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Comparison of recorded and simulated ground motions for NZS1170.5-based 3D building response analysis

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ABSTRACT

Validating dynamic responses of engineered systems subjected to simulated ground motions is essential in scrutinising the applicability of simulated ground motions for engineering demand analyses. This paper compares the responses of two 3D building models subjected to recorded and simulated ground motions scaled to the NZS1170.5 design response spectrum, in order to evaluate the applicability of simulated ground motions for use in conventional engineering practice in New Zealand. The buildings were designed according to the NZS1170.5 and physically constructed in Christchurch prior to the 2010-2011 Canterbury earthquakes. 40 recorded ground motions from the 22 February 2011 Christchurch earthquake, along with the simulated ground motions for this event from Razafindrakoto et al. (2018) are considered. The seismic responses of the structures are principally quantified via the peak floor acceleration and maximum inter-storey drift ratio. Overall, the results indicate a general agreement in seismic demands obtained using the recorded and simulated ensembles of ground motions and provide further evidence that simulated ground motions using state-of-the-art methods can be used in code-based structural performance assessments in-place of, or in combination with, ensembles of recorded ground motions.

1 INTRODUCTION

One of the main applications of earthquake ground motion time series (herein GM for brevity) is to conduct the response history analysis for designing new buildings or assessing the performance of existing structures. Despite the conventional use of scaled recorded GMs from past earthquakes to obtain the response of structures, the most critical issue in utilizing them is the paucity of GMs representing the specific-site hazard conditions, especially in the near-fault region. Another restriction is the incompatibility of selected GMs in terms of causal parameters (i.e. faulting style, magnitude, source to site distance, basin effects, etc.) with respect to the ruptures affecting the seismic hazard at the site of interest.

To overcome these limitations, supplementing the ensemble of recorded GMs with simulated ones is considered as a viable option in seismic response analysis of engineered systems. Ground motion simulation methods have been significantly improved in the last decade, enabling engineers to utilize the simulated GMs in obtaining seismic responses of engineered systems (Bijelić et al. 2018). Although using simulated GMs has been accepted by some design codes (Hachem et al. 2010) such as ASCE/SEI7 (ASCE-7 2017), Eurocode 8 (CEN 2004) and NZS1170.5 (Standard New Zealand 2004), they still do not represent the majority of the use cases in engineering practice due to ambiguity of instructions, lack of detailed validations of simulated GMs, and uncertainty in the obtained responses in comparison to those based on recorded GMs. Besides, validation of simulated GMs is essential to inform GM simulators about the quality of simulation methods in terms of their potential shortcomings based on their application in engineering demand analysis. Therefore, a systematic procedure for validation is needed to address the aforementioned concerns, which will help to confidently address the doubts regarding the utilization of simulated GMs in response history analysis of structures and will provide valuable insights and feedback to improve the simulation methods.

To scrutinise the step-by-step validation procedure, a validation matrix was developed by Bradley et al. (2017) which shows the complexity of metrics used in the validation process in terms of spatial extent and intensity measures (IMs). In this matrix, the validation of simulated GMs is addressed with respect to complexity in different levels of IMs, including comparison of qualitative waveforms, elastic/inelastic response spectra, and multi-degree of freedom (MDoF) system responses. Previous studies considered validation at different levels of IM complexity. For example, Bazzurro et al. (2004), Iervolino et al. (2010), Atkinson and Goda (2010), and Galasso et al. (2012) compared the responses of nonlinear single degree of freedom (SDoF) system excited by simulated and recorded GMs. Jayaram and Shome (2012), Galasso et al. (2013), Bijelic et al. (2014), Burks et al. (2015), and Bijelić et al. (2018) validated simulated GMs in terms of the response of MDoF systems.

In the context of the above sentiments, this study is focused on comparing seismic demand of 3D building models designed according to the NZS1170.5 subjected to recorded and simulated GMs of the 22 February 2011 Christchurch event. Two buildings that have been designed and physically constructed based on NZ standards are considered. The 3D nonlinear response history analysis models by consulting engineers are directly utilized (Holmes consulting), and their seismic response is quantified through commonly adopted Engineering Demand Parameters (EDPs) (i.e. peak floor acceleration and inter-storey drift ratio). As-recorded and simulated GMs from the 22 February 2011 Mw6.2 Christchurch earthquake are scaled following the NZS1170.5 procedure, and the computed seismic demands are compared. Through analysis and interpretation of the results, we address whether the structural responses from simulated GMs are comparable to recorded GMs for the considered structural systems, and hence explore the opportunity for their use in engineering design.

2 BUILDING PROPERTIES AND MODELLING

Two buildings were selected as the case-studies for nonlinear dynamic analyses (with their 3D view presented in Figure 1a-b.). The mid-rise building, denoted as Building A, is a six-storey reinforced concrete (RC) with moment resisting frames in both directions, a boundary wall system in the North-South direction, and shear walls in the East-West direction. Also, a lightweight storey has been added to the roof of this building. The high-rise building, denoted as Building B, is a 13-storey with ductile RC walls in the East-West direction and ductile RC coupled walls in the North-South direction. For this building, precast concrete elements were designed for the perimeter truss beams. The fundamental periods, for Buildings A and B, are 0.5 sec and 2.0 sec, respectively.

The 3D nonlinear response history analysis models are analysed by Holmes Consulting Engineers finite element software (ANSR). Beams and columns of these buildings were modelled using the lumped plasticity method, and the wall elements are modelled using the effective fibre-model approach at the ground level interface which can capture flexural yielding. The general nonlinear behaviour of concrete elements are assumed based on FEMA 356 (2000) and ASCE/SEI-41 (2017). The design ductility of 6 for the era of construction and the level of detailing in the elements is considered.

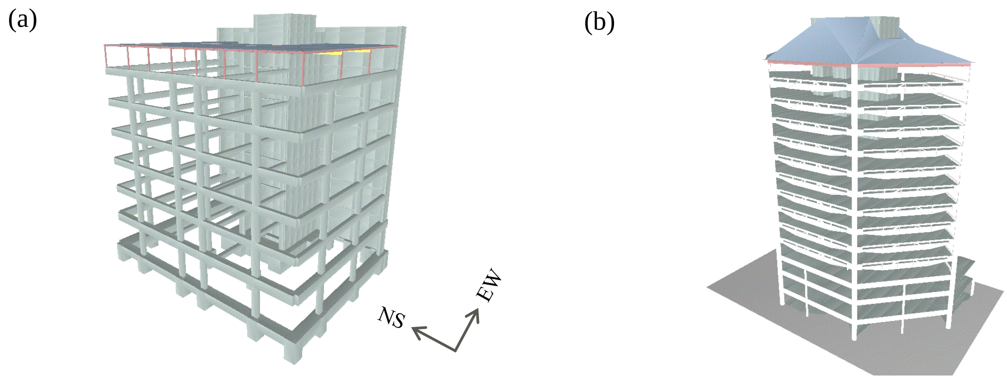

Figure 1: 3D view of (a) Building A; (b) Building B.

3 OBSERVED AND SIMULATED GROUND MOITION SETS

The selected buildings are subjected to GMs from 22 February 2011 Christchurch Earthquake (as one of the devastating events in the seismic history of New Zealand with 185 fatalities and 15 billion USD damage (Quigley et al. 2016)). Records from 40 stations located in the Canterbury region on different soil types from soft soil to rocks (155 m/s < Vs_{30} < 800 m/s) are considered.

Simulated GMs from Razafindrakoto et al. (2018) for this specific event are utilized. This simulation has been conducted using the hybrid broadband method (Graves and Pitarka 2010, 2015). Figure 2a-b illustrate the 5% damped (pseudo) spectral acceleration of the observed and simulated GMs at the considered stations; indicating that the considered GMs span a wide range of GM intensity as a result of variation in the source-to-site distance, soil conditions, among others. Herein, the geometric mean of the two components is considered as the representative of GM in each station.

Simulated and observed GMs are scaled to represent the NZS1170.5 response spectrum at 475 years return period. The scaling is performed such that the differences between scaled and the NZS1170.5 target spectrum is minimized for $0.4T_n$ and $1.3T_n$ range (T_n being the fundamental period of structure). Figure 2c-d compare the median of the scaled GMs with respect to the NZS1170.5 response spectrum. The median of the observed GMs is greater than the simulated ones for periods larger than the scaling range ($1.3T_n$<T<1.4 s) for Building A, while the medians are close for Building B.

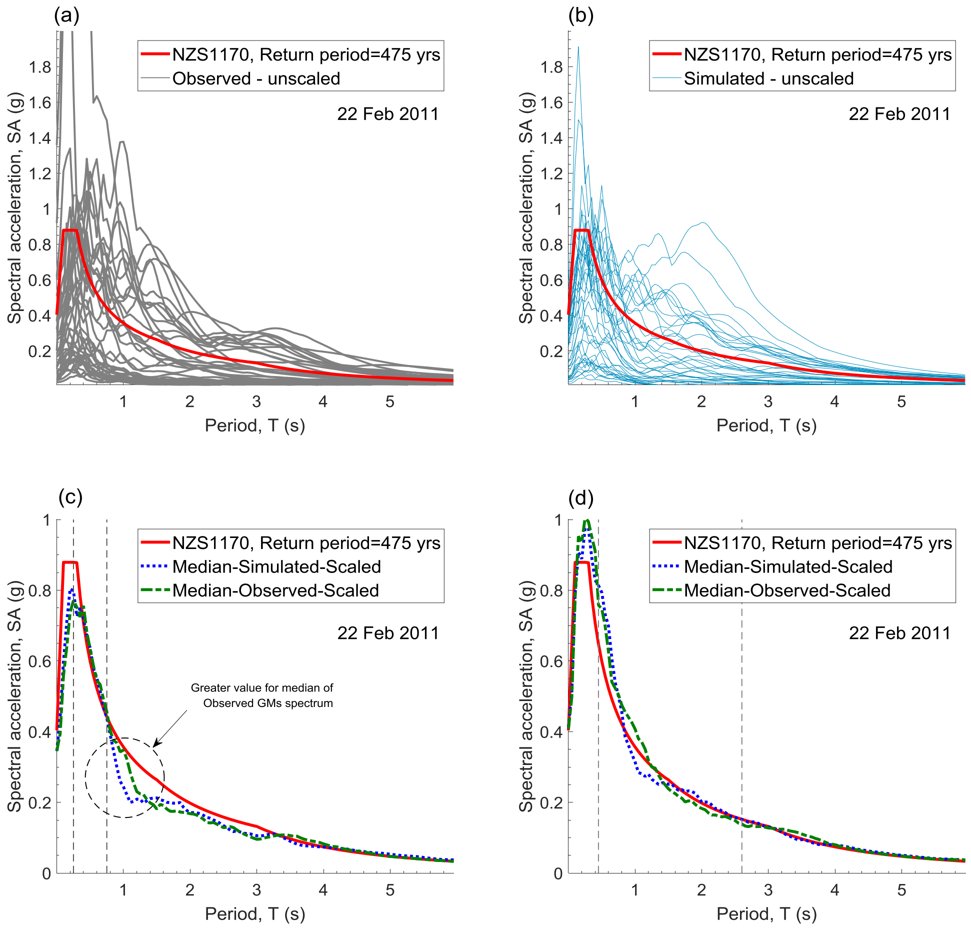

Figure 2: (a-b) Unscaled response spectra for the 40 observed and simulated GMs; (c-d) median of scaled response spectra for Building A and B.

4 COMPARISON OF ENGINEERING DEMAND PARAMETERS

4.1 Considered engineering demand parameters

The peak floor acceleration (PFA) and inter-storey drift ratio (IDR) at each floor are selected as the principal EDPs which are appropriate representatives of damages in non-structural and structural components and commonly used by codes to satisfy the defined design criteria. The considered EDPs are assumed to be lognormally distributed and their geometric mean, 16th, and 84th percentiles are considered to compare responses.

4.2 Comparison between the EDPs

Figure 3 shows the ratio of simulated to observed EDPs (i.e. sim/obs) at each floor centre of mass and the corresponding geometric mean, 16th and 84th percentiles along the height of the buildings. As shown in Figure 3a-b, there is an underestimation from simulated results in the peak floor acceleration and inter-storey drift ratio for Building A, which is higher for inter-storey drift ratio in comparison to peak floor acceleration. This underestimation can potentially be attributed to the greater value for the median of observed GM spectrum as shown in Figure 2c. Since the structure experiences nonlinearity the fundamental period of the system increases out of the GM scaling range where the median of observed GMs are higher than the simulated ones. The maximum difference between the geometric mean of sim/obs values and the one-to-one line (i.e. red line in Figure 3) is 0.89 for peak floor acceleration (at the sixth storey) and 0.77 for inter-storey drift ratio (at the first storey). As shown in Figure 3a-b, the range between the 16th to 84th percentiles for peak floor acceleration is smaller than that for inter-storey drift ratio. Figure 3c-d present a good agreement between the EDPs of simulated and observed responses along the height of the structure for Building B. In contrast to Building A, the range between the 16th to 84th percentiles for peak floor acceleration is higher than that for inter-storey drift ratio for this building. The maximum difference between the geometric mean of sim/obs values and the one-to-one line is 0.93 for peak floor acceleration (at the 12th storey) and 1.064 for inter-storey drift ratio (at the sixth storey).

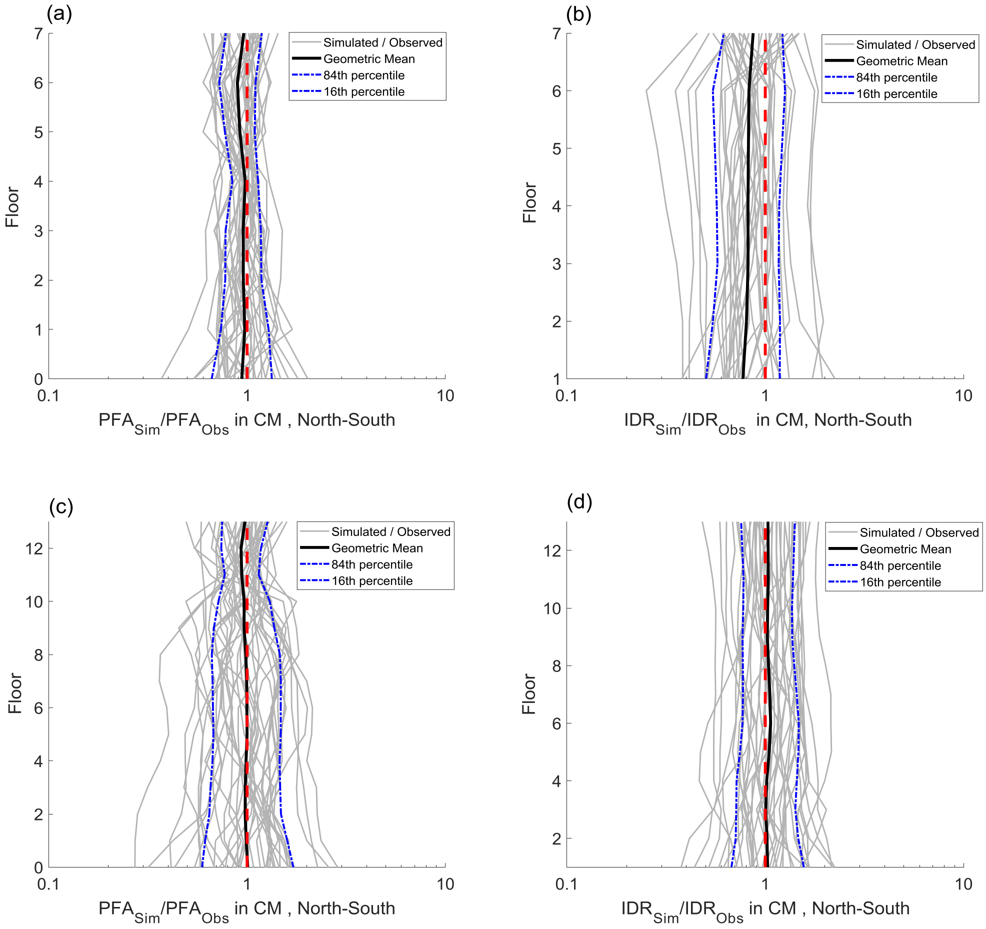

Figure 3: The ratios, geometric mean, and percentiles of simulated to observed responses (a) Building A peak floor acceleration (PFA); (b) Building A inter-storey drift (IDR); (c) Building B PFA; (d) Building B IDR.

4.3 EDPs trends with respect to data sample size

The effect of the sample size of the utilized GMs on the differences between the EDPs from the observed and simulated GMs is scrutinised using the bootstrap technique (Efron and Tibshirani 1994) and hypothesis testing. Bootstrapping helps to find how the observed differences can be explained by a large data set in contrary to the utilized data with a limited size. This is done by drawing random realizations with replacement from the initial set to generate a large ensemble. Also, hypothesis testing is used to check whether there is a systematic difference between two sets of responses. A two-tailed t-test is performed under the null hypothesis at the significance level of 0.05 following the suggested algorithm by Efron and Tibshirani (1994). Since 11 records are suggested as the minimum number of records for nonlinear response history analysis by the recent versions of building codes (ASCE-7 2017), in view of this, 11 records are considered for each random realization. Then the geometric mean of each realization is obtained and compared from 5000 bootstrapped samples. To assess the statistical significance of evidence, a p-value is calculated under the null hypothesis. A p-value less than 0.05 demonstrates that there is significant evidence for rejecting the null hypothesis (i.e. observed and simulated GMs result in significantly different responses). Conversely, there is weak evidence to reject the null hypothesis for the p-values greater than 0.05 (i.e. observed and simulated GMs results are similar).

Figure 4a-b show the geometric mean of all realizations and the 16th and 84th percentiles for selected EDPs of Building A. Figure 4b shows a considerable difference in the geometric mean of inter-storey drift ratio (with simulated results smaller than the observed) while there is a good agreement in peak floor acceleration (Figure 4a). As discussed, this is potentially related to the large median spectral acceleration of observed GMs (Figure 2c) compared to simulated GMs for periods outside the scaling range (T>1.3Tn). The maximum difference between the peak floor acceleration (Figure 4a) occurs at the sixth storey where the value for observed GMs (0.41g) is 10.2% greater than that for the simulated GMs (0.371g) which is statistically significant (p-value=0.0076). The p-values for peak floor acceleration in all stories of Building A except for the fifth and sixth stories are greater than the significance level (0.05), which shows the difference for two set of GMs is not significant in these stories. The maximum difference between inter-storey drift ratios for two groups occurs at the first storey (Figure 4b) where the value for observed GMs (0.00349) is 24.5% greater than that for the simulated GMs (0.00263) which is statistically significant (p-value=0.0016). Overall, the calculated p-values for inter-storey drift ratio demonstrate that the difference between the two sets of responses is statistically significant for all stories.

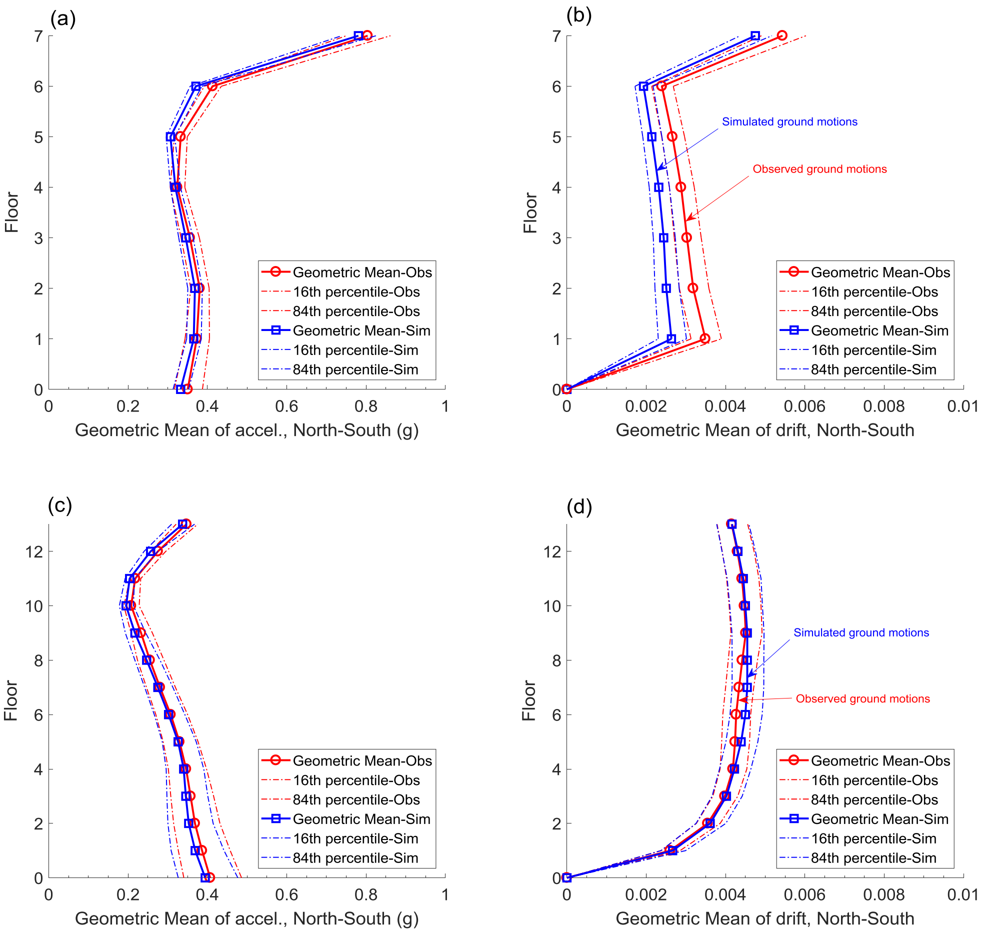

Figure 4: Geometric mean and percentiles of bootstrapped samples (a-b) Building A peak floor acceleration (PFA) and inter-storey drift ratio (IDR); (c-d) Building B PFA and IDR.

Figure 4c-d show the geometric mean of all realizations and the 16th and 84th percentiles for selected EDPs of Building B. These figures generally demonstrate a good agreement between the EDPs of two GM sets. The maximum difference between the peak floor acceleration and inter-storey drift ratio of two groups of the observed and simulated responses is significantly small (i.e. 1%). Performing hypothesis testing and calculating p-values, shows that the differences are not statistically significant for both EDPs for Building B (as all values are greater than 0.05).

5 CONCLUSION

State-of-the-art ground motion (GM) simulation methods provide opportunities to utilize simulated GMs as a supplement or in-place of recorded GMs ensembles for seismic hazard and response history analysis (specifically when there is a paucity of recorded GMs consistent with site-specific hazard in the database). However, validation needs to be conducted to scrutinise the quality of simulated GMs prior to their utilization. The validation process (which can be performed at different levels), gives valuable insights for engineers in using simulated GMs as inputs for response history analysis and valuable feedback for GM simulators to improve the simulation methodologies.

In the context of validation for GMs compatible with the design code spectrum, this paper compares responses of two 3D structural models, a 7-storey and a 13-storey building, subjected to 40 recorded and 40 simulated GMs of the 22 February 2011 Christchurch earthquake scaled to the NZS1170.5 spectrum. The models represent real buildings, which were designed based on the NZS1170.5 and physically constructed before the Canterbury earthquake sequences. Attempts are made to investigate the similarities and differences between the peak floor acceleration and inter-storey drift ratio of the systems subjected to observed and simulated GMs when the code instructions are followed in the analysis and design.

The results indicate a general agreement between the peak floor acceleration calculated by the simulated and recorded GMs for the two buildings. For the 13-storey building, the hypothesis test results indicate that the differences in inter-storey drift ratio are statistically small while they are statistically significant for the 7-storey building (which can potentially be attributed to the greater value of the median of the observed GMs spectrum outside the applied scaling range suggested by the NZS1170.5). The findings of this paper demonstrate the applicability of simulated GMs in the context of New Zealand-specific events and buildings when the code-based approach in response history analysis is followed.

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