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ESTIMATION OF SEISMIC ACCELERATION DEMANDS IN BUILDING COMPONENTS

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SUMMARY

A method to estimate floor acceleration demands in multi-story buildings subjected to earthquakes is presented. In the proposed method, buildings are modeled as a combination of a shear and flexural beams. The model is defined by three parameters: the fundamental period of structure, damping ratio and lateral stiffness ratio. The accuracy of the method is then evaluated by comparing accelerations computed with the method to those measured in three instrumented buildings in California. A parametric study to evaluate the effects of these three parameters on seismic acceleration demands of buildings including peak floor acceleration and floor response spectra is also presented.

INTRODUCTION

Nonstructural components typically represent a major portion of the total cost of buildings. Furthermore, nonstructural damage often occurs at response intensities that are smaller to those required to produce structural damage. Therefore, it is not surprising that when losses due to structural and nonstructural components are separated, losses due to nonstructural components have consistently been reported to be far greater than those resulting from structural damage (Ayers et al. [1], Whitman et al. [2], Rihal [3]). A large portion of nonstructural components and building contents are damaged primarily as a result of being subjected to large floor acceleration demands. Components such as suspended ceilings, light fixtures, fire sprinklers and parapets are examples of acceleration sensitive components. Figure 1 shows photos of a few acceleration sensitive nonstructural components. The functionality of many facilities such as hospitals depends on functionality of these components. In the Olive View Medical Center in Sylmar, California, during the 1994 Northridge earthquake, the water leakage from broken fire sprinkler and chilled water caused the facility to shut down and forced patients to be evacuated (OSHPD 1995 [4]). Similarly, the San Francisco International airport was shut down for thirteen hours as a result of the 1989 Loma Prieta earthquake because of a power failure and nonstructural damage in the control tower, such as falling ceiling tiles and several broken windows.

Despite their significance to control economic losses and downtime, seismic behavior and design of nonstructural components has received relatively small attention from researchers and practicing

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engineers compared to the attention that has been devoted to understand and improve the seismic behavior of structural members. Seismic provisions provide simplified procedures to estimate acceleration demands on nonstructural components. In particular, current U.S. seismic provisions recommend the use of a trapezoidal distribution of peak floor accelerations along the height of the building and a floor response spectrum regardless of the number of stories in the building or its lateral resisting system. Some limited evidence suggests that this variation may be inadequate for some structures (Singh [5]; Soong et al. [6]). Villaverde [7] noted that these problems are due to the fact that these design-oriented methods still do not account for all the factors that significantly affect the response of nonstructural components. More recently, these provisions have seen severely criticized by some practicing structural engineers (Kehoe and Freeman [8]; Searer and Freeman [9]) who concluded that the intensity and distribution of floor accelerations over the height of the building appears to be influenced by the predominant period of vibration of the building and the mode shapes.

Figure 1 – Examples of acceleration sensitive nonstructural components

In this paper, a simplified method to estimate peak floor acceleration demands and floor spectra ordinates in buildings that are expected to remain elastic or practically elastic when subjected to earthquake ground motions is presented. The approximate method is directly relevant to the estimation of seismic demands on acceleration-sensitive nonstructural components attached to conventional buildings during small and moderate earthquakes in which the structure is expected to remain elastic or practically elastic, as well as to the estimation of seismic demands on acceleration-sensitive nonstructural components in critical building facilities which are designed to remain elastic or practically elastic even during severe ground motions. The efficiency and accuracy of the method is evaluated by a few examples. A brief parametric study to evaluate effects of fundamental period of vibration of buildings and lateral resisting systems as well as stiffness reduction on peak floor acceleration and floor response spectra is also presented.

APPROXIMATE ESTIMATION OF ACCELERATION DEMANDS

Simplified model of building

In the method proposed here, multi-story buildings are modeled using an equivalent continuum model consisting of a flexural cantilever beam and a shear cantilever beam deforming in bending and shear configurations, respectively (Figure 2). It is assumed that lateral deformation of flexural and shear beams

are identical. Floor masses are assumed to remain constant along the height of the building. As shown in figure 3, the approximate model used here has the advantage of being able to consider not only the two extremes of deformation (pure shear and pure flexure), but in addition it can consider buildings whose lateral deformations are a combination of flexural and shear deformation.

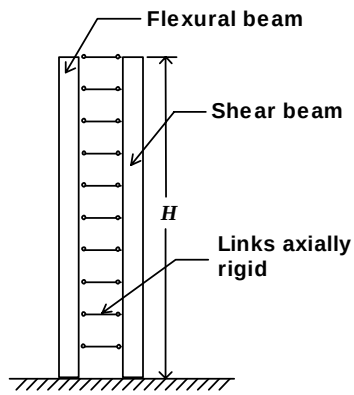

Figure 2 - Simplified model to estimate the dynamic properties of multistory buildings

The differential equation of the combined shear-flexural model used here was first developed by Traum and Zalewski [10] and by Heidebrecht and Stafford Smith [11]. More recently, Miranda [12] used the model to estimate maximum interstory drift demands in buildings subjected to earthquakes. He derived closed-form solutions for the lateral displacements normalized by the displacement at the top of the structure and for the ratio of the maximum rotation demand to the roof drift ratio (lateral displacement at the top divided by the total height) when subjected to a wide variety of static lateral forces. His approximate method was more recently extended to buildings with non-uniform lateral stiffness (Miranda and Reyes [13]).

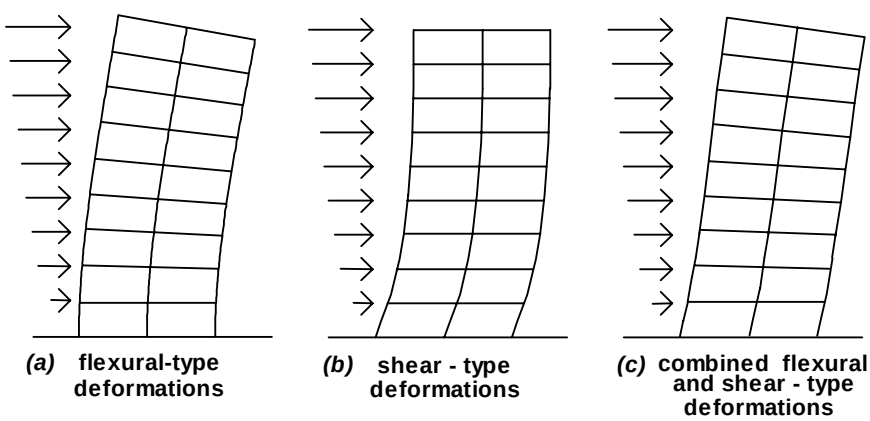

Figure 3 - Overall lateral deformations in multistory buildings

The governing dynamic equation of motion of the continuum system with uniform lateral stiffness shown in Figure 2 when subjected to a horizontal base acceleration of $\ddot{u}_g(t)$ is given by the following equation:

$$\frac{\rho}{EI_0}\frac{\partial^2 u(x,t)}{\partial t^2}+\frac{c}{EI_0}\frac{\partial u(x,t)}{\partial t}+\frac{1}{H^4}\frac{\partial^2}{\partial x^2}\left(\frac{\partial^2 u(x,t)}{\partial x^2}\right)-\frac{\alpha_0^{\,2}}{H^4}\frac{\partial}{\partial x}\left(\frac{\partial u(x,t)}{\partial x}\right)=-\frac{\rho}{EI_0}\frac{\partial^2 u_g(t)}{\partial t^2} \tag{1}$$

where $\rho(x)$ is the mass per unit length in the model, $u(x\ ,\ t)$ is the lateral displacement at non-dimensional height x (varying between zero at the base of the building and one at roof level) at time t, H is the total

height of the building, $c(x)$ is the damping coefficient per unit length, EI_0 is the flexural rigidity at the base of the structure and α_0 is the lateral stiffness ratio defined as:

$$\alpha_0 = H\left(\frac{GA_0}{EI_0}\right)^{1/2} \tag{2}$$

where GA_0 is the shear rigidity at the base of the structure. The lateral stiffness ratio is a dimensionless parameter α_0 that controls the degree of participation of overall flexural and overall shear deformations in the simplified model of multi-story buildings and thus, it controls the lateral deflected shape of the building. A value of α_0 equal to zero represents a pure flexural model (Euler-Bernoulli beam) and a value equal to ∞ corresponds to a pure shear model. Intermediate values of α_0 correspond to multi-story buildings that combine shear and flexural deformations.

Dynamic characteristics of simplified model

In the method proposed here, the dynamic properties of multistory buildings are approximated by those of the simplified model discussed in the previous section. For the case of uniform lateral stiffness, the dynamic characteristics can be obtained in closed form. In particular, the mode shape associated to the ith mode of vibration is given by (Miranda and Taghavi [14]):

$$\phi_i(x) = \frac{\sin(\gamma_i x) - \gamma_i\left(\alpha_0^2+\gamma_i^2\right)^{-1/2}\sinh\left(x\sqrt{\alpha_0^2+\gamma_i^2}\right) + \eta_i\left[\cosh\left(x\sqrt{\alpha_0^2+\gamma_i^2}\right) - \cos(\gamma_i x)\right]}{\sin(\gamma_i) - \gamma_i\left(\alpha_0^2+\gamma_i^2\right)^{-1/2}\sinh\left(\sqrt{\alpha_0^2+\gamma_i^2}\right) + \eta_i\left[\cosh\left(\sqrt{\alpha_0^2+\gamma_i^2}\right) - \cos(\gamma_i)\right]} \tag{3}$$

where η_i is defined as:

$$\eta_i = \frac{\gamma_i^2\sin(\gamma_i) + \gamma_i\sqrt{\alpha_0^2+\gamma_i^2}\,\sinh(\sqrt{\alpha_0^2+\gamma_i^2})}{\gamma_i^2\cos(\gamma_i) + \left(\alpha_0^2+\gamma_i^2\right)\cosh(\sqrt{\alpha_0^2+\gamma_i^2})}$$

(4)

γ_i is an eigenvalue parameter associated with mode i and the root of the following characteristic equation:

$$2 + \left[2 + \frac{\alpha_0^4}{\gamma_i^2(\gamma_i^2+\alpha_0^2)}\right]\cos(\gamma_i)\cosh\left(\sqrt{\alpha_0^2+\gamma_i^2}\right) + \left[\frac{\alpha_0^2}{\gamma_i\sqrt{\alpha_0^2+\gamma_i^2}}\right]\sin(\gamma_i)\sinh\left(\sqrt{\alpha_0^2+\gamma_i^2}\right) = 0 \tag{5}$$

Once γ_i is known for i^{th} mode of vibration, the modal participation factor and the period ratio of ith mode are given by:

$$\Gamma_i = \frac{\int_0^1 \phi_i(x)dx}{\int_0^1 \phi_i^2(x)dx} \tag{6}$$

$$\frac{T_i}{T_1} = \frac{\gamma_1}{\gamma_i}\sqrt{\frac{\gamma_1^2+\alpha_0^2}{\gamma_i^2+\alpha_0^2}} \tag{7}$$

Examination of equations 3 to 7 shows that mode shapes, modal participation factors and period ratios are fully defined by a single parameter, the lateral stiffness ratio, α_0 (see figures 4 and 5). Miranda and Reyes [13] have indicated that this parameter can be estimated based on the type of lateral resisting system in the building. Shear wall and braced frame buildings usually have values of α_0 between 0 and 1.5; buildings

with dual structural systems consisting of a combination of moment-resisting frames and shear walls or a combination of moment-resisting frames and braced frames usually have values of α_0 between 1.5 and 5; whereas moment-resisting frame buildings usually have values of α_0 between 5 and 20. Hence, the simplified model presented in the previous section has the important advantage of allowing estimation of the dynamic characteristic of a multi-story building based only on its lateral resisting system and its fundamental period of vibration.

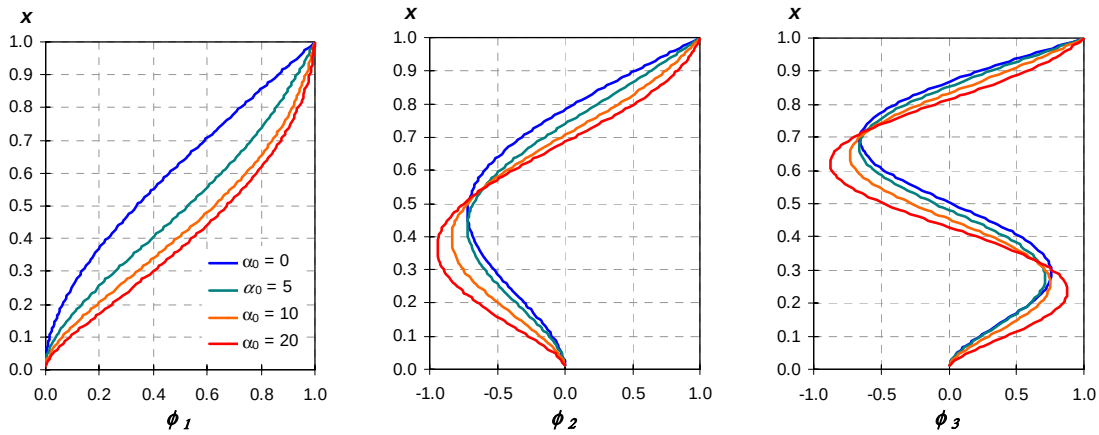

Figure 4 - Effect of α_0 on mode shapes when the lateral stiffness remains constant.

In the proposed method, floor acceleration demands are approximated by only including the first few modes of vibration. Therefore, using modal analysis equations, the absolute (total) floor acceleration at non-dimensional height x can be approximated as:

$$\ddot{u}^t(x,t) \cong \ddot{u}_g(t) + \sum_{i=1}^{N} \Gamma_i \, \phi_i(x) \ddot{D}_i(t)$$

(8)

where $\ddot{D}_i(t)$ is the relative acceleration of the ith mode SDOF system subjected to ground acceleration.

In equation 8, modal participation factors and mode shapes are functions of lateral stiffness ratio, α_0 and $\ddot{D}_i(t)$ is a function of period of the ith mode which is a function of α_0 and T_1, and modal damping ratio ξ. Therefore total acceleration at a certain location x can be computed by knowing fundamental period of vibration of the building T_1, lateral stiffness ratio α_0, modal damping ratio ξ and ground acceleration.

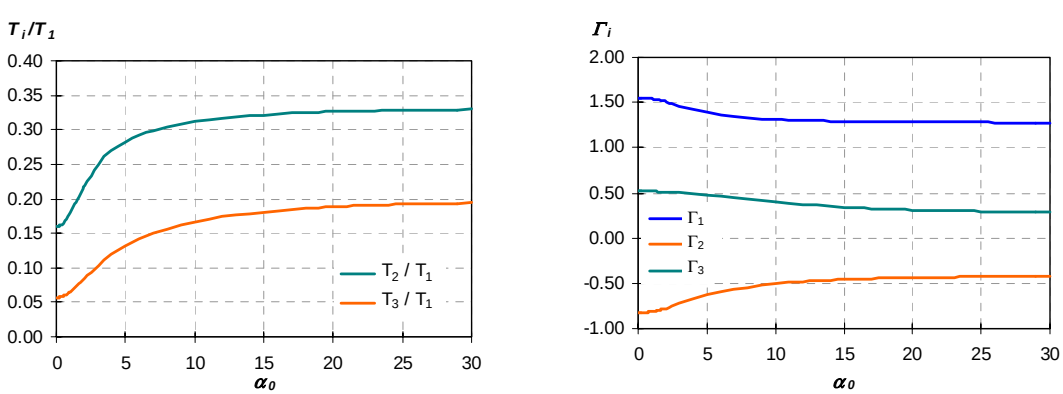

Figure 5 - Effect of α_0 on modal participation factors and period ratios.

The computational effort in the proposed method is very small. In particular, the computational effort is much smaller than the computational effort involved in the computation of a linear elastic response spectrum.

Equations 3 to 8 assume that the lateral stiffness of the building remains constant along the height of the building. With the exception of one to three story buildings such assumption is not usually realistic. Miranda and Taghavi [14] studied the effect of reduction of stiffness along the height on the product of the mode shape and the modal participation factor (product of equations 3 and 6) and on period ratios (equation 7). They considered linear and parabolic reductions of stiffness along the height and up to 75% reduction in lateral stiffness from the base to the roof. Their study showed that reductions in lateral stiffness along the height have a relatively small effect on the product of the mode shape and the modal participation factor (product of equations 3 and 6) and on period ratios (equation 7). Hence, using $\Gamma_i\ \phi_i(x)$ and $T_i\ /\ T_1$ computed from a uniform model provides a relatively good approximation to these dynamic properties in non-uniform buildings.

VALIDATION OF THE PROPOSED METHOD

Accuracy of the proposed method is evaluated in this section by comparing floor acceleration demands computed with the continuum model to those recorded in three instrumented buildings in California. The first building is a 30-story reinforced concrete building in Emeryville that recorded the 1989 Loma Prieta earthquake. The second and third buildings are 13-story reinforced concrete building and 6-story steel building that were shaken by the 1994 Northridge earthquake.

When the lateral stiffness is assumed to remain constant along the height of the building, the continuum model used in the method is fully defined with knowledge of only three parameters: the fundamental period of the structure, the damping ratio and the lateral stiffness ratio α_0. As mentioned before, α_0 can be approximated based on knowledge of the lateral resisting system. The parameters used for each of the buildings are shown in table 1 for each component. The fundamental period of vibration and the damping ratio of these buildings correspond to those available in the literature and the lateral stiffness ratio is based on the lateral resisting system of each building.

Table 1 – Information of the buildings used for evaluation of the method

Bldg.	Location	No. of stories	Earthquake	Dir	T_1 (s)	ξ (%)	Structural System	α_0	Reference
1	Emeryville	30	Loma Prieta	N-S	2.59	3	MRF	12.5	[15]
				E-W	2.69	3	MRF	12.5	
2	Sherman Oaks	13	Northridge	N-S	3.0	5	MRF	12.5	[16]
				N-S	2.8	8	MRF	12.5	
3	Sylmar	6	Northridge	N-S	0.33	12	Dual System	3.1	[17]
				E-W	0.33	18	Dual System	3.1	

Comparison of acceleration demands

Figure 6 shows a comparison of peak floor acceleration predicted with the proposed method using the parameters listed in table 1 with peak recorded accelerations. It can be seen that for all three buildings and for both directions the proposed method produces very good estimates. Also shown in the figure are the peak floor accelerations the floor accelerations computed according to the FEMA-368 [18] assuming that the peak ground acceleration is known. As shown in the figure, these provisions recommend a linear variation of lateral acceleration demands varying from an acceleration equal to peak ground acceleration at the base to three times the peak ground acceleration at the roof. As shown in the figures, in many cases these recommendations can lead to significant errors.

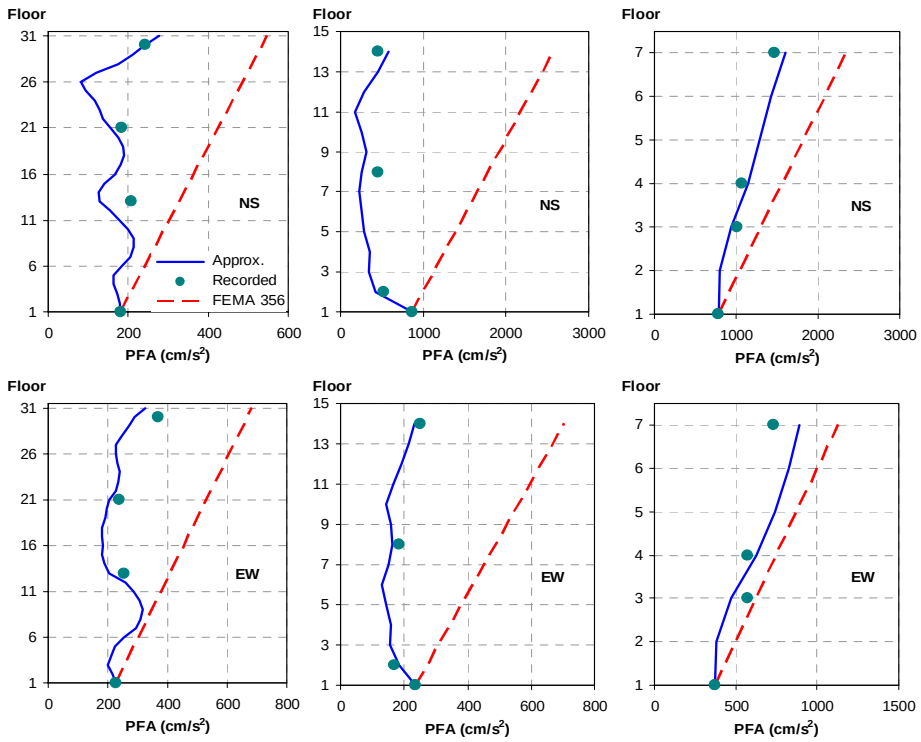

Figure 6 – Comparison of recorded peak floor accelerations with those of simplified model and NEHRP provisions

In addition to estimation of peak floor acceleration demands, the proposed method can also be used to estimate floor spectra and floor acceleration time histories. Figure 7 shows comparison of recorded and estimated acceleration time histories at roof level in the 6-story Sylmar Medical Center and the 13-story Sherman Oaks building. Considering the simplicity of the method, the results are very promising. Floor response spectra at roof level for the perpendicular direction of the same buildings are shown in figure 8. As shown in this figure, the proposed method is also able to estimate floor spectra relatively well.

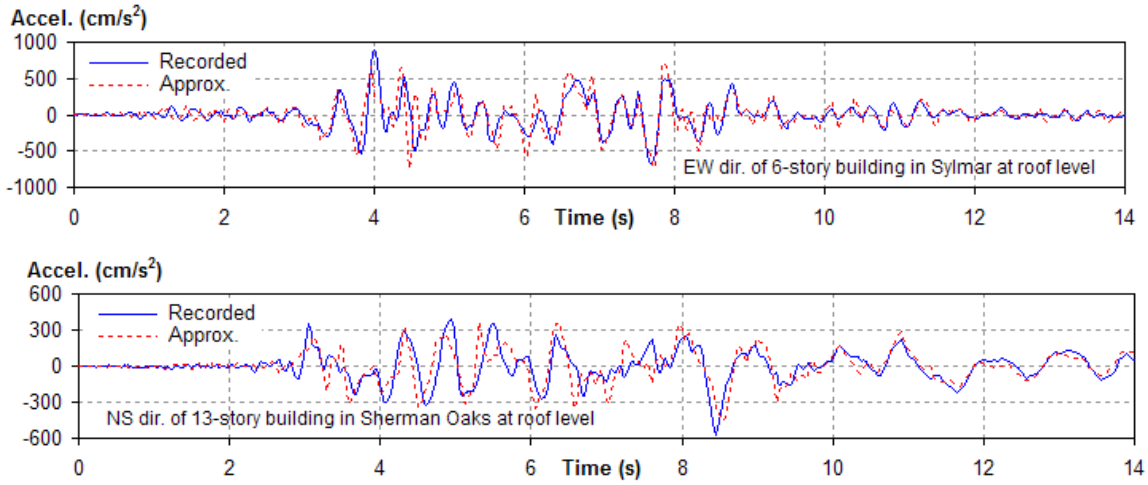

Figure 7 – Comparison of recorded acceleration time histories with those of simplified model at roof levels of 6-story Sylmar and 13-story Sherman Oaks Buildings

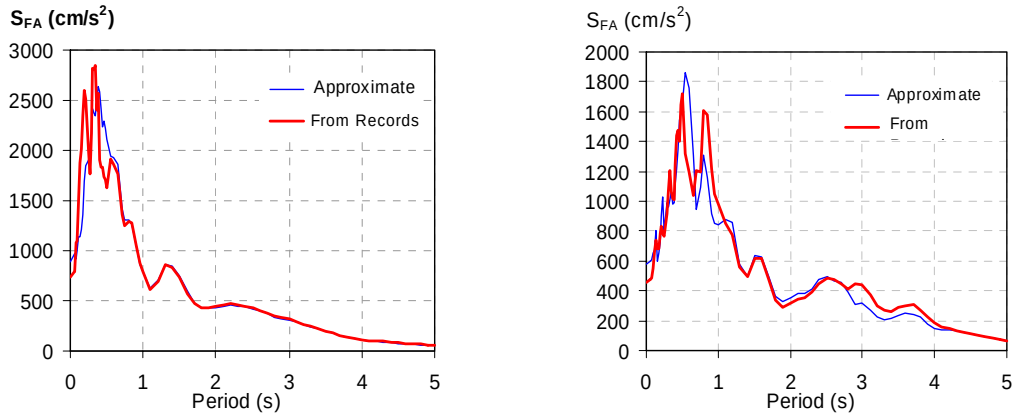

Figure 8 – Comparison of floor response spectra computed from recorded accelerations and those calculated by the method at roof levels of 6-story Sylmar and 13-story Sherman Oaks Buildings

PARAMETRIC STUDY ON PEAK FLOOR ACCELERATIONS

Some studies have suggested that the variation of acceleration demands along the height of buildings and in particular the ratio of the peak floor acceleration demand to peak ground acceleration is independent of the period of vibration of the structure (Bachman and Drake [19] and Drake and Gillengerter [20]). However, as shown figure 6, the acceleration profile can change significantly from one building to another. Kehoe and Freeman [8] have criticized the NEHRP provisions to estimate floor accelerations in buildings and have indicated that the period of vibration may influence the distribution of accelerations along the height of the building, but have not provided specific recommendations on how this parameter should be taken into account. In the following paragraph the results of a parametric study of the effects of fundamental period of vibration, lateral stiffness ratio and stiffness reduction along the height on seismic peak floor acceleration demands are summarized and discussed.

Structural parameters

For building with uniform stiffness along the height the simplified model is defined by three parameters: fundamental period of the structure, modal damping ratio and lateral stiffness ratio. For buildings with non-uniform stiffness, a fourth parameter corresponding to the ratio of the lateral stiffness at roof to the lateral stiffness at the base. (Miranda and Taghavi [14]). In this study, all models are assumed to have the modal damping ratio equal to 5 percent. The fundamental period of the structure was varied from 0.5s to 4.0s with increment of 0.25 s. The lateral stiffness ratio, α_0, was varied from 0 (flexural behavior) to 20 (nearly shear behavior) with increments of 2. Finally, the stiffness reduction parameter was varied from 0 to 75 percent with increment of 25 percent.

Ground motions considered

Eighty recorded ground motions were used in this study. The ground motions were recorded on sites classified as class D according to recent NEHRP provisions. These ground motions were then classified into four bins according to their earthquake magnitude and epicentral distance as follows: (1) SMSR (Small Magnitude, Small Distance); (2) SMLR (Small Magnitude, Large Distance); (3) LMSR (Large Magnitude, Small Distance); (4) LMLR (Large Magnitude, Large Distance). The earthquakes with magnitude of 5.8 to 6.5 are referred as small magnitude and from 6.6 to 6.9 are referred as large magnitude. The distance of recording station to epicenter from 13 to 30 km is referred to as small distance and from 30 to 60 km is referred to as large distance. The ground motions have *PGAs* ranging from 0.03g to 0.44g. More information regarding the ground motions can be found in Medina [21].

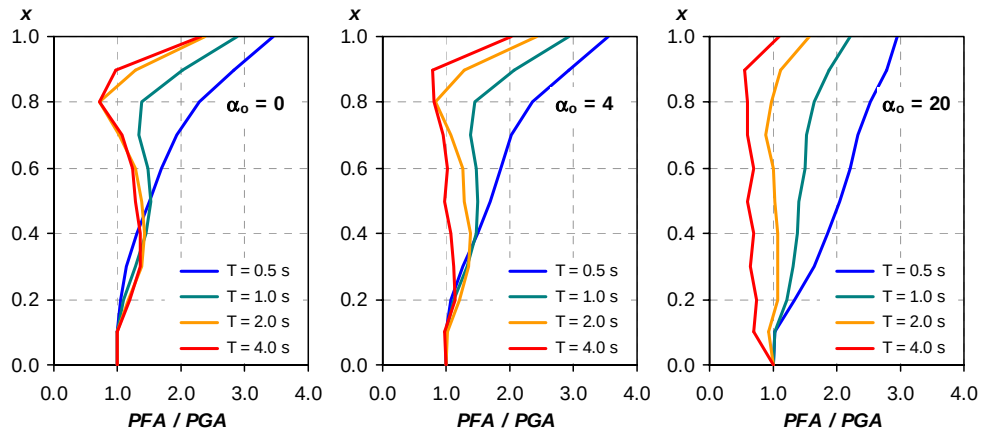

Figure 9 – Effect of T_1 of peak floor acceleration profile

Effects of fundamental period of vibration and lateral stiffness ratio on PFA profile

Figure 9 shows the effect of the fundamental period of vibration on the variation of peak floor accelerations along the height of the building. Results shown in this figure correspond to mean ratios (average of 80 records) of peak floor acceleration demands to peak ground acceleration. It can be seem that floor accelerations are amplified as the period of vibration decreases. In particular, short period structures exhibit large amplification of acceleration demands as height increases. For buildings with small values of α_0 mean amplifications at roof level can be larger than those currently recommended in NEHRP provisions. It can also be observed that the effect of the fundamental period of vibration of the structure is larger in buildings that deflect laterally like shear beams than those that deflect laterally like flexural beams. However, the latter buildings are more likely to experience sharp local amplifications near the top of the building as a result of higher modes.

Figure 10 shows the effects of the lateral stiffness ratio α_0 on the variation of peak acceleration demands a long the height of buildings. It can be seen that for short period structures, floor acceleration increase as height increases regardless of the lateral stiffness ratio. It can be seem that long period buildings that deflect laterally like shear beams on average will have acceleration demands that are smaller than those occurring at the base.

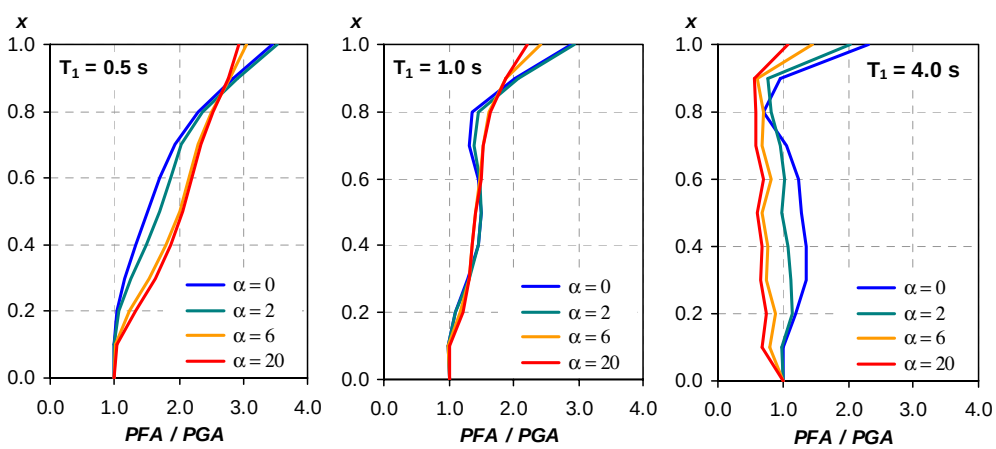

Figure 10 – Effect of α_0 on the variation of peak floor accelerations along the height of the building

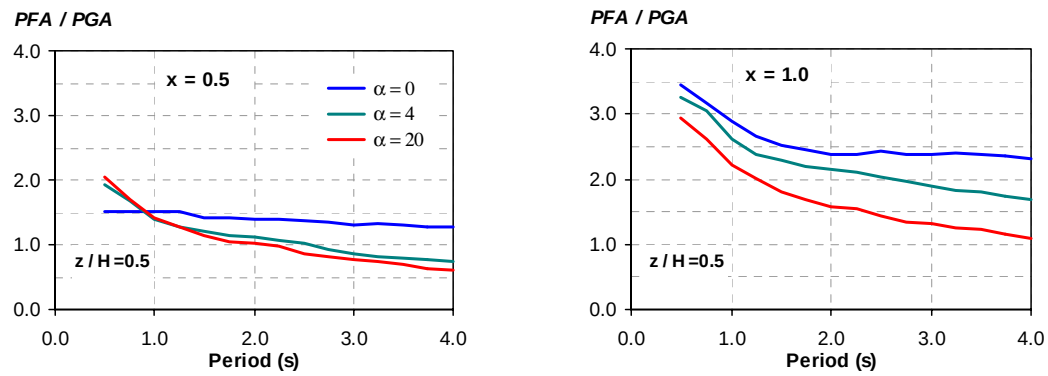

Figure 11 – Variation of peak floor acceleration at mid height and roof level with changes in the fundamental period of vibration T_1 for different lateral stiffness ratios.

Figure 11 shows changes in peak floor accelerations normalized by peak ground accelerations at mid-height and roof levels with changes in the fundamental period of vibration. It can be seen that mean PFA to PGA ratios tend to decrease as the fundamental period of vibration increases. However, reductions are more important for buildings with large values of α_0 and are more pronounced at roof level than those at mid-height. In some cases the mean reductions are substantial. For example for buildings with large values of α_0 the PFA to PGA ratio decreases from approximately 3.0 for a period of 0.5s to approximately 1.0 for period of vibration of 4.0s.

Effect of stiffness reduction on PFA profile

Figures 9, 10 and 11 correspond to buildings in which the lateral stiffness was assumed to remain constant along the height. Miranda and Taghavi [14] studied the effect of the reduction of lateral stiffness on the dynamic properties required to estimate lateral acceleration in buildings. They considered variations in lateral stiffness defined by two parameters, δ that controls the lateral stiffness at roof to that at the base and λ that controls the shape of the stiffness profile (see figure 12). They showed that the effect of λ is negligible so only the effect of δ was considered in the parametric study. Figure 13 shows the effect of stiffness reduction for moment frame buildings. *PFA* is plotted for δ = 1.0 (uniform stiffness), 0.75, 0.50 and 0.25. It is seen that regardless of fundamental period of the structure, stiffness reduction does not has a significant effect on the variation of acceleration demands along the height of the building for most of the height. A small effect is observed near the top of the structure.

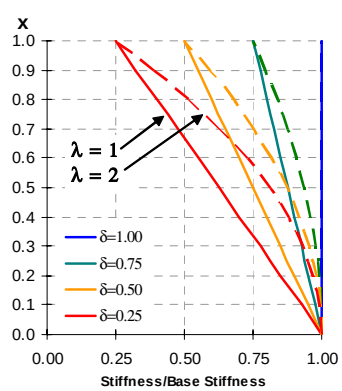

Figure 12 – Lateral stiffness profile

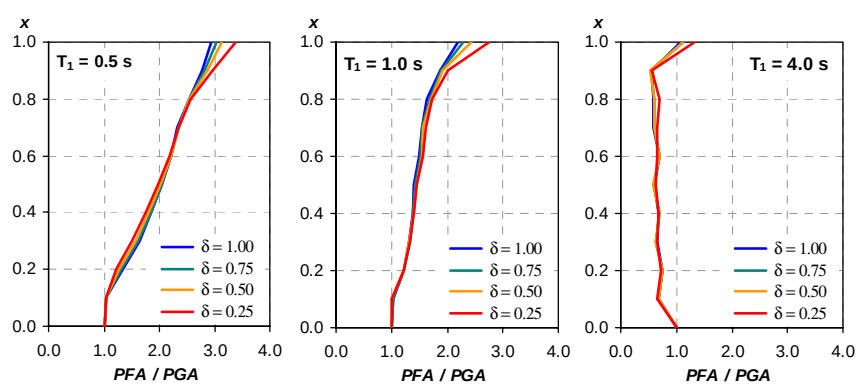

Figure 13 – Effect of stiffness reduction on peak floor acceleration profile

PARAMETRIC STUDY OF FLOOR SPECTRA

Floor response spectra are useful to estimate seismic demand of flexible acceleration sensitive components mounted on floors of buildings whose mass is significantly smaller to that of the building. Various studies have shown that acceleration demands can be greatly amplified for building components whose period of vibration coincide with those of the primary structure. A parametric study was conducted to study the effects of the fundamental period of vibration, the lateral stiffness ratio and the reduction of stiffness along the height on floor spectra.

Effects of fundamental period of vibration and lateral stiffness ratio on floor spectra ordinates

Figure 14 shows mean floor response spectra at roof level for buildings with fundamental periods T_1 equal to 1.0, 2.0 and 3.0 s and with lateral stiffness ratio of α_0 = 0 and 20. All floor spectra are normalized by peak floor acceleration (sometimes also referred to as zero period acceleration). It can be seen that the amplitude and location of the peaks in the floor spectra change with changes in the fundamental period of vibration. The amplification at a period equal to the fundamental period decreases as the fundamental period of the building increase. This amplification is approximately 3.2 for T_1 = 1.0s, 2.5 for T_1 = 2s and 1.0 for T_1 = 3 s. This trend does not hold for higher modes. It can be seen that the amplification for a period equal to the second mode of vibration of the building is 4.0 when T_1 = 1.0, 4.4 when T_1 = 2.0 and 3.8 when T_1 = 3.0 s. In all three plots, it can be observed that normalized spectral ordinates increase around T_1 when lateral stiffness ratio increases. In other word, there is a slight increase of floor spectra around the first mode of structure in buildings deflecting laterally as shear beams (e.g. moment frame buildings) compared to that in buildings that deflect laterally like flexural beams (e.g. shear wall buildings). However, around the second mode, this trend is reversed.

Figure 15 shows the variation of spectral amplifications along the height for flexible nonstructural components whose periods coincide to those of the first and second periods of vibration of the building. As shown in this figure the amplification in spectral ordinates changes not only with the fundamental period of vibration of the structure but also with the height level. In general, spectral amplifications for periods around T_1 are larger in the upper part of the building and can be on average as large as five for short period structures at two third of the height. For long period structures, this value reduces to 1.0. Also it is clear that for buildings with longer fundamental period, the demand is lower around their fundamental period. The spectral amplification around T_2 also varies significantly along the height of the building. Maximum amplification in this case are expected to occur at one third of the height. The mean spectral acceleration ordinate can be as low as *PFA* and increase to values as large at 4 times *PFA* with changes in height location within the building. The lowest amplifications occur, as expected, at the building height where the mode shape of the second mode has a node.

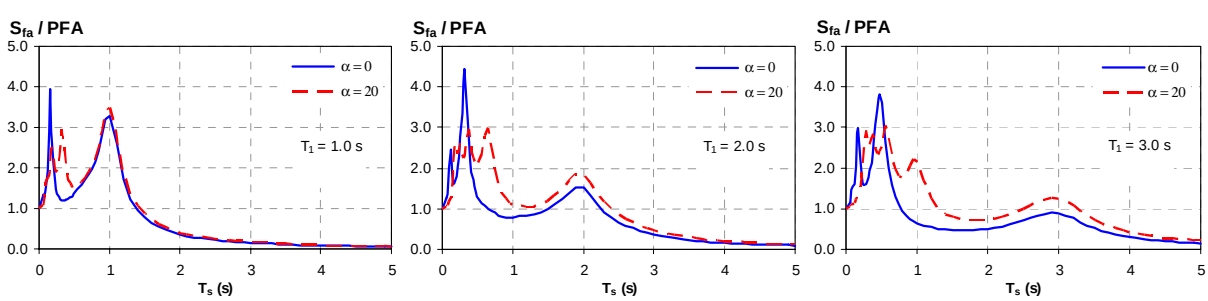

Figure 14 – General observations of effects of T_1 and α_0 on floor response spectra

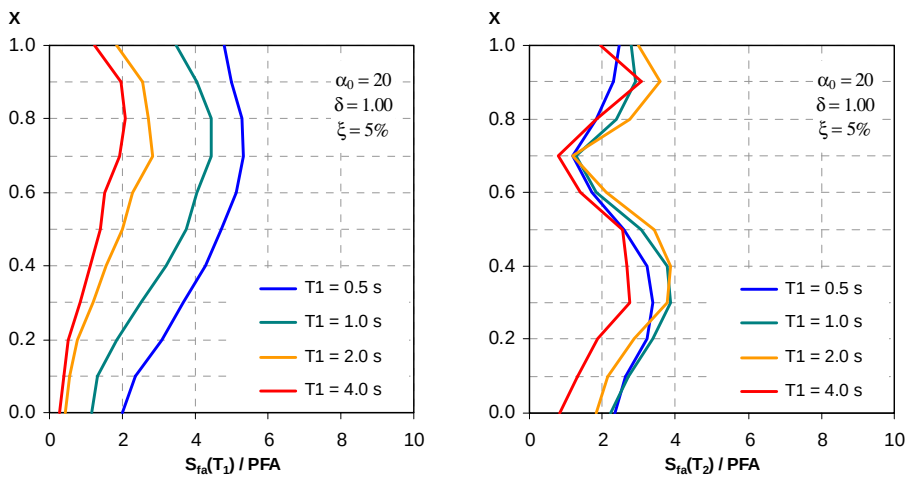

Figure 15 – Variation of floor response spectra peaks around the first and second modes of the main structure

Figure 16 shows the variation of floor spectral amplifications at roof level with changes in the lateral stiffness ratio. As shown in this figure, an increase in lateral stiffness ratio increases the floor spectral acceleration around the fundamental period and decreases it around the second mode. The effect of lateral stiffness ratio is smaller for buildings with short periods of vibration than for buildings with long fundamental periods of vibration.

Effects of reduction of lateral stiffness along the height of the building on floor spectra

Figure 17 shows the effect of the reduction of lateral stiffness along the height of the building on floor spectra ordinates at the roof level in buildings with a fundamental period of vibration of 4.0s. The figure compares floor spectra computed for buildings with uniform stiffness along the height to those computed in buildings where the lateral stiffness at the top of the building is one fourth of the lateral stiffness at the base of the building ($\delta = 0.25$). can increase the period of higher modes up to 20 percent and therefore the peaks in floor response spectra move slightly. It is seen that the floor spectra are not affected significantly due to stiffness reduction. The peak on the first period of the structure is reduced by about 10 percent for $\delta = 0.25$ in a building with flexural behavior and fundamental period of 4.0 seconds. Changes in spectral ordinates are primarily due to changes in building period ratios. A behavior similar to that shown in this figure was observed for buildings with other periods of vibration.

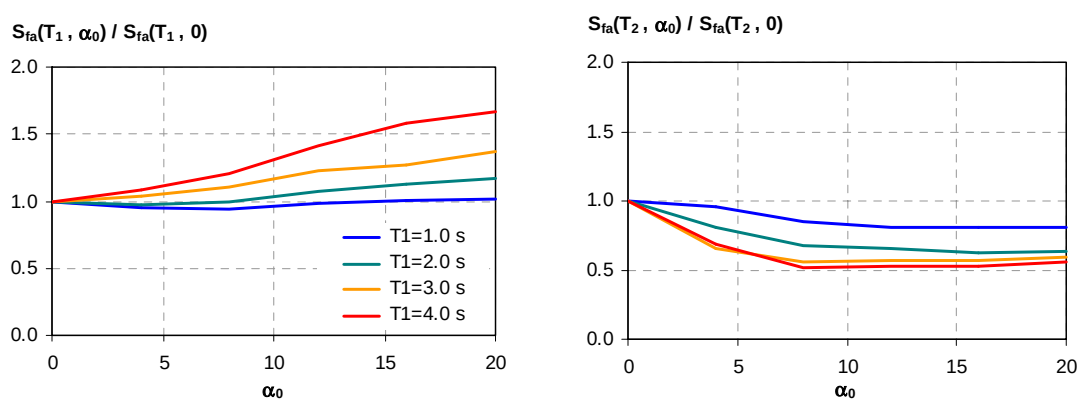

Figure 16 – Effects of lateral stiffness ratio on amplification of floor spectra ordinates at the roof.

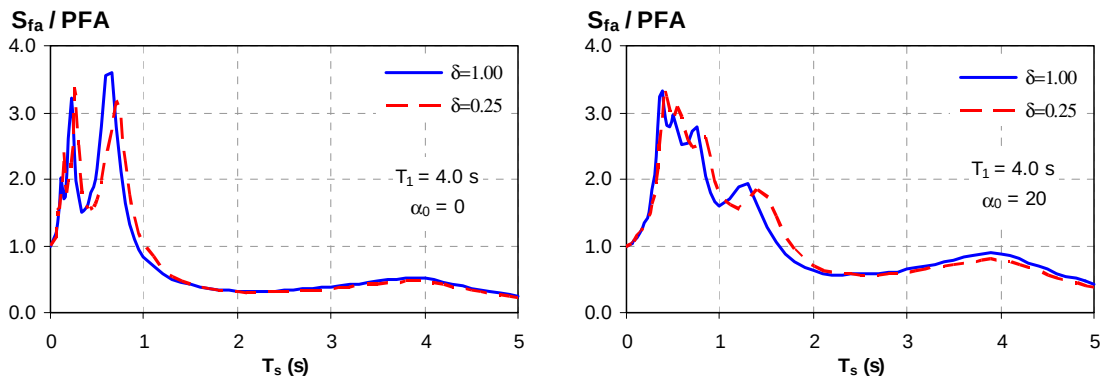

Figure 17 – Effect of stiffness reduction on floor response spectra ordinates at roof level.

SUMMARY AND CONCLUSIONS

A method to estimate floor acceleration demands in buildings subjected to earthquakes was presented. The method uses a simplified model consisting on two continuous beams. A close form solution of the dynamic characteristics of the model was presented when the lateral stiffness of the model is uniform. The model is fully defined with only three parameters: the fundamental period of the structure, a modal damping ratio and the lateral stiffness ratio. The accuracy of the method was evaluated by comparing the peak floor acceleration demands, time histories and floor spectra computed with the method to those obtained from acceleration records in three instrumented buildings. It was shown that the method is able to capture acceleration demands with reasonable accuracy with a very small computational effort.

A parametric study was performed to study the effects of various parameters on acceleration demands in buildings. The parameters that were studied are: the fundamental period of the structure, the lateral stiffness ratio and a parameter describing the amount of reduction of lateral stiffness along the height. Variation of these parameters was studied together with 80 ground motions recorded on firm sites in various earthquakes in California. It was observed that both the fundamental period of the structure and the lateral stiffness ratio can significantly change acceleration demands in buildings. On the other hand, results indicate the reduction in lateral stiffness along the height of the building do not have a significant effect on acceleration demands. A similar parametric study was performed to investigate the effect of these parameters on floor spectra ordinates. Results indicate that spectral amplifications around the periods of the main structure can change significantly with change in fundamental period of the structure, lateral stiffness ratio as well as floor level. Spectral amplifications around the first mode of the structure decrease as the fundamental period of vibration increases and increase as the lateral stiffness ratio increases. Effects of structural nonlinearity are currently being investigated.

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