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RECORD-TO-RECORD VARIABILITY IN THE SEISMIC RESPONSE OF RC WALLS BUILDINGS SUBJECTED TO GROUND MOTIONS MATCHED TO THE CONDITIONAL SPECTRUM

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Abstract. *In Performance-Based Earthquake Engineering, it is often necessary to be able to estimate the response of a structure whilst accounting for various sources of uncertainty. A number of simplified procedures have been proposed whereby record-to-record variability is accounted for using empirical estimates of dispersion. It is shown herein how the calibration of empirical dispersion estimates may prove challenging due to the numerous factors that influence dispersion. An alternative simplified numerical approach is examined, in which record-to-record variability is accounted for through use of the conditional spectrum. This simplified procedure is evaluated through a comparison with results obtained from nonlinear response-history analyses. Both inter-storey drift ratio and base shear are examined, with promising initial results.*

1 INTRODUCTION

Traditionally, in the field of earthquake engineering, the seismic design or assessment of structures has been based around rather prescriptive code provisions. However, over the last couple of decades there has been a realisation that these more simplistic approaches may not be satisfactory and thus there is an ongoing shift towards Performance-Based Earthquake Engineering (PBEE). From the seismic assessment point of view, the core concept of PBEE is that the expected performance of a structure should be able to be expressed to an end user in terms of useful performance metrics. To this extent it is becoming evident that an effective means of expressing performance is to use time-dependent performance metrics such as expected annual economic loss or the annual rate of exceedance of a predefined limit state.

To calculate time-dependent performance metrics, a conditional approach is typically adopted, whereby information on the fragility of the structure is combined with hazard information. For example, the annual rate of exceeding a limit state inter-storey drift ratio, *idr*, can be determined through numerical integration of Equation 1, adapted from [1]:

$$\lambda(IDR > idr) = \int_x P(IDR > idr \mid IM = x)\left|d\lambda(x)\right| \tag{1}$$

where $P(IDR>idr|\text{IM}=x)$ is the probability that IDR exceeds the limit state value (*idr*) conditioned on the occurrence of an event with intensity IM=x, in which IM is the specific intensity measure being considered (*e.g.* PGA), and $|d\lambda(\text{x})|$ is the absolute value of the derivate of the hazard curve times dx. In simplistic terms $|d\lambda(\text{x})|$ can be considered as the rate of events occurring within an infinitesimal intensity range dx about x. Alternatively, closed form solutions exist whereby the annual probability of exceeding a given level of demand can be approximated. For example, as shown in Equation 2, again adapted from [1]:

$$P(EDP > edp) = \lambda\left(x^{edp}\right)\exp\left(\frac{1}{2}\frac{k^2}{b^2}\beta^2\right) \tag{2}$$

where $\lambda(x^{edp})$ is the hazard rate corresponding to a median response equal to *edp*, k is related to the slope of the hazard curve, b is used to described the relationship between intensity and the median conditional response and β is the dispersion in the engineering demand parameter (EDP) being examined.

Regardless of whether the rigorous numerical solution (Equation (1)) or the simplified closed form solution is used, a critical aspect is that for one or more intensity levels the engineer must determine not only the mean (or median) response of the structure but also the dispersion in response. In the case of Equation 1 this uncertainty manifests itself in the $P(IDR>idr|\text{IM}=x)$ term, whereas in Equation 2 the uncertainty is incorporated directly as the term β, which is the standard deviation of the logarithms of the EDP. Note that both epistemic and aleatory uncertainty in demand and capacity should be considered [1]; however, this paper will be limited to consideration of the aleatory component in demand, or record-to-record variability, only.

To obtain the probability distribution of an EDP at a given intensity, it is possible to conduct nonlinear response-history analyses (NLRHAs) using an appropriate set of ground-motions. However, both the numerical modelling and ground-motion selection aspects of this approach may be too demanding in practice. Furthermore, for large structural models the computational time required for the step-by-step integration schemes can be significant. Alternatively, a number of simplified approaches exist in which the median value of an EDP is obtained through simplified analyses, for example using the N2 method [2,3] or Direct Displacement-Based Assessment [4], and then combining the median response with an empirical estimate of dispersion. In most cases a lognormal distribution can be assumed [5]. Some notable examples of the aforementioned simplified approach include FEMA P-58 [5], Fajfar and Dolšek [6], Welch *et al.* [7] and Sullivan *et al*. [8]. The ability to estimate the dispersion, β, is the focus of the remainder of this paper. Section 2 starts by looking at the factors that influence dispersion. Section 3 then discusses a proposed simplified method for determining dispersion numerically, rather than relying on empirical estimates. Section 4 then covers a case study application of this method.

2 FACTORS INFLUENCING DISPERSION

As mentioned in the previous section, a number of proposed simplified procedures rely on empirical estimates of dispersion, such as those provided in FEMA P-58[5]. The values provided in FEMA P-58 [5] are given as a function of the relative intensity and different values are provided depending on the EDP being considered. In addition to the relative intensity and EDP being considered, there are additional factors that will also influence dispersion, in particular, building configuration and local site conditions.

To demonstrate the influence that these factors can have on dispersion, a number of simple numerical analyses are carried out:

- A regular three storey RC wall building, described in [9], is analysed in two configurations, one corresponding to a reduction factor of R=2 and the other corresponding to R=4, to show the influence of relative intensity on dispersion. IDR is investigated as the EDP.
- For the same three storey building with R=4, the dispersion in both IDR and wall curvature, φ_w, are compared to show that dispersion is dependent on which EDP is being considered.
- The same case study building for R=2 is increased in height to six storeys, but with the walls stiffened and strengthened so that the same fundamental mode period and strength reduction factor are retained. The dispersion in IDR is then compared to that for the three storey building to show the influence of structural configuration on dispersion.
- Finally, the three storey building for R=4 is considered at two different sites with different ground types and seismic hazard characteristics.

In all cases the buildings are modelled using Ruaumoko3D [9] and analysed using NLRHA. A set of 40 ground motions (Set 1), selected to match the conditional spectrum [11, 12, 13] is used in the analyses. For the final case, a second set of 40 ground motions (Set 2), selected assuming different site characteristics, is used. The conditional spectra and displacement response spectra of the selected ground motions are shown in Figure 1. The different site characteristics are provided in Table 1. Note that these characteristics have been selected so that the two different conditional spectra differ significantly in their shape, but possess the same value of spectral displacement at the conditioning period, T^*, of 1s (equal to the fundamental period of the case study building). The choice to use the conditional spectrum as a target for ground motion selection was made as it is considered a superior alternative to other target spectra, such as the uniform hazard spectrum. In particular, because the conditional spectrum accurately captures the record-to-record uncertainty on a spectrum level (conditioned on $Sd(T^*)$) it is consider as a robust starting point for obtaining estimates of record-to-record variability in structural response. Further discussion relating to the ground–motion sets used in this work and the target conditional spectra is provided in [9].

Set	V_{S30} (m/s)	$\overline{M}$	$\overline{R}$ (km)	ε
1	400	6.0	20	1.45
2	200	7.0	50	0.72

Table 1: Key parameters for conditional spectra and ground-motion selection (from [9]).

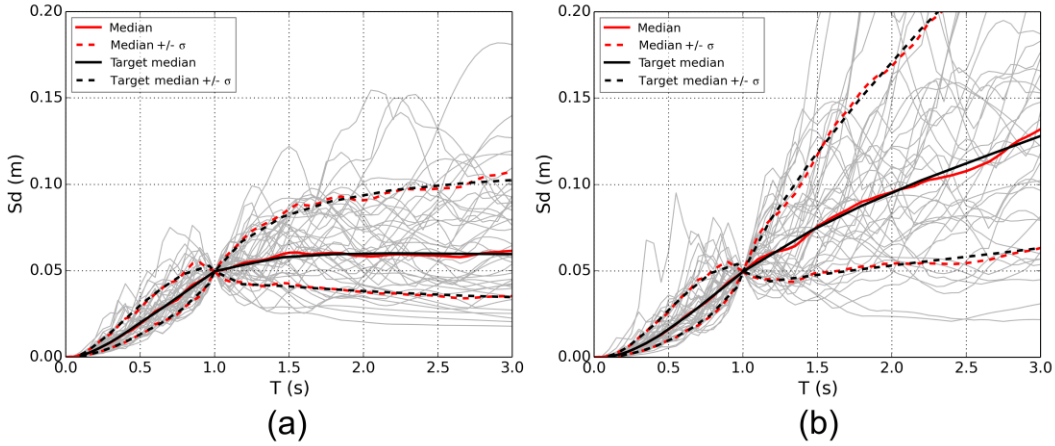

Figure 1. Conditional spectra and displacement response spectra of selected ground motions. (a) Set 1. (b) Set 2 (from [9]).

The results from the four different analyses are presented in Figure 2. In each case the results are plotted as empirical cumulative distribution functions of the EDP being considered. However, in order for the analyses to focus on the differences in dispersion, the results are first normalized by the median value of the EDP being considered. It can be seen that in each case the dispersion of the EDP distributions changes as different factors are varied. In Figure 2a, dispersion increases as R increases. This reflects how the empirical dispersion estimates in FEMA P-58 [5] vary with relative intensity. In Figure 2b the dispersion in curvature is observed to be larger than the dispersion in IDR. This is a consequence of curvature starting to increase rapidly once the walls start to yield, whereas the effect of yielding is not as dramatic on drift response. In Figure 2c it is observed that dispersion is larger for the six storey building. This is due to the stronger influence of higher-modes in the taller building, which contribute significantly to the total base shear. Finally, in Figure 2d it is observed that the different sets of ground motions produce very different levels of dispersion in drift response. This is despite the fact that the parameters used to construct the target conditional spectra do not vary too dramatically between the different ground-motion sets.

It is difficult to envisage how all these different factors could be taken into account when compiling empirical distribution data, in particular, the last aspect considering different site characteristics. As such, Fox and Sullivan [9] proposed a simplified numerical assessment procedure capable of estimating, with reasonable accuracy, the dispersion in the structural response of simple RC wall buildings.

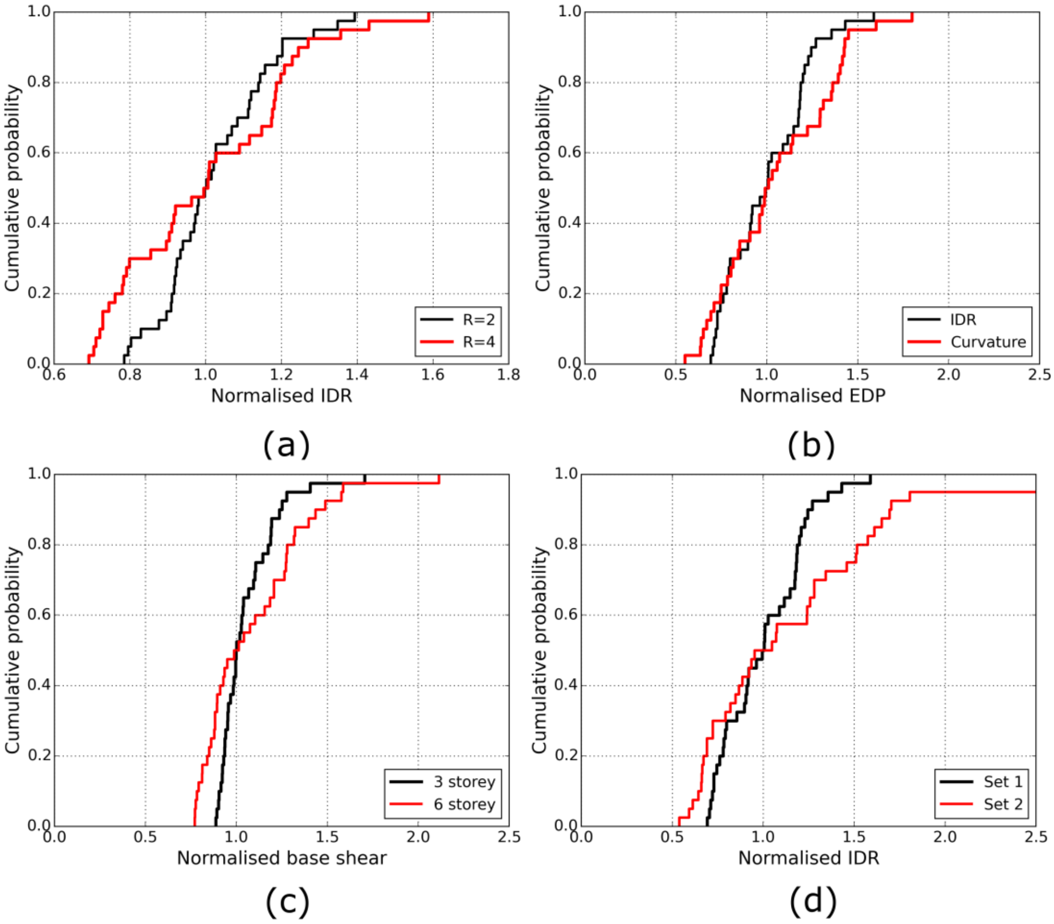

Figure 2. EDP distributions for examining different factors influencing dispersion. (a) Relative intensity, (b) different EDPs, (c) different building configurations, and (d) different site characteristics.

3 SIMPLFIED ASSESSMENT PROCEDURE INCORPORATING RECORD-TO-RECORD VARIABILITY IN STRUCTURAL RESPONSE ESTIMATES

To avoid the need for empirical dispersion estimates, Fox and Sullivan [9] proposed a simplified numerical assessment procedure for estimation of structural response whilst accounting for record-to-record variability. The procedure is considered "simplified" in the sense that ground-motion selection and NLRHA are not required. The key aspect of the procedure is that it relates the displacement response of an equivalent single-degree-of-freedom (SDOF) system to the spectral displacement at the effective period of the structure, $Sd(T_e)$, sampled from the conditional spectrum. The effective period is defined in accordance with [5] as the period of an equivalent linear system with the stiffness equal to the secant stiffness to the maximum displacement of the inelastic system. As this relation between displacement demand and spectral displacement is formed at the effective period, the procedure can capture the influence of the dispersion that is already defined within the conditional spectrum.

To start the procedure it is necessary to convert the multi-degree-of-freedom (MDOF) wall system to an equivalent SDOF system. This can be done using numerous existing approaches, for example, through the equations provided in [14]. Once the equivalent SDOF properties are obtained the following step-by-step procedure is used:

1) Construct the conditional (displacement) spectrum using T_1 as the conditioning period.

2) If $Sd(T_1)$ is greater than the yield displacement, Δ_y, the SDOF system will respond in the inelastic range and so the procedure moves on to step 3. If instead the structure remains in the elastic range then the solution is trivial *i.e.* $\Delta_d = Sd(T_1)$.

3) Make an initial estimate of the displacement demand. A good initial guess is $\Delta_d = Sd(T_1)$ *i.e.* the 'equal displacement rule' [15].

4) Calculate the displacement ductility demand and subsequently the effective period using Equations 3 and 4 respectively.

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{3}$$

$$T_e = \sqrt{\frac{\mu}{(1 - r\mu + r)}} \tag{4}$$

where r is the post-yield hardening ratio.

5) Obtain a randomly sampled value of spectral displacement at the effective period using Equation 5:

$$Sd(T_e) = \exp\left(\overline{\ln Sd}(T_e) + K.\sigma_{\ln Sd(T_e)}\right) \tag{5}$$

where $\overline{\ln Sd}(T_e)$ and $\sigma_{\ln Sd(Te)}$ are the mean and standard deviation of the natural logarithms of spectral displacement obtained from the conditional spectrum at the effective period and K is a randomly sampled value from the standard normal distribution.

6) Calculate a ductility dependent displacement reduction factor (DRF), which is defined as $\eta = \Delta_d / Sd(T_e)$. This can be done in using the concept of equivalent viscous damping as per Priestley *et al.* [4]. Equation 6, which is based on a thin Takeda hysteresis rule and considered appropriate for well detailed RC wall systems, is used to calculate the equivalent viscous damping ratio. Equation 7 is then used to adjust the 5% damped displacement spectrum to that corresponding to the new damping ratio ξ_{eq}.

$$\xi_{eq} = 0.05 + 0.444\sqrt{\frac{\mu - 1}{\mu\pi}} \tag{6}$$

$$\eta = \sqrt{\frac{0.07}{0.02 + \xi_{eq}}} \tag{7}$$

An alternative approach to this step is to use equations that provide DRFs as a function of ductility directly, such as those provided by Pennucci *et al.* [16].

7) Using the DRF found in the previous step, calculate a revised displacement demand from Equation 8. If the difference between the initial estimate of displacement demand (made in step 3) and the revised value is small, say within 2%, then continue to step 7. If not, then start again at step 3 using the revised estimate of displacement demand. Note that within each loop from steps 2 to 6, the same value of K should be used.

$$\Delta_d = \eta.Sd(T_e) \tag{8}$$

8) Record the calculated value of displacement demand and then start again at step 2 with a newly sampled value of K. This process is repeated until the desired number of estimates for displacement demand is obtained. Note that each estimate roughly corresponds to a displacement value calculated using NLRHA for a single ground motion. It is therefore recommended that the number of samples of K be comparable to the number of ground motions one might use in NLRHA.

With the set of displacement estimates, each corresponding to different sampled values of K, the distribution of different EDPs can be obtained. This is achieved by, for each different displacement estimate, converting the SDOF system back to a MDOF system and extracting values of the relevant EDPs. Any desired statistics, such as the median and dispersion, can then be calculated for each EDP. An example of the core part of the procedure is illustrated in Figure 3. In Figure 3a two iterations of steps 2-7 are shown for the case where K=+1.2, which results in a single estimate of displacement response. Figure 3b then shows 10 iterations of steps 2-8, which results in 10 different estimates of displacement demand, each corresponding to a different value of K.

The aforementioned procedure is suitable for obtaining estimates of EDPs that are not strongly influenced by higher-modes of vibration. To account for higher-mode effects Fox and Sullivan [9] proposed an additional procedure based on similar principals; however, it is limited to elastic structures only. To account for higher-mode effects, the contributions from multiple modes of vibration are combined using the square-root-sum-of-the-squares rule. As the conditional spectrum is chosen to be conditioned on $Sd(T_1)$ the first mode (elastic) response is deterministic. On the other hand, for the second mode of vibration there is a known distribution of $Sd(T_2)$ conditioned on the occurrence of $Sd(T_1)$. Therefore, a sampling procedure can again be used to estimate the distribution of different EDPs in the elastic range, considering the distribution of $Sd(T_2)$. It is assumed then that the contribution from the third mode and higher is not significant. Extension of this approach to account for higher-mode effects is a subject of ongoing research; however, for illustrative purposes it be assumed in this work that the SRSS modal combination rule can be extended to inelastic structures. This is approximately equivalent to what is done in Modal Pushover Analysis [17].

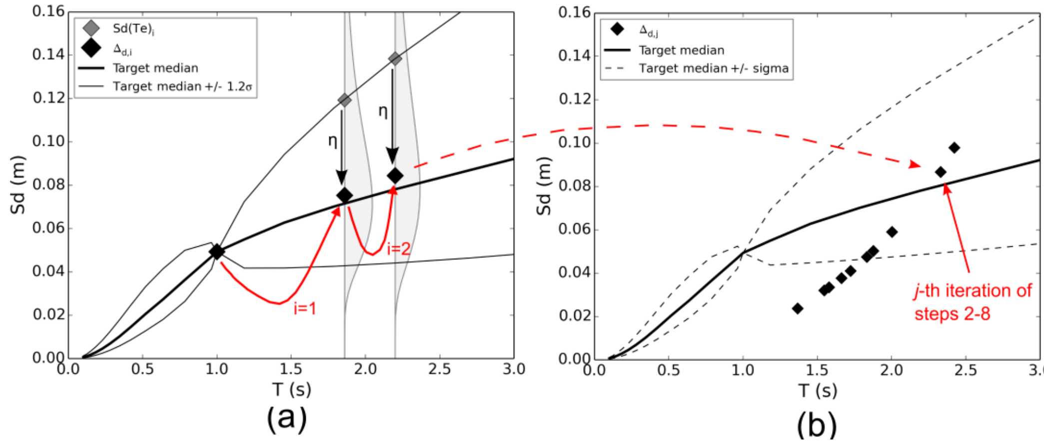

Figure 3. Example of the sampling procedure for an SDOF system. (a) Two iterations of steps 1-7. (b) 10 iterations of steps 1-7 (from [9]).

4 APPLICATION EXAMPLE

To illustrate the performance of the simplified approach of Fox and Sullivan [9], a case study application is examined. The building is the same three storey building used in section three, but with the walls having flexural strength corresponding to a force reduction factor of R=3 for $Sd(T_1)$=0.05m. The site under consideration is in Campobasso, Italy and it is assumed the building is located on Rock/Stiff Soils. The hazard curve for the site is obtained from the Italian National Institute of Geophysics and Volcanology (INGV) website [18] and the corresponding disaggregation data obtained from Barani *et al.* [19].

Conditional spectra are constructed using the same methodology and tools as in Fox and Sullivan [9]. A limitation of this is that the GMPE used for constructing the conditional spectra [20] is inconsistent with those used in the probabilistic seismic hazard analyses (PSHA). Furthermore, hazard is defined in terms of envelope spectral acceleration, whereas it is herein assumed to correspond to a random component of spectral acceleration. Although these limitations mean the resulting conditional spectra are not consistent with the PSHA, they are sufficient for a comparative evaluation of the method of [9].

Nine different levels of hazard are considered, corresponding to those provided by INGV and shown in Figure 4a. The resulting conditional spectra are expressed as conditional mean spectra and dispersion, shown in Figures 4b and 4c respectively. Once the conditional spectra were constructed, ground-motion selection was carried out using the same tools and methodology discussed in [9].

Multiple-stripes analyses were conducted using NLRHA and the simplified [9] approach was used to evaluate the inter-storey drift ratio at roof level and the maximum shear at the base of the wall. The results for the nine intensity levels are shown in Figure 5. It can be observed that the results from the simplified approach match those of the NLRHA rather well.

For drift, the simplified approach tends to give a lower prediction of the median response, but with larger dispersion than the NLRHA results. It is interesting to note that after yield, the inter-storey drift ratio tends to increase less quickly (with increasing demand) than prior to yield. This is a result of post-yield wall deformations being due to essentially rigid body rotation only (refer [21]). Figure 5b shows that after flexural yielding, the shear force at the base of the wall continues to increase, albeit at a slower rate. This is due to the contribution of higher modes, which play a significant role, even though the building is only three storeys. As the first mode contribution to shear cannot increase post-yield, a more appropriate intensity measure for evaluation of shear forces may be $Sd(T_2)$, as discussed in [22].

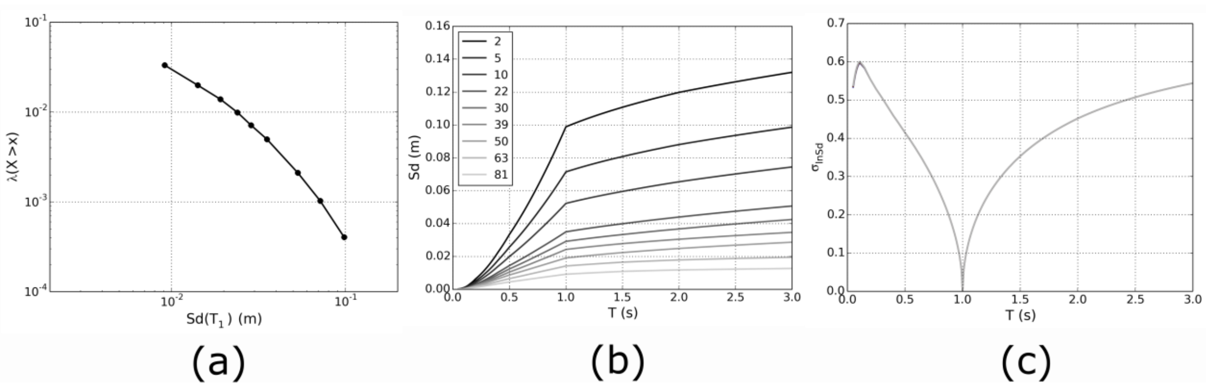

Figure 4. (a) Hazard curve for Campobasso, Italy. (b) Conditional mean spectra.
(c) Conditional spectrum dispersion

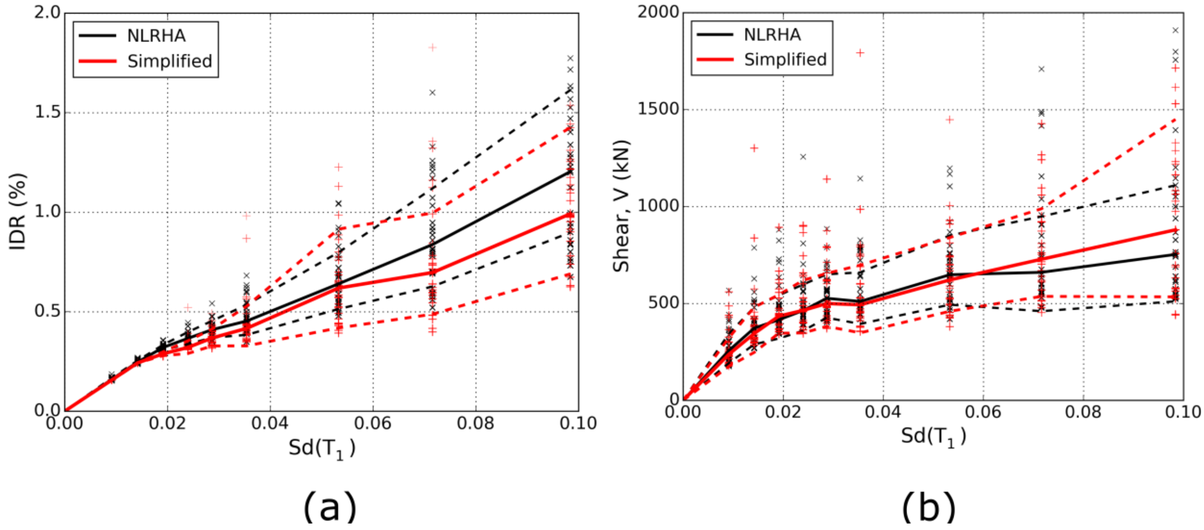

Figure 5. Multiple-stripes analysis curves showing individual data points, median response, and median +/- one standard deviation for (a) inter-storey drift ratio, and (b) base shear.

Using the information obtained from the multiple stripes analyses, fragility curves were constructed for the exceedance of several levels of drift and shear. Specifically, inter-storey drift ratios of 0.4, 0.6, 0.8 and 1.0% are considered and shear forces of 400, 600, 800 and 1000kN. Fragility curves are assumed to be well represented by a lognormal distribution and the data is fitted using maximum likelihood estimation, as discussed in [23]. For drift, the match between the NLRHA results and those obtained using the simplified method is not particularly good, but is still deemed reasonable considering the minimal effort required for the simplified approach. Furthermore, one must keep in mind the inherent uncertainty associated with the use of only 30 ground-motions at each intensity level. For shear, the match between the simplified and NLRHA results is extremely good. One will note, however, the large dispersion for the higher shear limits.

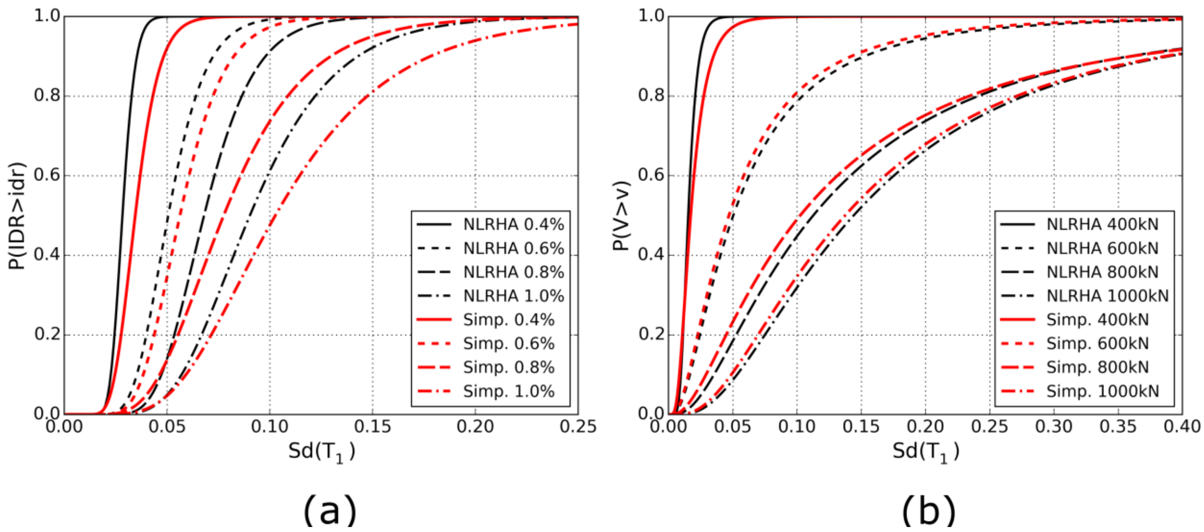

Figure 6. Fragility curves constructed from multiple-stripes analyses. (a) Inter-storey drift ratio. (b) Base shear.

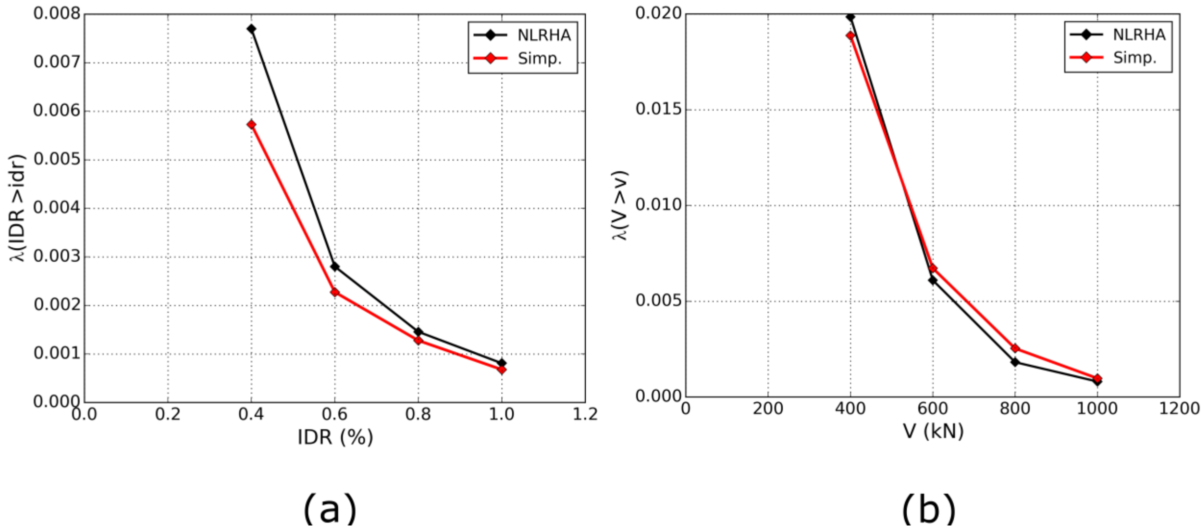

Figure 7. Annual rates of exceedance for various (a) inter-storey drift ratios, and (b) base shear forces.

Using the fragility curves of Figure 6 and the hazard curve from Figure 4a, the annual rate of exceedance is calculated for the different drift and shear limits. This calculation is done numerically using Equation 1. The resulting curves are shown in Figure 7. In general, the curves obtained using the two approaches match rather well. The exception to this is at the low levels of drift (0.4% and 0.6%). This is a result of poor fitting of the fragility curves for large probabilities of exceedance. Because the dispersion for the low drift levels is small, the most significant contribution to the annual rate of exceedance comes from the region above the median intensity *i.e.* the region in which there are large errors.

5 CONCLUSIONS

This paper has examined the record-to-record variability in the response of RC wall buildings subjected to ground motions matched to the conditional spectrum. It was shown through numerical examples that dispersion is a function of relative intensity, the EDP being considered, properties of the structure, and site characteristics. This presents a challenge in the calibration of empirical dispersion estimates for use in simplified probabilistic seismic assessment procedures.

The simplified approach of Fox and Sullivan [9] was investigated as an alternative to overcome this difficulty. For a case study example it was shown to give predictions of response that were comparable to those obtained from NLRHA, in most cases. The results were used to construct fragility curves, which for shear gave an excellent match to those obtained for NLRHA. For drift, the match in fragility curves was not as good. The annual rate of exceedance was calculated for a number of drift and shear limit states. Again in most cases the match between the NLRHA results and those obtained using the simplified method was good, the exception being for low levels of drift.

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