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FLOOR AND ATTACHED COMPONENT SEISMIC AMPLIFICATION FACTORS FROM NONLINEAR TIME-HISTORY ANALYSIS

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ABSTRACT :

Seismic forces for attached nonstructural building components can be quite different from forces obtained from the current design code. This is because the current code is based on measured building motions where the buildings remained largely linear and past analysis studies that focused on linear building response – clearly a significant limitation since buildings are designed for nonlinear response. Maximum floor accelerations of frame buildings subjected to major earthquake loading are often amplified over the peak ground acceleration (PGA). Furthermore, maximum accelerations of nonstructural components that are attached to a given floor can be amplified several times over these already large maximum floor accelerations. Hence, attached component accelerations of above 5g are possible, even under design-basis earthquake events. This paper provides floor and attached component acceleration and amplification results determined from nonlinear time-history analyses of a suite of low and mid-rise steel moment frame buildings designed to current US code provisions. All possible attached components from flexible to stiff were considered at each floor level. Ten significant earthquake motions were selected and scaled to match a design ARS curve for the highest seismic regions in California. A simple hand equation for determining maximum possible floor accelerations is presented and validated based on the local strength characteristics of the structure above and below the floor of interest. Of particular note is that the maximum possible floor acceleration is independent of earthquake loading, as it is based on the plastic capacity of the girder hinges.

KEYWORDS: Nonlinear time-history analysis, building response, floor accelerations, attached nonstructural components, seismic response, amplification factors

1. INTRODUCTION

Strong ground shaking caused by a large magnitude earthquake forces frame buildings to respond in the nonlinear range, with plastic hinges developing at predetermined locations, typically at the girder ends and at the base of the 1st floor columns. This paper examines maximum floor and attached nonstructural component accelerations for steel moment frame buildings subjected to low to high intensity earthquakes. Three constant hazard levels were looked at with target spectra of 2%, 10% and 50% probability of exceedance in 50 years, representing the maximum credible earthquake, the design level earthquake, and a much smaller earthquake, respectively. Target design spectra were constructed by obtaining key spectral ordinates from the NEHRP maps [3] for a site class *D* in the San Francisco and Los Angeles seismic regions. A total of 10 measured ground motions were amplitude-scaled so that the mean of their spectra matched each of the three different hazard level response spectra. Nonlinear time-history analyses were conducted for 1-story through 5-story buildings as well as a 10-story building. This required 30 nonlinear time-history analyses for each building, resulting in a total of 180 nonlinear building analyses. From these analyses, floor time-history responses were used as input to construct acceleration response spectrum (ARS) curves at each floor level. The ARS then gives peak accelerations of all possible SDOF linear-elastic nonstructural components attached to that floor.

Maximum floor accelerations are often larger than peak ground accelerations (PGA) due to amplification demonstrated by a typical ARS curve. Building modes that have similar frequency to the dominant earthquake motion frequency are amplified more than other modes. Rather than the linear-elastic behavior assumed in construction of an elastic ARS curve, a building is designed to respond nonlinearly from a large seismic event, often limiting floor accelerations to below what would be expected from an elastic ARS analysis, with an upper limit based on the strength of the building. This upper limit is explored in the present study, resulting in a simple hand calculation for maximum possible floor accelerations. Peak floor accelerations from nonlinear time-history analyses were all below this upper bound limit, as expected. Additional amplification from a given floor to a linear-elastic attached component can be found from a traditional ARS approach. However, total amplification from the ground to the component includes the strength-limited ground-to-floor amplification and, hence, the building strength also plays a limiting role in the maximum possible accelerations and forces of attached nonstructural components. This study is important because it is based on nonlinear building response while other studies of floor and attached component accelerations and amplification factors have often relied upon linear-elastic building analysis. Various national and international building design codes consider added amplification for attached, flexible nonstructural components [1, 2, 3, 4, 5, 6, 7 and 8]. Nonlinear characteristics of the building have been considered to determine maximum floor accelerations in [9 and 10].

2. GROUND MOTION SELECTION AND SCALING

The goals of the ground motion selection and scaling was to obtain a suite of ground motions that (1) are from a magnitude 6 or larger earthquake, (2) are within 20 miles of the epicenter, (3) are from strike-slip faults in California and (4) have spectra of the scaled records that adequately represent the target response spectrum curve for three constant hazard levels of 2%, 10% and 50% probabilities of exceedance in 50 years - see Figure 1(a). This study focuses on regions of high seismicity in California. For the 2% in 50 year hazard level, the spectral response accelerations at the shorter period (S_S) and longer period of 1 second (S_1) are 2 g and 1g, respectively [3]. The S_S and S_1 values for 10% and 50% probabilities of exceedance in 50 years are calculated by multiplying the 2% probability values by 2/3 and 1/3, respectively. A site classification of *B* is assumed, permitting F_a and F_v to be taken as 1.0. Note that the intent of this study was to generalize results for regions of high seismicity in California. There are several locations in the vicinity of Los Angeles and San Francisco with site classifications of *B*, *C* and *D* that have the same design spectra shown in Figure 1(a).

In this study, 40 ground motion records were initially selected from stations in California. Record locations were limited to stations with site classifications *B*, *C* and *D*. The suite was reduced to the 10 measured ground motions listed in Table 1. While shear wave velocities given in Table 1 are lower than defined for site class B, the code allows deviation to an adjacent class, to site class C in this case. The suite of measured free-field ground motions were chosen so that the average spectrum would most closely match the target spectrum, with best results in the shorter period range below two seconds where more of the interest in this study lies. The most flexible of the buildings studied in this project has a 1st mode period of 2.25 seconds, with all other modes for this building and all other building 1st mode and higher mode periods fall below 1.5 seconds.

Table 1. Measured ground motions used in analyses (unscaled)

Record Name (earthquake location, station location, horizontal channel, abbreviation)	Magnitude	Epic. Dist. (km)	Peak Ground Accel. (g)	Scale Factor for 2%/50yr	Shear Wave Velocity (m/s)
Parkfield, Fault Zone 1, Ch. 1, (PFFZ1-1)	6.0	8.8	0.592	1.57	339
Northridge, UCLA, Ch. 1, (NUCLA-1)	6.4	18.0	0.278	3.78	398
Northridge, UCLA, Ch. 3, (NUCLA-3)	6.4	18.0	0.474	2.57	398
Northridge, Santa Monica City Hall, Ch. 3, (NSM-3)	6.4	23.0	0.370	2.89	336
Northridge, Pacoima-Kagel Fire Sta., Ch. 1, (NPAC-1)	6.4	18.0	0.301	2.43	508
Northridge, Pacoima-Kagel Fire Sta., Ch. 3, (NPAC-3)	6.4	18.0	0.432	1.99	508
Landers, Joshua Tree Fire Sta., Ch. 3, (LJ-3)	7.3	14.0	0.274	2.65	379
Palm Springs, Desert Hot Springs Fire Sta., Ch. 3, (PS-DHS-3)	6.1	17.0	0.300	2.53	345
Loma Prieta, San Jose-Santa Teresa, Ch. 1, (LPSJT-1)	7.0	21.0	0.274	3.32	672
Loma Prieta, San Jose-Santa Teresa, Ch. 3, (LPSJT-3)	7.0	21.0	0.228	3.76	672

Unit conversion: 1 mile = 1.61 km, 1 ft = 0.3048 m

The geometric mean scaling method [11] is used to scale the motions to the target response spectrum at the selected hazard level. This method was chosen because it matches the design spectrum over a wide range of frequencies. This characteristic is important to investigate the higher building mode response and associated nonstructural equipment amplification. In this approach the ground motions are amplitude-scaled to minimize the sum of the square errors between the target spectrum and the scaled geometric mean of al motions.

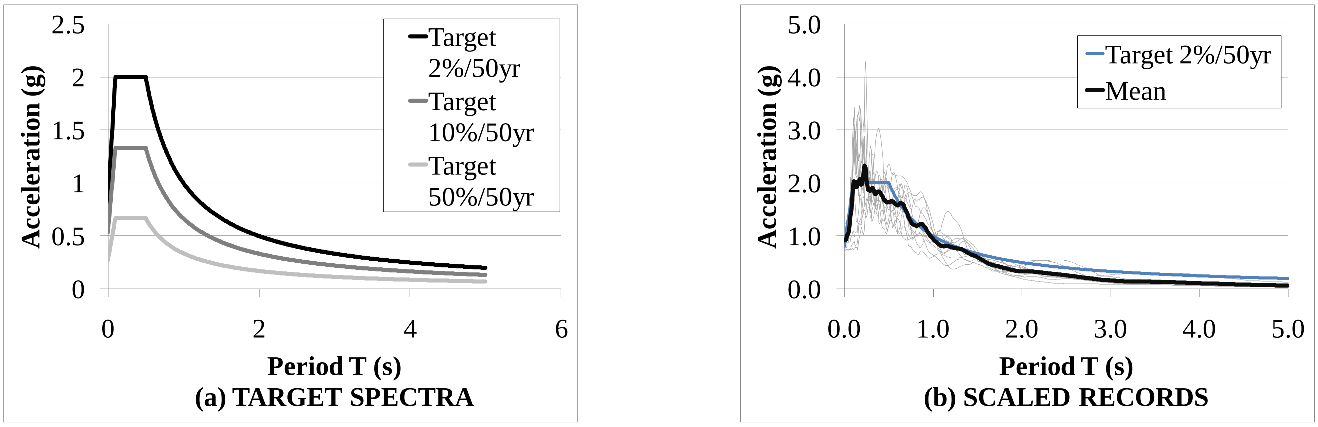

Figure 1. Elastic (5% damped) acceleration response spectra

As shown in Figure 1(b), the mean of the spectra from the scaled ground motions is fairly close to the 2%/50 year target spectrum for the period range from zero to two seconds. Spectra from all 10 scaled motions are also shown in light gray in Figure 1(b).

3. BUILDING DESIGN

Several steel moment-resisting frame buildings, with concrete slabs, ranging in height from one through five stories, and 10 stories, were designed for dead and live load, and to withstand major earthquake loading [4]. Plastic hinges are designed to occur at the girder ends and at the base of the 1st story columns. A single bay is assumed with column spacing of 24 ft (7.3 m) and floor spacing of 12 ft (3.6 m). One building frame is modeled in 2-D with tributary load and mass from adjacent frames included. The tributary dead load to the floor members is 2.5 kips/ft (36 kN/m) with associated line mass. The buildings are designed to have a lateral strength equal to 16.7 percent of the total building weight – with a peak design ARS value of 1.33g divided by a response modification coefficient *R* of 8 for the moment frame - applied at the assumed inertial force centroid location of two-thirds the building height. Thus if the building base shear reaches 0.167 of the building weight, the structure will be in the nonlinear range with plastic hinges formed. Based on a simple hand calculation (schematic in Figure 2), the required plastic moment capacity expression can be given as a function of the number of floors *n*, spacing between columns *a*, distance between floors *s* and distributed floor weight *w* as $M_p = \frac{was}{8}\left(\frac{n^2}{n+1}\right)$. With all of the plastic hinges formed the simple hand calculation provides the same lateral force capacity as a nonlinear computer pushover analysis.

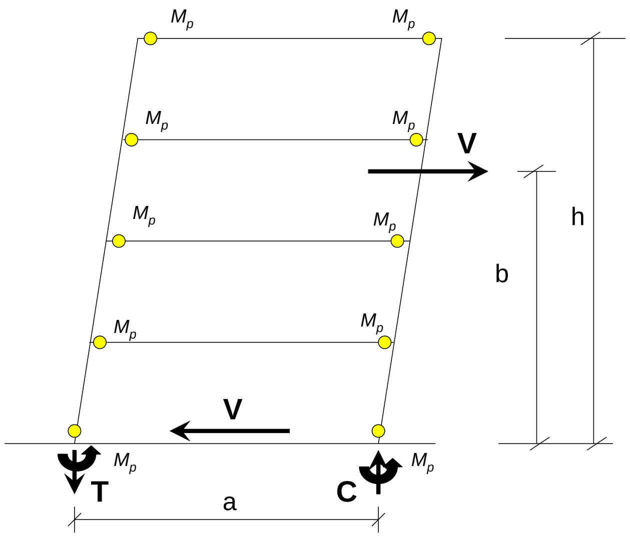

Figure 2. Schematic of pushover analysis of building to determine required plastic moment capacity M_p

Table 2 gives the required plastic beam moments for each of the designed buildings based on the above expression. Also provided in the table are maximum factored beam moments from dead load and live load, which are approximately the same for each building design as the dominant consideration is the span length and the loading (which are the same for all of the building designs). Slight differences in dead and live load moments are due to small changes in boundary conditions at the beam ends from frame action. Uniform dead and live load on each floor are assumed to be 2.5 kips/ft (36 kN/m) and 1.2 kips/ft (18 kN/m), respectively. The beam size of W18x40 was chosen for all building designs, with plastic modulus Z = 78.4 in.^{3} (1226 cm^3). Using this plastic modulus and the yield strength of 50 ksi (340 MPa), the chosen W18x40 beam plastic moment capacity is 327 kip-ft (444 kN-m). Thus gravity loads controlled the size of the members for all of the buildings, with similar requirements from earthquake and gravity loads for the 5-story building. The natural periods were so long for the 10-story building, shifting far off the peak ARS value, that the 0.167 base shear factor was significantly reduced, with dead and live load still controlling the member size for the taller structure.

Table 2. Beam moments from dead and live loads and required plastic moment from earthquake loads

Number of Stories *n*	**Required Plastic Moment from EQ** M_p ***(kip-ft)***	**Dead Load Moment** M_D ***(kip-ft)***	**Live load Moment** M_L ***(kip-ft)***	**Factored Moment (1.2D+1.6L)** M_u ***(kip-ft)***	**Chosen Beam Size (50 ksi)**
1	20	93.4	44.8	184	W18x40
2	53	110	52.8	216	W18x40
3	90	112	53.8	220	W18x40
4	128	112	53.8	220	W18x40
5	167	112	53.8	220	W18x40

Units: 1 kip-ft = 1.36 kN-m, 1 ksi = 6.8 MPa

4. DERIVATION OF PLASTIC FLOOR ACCELERATIONS BASED ON BUILDING STRENGTH

In this section it is shown that maximum possible floor accelerations are directly related to the building strength and are independent of the applied base loading, so long as the earthquake motion is large enough to cause plastic hinging of the girder ends and nonlinear building response. Since buildings are typically designed to perform nonlinearly under severe seismic attack this is a reasonable assumption. The strength-based approach given in this section provides an upper bound to the floor accelerations and the ground-to-floor amplification factors, based on the plastic moment capacity of the girders. For the buildings designed in the prior section, all the girders have the same plastic moment capacity. If any given floor level of a frame building is examined the maximum possible absolute floor acceleration can be determined from the plastic moment capacity of the girders. The results are unchanged if dead and live load are included. It is reasonable to assume that the girder plastic moment is evenly distributed into the column ends that are above and below the joint where they meet the girder. Allowing for higher mode effects, the maximum possible shear that can be applied to a given floor from the column ends is found when the floor of interest is going one way, and the two adjacent floors (above and below the floor of interest) are opposing this response by going in the opposite direction. The maximum possible longitudinal floor force is found by summing the shear forces applied to the floor from the columns, with

$$F_p = \frac{4M_p}{s} \tag{1}$$

This represents the maximum plastic floor force. The maximum absolute acceleration is then found by dividing the plastic floor force F_p by the tributary mass m of the floor.

$$a_p = \frac{4M_p}{sm} = \frac{4M_p g}{sW_F} \tag{2}$$

where W_F is the total tributary weight to the floor and g is the acceleration of gravity. For the designed buildings the girder plastic moment M_p is 327 kip-ft (444 kN-m), spacing s between floors is 12 ft (3.6 m) and the tributary weight W_F is (2.5)(24) = 60 kips (267 kN). Hence the maximum possible floor acceleration (upper bound) for all floors of the designed buildings, based on strength considerations and higher mode effects, is

$$a_p = \frac{4(327)(g)}{(12)(60)} = 1.82g = 58.5\, ft/s^2 \text{ (17.8 m/s}^2\text{)}$$

It is of interest to note that this upper bound floor acceleration was calculated without any reference to an earthquake record and is independent of the total building height. As long as significant enough ground shaking occurs, girder plastic hinging will take place and the floor accelerations should not exceed the 1.82g value given above. If different member sizes are used along the building height, or variable heights or mass distribution, then the upper bound floor accelerations will also vary.

5. NONLINEAR TIME-HISTORY ANALYSES

Time-history analyses were conducted for each of the 6 buildings considered, using the 10 ground motions scaled to the three different hazard levels (2%/50 year, 10%/50 year and 50%/50 year), resulting in 180 nonlinear analyses. All 10 ground motions were required for a given hazard level based on the geometric mean approach of scaling ground motions, with average results from the 10 analyses representing that hazard level. Plastic hinging at girder ends and at the base of the 1st floor columns are the only nonlinear features of the models. All analyses were conducted using SAP2000 [12]. Modal periods for the different buildings considered here are given in Table 3.

Table 3. Modal periods for all buildings

No. of Stories	Mode1	Mode2	Mode3	Mode4	Mode5
1	0.289	NA	NA	NA	NA
2	0.574	0.163	NA	NA	NA
3	0.875	0.255	0.137	NA	NA
4	1.19	0.356	0.186	0.127	NA
5	1.5	0.461	0.244	0.159	NA
10	2.26	0.716	0.39	0.261	0.19

6. FLOOR AND COMPONENT ACCELERATIONS AND AMPLIFICATION FACTORS

Typical results for component accelerations and amplification factors versus component period are given in Figures 4 through 9. In these example analyses results, the component is attached to the roof of the 1-story building (Figures 4 through 6) or the 2nd floor of the 2-story building (Figures 7 through 9). All linear-elastic components are considered in these plots since the horizontal axis is the component period, allowing for any combination of weight and stiffness. Similar plots were developed showing component acceleration, total amplification from ground to the component and amplification from the floor to the component for each floor of the buildings, but not shown here due to lack of space. These plots were developed by performing nonlinear time-history building analyses and then using response spectrum analysis for each floor. From all of these graphs (Figures 4 through 9 and similar plots for all other floors of the buildings not shown here), maximum component and floor accelerations and amplification factors were determined between component periods of zero and one second. Maximum results for all building floors at the three hazard levels are shown in Figures 10 through 13. Peak floor accelerations are less than 1.82g at all floors (see Figure 10), as expected from the upper bound plastic analysis approach. In the geometric mean approach the average result from all 10 ground motions represents the given hazard level, with higher results from individual earthquake motions.

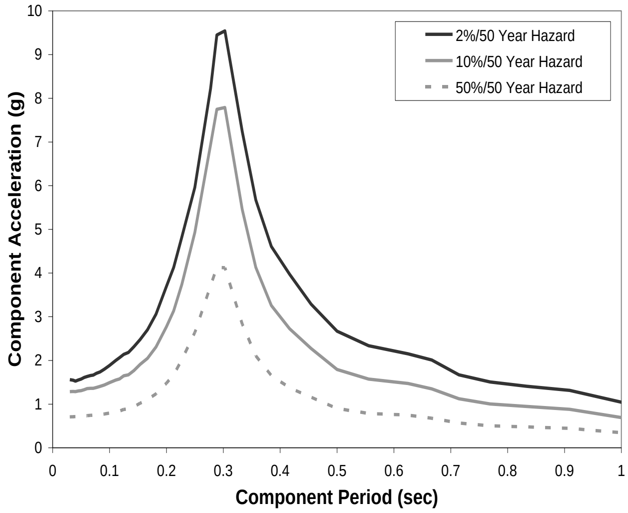

Figure 4. Acceleration (Roof of 1-Story)

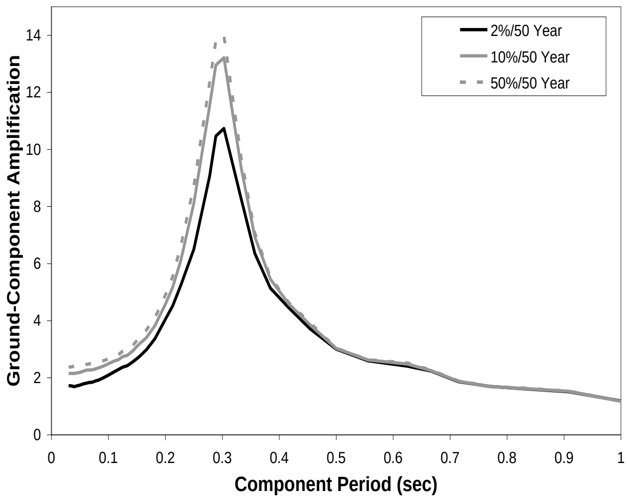

Figure 5. G-Comp. amplification (Roof of 1-Story)

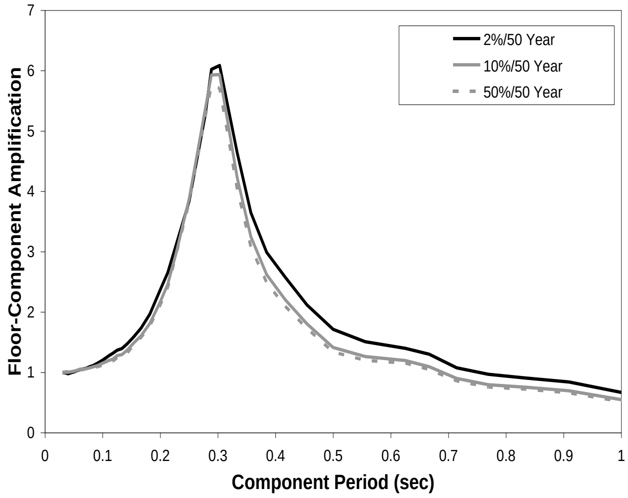

Figure 6. F-Comp. amplification (Roof of 1-Story)

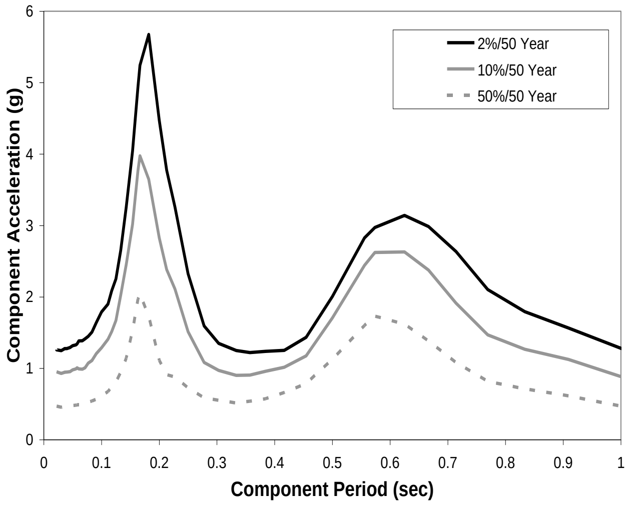

Figure 7. Acceleration (Floor 2 of 2-Story)

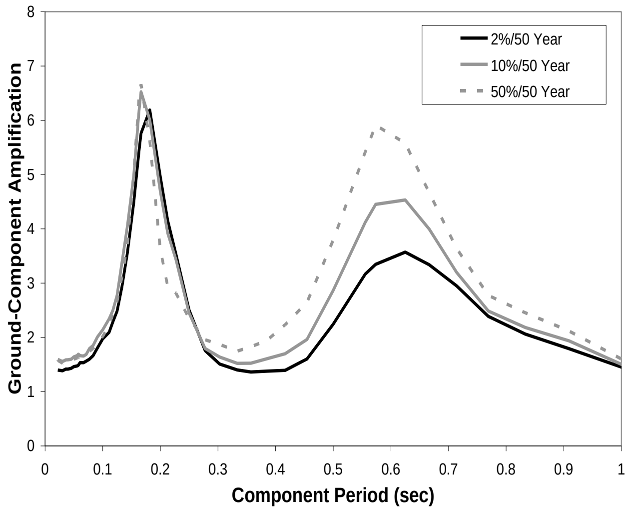

Figure 8. G-Comp. amplif. (Floor 2 of 2-Story)

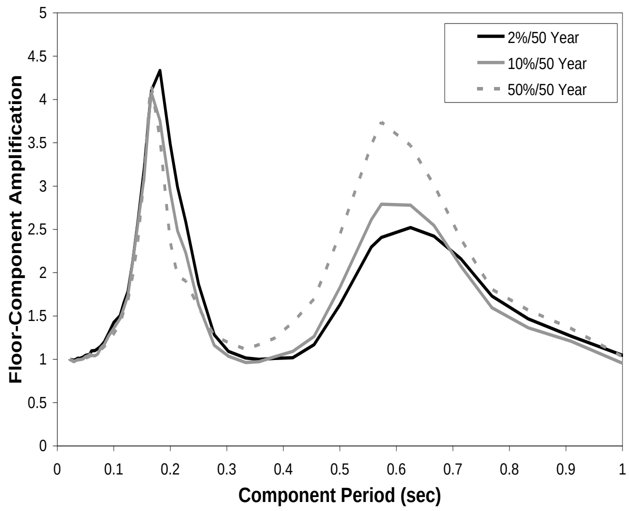

Figure 9. F-Comp. amplif. (Floor 2 of 2-Story)

For the 1-story structure the maximum component acceleration at the 2%/50 year hazard level of almost 10g occurs close to the 1st natural period of the building, whereas much smaller accelerations are expected at other component periods. This reduces to less than 8g and 4g at the 10%/50 year and 50%/50 year hazard levels, respectively. Maximum total ground-to-component amplification factors are about 10, 13 and 14 at the reducing hazard levels. It is interesting that the ground-to-component amplification factors increase with reduced hazard level and reduced earthquake size. This is because amplification factors are found by dividing the maximum component acceleration by the PGA, and with larger earthquakes the PGA increases more rapidly than the maximum component accelerations - nonlinear building response limits the increase in floor accelerations which limits the increase to component accelerations. Floor-to-component amplification is almost identical for the three hazard levels, and this is expected as the response from the floor to the attached component is linear. Similar trends are seen for components attached to the 2nd floor of the 2-story building (Figures 7 through 9). However, here two dominant peaks are seen in the graphs, approximately occurring at the 1st two natural periods of the 2-story building.

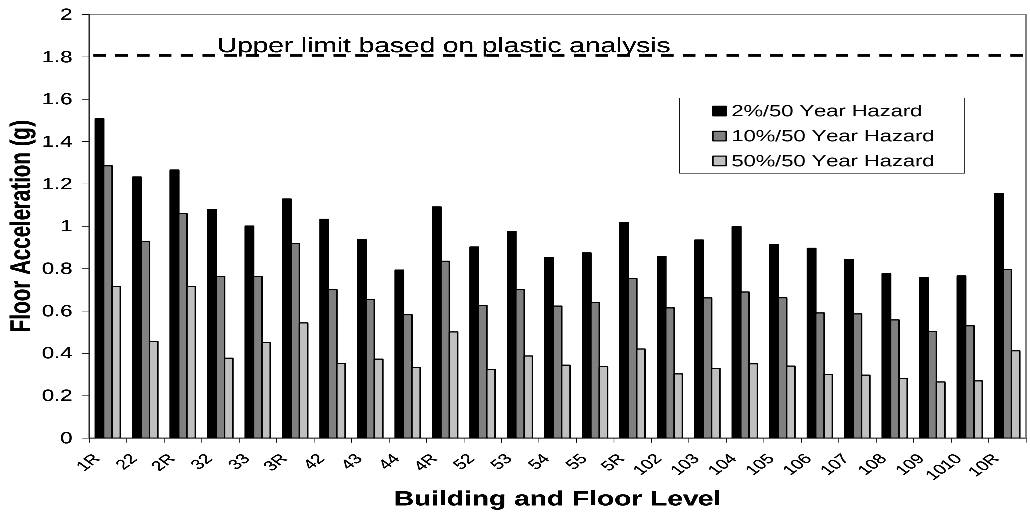

Figure 10. Peak floor accelerations from nonlinear analyses (all buildings and all floors)

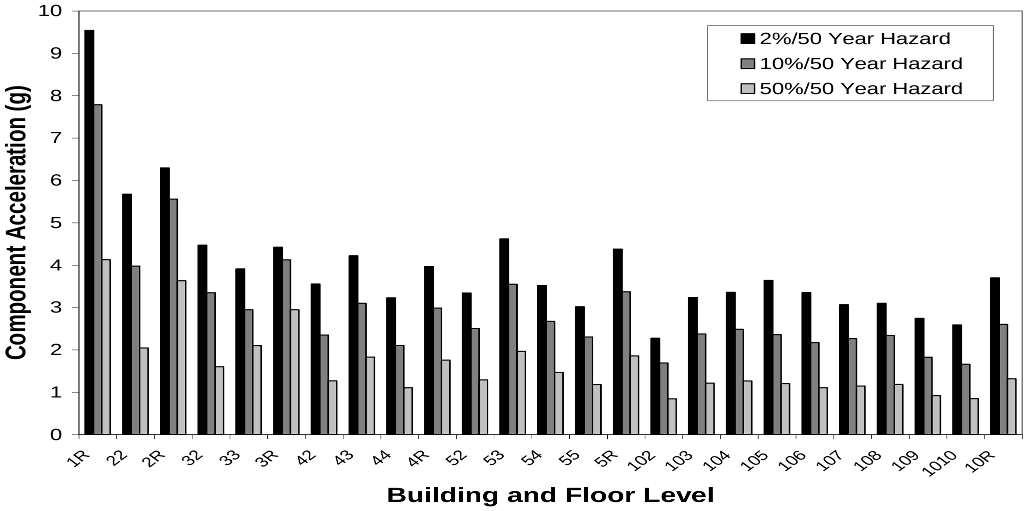

Figure 11. Peak component accelerations from nonlinear analyses (attached to all buildings and all floors)

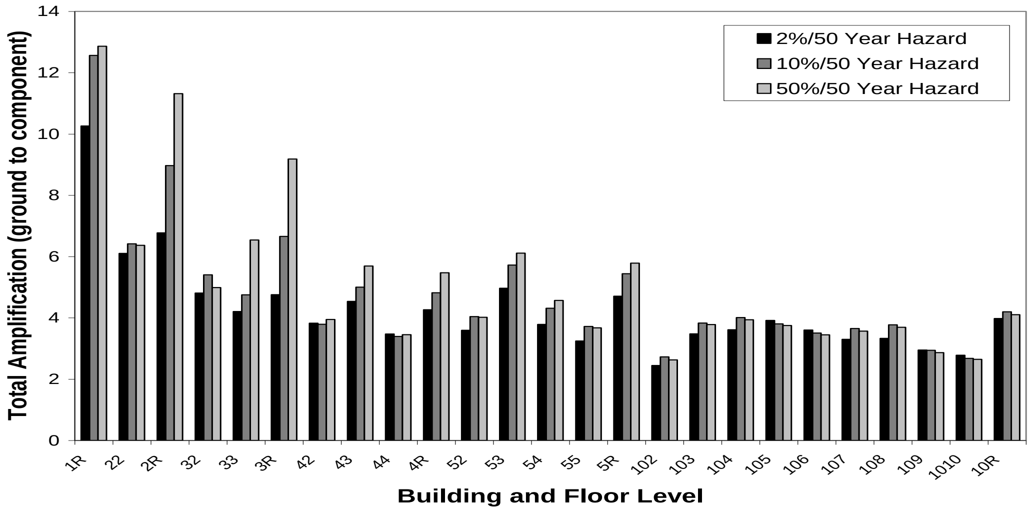

Figure 12. Total amplification from ground to component from nonlinear analyses

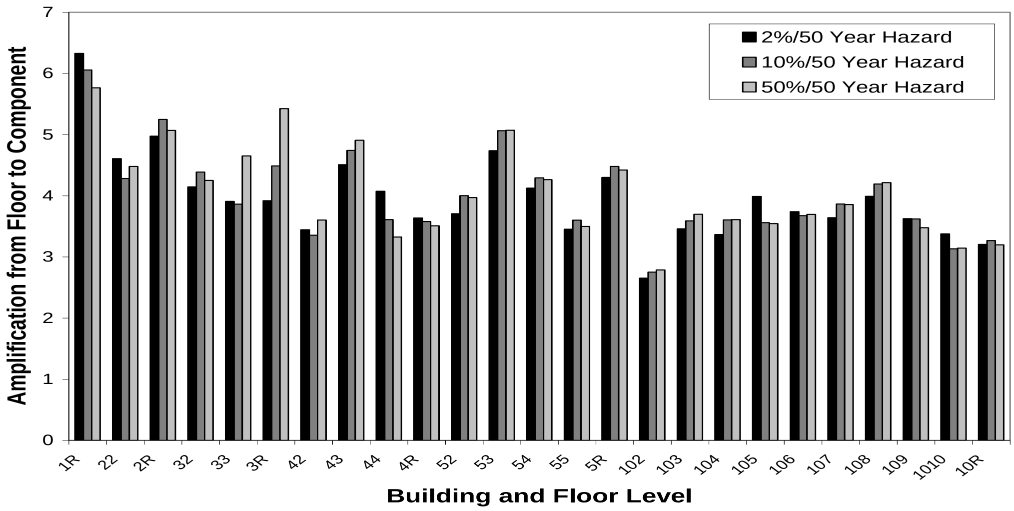

Figure 13. Amplification from floor to component from nonlinear analyses

Of concern is that the largest peak, which is also the peak closest to the typical period range of attached nonstructural components, is related to the 2nd mode of the building and not the 1st or fundamental mode as implied in the design codes. Higher mode effects have also been shown by others to be important [13 and 14]. For the 2-story structure the maximum component accelerations are about 5.5g, 4g and 2g at the three hazard levels. Total amplification factors from the ground to the component have dropped below 7 and floor-to-component amplification is just above 4. Similar results were found from other building analyses and other floors studied (see Figures 10 through 13 for peak results at all floors of the buildings), with the number and periods of prominent peaks in the graphs corresponding to the number of stories and modal periods of the building.

7. CONCLUSIONS

Flexible nonstructural component (NC) accelerations can be several times larger than the floor values, while stiff NC move with the floor they are attached to and, hence, have little to no amplification. Therefore the design codes are reasonable in providing additional amplification for flexible components and no additional amplification for rigid components. The cutoff for the definition of stiff components at 0.06 seconds also appears reasonable as little amplification develops at periods below this value. Maximum floor and attached component accelerations occur from higher mode effects and not from the 1st mode response that is assumed in design. While buildings are designed to respond nonlinearly to a major seismic event the design code recommendations are based on linear-elastic building studies and small measured responses where buildings remained linear-elastic. By including nonlinear building response in the present study amplification factors have dropped significantly. A simple hand approach was developed to determine upper bound floor accelerations based on the local strength and mass of the building, found to be 1.82g for all floors of the buildings in this study. Constant accelerations up the building height have been discussed by others [15 and 16], while design codes assume a 1st mode response with linear variation of accelerations up the building height.

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