Validation of NZ small-magnitude ground-motion simulations using complex structural systems

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ABSTRACT

This study assesses the need to consider complex structural models in ground-motion simulation validation. It develops a novel statistical framework to quantify the proportion of the differences in structural responses under simulated and observed ground motions that can be explained by corresponding differences in simple ground motion intensity measures.

Two steel special moment frames are analysed under simulated and observed ground motions from 496 small-magnitude events ($3.5 \leq Mw \leq 5.0$) across New Zealand. The seismic responses of the structures are quantified using peak inter-storey drift ratio and peak floor acceleration. Application of the statistical framework to the analysis results indicates that a large fraction (> 90%) of the differences between the simulated and observed responses can be explained by differences in their simple intensity measures like spectral acceleration at the main modes of vibration. Future work will extend on this study to examine the application of this framework to moderate and large magnitude events and different structural systems.

1 INTRODUCTION

Response history analysis is an advanced method used for seismic design and performance assessment of engineered systems. Considering the limitations of using recorded ground motions in this application, particularly the scarcity of large-magnitude ground motions in the near-fault region, necessitates implementing alternatives methods such as using simulated ground motions.

Validation is an essential step in evaluating the applicability of simulated ground motions for utilisation in engineering practice. It also provides valuable insights towards improving the simulation methodologies by highlighting specific limitations of simulation methods. Simulated ground motions are validated considering a range of model complexity from a single degree of freedom through to complex 2D/3D systems (Bradley et al. 2017). Although the use of simplified intensity measures, e.g. Sa(T), PGA, PGV… for validation is common (Graves and Pitarka 2015; Razafindrakoto et al. 2018; Lee et al. 2020), they are unable to capture the complexities of real engineered systems (Bradley et al. 2017).

Ground-motion simulation validation is developing to consider advanced intensity measures (e.g. building responses) as the simple intensity measures have almost been depleted in providing feedback to improve the ground motion simulation. More recently, studies have attempted to validate simulated ground motions by comparing the response of complex structural models (Galasso et al. 2013; Bijelić et al. 2014; Burks et al. 2015; Bijelić et al. 2018; Loghman et al. 2019). Nevertheless, these attempts mostly focused on the application of simulated ground motions in engineering practice than considering the responses of complex systems to provide insights to improve the simulation procedures.

This research has been conducted to address the source of differences in the response of complex systems in terms of ground-motion intensity measures (denoted as simplified IMs) via a novel statistical framework. In other words, this study assesses the need to consider complex structural models in validation studies by developing a novel statistical framework to quantify the proportion of differences in the structural responses that can be explained by corresponding differences in simple ground motion intensity measures. The application of this framework is demonstrated by analysing two steel special moment frames subjected to pairs of simulated and observed ground motions from small-magnitude ($3.5 \leq Mw \leq 5.0$) earthquakes across New Zealand.

2 GROUND MOTIONS CONSIDERED

5349 pairs of observed and subsequently simulated ground motions from 496 small-magnitude events ($3.5 \leq M_w \leq 5$) at 294 station locations across New Zealand are used in this study. Figure 1a shows the strong-motion stations and the schematic observed ground-motion ray paths of the selected records. Figure 1b plots the distribution of magnitude and source-to-site distance of the records. Figure 1c shows the histogram of source-to-site distance explicitly, from which it is observed that the majority of ground motions have a source-to-site distance of less than 60 km.

Simulations are conducted using the hybrid broadband method (Graves and Pitarka 2010, 2015), utilising a comprehensive physics approach for the low-frequency part and a simplified physics approach for the high-frequency part of the simulation.

The use of ground motions recorded from small-magnitude events permits benchmarking the statistical framework for linear structural response. This study will extend to moderate and large magnitude events to consider other effects (e.g. nonlinear structural response) in the validation context, which cannot be captured by the small-magnitude events. The geometric mean of the intensity measures computed from the two horizontal ground motion components is considered representative of the ground-motion intensity measure at each site.

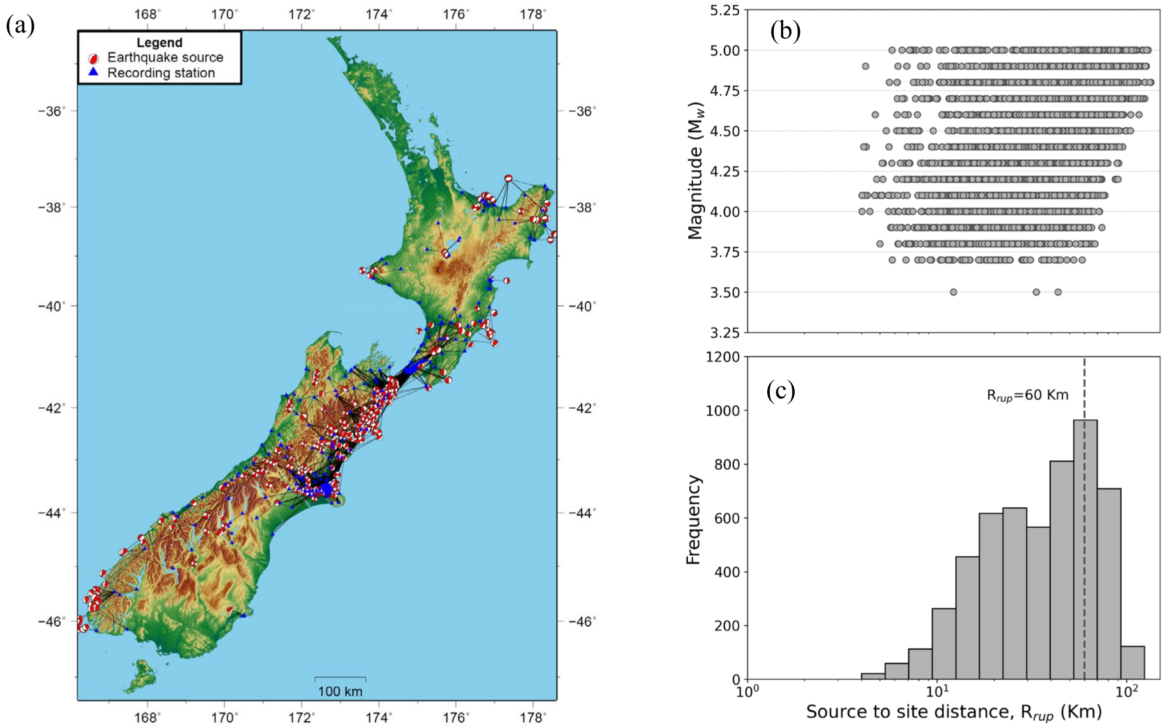

Figure 1: Descriptions of the simulated and observed ground motion sets considered in this study a) 496 small-magnitude events, 5349 strong motion stations, and schematic observed ground motion ray paths; b) distribution of magnitude versus source-to-site distance; and c) source-to-site distance histogram.

3 STRUCTURES ANALYSED

Two steel special moment frame buildings were selected for analysis (Fig. Figure 2a). These buildings were designed for a site in Seattle based on US standards (UBC 1994) as part of the SAC Steel Project (FEMA 2000). The low-rise building, denoted as Building A, has three stories. The high-rise building, denoted as Building B, has nine stories above the basement. The fundamental periods for Buildings A and B are 0.98 s and 2.95 s, respectively. Beams and columns were modeled as elastic elements with concentrated plastic hinges at their end (Fig. 2b). Both structures were analysed subjected to all pairs of simulated and observed ground motions using OpenSees (McKenna et al. 2006). The peak inter-storey drift ratio (IDR_k) and the peak floor acceleration (PFA_k) for storey k are the engineering demand parameters (EDPs) recorded at each storey. The geometric mean of maximum responses in two directions is considered representative of the response at each story.

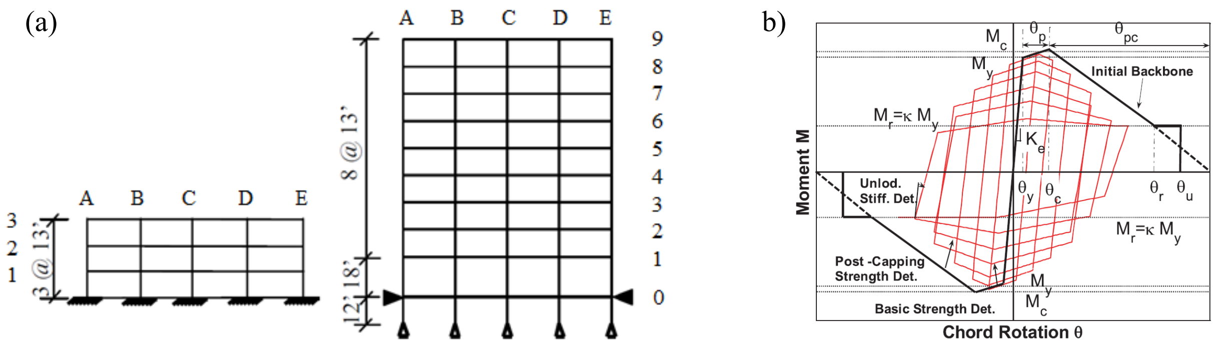

Figure 2: a) Schematics of SAC steel frames (FEMA 2000); b) Hysteretic behaviour of the plastic hinges (Lignos 2008).

4 STATISTICAL FRAMEWORK

The basis of the adopted framework for interpreting the structural response of complex systems is the notion that a portion of the differences in the response is directly due to the differences in the simulated and observed ground motions as measured through simplified intensity measures (IMs); with the remainder is due to other complexities. The relative proportion of these two parts is of interest to understand the additional information that considering complex structural systems can provide. For example, if the complex response is entirely explainable due to the differences in simplified intensity measures, then the response itself does not provide any additional information for validation insights.

A rigorous statistical framework is needed to address the source of differences in engineering demand parameters (EDPs) and the proportion of variability they explain. Specifically, a multiple linear regression method can be used to represent this relationship (Equation 1).

$$\Delta EDP = a_0 + a_i \Delta IM_i + \cdots + a_j \Delta IM_j + \epsilon \quad (1)$$

where ΔEDP represents the difference in the natural logarithms of structural response under a pair of simulated and observed ground motions at the same site, ΔIM_i to ΔIM_j represent the difference in the natural logarithms of the i^{th} to the j^{th} ground-motion intensity measures computed from the same pair of simulated and observed ground motions, a_0 is the intercept, a_i to a_j are the regression coefficients corresponding to the related intensity measures, and ε is the error term. Herein, the response spectral ordinates at all periods from 0.01s to 10s are considered as the possible predictors.

This approach is graphically illustrated in Figure 3, which shows the differences between the response spectral ordinates of a pair of simulated and observed ground motion (Fig. 3a) and the peak inter-storey drift ratio at each storey of Building B under the same pair of ground motions (Fig. 3b). In this case, for the considered EDPs and IMs, the regression model can be written as:

$$\Delta IDR_k = a_0 + a_i \Delta Sa(T_i) + \cdots + a_j \Delta Sa(T_j) + \epsilon \quad (2)$$

Note that the coefficients a_0-a_j are also a function of k, but this is suppressed for brevity. There are several statistical methods to select a subset of IMs that contributes to the EDP of interest. In this study, a forward stepwise model selection procedure (James et al. 2013) was preferred due to its simplicity and computational efficiency over the other methods. This procedure is explained via Algorithm 1.

Algorithm 1: Algorithm for forward stepwise model selection.

1) Choose the list of predictors, ΔIM_i, relevant to the considered structural response, ΔEDP
2) Determine the first IM among all predictors such that gives the highest correlation
   $\Delta \mathrm{EDP}_{predict} = a_0 + a_1 \Delta IM_1$
3) Calculate the residual: $residual = \Delta \mathrm{EDP}_{actual} - \Delta \mathrm{EDP}_{predict}$
4) Identify the best IM (with the highest correlation) among remained predictors to predict *residual*
   $residual = b_0 + b_1 \Delta IM_i$
5) Check the stopping criterion, p-value (the dependency of *residual* on ΔIM_i)
   -If there is no dependency terminate the algorithm.
6) Compute the $\Delta \mathrm{EDP}_{predict}$ considering all selected ΔIM
7) Repeat from Step 3.

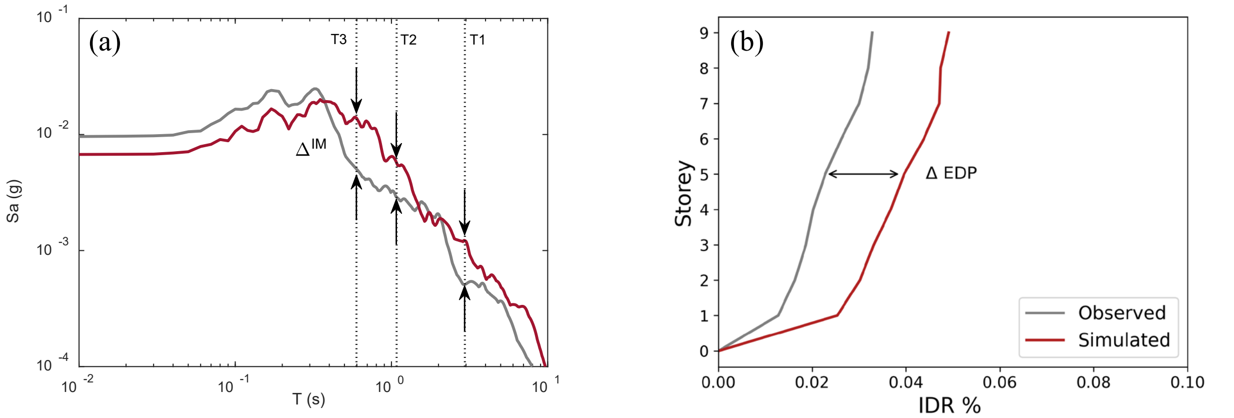

Figure 3: Comparison between the simulated and observed a) response spectra; b) structural response (IDR) along the height of 9-storey model.

Following this procedure, the results for one example (IDR_3, Building A) are shown in Figure 4. First, the most correlated IM is selected as IM_1 (herein $Sa(T_1)$, shown in Fig. 4a). Figure 4b shows the relation between the ΔIDR_3 and the IM_1. The other IMs are selected based on the residual analysis (*Algorithm **1***). Figure 4c shows the relationship between the residual from the first step and the second IM. This procedure is continued while no dependency (p-value > 0.05) between the residuals from the previous step and the candidate IM is captured (Fig. 4d).

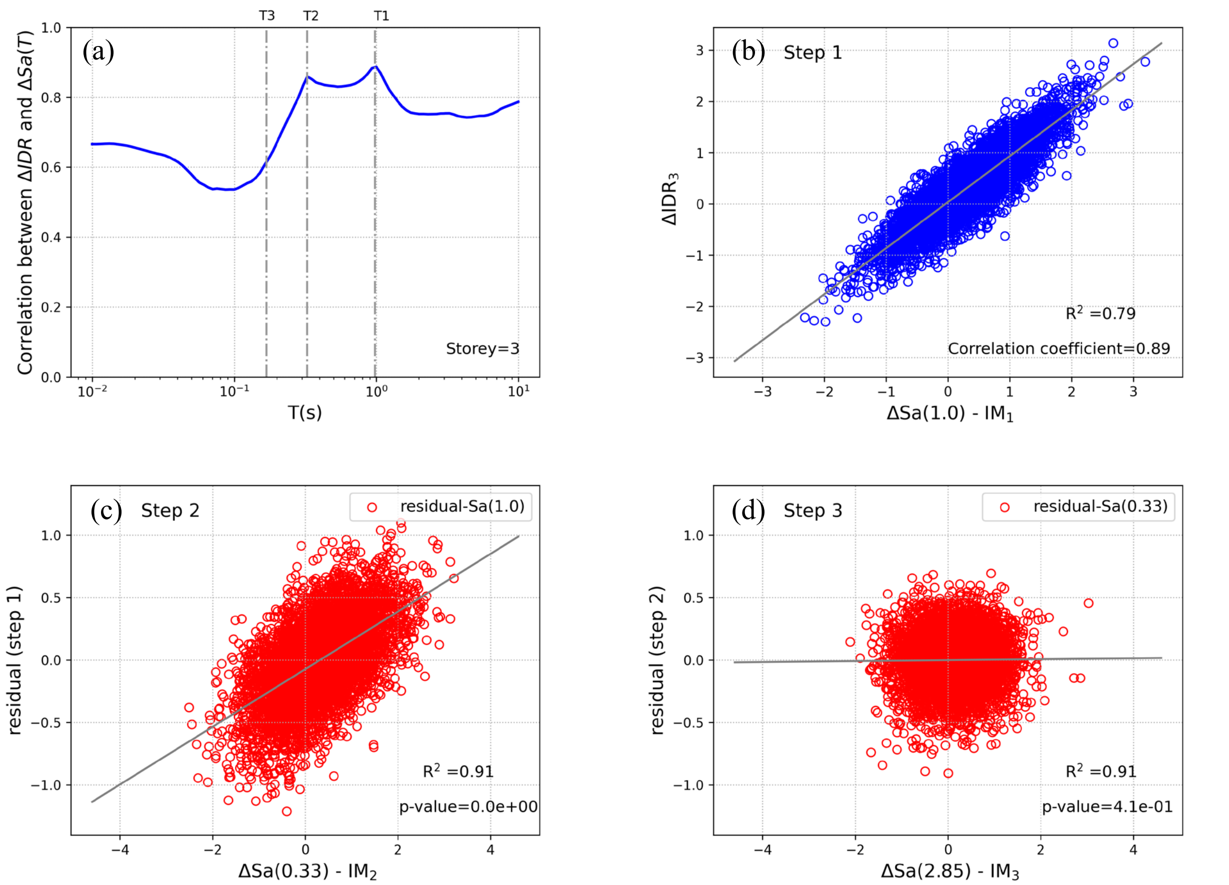

Figure 4: Variable selection procedure for IDR at Building A third storey a) correlation between ΔIDR and $\Delta Sa(T_1)$; b) ΔIDR_3 versus ΔIM_1; c-d) relation of residuals with the selected ΔIM at steps 2 and 3.

Checking the stopping criterion is necessary to avoid adding extra predictors which there is insufficient evidence to deduce that there is a correlation between them and the dependant variable (ΔEDP). The regression model using Equation 2, for this example, can be written as:

$$\Delta IDR_3 = a_0 + a_1 \Delta Sa(1.0\ s) + a_2 \Delta Sa(0.33\ s) + \epsilon \quad (3)$$

5 RESULTS

5.1 Selected predictors

Following the above procedure, the selected predictors are shown in Figures 5a, c for Building A IDR and PFA, respectively. The order of selected IMs is highlighted in different colours. As shown, the difference in the structural responses is explained by the difference in spectral acceleration at the first few modes of vibration. This is expected since the selected IMs are likely most closely correlated to the considered EDP. The findings here are comparable to results from a modal analysis at the linear level, where the structural responses are estimated by the contribution of different modes of vibration. The linear behaviour of structures is a valid assumption as the models are subjected to small-magnitude events.

The results from the same procedure are shown in Figures 6a, c for Building B IDR and PFA, respectively. The number of selected IMs increases when the building height increases. As expected, the higher modes contribute more to the response of the taller building (Building B).

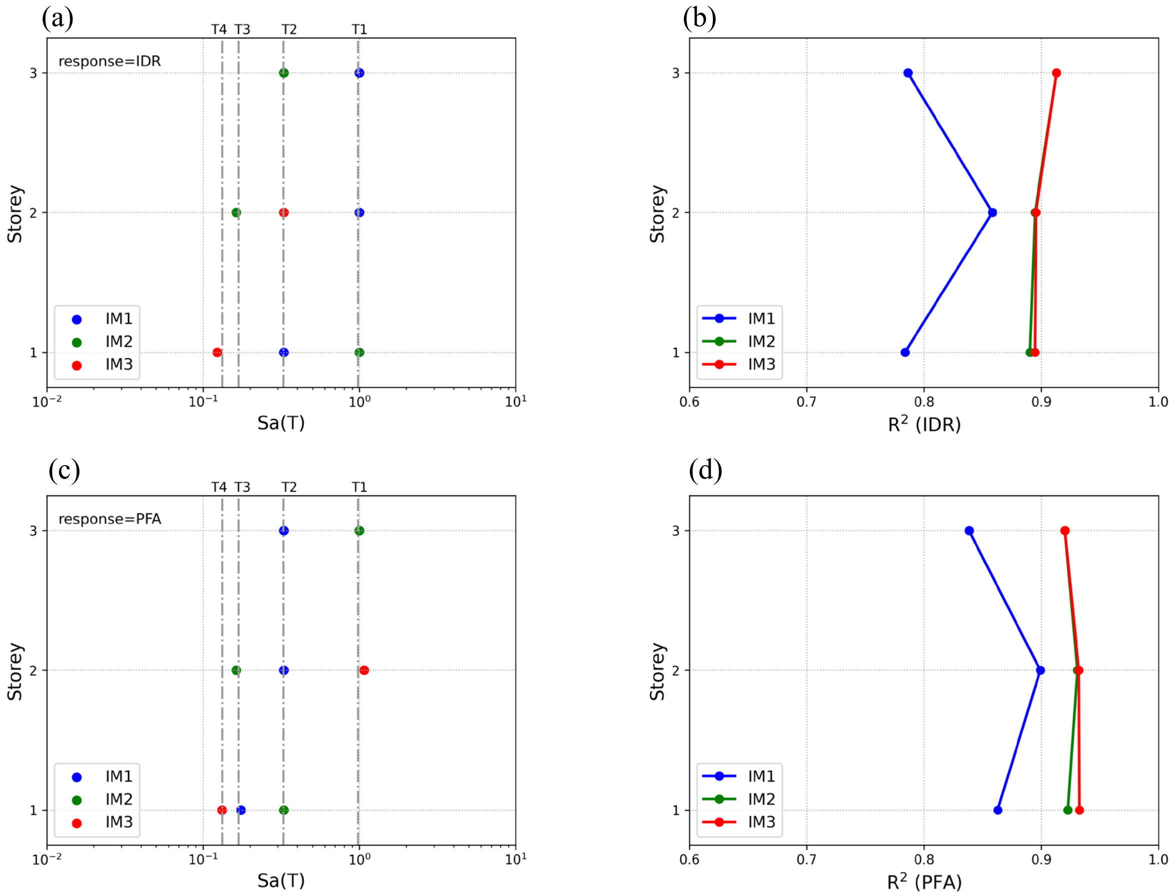

Figure 5: Variable selection and coefficient of determination (R^2) for Building A a-b) IDR; c-d) PFA.

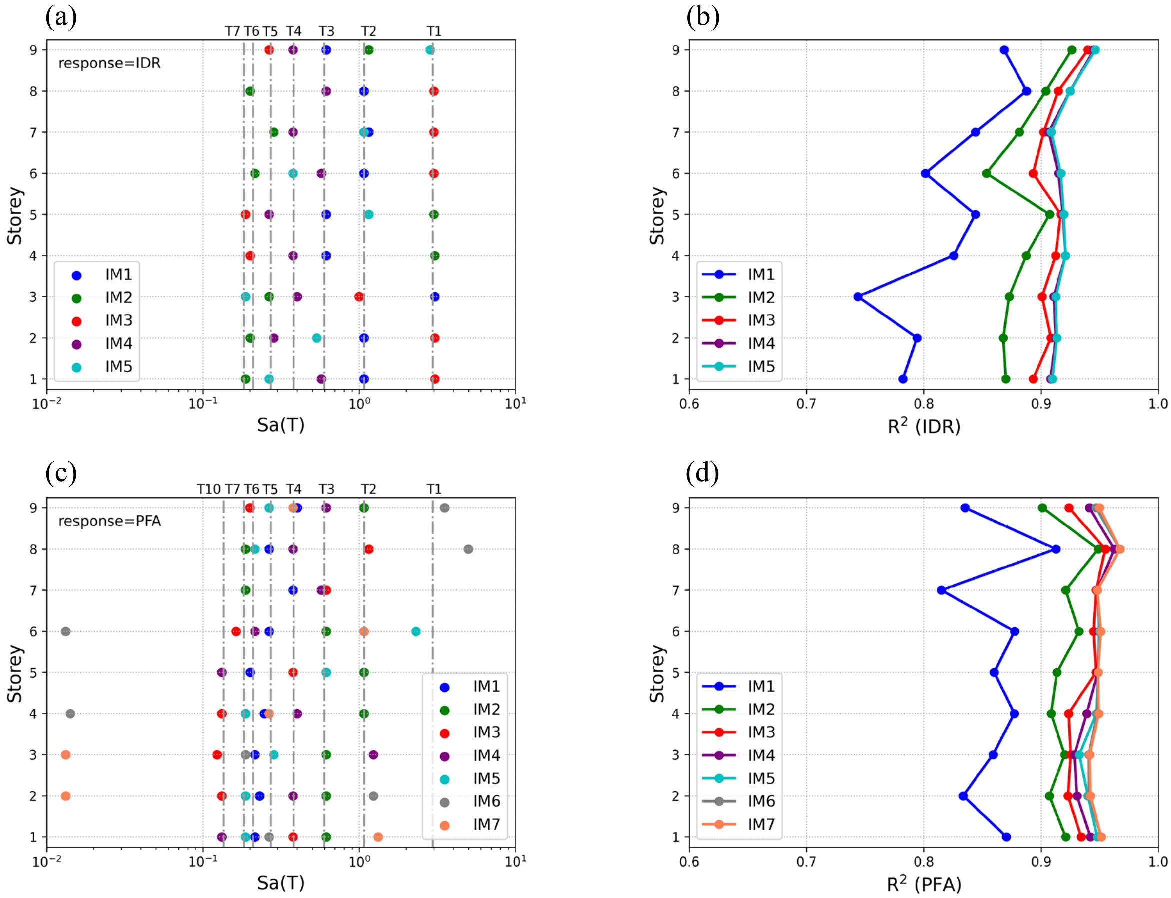

Figure 6: Variable selection and coefficient of determination (R^2) for Building B a-b) IDR; c-d) PFA.

5.2 Predictive capability of the regression model

Figures 5b, d and similarly Figures 6b, d show how much of the differences (quantified by the coefficient of determination, R^2) in IDR and PFA for Building A and B are explained by the selected IMs, respectively. As shown, the large fraction of differences in responses (at least 75%, Building B IDR_3) is explained by the first IM (Fig. 6b). The explained part of the difference in responses (IDR and PFA) is increased by considering more IMs (> 90%). Nevertheless, the incremental improvement in the predictive capability of the regression model decreases with the addition of each subsequent IM. Comparing the R^2 corresponded to PFA and IDR indicates that the higher percentage of variance can be explained by the PFA regression model than the IDR regression model for both buildings.

6 CONCLUSION

A novel statistical framework was developed to quantify the proportion of the differences in structural responses (e.g. peak inter-storey drift ratio and peak floor acceleration) under simulated and observed ground motions that can be explained by corresponding differences in simplified ground-motion intensity measures (e.g. spectral acceleration at different periods). This framework enables us to assess the importance of comparing the response of complex systems under simulated and observed ground motions in future validation studies.

As a case study, the response of two steel special moment frame models (a 3-storey and a 9-storey building) were considered subjected to 5349 unscaled pairs of simulated and observed ground motions from 496 small-magnitude ($3.5 \leq M_w \leq 5$) events across New Zealand. The results indicate that the large fraction (90%) of the difference between the simulated and observed responses can be explained by the difference of the spectral acceleration at the first few modes of vibration contributing to the selected response. This implies that the simulated ground motions which can capture the response spectra at the main modes of vibration can capture the response of structure well at the linear level. Future study will examine the application of this framework to the moderate and the large magnitude events where the structural responses are potentially affected by other effects (e.g. nonlinearity) that cannot be captured by the small-magnitude events.

7 ACKNOWLEDGEMENT

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