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SEISMIC RESPONSE OF A TALL BUILDING TO RECORDED AND SIMULATED GROUND MOTIONS

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ABSTRACT

Seismological modeling technologies are advancing to the stage of enabling fundamental simulation of earthquake fault ruptures, which offer new opportunities to simulate extreme ground motions for collapse safety assessment and earthquake scenarios for community resilience studies. With the goal toward establishing the reliability of simulated ground motions for performance-based engineering, this paper examines the response of a 20-story concrete moment frame building analyzed by nonlinear dynamic analysis under corresponding sets of recorded and simulated ground motions. The simulated ground motions were obtained through a larger validation study via the Southern California Earthquake Center (SCEC) Broadband Platform (BBP) that simulates magnitude 5.9 to 7.3 earthquakes. Spectral shape and significant duration are considered when selecting ground motions in the development of comparable sets of simulated and recorded ground motions. Structural response is examined at different intensity levels up to collapse, to investigate whether a statistically significant difference exists between the responses to simulated and recorded ground motions. Results indicate that responses to simulated and recorded ground motions are generally similar at intensity levels prior to observation of collapses. Collapse capacities are also in good agreement for this structure. However, when the structure was made more sensitive to effects of ground motion duration, the differences between observed collapse responses increased. Research is ongoing to illuminate reasons for the difference and whether there is a systematic bias in the results that can be traced back to the ground motion simulation techniques.

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ABSTRACT

Seismological modeling technologies are advancing to the stage of enabling fundamental simulation of earthquake fault ruptures, which offer new opportunities to simulate extreme ground motions for collapse safety assessment and earthquake scenarios for community resilience studies. With the goal toward establishing the reliability of simulated ground motions for performance-based engineering, this paper examines the response of a 20-story concrete moment frame building analyzed by nonlinear dynamic analysis under corresponding sets of recorded and simulated ground motions. The simulated ground motions were obtained through a larger validation study via the Southern California Earthquake Center (SCEC) Broadband Platform (BBP) that simulates magnitude 5.9 to 7.3 earthquakes. Spectral shape and significant duration are considered when selecting ground motions in the development of comparable sets of simulated and recorded ground motions. Structural response is examined at different intensity levels up to collapse, to investigate whether a statistically significant difference exists between the responses to simulated and recorded ground motions. Results indicate that responses to simulated and recorded ground motions are generally similar at intensity levels prior to observation of collapses. Collapse capacities are also in good agreement for this structure. However, when the structure was made more sensitive to effects of ground motion duration, the differences between observed collapse responses increased. Research is ongoing to illuminate reasons for the difference and whether there is a systematic bias in the results that can be traced back to the ground motion simulation techniques.

Introduction

Modern performance-based earthquake engineering (PBEE) methods rely heavily on the use of nonlinear response-history analysis to determine engineering demand parameters from the onset

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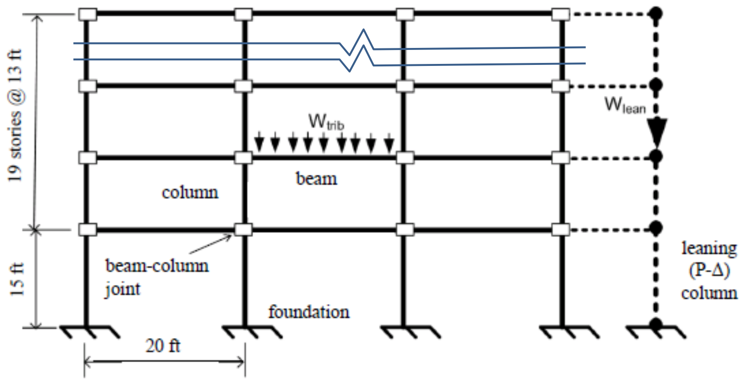
of damage up to collapse [1]. The use of appropriate ground motions, alongside proper modeling of nonlinear structural behavior and inherent uncertainties [2, 3], is a crucial link between seismic hazard and structural response and has received much research attention in recent years [4, 5]. Constrained by a limited database of recorded ground motions on one side and driven by the need of practicality on the other, most engineering applications to date utilize modification of recorded ground motions, e.g. by spectral matching or by amplitude scaling based on the intensities estimated by empirical ground motion prediction models (GMPMs) [6]. Particularly for collapse assessment, this process may involve amplitude scaling of records to several times their original intensity. Although very practical, such approaches can potentially lead to biased estimates of structural response. For example, it has been demonstrated [7] that improper execution of spectral matching can introduce bias in structural response due to artificial reduction of the spectral variability. Moreover, simple amplitude scaling of recorded motions overlooks fundamental seismological aspects that influence the frequency content, duration, and other characteristics of ground motions.

Parallel to advances in earthquake engineering, significant research breakthroughs and enabling technologies have been made within the earthquake science community. In particular, the recent development of wave propagation simulations [8, 9] that incorporate fundamental fault rupture and site-specific characteristics provide an attractive alternative to the use of recorded ground motions that are modified based on idealized parameters predicted by GMPMs. Ultimately, simulated ground motions offer tremendous potential to characterize extreme earthquake ground motions, including spatial correlations that are necessary to simulate regional effects on distributed infrastructure and communities. For more information on current thrusts in ground motion simulation, the reader is referred to [10] and references therein.

An important step toward utilizing simulated ground motions in performance-based engineering is validation to demonstrate that simulated ground motions can reliably capture features that have a significant effect on structural response. As part of a broader objective towards exploring the needs and opportunities for using simulated ground motions in performance-based engineering, this paper examines the structural response of a tall building subjected to recorded and simulated ground motions. Recognized important aspects of ground motions – namely, spectral shape and significant duration [11, 12] – are explicitly taken into account during selection of the ground motions to help ensure that the ground motion sets are comparable, insofar as can be assessed using idealized parameters. Structural response, evaluated by nonlinear dynamic analysis, is examined at different intensity levels from the onset of damage up to collapse, to investigate whether a statistically significant difference exists between the responses to simulated and recorded ground motions.

Tall Building Model Description

Tall building used in this study is an archetype model of 20-story reinforced concrete special moment frame that is representative of office buildings in California. The building was designed as part of a previous benchmark study [13], according to the governing provisions of the 2003 IBC, ASCE7-02 and ACI 318-02. As shown in Fig. 1, the frame is idealized as a 2D analysis model using OpenSees [14]. The nonlinearities are captured in concentrated plasticity models in panel zones and plastic hinges at the ends of columns and beams. Lumped plastic hinges are modeled using the phenomenological Ibarra-Medina-Krawinkler model [15], which has been previously calibrated to capture the monotonic and cyclic deterioration of concrete members that can lead to sidesway collapse. When performing response history analyses, unmodeled energy dissipation is approximated using Rayleigh damping of 5% critical assigned to the fundamental mode period T_1 and to one fifth to the period, $T = 0.2T_1$.



Model periods: $T_1 = 2.63$s; $T_2 = 0.85$s; $T_3 = 0.45$s

Figure 1. Analysis model of the frame [13]

Source of Ground Motions and Selection Procedure

For this study, two groups of comparable ground motion sets were assembled (designated as groups CS and CSDS), where each group contains two sets of 48 ground motions. The first set within a group consist of recorded ground motions while the second set contains simulated ground motions. In all cases the recorded ground motions were selected from the PEER Next Generation Attenuation (NGA) database [16] while the simulated motions were chosen from a database of motions developed by the Southern California Earthquake Center (SCEC) using what is termed their Broadband Platform (BBP) [17]. The database of ground motions consists of simulations of historical earthquake events that were generated using the SCEC BBP as part of a large ground motion simulation and validation project. As summarized in Table 1, this included historical earthquakes with magnitudes ranging from Mw of 5.9 to 7.3. As part of the SCEC simulation effort, fifty realizations for each of the five historical scenarios were generated, and two horizontal ground motion acceleration time histories were developed at about forty ground motion stations in each scenario. In the broader SCEC validation study, six different earthquake simulation models were used, but our study only considered the simulated ground motions run using the Graves and Pitarka (2010) model [8]. In total, the database used in our study includes about 18,800 simulated ground motion records (over five events, fifty realizations, and forty sites with two horizontal ground motion components).

Table 1. Source of simulated ground motions

Scenario	**Magnitude (Mw)**	**BBP run**	**Ground motion model**
Loma Prieta	6.9	13.5	Graves and Pitarka (2010)
Northridge	6.7	13.5	
Whittier Narrows	5.9	13.5	
North Palm Springs	6.1	13.6	
Landers	7.3	13.5	

*included all realizations of each scenario → 18,800 simulated GMs

The two groups of ground motion sets were each developed based on a hypothetical site scenario event with the mean M, R and ε(2.6s) values of 6.5, 10km and 1, respectively. Such values were chosen to be within range of available BBP simulations. The Campbell and Bozorgnia [18] GMPM was used for spectral amplitudes, while Kempton and Stewart [19] prediction model was used for significant durations. Correlations between spectral amplitudes at different periods as well as between spectral amplitudes and significant durations were obtained using [20, 21]. For each of the two groups, a set of recorded ground motions and a set of simulated ground motions were selected to match either a specified conditional spectrum (CS) target [4, 5] or a generalized conditional intensity measure (GCIM) [22] target, here referred to as CSDS target, that additionally included 5-75% significant durations (Ds5-75). Such selection scheme was used to emulate the procedure by which recorded ground motions are selected in practice and to allow for consideration of ground motion properties that primarily affect structural response. The matching was based on a weighted comparison of conditional spectra and significant durations, as summarized in Table 2.

Shown in Fig. 2 is an example of the response spectra of selected recorded and simulated ground motions in group CS, and shown in Fig. 3 is a comparison of the mean logarithmic Sa(T), standard deviation of logarithmic Sa(T) and significant durations for selected recorded and simulated ground motions in the CS group along with distribution of significant durations for the CSDS group. Given the large databases of recorded and simulated ground motions to choose from, it is possible to obtain good agreement with the target scenarios (equally good fits of mean Sa values and variances were obtained for the CSDS group as well). Finally, it can also be noted that the distribution of significant durations of selected BBP motions is very close to the conditional target for the hypothetical scenario whereas the durations of NGA motions significantly deviate from it. This was expected given the seismological properties of BBP simulation scenarios and the fact that durations were not explicitly considered when performing the selection for the CS group. In contrast, when durations became part of the selection criteria in the CSDS group, the distributions of significant durations of both NGA and BBP motions match well with the target conditional distribution of significant durations.

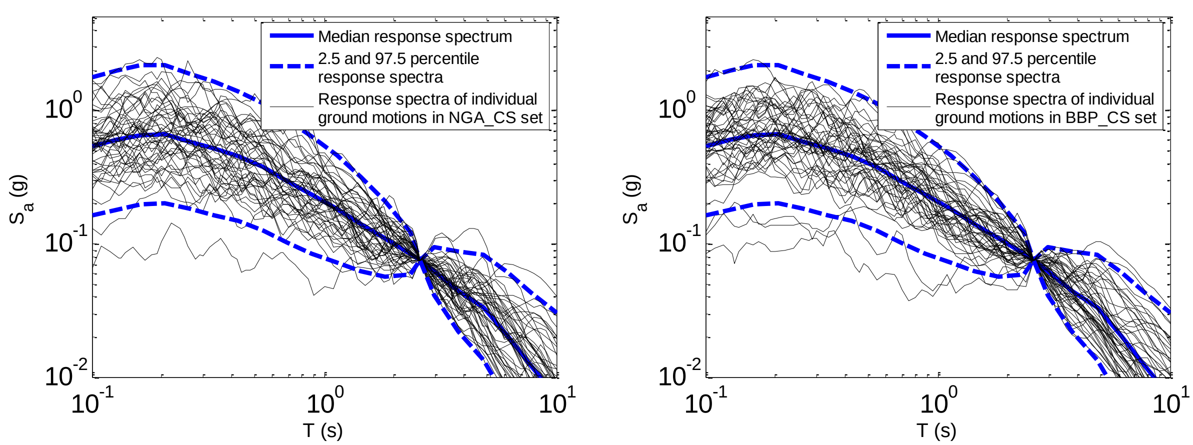


Figure 2. Response spectra of selected (a) recorded and (b) simulated ground motions (CS group)

Table 2. Details of ground motion selection for selected groups of ground motion sets

CS group	**Set name:**	**NGA_CS**	**BBP_CS**
	IM target	Conditional spectrum	Conditional spectrum
	Weights	Sa: 100%	Sa: 100%
CSDS group	**Set name:**	**NGA_CSDS**	**BBP_CSDS**
	IM target	Conditional spectrum & duration (5-75%)	Conditional spectrum & duration (5-75%)
	Weights	Sa: 80%, Ds: 20%	Sa: 80%, Ds: 20%

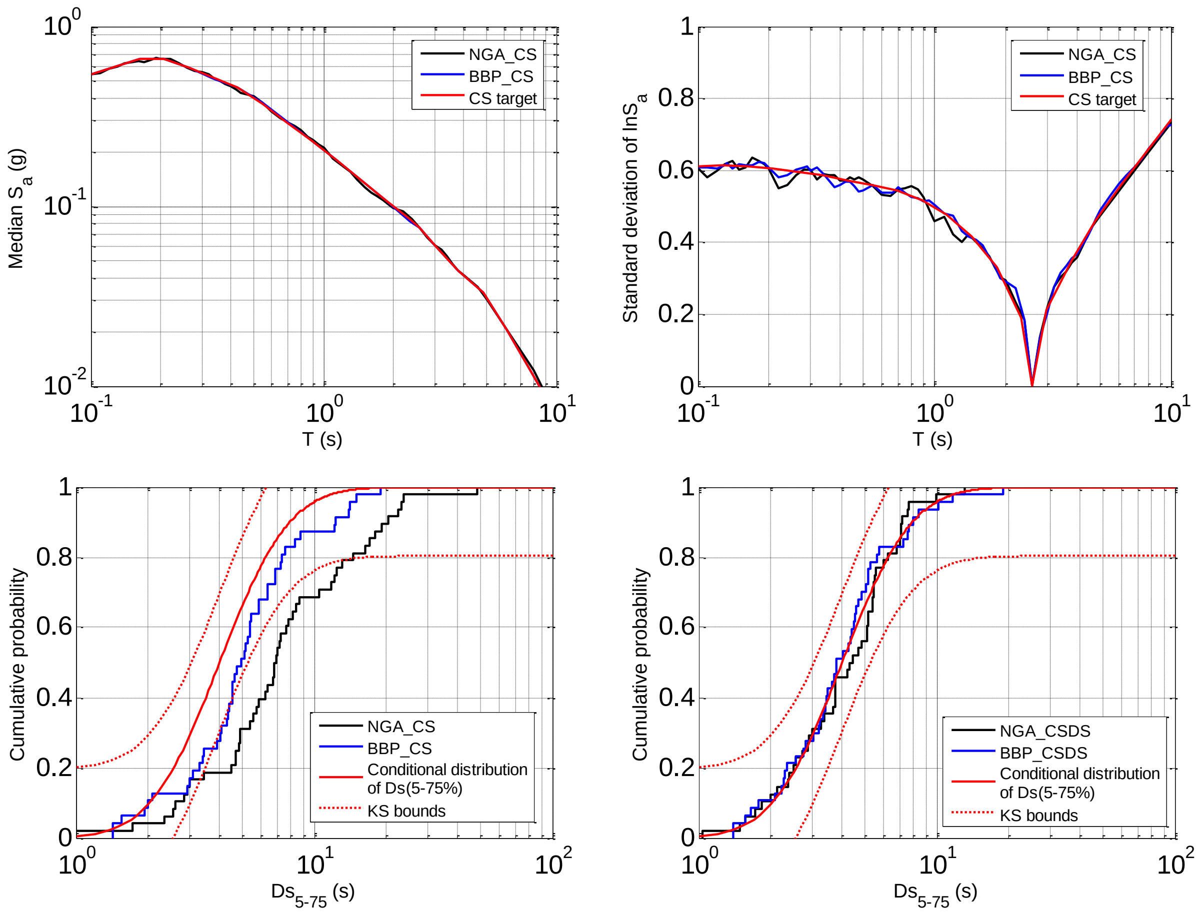


Figure 3. Match between selected sets of ground motions: (a) exponential of the logarithmic mean spectra (CS group), (b) standard deviation of logarithmic spectrum (CS group), (c) significant duration Ds 5-75% (CS group), (d) significant duration Ds 5-75% (CSDS group); a modified version of the Jayaram et al. [23] algorithm was used to perform ground motion selection; KS bounds shown in figures (c) and (d) indicate 95% confidence bounds for the Kolmogorov-Smirnov test

Response History Analysis and Hypothesis testing

Ground motions from the selected sets were systematically scaled to different intensity levels and response history analyses were performed to obtain engineering demand parameters (EDPs) for the 20-story moment frame. To evaluate EDPs of story drift, floor accelerations, and story shears at selected intensities, each of the motions was scaled to target intensities based on their spectral acceleration at the fundamental period. To evaluate collapse capacity, each of the motions was scaled up to the point of collapse following an incremental dynamic analysis (IDA) [24] approach. All of the analyses were performed on Texas Advanced Computing Center (TACC) Stampede supercomputer using OpenSeesMP version 2.4.0.

To establish whether there is a statistically significant difference between structural responses to recorded and simulated ground motions, a hypothesis testing method was used, as proposed in [25]. In this approach, hypothesis testing determines whether the observed differences in the calculated structural response to recorded and simulated motions are statistically significant. For example, assuming that the difference between mean responses equals zero (null hypothesis) implies that the differences between responses are solely due to finite sample sizes and not the result of inherent differences in the simulation data. This null hypothesis can be rejected if the sample means are significantly apart such that the difference is unlikely to have been observed if the true means were the same. It is assumed that, under the null hypothesis, the difference between sample mean of EDP_{sim} and EDP_{rec} follows a normal distribution with mean zero and sample standard deviation of EDP (pooled estimate of the standard deviation is used here due to it having a lower standard error). The null hypothesis can be rejected if the observed difference in the mean values falls outside of pre-specified percentiles of the assumed normal distribution (2.5 and 97.5 percentiles were used in this paper), which enables the derivation of rejection region boundaries. It should be noted that failure to reject the null hypothesis does not mean that the null hypothesis is accepted; it only implies insufficient evidence for its rejection.

Hypothesis tests as described above were carried out for the medians of peak story drift ratios, peak floor accelerations and peak story shears at different intensity levels. The results are presented in the following section.

Results and Discussion

Since any differences in the response quantities are expected to increase with the degree of nonlinearity, the EDPs are compared at the highest $Sa(T_1)$ intensity at which point no collapses occur. Results are described for ground motion sets in group CS, which are similar to findings for sets in group CSDS. The calculated median and dispersion for the peak story drifts, peak floor accelerations, and peak story shears are shown in Figs. 4, 5 and 6, respectively. Included in the plots of median values are the 2.5 and 97.5 percentile rejection boundaries for the null hypothesis. Overall, the responses are quite similar for the recorded (NGA) and simulated (BBP) sets, where both responses generally lie within the rejection boundaries. For peak story drift ratios in the upper (15-20) stories, the simulated motions produce smaller responses than the recorded motions with values being very close to the rejection region boundary. Analyses of response at lower intensity levels confirm that these slight differences observed in Figs. 4 through 6 are even smaller when the behavior is less nonlinear. This is expected, since the simulated and recorded ground motions were selected to match their elastic response spectra.

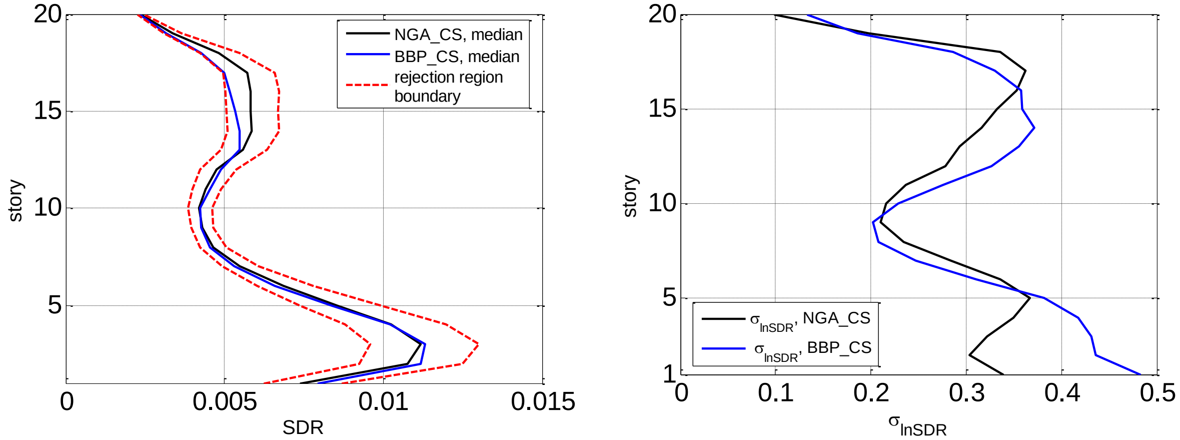


Figure 4. (a) Median and (b) dispersion of story drift ratios (CS group)

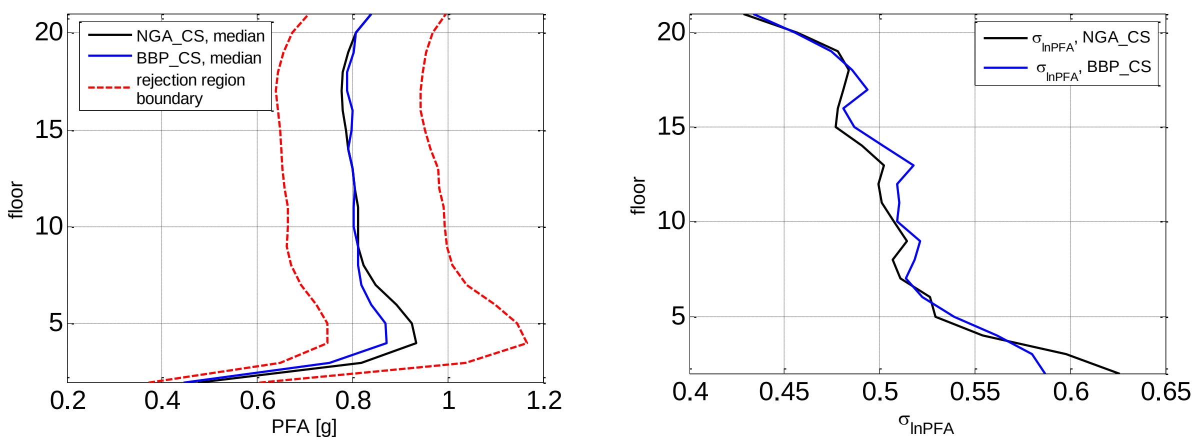


Figure 5. (a) Median and (b) dispersion of peak floor accelerations (CS group)

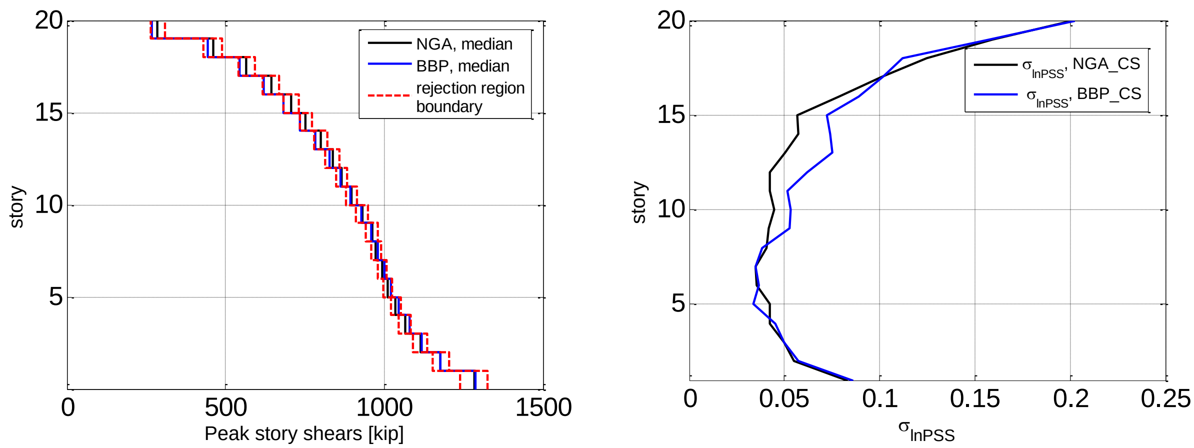


Figure 6. (a) Median and (b) dispersion of peak story shears (CS group)

The results of the IDA up to collapse for both groups of ground motion sets are shown in Fig. 7 and summarized in Table 3. A very good agreement between median collapse capacities and dispersions obtained using the recorded and simulated ground motions can be seen. Differences between median collapse capacities range from 2% to 4% and no statistically significant differences were observed.

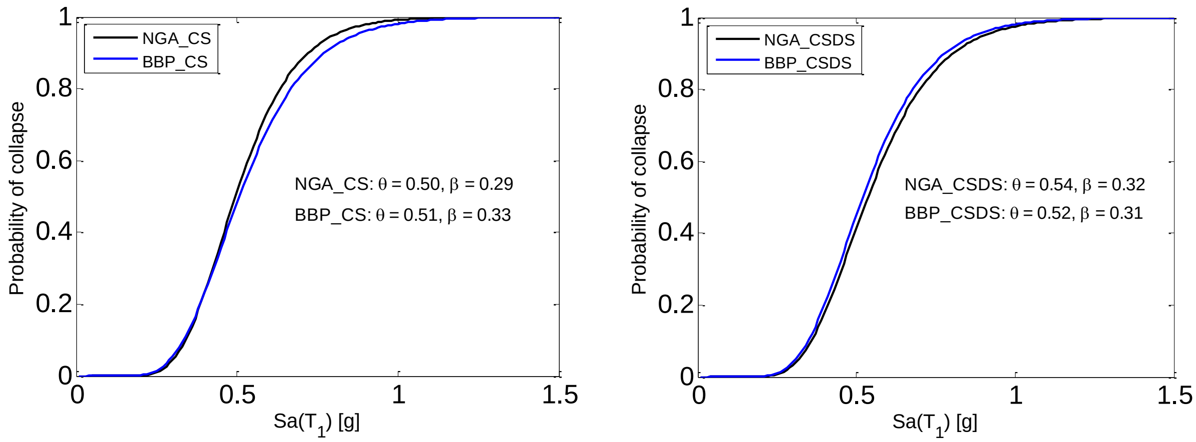


Figure 7. Collapse fragilities for ground motion sets from CS and CSDS groups, $\lambda / \lambda 0 = 1.0$; θ and β represent the median collapse capacity and dispersion, respectively

Since significant durations of ground motions in sets NGA_CS and BBP_CS are relatively different (Fig. 3c), a very close match between collapse fragilities for the two cases would suggest that the analyzed structure is not overly sensitive to effects of duration. To further investigate simulated ground motions under the circumstances where significant durations do play a larger role, the hysteretic energy dissipation capacity of the structure was reduced by uniformly scaling the hysteretic energy dissipation capacities of all plastic hinges in the structure to 0.4 of their original value (indicated as $\lambda / \lambda 0 = 0.4$), thus artificially making the structure more sensitive to effects of duration. Collapse analyses were then repeated and the results are given in Table 3 and Fig. 8. The resulting difference in median collapse capacities for the CS group ground motions increased from 2% to 17%, with BBP_CS set having larger median collapse capacity. This seems to be a reasonable result given that NGA_CS ground motions have longer significant durations. Contrary to expectation, the difference between median collapse capacities for the CSDS group also increased with the BBP_CSDS set having 11% larger median (note that for $\lambda / \lambda 0 = 1.0$ case the NGA_CSDS set has 4% larger median). It can be seen that the difference between dispersions also increased. Although the observed difference for the CSDS group is still not statistically significant, a better fit of the results was expected. Additional research is currently underway to illuminate and quantify the cause of observed differences and investigate whether there are legitimate reasons for the differences, or whether the ground motion simulation procedures should and can be modified to eliminate this potential bias.

Conclusions

Nonlinear dynamic analyses of a 20-story building were conducted using two groups of comparable sets of simulated and recorded ground motions to investigate whether there are systematic biases in the computed response quantities. Engineering demand parameters, including story drift ratios, floor accelerations and story shears, were calculated at various intensity levels up to the onset of collapse, and Incremental Dynamic Analyses were performed.

Recorded ground motions were selected from the PEER NGA database while the simulated motions were selected from the pool of ground motions generated for five historical earthquakes as part of a SCEC Broadband Platform validation exercise. To ensure that the selected sets are comparable, recorded and simulated ground motions are chosen to match the Conditional Spectrum (including mean and variance of the response spectra) and a target that additionally considers the 5-75% significant durations. Hypothesis testing was used to compare the structural responses to simulated and recorded ground motions. The results indicate that responses to simulated and recorded ground motions are generally similar at intensity levels prior to observation of collapses. In addition, collapse responses are also in good agreement for this structure. However, when the structure was artificially made more sensitive to effects of ground motion duration, the differences in observed collapse responses increased. Additional research is currently underway to illuminate the cause of the difference.

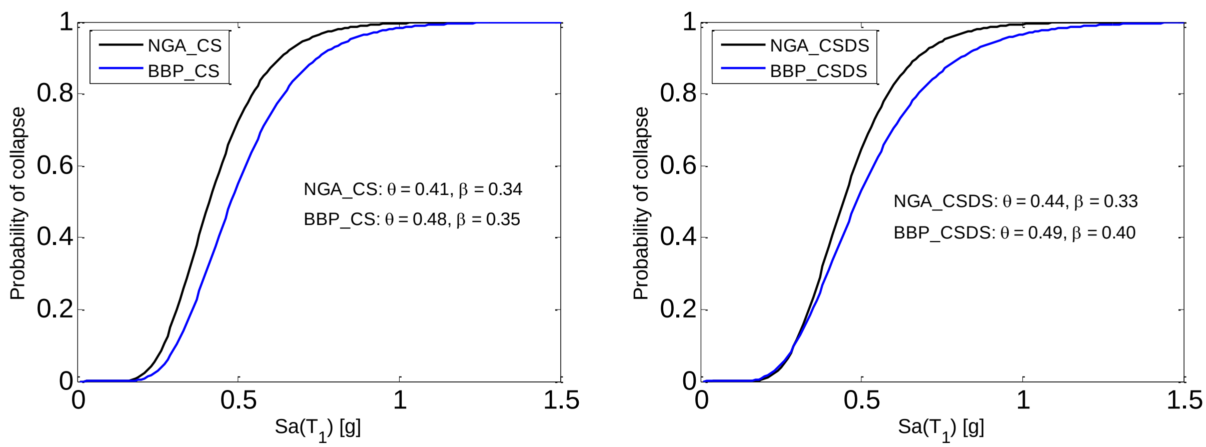


Figure 7. Collapse fragilities for ground motion sets from CS and CSDS groups, $\lambda / \lambda 0 = 0.4$; θ and β represent the median collapse capacity and dispersion, respectively

Table 3. Results of collapse analyses

λ/λ0	**NGA_CS**		**BBP_CS**	
	median Sa, col [g]	σlnSa	median Sa, col [g]	σlnSa
1.0	0.50	0.29	0.51	0.33
0.4	0.41	0.34	0.48	0.35
λ/λ0	**NGA_CSDS**		**BBP_CSDS**	
	median Sa, col [g]	σlnSa	median Sa, col [g]	σlnSa
1.0	0.54	0.32	0.52	0.31
0.4	0.44	0.33	0.49	0.40

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