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PROBABILISTIC SEISMIC DEMAND EVALUATION OF ACCELERATION SENSITIVE NONSTRUCTURAL COMPONENTS MOUNTED ON STRUCTURAL WALL AND FRAME STRUCTURES

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ABSTRACT

This paper introduces a probabilistic approach to quantify peak component acceleration (PCA) demands for acceleration-sensitive nonstructural components attached to elastic and inelastic structural walls and moment-resisting frames. The method uses incremental dynamic analyses and site-specific ground motion hazard curves to estimate PCA hazard curves and component hazard spectra based on various structural and nonstructural parameters. The probabilistic methodology is illustrated in this paper with models of structural walls and frame structures. The results obtained are compared to ASCE/SEI 7 estimates of peak component acceleration values for a site located in Los Angeles, CA. The results demonstrate that probabilistic estimates of PCA demands may differ significantly from the demands obtained with ASCE/SEI 7. This discrepancy highlights the need for robust probabilistic seismic demand estimation methodologies for nonstructural components in which the major sources of uncertainty are incorporated in the process.

Introduction

This paper introduces a probabilistic method for quantifying peak component acceleration (PCA) demands experienced by acceleration-sensitive nonstructural components (NSCs) during seismic events. NSCs correspond to architectural, electrical, and mechanical components that are rigidly attached to or suspended from the structure. These components could fully or partially detach from the main structure during seismic events, leading to life safety risks, property damage, and economic losses. Seismic design procedures for NSCs and their attachments to the structure focus primarily on the life safety level, and in many cases, economic losses and financial considerations are considered to be secondary in the design (Ghobarah 2001). Adequate estimation of seismically induced forces in components is especially critical considering that damage costs of secondary systems may account for up to 82%, 87%, and 92% of total constructions costs for office buildings, hotels and hospitals, respectively (Taghavi and Miranda 2003). Appropriate quantification of these forces also plays an important role in the design of critical facilities such as hospitals in which the structure is to remain functional after the seismic event. Damages from recent earthquakes have demonstrated the need for a better understanding and an improved quantification of seismic-induced component responses (Ghobarah 2001).

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Past research efforts to quantify seismic demands on acceleration-sensitive NSCs include simplified methodologies that do not explicitly account for the most important sources of uncertainty in the process (Singh et al. 1993, Villaverde 1997). In addition, current seismic design provisions for acceleration-sensitive NSCs are based on probabilistic estimates of peak ground accelerations that are combined with amplification factors based on the location of the component within the structure as well as the dynamic characteristics of the supporting structure and the component. These amplification factors do not necessarily account for the most important sources of uncertainty in the estimation of component seismic demands and do not explicitly address the influence that inelasticity of the primary structure has on the dynamic responses of acceleration-sensitive NSCs. Sankaranarayanan and Medina (2007) have shown that inelasticity in the primary structure plays an important role in terms of the magnitude and distribution of peak component acceleration demands, and hence, this study addresses both elastic and inelastic primary structures. The objective of this paper is to illustrate a probabilistic methodology geared towards the quantification of PCA demands through the development of structure-specific component acceleration hazard curves. This methodology is based on the application of Probabilistic Seismic Demand Analysis through the implementation of the Total Probability Theorem. Although this methodology is able to incorporate both the aleatory and epistemic uncertainties inherent in the quantification of PCA demands, this study focuses on the development of PCA hazard curves and PCA uniform hazard spectra that explicitly account for the aleatory variability (record-to-record variability) present in the process. The sensitivity of PCA hazard curves and PCA uniform hazard spectra to specific structural system and nonstructural component parameters is briefly addressed.

Structural Models and Ground Motions

It is well established that a variety of structural and nonstructural parameters influence the acceleration demands experienced by nonstructural components (Medina et al. 2006, Sankaranarayanan and Medina 2007). These parameters include but are not limited to first mode and higher mode frequencies, structural damping levels, inelasticity of primary structural system, component damping, location of component within the structure, and component stiffness. Thus, in this study, numerical models of structural systems and nonstructural components that account for variations in these characteristics are used to quantify the contribution of each parameter to the overall response of NSCs in terms of peak component acceleration demands.

Two-dimensional models of shear walls and moment resisting frames are used. Elastic beam column elements with nonlinear rotational springs at the ends are used to represent shear wall structures and single-bay frames with nonlinear rotational springs at the beam ends and at the bottom of the first-story columns are used to represent moment-resisting frames. The nonlinear rotational springs exhibit peak-oriented (stiffness-degrading) hysteretic behavior. The base shear strength of each model and the distribution of strength over the height are determined using the equivalent lateral force procedure outlined in FEMA 450 (FEMA 2006) for a location near downtown Los Angeles, CA. For the structural wall systems, yielding is confined at the bottom of the structure. Models with number of stories, N, ranging from 3 to 18 are used in which constant story heights are used throughout the height of the structures (the 18-story structural walls are included to provide a direct comparison to the 18-story moment resisting frame). Stiffnesses for each model are tuned such that the first mode period corresponds to 0.05N, 0.075N, and 0.1N for shear walls and 0.1N and 0.2N for moment-resisting frames. In all models, five percent Rayleigh damping is assigned to the first mode and the mode at which 95% cumulative mass participation is attained.

Nonstructural components are modeled as single-degree-of-freedom (SDOF) systems with component periods ranging from 0.01 to 5 seconds. Components remain in the linear elastic range. Component masses are assumed to be negligible in comparison to the overall mass of the structure; therefore, the results apply to relatively light components. The focus is on the evaluation of components located at the second floor of the structure and at the roof level to account for the contribution of relative height to the overall component response.

The structural models are exposed to 40 ground motions with moment magnitude from 6.5 to 6.9, where the closest source-to-site distance is within the range of 13 to 40 km. The simulations represent site classifications with stiff soils (NEHRP Soil Classification D) (FEMA 2006).

Probabilistic Model

Peak component acceleration hazard curves (i.e., mean annual rate of exceedance of a component acceleration demand versus peak component acceleration demand, $\lambda_{SaC}(s_{aC})$) are estimated for a *given site* and a *given structural* system based on the following probabilistic model:

$$\lambda_{SaC}(s_{aC}) = \int_0^\infty P[S_{aC} > s_{aC} \mid S_a = s_a] \, | d\lambda_{Sa}(s_a) | \tag{1}$$

Where $P[S_{aC} > s_{aC} \mid S_a = s_a]$ is the probability of the peak component acceleration demand exceeding a value, s_{aC}, given the spectral acceleration at the first mode period of the primary structure (S_a). This probabilistic distribution is also conditioned on parameters such as component damping ratio (ξ_c), the ratio of the period of the component to the i^{th} modal period of the primary structure (T_C/T_{Bi}), the location of the component in the structure (relative height, RH), the modal damping ratio of the supporting structure (ξ), and the base shear strength of the supporting structure (γ). For simplicity, this conditional probability is denoted as presented in Eq. 1. Moreover, λ_{Sa} (S_a) is the mean annual frequency of exceedence of first-mode spectral acceleration, which is commonly known as the seismic hazard curve in terms of S_a.

Methodology

The methodology used to produce PCA hazard curves consists of conducting nonlinear time history analyses with scaled ground motions, obtaining floor response spectra for each ground motion intensity level, generating incremental dynamic analyses (IDA) on the component responses, utilizing the IDA curves to estimate $P[S_{aC} > s_{aC} \mid S_a = s_a]$, and combining this information with seismic hazard curves (i.e., implementing Eq.1) to generate peak component acceleration hazard curves and component uniform hazard spectra. Each step in the methodology is described in greater detail in the following subsections. The structural system used to illustrate the methodology corresponds to a 9-story shear wall model with a fundamental period of 0.45s, and components located at the roof level with a 5% component damping ratio.

Nonlinear Time History Analyses

Each structural model is exposed to a scaled ground motion whose intensity is defined by a factor denoted as relative intensity, $RI = S_a(g)/\gamma$, which is equal to the ratio of the ground motion spectral acceleration at the first mode period of the supporting structure, $S_a(g)$ to the seismic base shear coefficient, $\gamma = V_y/W$, where V_y is the base shear strength, and W is the seismically effective weight. The 9-story structural wall used for illustration in this paper has $\gamma = 0.27$.

In this study, 21 values of relative intensity of (.25, .5, 1.0, 1.5…10) are investigated, with RI = .25 representing elastic behavior and RI = 10 representing highly inelastic behavior. Nonlinear time history analyses are conducted using DRAIN-2DX (Prakash et al. 1993). Structural models with various combinations of number of stories and fundamental periods are exposed to the suite of 40 ground motions used in this study, each of which is scaled using all aforementioned relative intensities. For each time history analysis, floor acceleration time histories are recorded at the second-floor and at the roof level of each structure to evaluate the influence of the location of the NSC in the probabilistic quantification of peak component acceleration demands.

Floor Response Spectra

Each of the floor response acceleration time histories obtained from the numerical simulations of the primary structure exposed to ground motions of various intensities is used to develop floor response spectra (FRS). FRS are produced for component damping ratios equal to 1, 2, and 5%. In order to quantify the dependence of PCA demands on the relative ratio of the periods of vibration of the NSCs and the primary structure, two distinct regions are defined to quantify the maximum accelerations experienced by a component. PCA hazard curves are developed for each one of these regions, which are denoted as the Short-Period Region (SPR) ($0 < T_C < 0.5T_{B1}$) and the Fundamental-Period Region (FPR) ($0.5T_{B1} < T_C < 2.0T_{B1}$), where T_{B1} is the fundamental period of the structure and T_C is the period of vibration of the component (Fig. 1). Vertical lines in Fig. 1 represent the boundaries for the short- and fundamental-period regions.

Fig. 1 depicts floor response spectra for the roof level of a 9-story structural wall system with a fundamental period of 0.45s with a base shear coefficient of 0.27 and 5% component damping ratio. Each curve represents a median floor response spectrum for an individual relative intensity value. Median values were calculated using the set of 40 ground motions utilized in this study. Peaks in component spectral accelerations can be observed in the short-period regions for both the second floor level and the roof level, but are less apparent in the fundamental-period region for the second floor level. In the short-period region, peaks for both the second and third mode periods are evident, which cause amplifications in acceleration relative to the peak floor accelerations. These amplifications may be greater than the amplifications in the FPR when the primary structure experiences significant levels of inelastic behavior (e.g., Fig. 1(a)). Within the fundamental-period region, maximum component accelerations do not always increase with increasing inelasticity, and accelerations eventually reach a cap, in this case near 2.5g (e.g., Fig. 1(a)). Fig. 1(b) shows median floor response spectra for the second-floor level. An increase in ground motion intensity generally yielded higher component spectral accelerations. The observations presented in this paragraph illustrate the importance of considering higher mode contributions and inelasticity to the overall response of NSCs to ground motions.

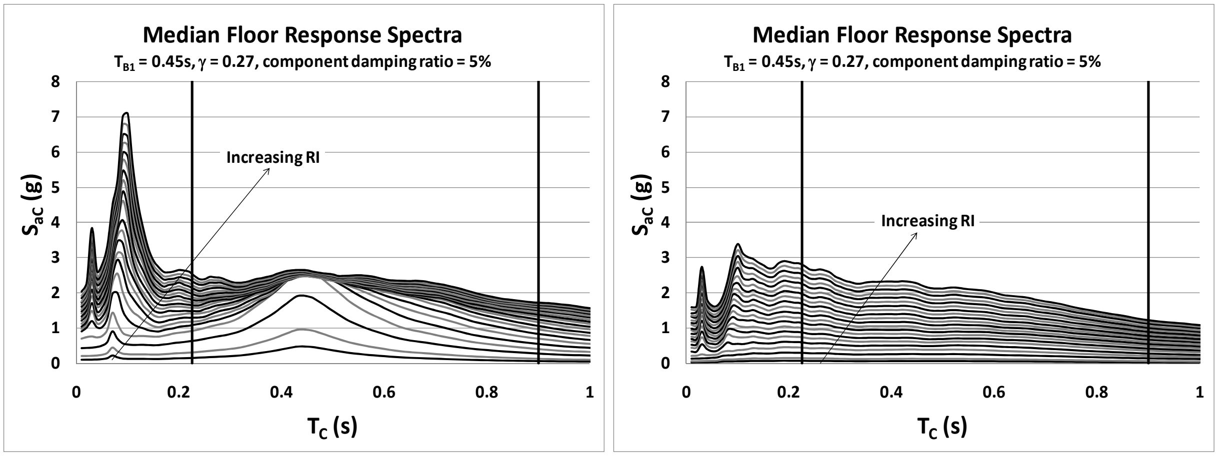


Figure 1. Floor response spectra for 9-story shear wall (a) roof level, (b) 2nd floor level.

Incremental Dynamic Analysis (IDA)

Floor response spectra for each combination of number of stories, fundamental period, component damping, component period region (denoted as SPR and FPR), relative height (i.e., 2nd floor level or roof level), and relative intensity are used to construct IDA curves. IDA curves can be used to numerically evaluate the integrand in Eq. 1 and estimate the probability of exceeding a component spectral acceleration given the spectral acceleration at the first mode period of the supporting structure and the properties of the primary structure.

In this study, for a given site and supporting structure of interest, IDA curves, and hence peak component acceleration hazard curves, are developed for each component period range (FPR or SPR). This implies that for each individual floor response spectra, only one representative peak component acceleration value per component period range is used in the implementation of Eq.1. In each region, this representative value corresponds to the maximum component acceleration demand in the interval of interest. Thus, for an individual ground motion, a point in an IDA curve is obtained by plotting the maximum component acceleration demand for a given ground motion intensity level (i.e., RI) and a given component period range (FPR or SPR) versus Sa(g).

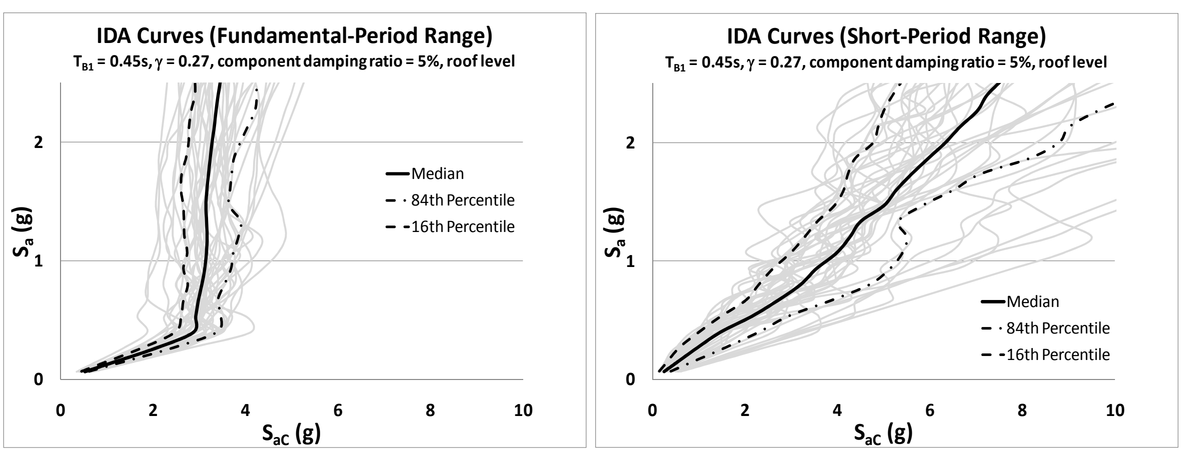


Figure 2. Incremental dynamic analysis curves, 9-story shear wall structure.

As an example, Fig. 2(a) presents IDA curves for 40 ground motions for the fundamental-period range of the 9-story shear wall model used in Fig. 1. The thick line represents the median IDA curve. The record-to-record variability (i.e., aleatory variability) in the component responses is evident in Fig. 2 from the results of individual ground motions, shown in light grey lines. In the fundamental-period range, peak component acceleration demands tend to saturate with increasing ground motion intensity. For example, $S_a(g)$ values near 0.5g produce median component spectral accelerations close to 3g, while a ground motion with an intensity measure five times as strong produces a median spectral acceleration just under 3.75g. This behavior is common to all IDAs produced in this study for the fundamental-period range. IDAs generated for short-period ranges do not behave in the same manner, as increasing the ground motion intensity generally produces higher component spectral accelerations as shown in Fig. 2(b). This behavior is consistent with that of the results presented in Fig. 1(a).

Component Hazard Curves and Component Uniform Hazard Spectra

The implementation of Eq. 1 necessitates the combination of IDA results (i.e., integrand in Eq. 1) with a seismic hazard curve (i.e., variable of integration in Eq. 1). The seismic hazard curves used in this study were computed using OpenSHA (Field et al. 2003) for a site located in Los Angeles, CA. The attenuation relationship developed by Campbell and Borzognia (2008) for the random horizontal component of ground motion was used (see Fig. 3(a)). Peak component acceleration hazard would facilitate the evaluation of expected dollar losses due to damages to acceleration-sensitive NSCs and their attachments through their combination with component seismic fragility information. In addition, a family of peak component acceleration hazard curves of the type shown in Fig. 3(b) corresponding to various fundamental periods of the primary structure and a given component period range can be used to develop Component Uniform Hazard Spectra (CUHS). CUHS are useful to estimate peak component acceleration demands associated with a constant mean annual frequency of exceedance (annual probability of exceedance) as a function of the fundamental period of the supporting structure.

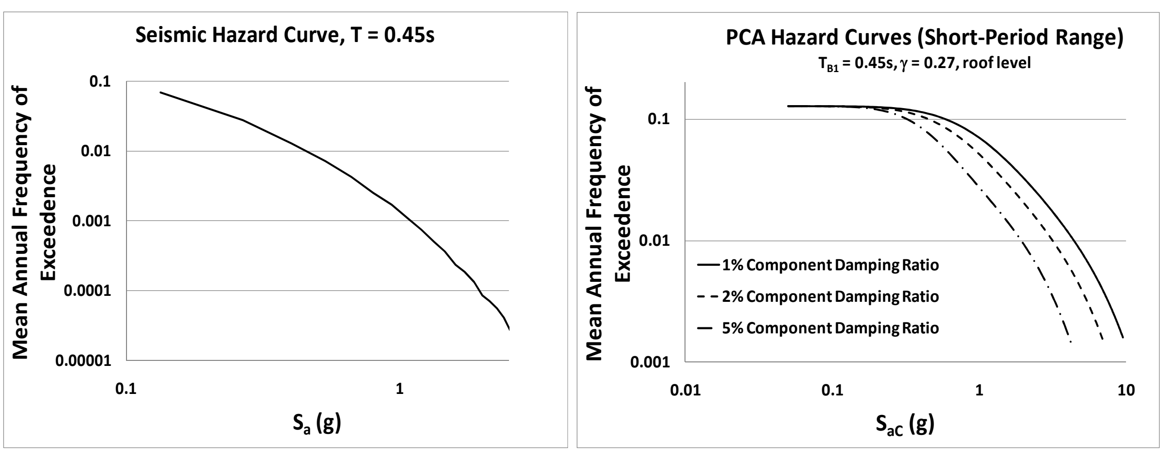


Figure 3. 9-story structural wall, (a) site-specific hazard curve for a location near downtown Los Angeles, CA and (b) peak component acceleration hazard curves.

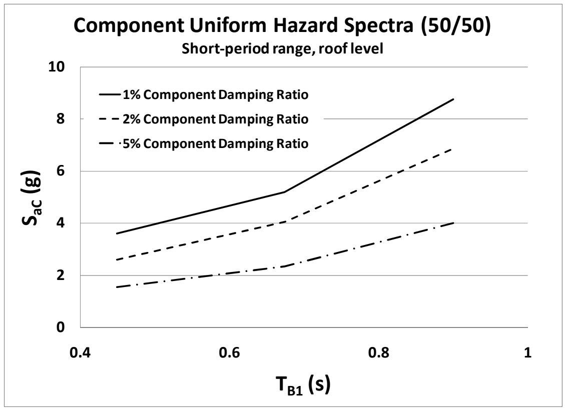
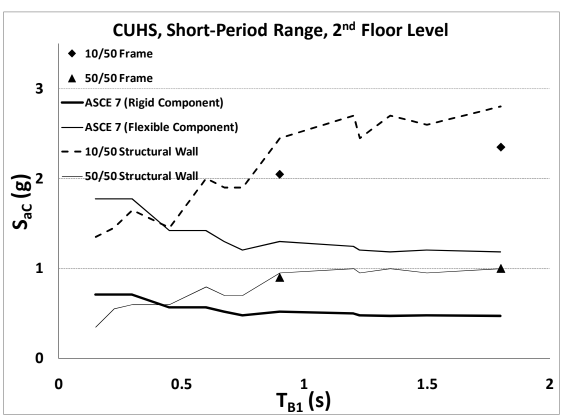


Figure 4. Component uniform hazard spectra for a 9-story shear wall structures, different fundamental periods, 50/50 hazard level.

Fig. 4 shows a 50/50 hazard level CUHS for the roof of various 9-story shear wall structures with T_{B1} = 0.45, 0.68, and 0.90s. Component periods correspond to the short-period region of floor response spectra. In this case, component acceleration demands increase with longer periods and with lower component damping ratios. CUHS would provide estimates of peak component acceleration demands that incorporate some of the most important sources of uncertainty.

Component Uniform Hazard Spectra: Structural Walls and Moment-Resisting Frames

CUHS are generated for the family of structural wall and moment-resisting frame systems used in this study following the methodology described in the preceding section. Representative results are depicted in Fig. 5. Only moment-resisting frames with fundamental periods of 0.9s and 1.8s are included. For those cases in which fundamental periods of vibration of the primary structure overlap (e.g., a 0.05N 6-story structure and a 0.1N 3-story structure), the maximum acceleration is plotted. In these circumstances, the smallest acceleration values were typically within 15% of the largest value. Although the short-period range is intended to represent rigid components, a small fraction of components would fall into the flexible component category described in ASCE/SEI 7 ($T_C \geq 0.06$s) (ASCE 2005).



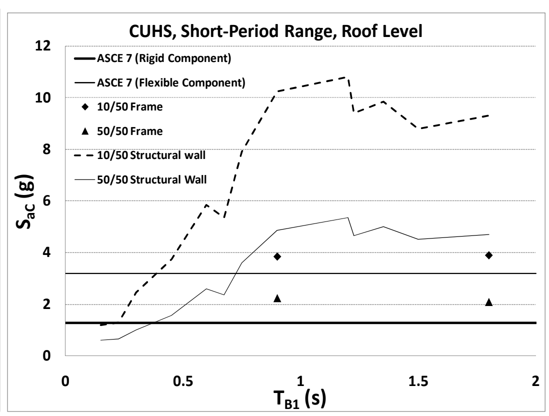


Figure 5. Component uniform hazard spectra, structural wall and frame models with various fundamental periods of vibration, T_{B1}, component damping ratio = 5%, (a) 2nd floor level and (b) roof level.

Fig. 5 shows CUHS values estimated in this study along with the code-prescribed acceleration values for NSCs located at the second-floor and roof levels. Equivalent code values for component acceleration were determined based on the ASCE/SEI 7 seismic design procedure used to estimate component seismic forces. ASCE/SEI 7 predicts peak ground accelerations using $0.4S_{DS}$, where S_{DS} is the design spectral response acceleration parameter at short periods. These values are amplified using a factor a_p, defined as 1 for rigid components and 2.5 for flexible components. To account for amplification along the height of a structure, these values are multiplied by a factor equal to $(1+2z/h)$, where h is the average roof height of the structure above the base and z is the height above the base of the point of attachment of the component. Both the importance factor and the component response modification factor are taken as one.

For shear wall structures and components located near the bottom of a structure (Fig. 5(a)), probabilistic estimates of component acceleration demands consistently exceed code values for rigid components. For flexible components in the short-period range, only the 10/50 level values exceed the code-specified seismic demands for $T_{B1} > 0.5$s. Components located at the roof level of the shear wall structures consistently exceed design levels for both rigid and flexible components, in many cases by approximately 50%. This is particularly true for $T_{B1} > 0.5$s.

For moment-resisting frames (individual data points) with fundamental periods of vibration of 0.9s and 1.8s, both 10/50 and 50/50 CUHS values in the short-period range exceed the design values for rigid components. Only the 10/50 values exceed the design values for flexible components, which represent the maximum design level prescribed by ASCE/SEI 7. A comparison of Fig. 5(a) and 5(b) demonstrates that in this case NSCs located at the roof level experience demands that are much closer to the code-specified values. This implies that code-prescribed seismic demands on rigid components and their attachments, as defined by the short-period region, could potentially be underestimated based on the results obtained in this study.

Probabilistic estimates of component accelerations within the short-period range are similar for shear wall and moment resisting frame models near the bottom of the structure (Fig 5(a)). However, for the roof level, estimates of accelerations are greater in structural wall models than in corresponding moment resisting frames, which is most likely due to the greater influence that higher modes have in the response of structural walls.

Probabilistic estimates of component accelerations within the fundamental-period range (not shown in Fig. 5) were typically below the design level in ASCE/SEI 7 when the hazard level is 50/50 and above the design level when the hazard level is 10/50.

In general discrepancies between CUHS and ASCE/SEI 7 values are likely the result of (a) the method used to determine design component accelerations; (b) the assumptions implicit in code equations, e.g., structural response dominated by one mode of vibration, lack of explicit consideration of uncertainties in component responses; (c) the fact that the probabilistic approach is highly site-dependent, so these results do not represent values obtained at other locations, and (d) analyses based exclusively on numerical models. Moreover, the relatively large CUHS values in Fig. 5 are due in part to the use of linear elastic NSCs during the simulations.

Conclusions

This paper describes a probabilistic approach useful to generate peak component acceleration (PCA) hazard curves and component uniform hazard spectra (CUHS) for acceleration-sensitive nonstructural components (NSCs). Current design criteria do not explicitly account for some of the major sources of uncertainty present in the estimation of PCA demands. This is an important issue given that a substantial portion of repair costs in structures following seismic events is associated with damages to NSCs. As illustrated in this paper, explicit consideration of the aleatory variability present in the quantification of peak component acceleration demands provides estimates of component seismic forces that may differ significantly from the seismic design forces on components and attachments estimated following seismic design code criteria. For instance, CUHS values for the specific case of NSCs attached to structural walls and frames with periods from 0.15s to 1.8s in a stiff-soil site in Los Angeles, CA demonstrated that, in some cases, PCA demands obtained using ASCE/SEI 7 design procedures may be underestimated. In this case, this was particularly important for NSCs with periods of vibration that are less than half of the fundamental period of vibration of the supporting structure, i.e., short-period range.

The probabilistic approach illustrated in this study is also useful for conducting seismic performance assessment studies in which specific performance levels are to be evaluated for acceleration-sensitive components. Moreover, the peak component acceleration hazard curves generated in the process can be combined with component seismic fragility curves for estimation of expected economic losses associated with seismic damage to NSCs. Another advantage of the proposed methodology is the explicit incorporation of structural and nonstructural parameters that significantly influence the response of acceleration-sensitive nonstructural components.

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