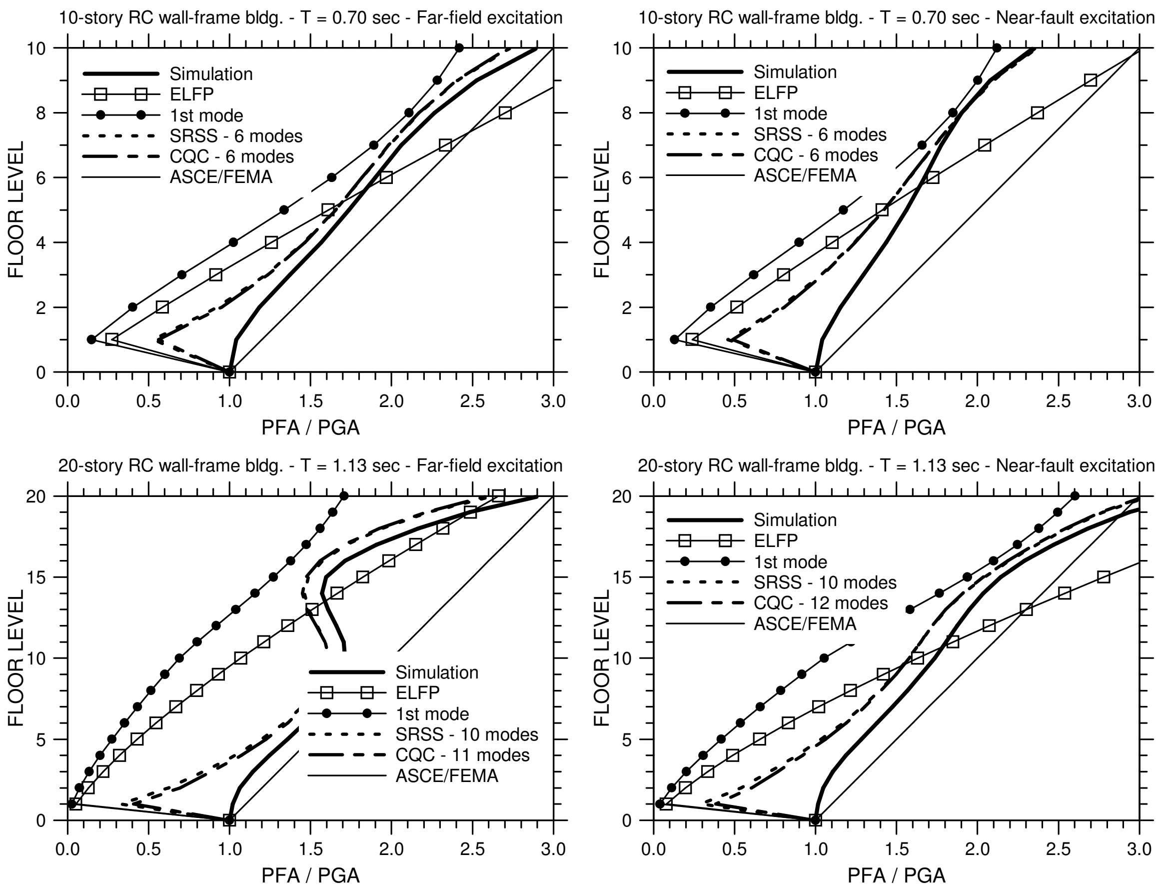

Figure 5 Peak floor accelerations: RC wall-frame building models

The study described in this paper is limited to linearly elastic buildings not exhibiting a significant torsion response. The characteristics of peak floor accelerations in building structures affected by inelastic deformations and by torsion effects are being investigated in ongoing research projects.

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