SELECTION AND SCALING OF GROUND-MOTION RECORDS FOR NONLINEAR RESPONSE-HISTORY ANALYSES BASED ON EQUIVALENT SDOF SYSTEMS

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ABSTRACT :

Nonlinear response-history analyses are used by the engineering community as the most reliable technique to estimate structural behaviour. In such analyses, besides an adequate structural model, a set of acceleration time-series is needed as the most realistic representation of the seismic action. Hence, the issue of adequately selecting and scaling an assemblage of ground-motion records that could enable a dependable determination of structural behaviour with fewer number of runs is of critical importance. In this paper, the possibility of using the outcome of equivalent SDOF models as an additional criterion for selecting and scaling real records for nonlinear analyses of buildings has been studied through extensive nonlinear response-history analyses over a wall-frame building. In general, the inclusion of equivalent SDOF peak roof drift estimates as an additional selection parameter proved useful in reducing the variability and improving the median estimate of global structural response. Finally, a simple yet effective method for selecting and scaling ground-motion records is proposed.

KEYWORDS: Equivalent SDOF, nonlinear response-history, ground-motion record selection, ground-motion record scaling.

1. INTRODUCTION

Several methods of different levels of accuracy exist for the structural analysis of buildings subjected to strong earthquake actions. Among them, nonlinear response-history analysis is widely recognized as the most reliable technique able to reflect the time dependent nature of the seismic behaviour. Furthermore, seismic design provisions specify nonlinear response-history analyses for some particular structures such as: irregular or complex-shaped buildings, structures tested under near-collapse conditions, structures designed to high ductility demand levels, etc. In order to perform such response-history analyses both an adequate computational model of the real structure and a representative set of acceleration time-series are essential.

In order to achieve an adequate use of real accelerograms in nonlinear analyses of structures, several issues need to be addressed: (a) the selection criteria, (b) the required number of records and (c) the scaling issue. Moreover, all these questions should be answered bearing in mind the engineer’s need for the smallest set of ground-motion records that provides a reliable estimate of the average structural response.

Several selection and scaling procedures of real ground-motion records have been put forward and the reader is referred to Bommer and Acevedo (2004) for a thorough review of the possible options. This conference paper, however, presents and compares a straightforward methodological alternative to tackle the issue of selecting and scaling real accelerograms for nonlinear analyses of buildings. The method is based on equivalent single degree of freedom (SDOF) representations of a more complex structure. The effectiveness of the proposed methodology is assessed in terms of median estimates of peak roof drift and its associated response variability.

This study is founded on and progresses the work completed by Hancock *et al*. (2008). It relies on extensive nonlinear analyses over equivalent SDOF models of a wall-frame structure, which provide a valuable suit of results from which conclusions are drawn. In fact, the inclusion of equivalent SDOF peak roof drift estimates as an additional selection parameter improves the median estimate and reduces the variability in the structural response, allowing fewer analyses to be performed whilst achieving the same level of estimation reliability.

2. MODELS AND ANALYSIS

2.1. Wall-frame Building Studied

The structure under consideration is presented in Figure 1. The building is an 8-storey regular reinforced concrete wall-frame building which model was initially developed by Mwafy (2001) and recently studied by Hancock and Bommer (2007) and Hancock *et al.* (2008). The building was originally designed to Eurocode 8 (CEN, 1995) with a behaviour factor of 2.625 and a design PGA of 0.15g and modelled in SeismoStruct (Seismosoft, 2005). Additional details of the structure can be found in Hancock and Bommer (2007).

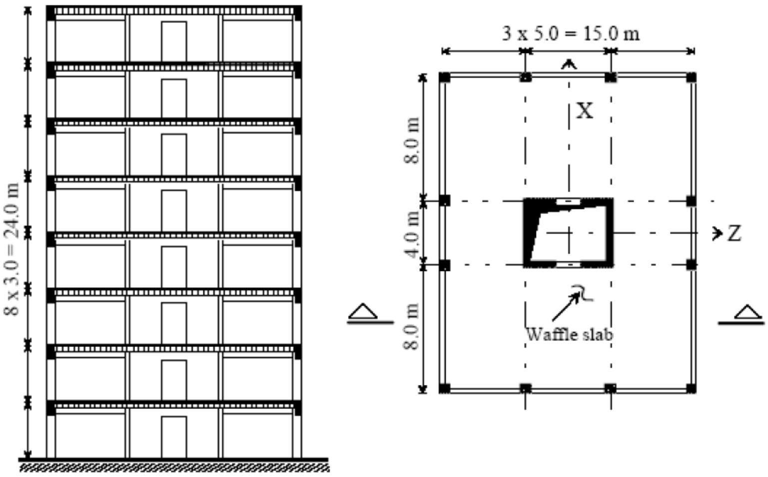

Figure 1 The 8-storey reinforced concrete wall-frame building studied (Mwafy 2001)

2.2. Equivalent SDOF Models

An equivalent SDOF representation of a multi degree of freedom (MDOF) system is obtained by condensation of the mathematical model and the subsequent definition of a hysteretic behaviour. Such an equivalent SDOF model enables us to estimate the displacement response at a significant point of the MDOF structure. Several methodologies for constructing equivalent SDOF models have been proposed and the reader is referred to Málaga-Chuquitaype (2007) for a complete review of them. In this study, on the other hand, the SDOF properties are defined from normalized displacement profiles obtained through static nonlinear (pushover) analyses.

Three nonlinear SDOF hysteretic rules were investigated: (a) bilinear, (b) trilinear and (c) modified Takeda. Also, three backbone curves were obtained by performing nonlinear static analysis with: (a) uniform and (b) triangular lateral load distributions. The third backbone corresponds to the use of an initial stiffness equal to the first elastic mode stiffness with post-yielding stiffness equal to the one obtained by means of a triangular force distribution. Only results for the bilinear and trilinear idealizations are presented here. Further details can be found in Málaga-Chuquitaype (2007).

2.4. Predictive Equations for SDOF Peak Displacement and Roof Drift

Regression analyses were performed using the same dataset employed by Hancock *et al.* (2008) in order to obtain predictive equations for peak roof drift based on equivalent SDOF models. The regression functional form is similar (although not exactly the same) as the one used by Hancock *et al.* (2008) and it is described by:

$$\log(y) = r_1 + r_2 M_w + r_3 M_w^{\ 2} + r4 \log(\sqrt{R_{jb}^{\ 2} + r_5^{\ 2}}) + r_6 S_1 + r_7 S_2 + r_8 F_1 + r_9 F_2 \pm \varepsilon \sigma_T \qquad (2.1)$$

where log(y) is the logarithm of the parameter sought, r_i is the ith coefficient determined by regression, M_w is the moment magnitude, R_{jb} is the closest distance from the site to the surface projection of the rupture, S_1 and S_2 are variables that consider the site classification (where S_1 is one for soft soils and S_2 is one for stiff soils), F_1 and F_2 are variables that consider the style of faulting (F_1 is one for normal faults and F_2 is one for reverse faults including reverse-oblique faults), σ_T is the total standard deviation of the residuals assuming they conform to a lognormal distribution and ε equals the number of standard deviations corresponding to the expected level of motion (i.e., equals +1 for the 84th percentile motion and -1 for the 16th percentile motion). The total standard deviation, σ_T , is the combination of the standard deviation from site to site variability, σ_S , and the standard deviation from the earthquake to earthquake variability, σ_E , in the following way:

$$\sigma_T = \sqrt{\sigma_S^{\ 2} + \sigma_E^{\ 2}} \qquad (2.2)$$

On the other hand, and despite the lack of consensus on how to combine the two horizontal components of the record (Beyer and Bommer, 2006, Beyer and Bommer, 2007), in the present study only the maximum component in terms of peak ground acceleration (PGA) was used as no more than 2D models were studied. Finally, the regression method used was the one-stage maximum likelihood method (Joyner and Boore, 1993). The coefficients of Eqn. 2.1 for peak roof drift estimations of several equivalent SDOF are presented in Table 2.1

Table 2.1 Regression coefficients for peak roof drift [%] of the wall-frame building for different equivalent SDOF

SDOF model	r_1	r_2	r_3	r_4	r_5	r_6	r_7	r_8	r_9	σ_E	σ_S	σ_T
Bilinear triangular load	-6.387	1.690	-0.108	-0.504	3.242	0.229	0.103	-0.075	0.044	0.175	0.292	0.341
Bilinear uniform load	-3.774	0.929	-0.057	-0.483	3.068	0.335	0.233	-0.086	0.050	0.167	0.278	0.324
Trilinear triangular load	-3.714	0.861	-0.054	-0.464	1.336	0.180	0.104	-0.124	0.023	0.120	0.239	0.267
Trilinear uniform load	-3.426	0.810	-0.051	-0.454	1.555	0.178	0.106	-0.122	0.023	0.123	0.234	0.264

2.4. Definition of Average Response

Peak-roof drift was chosen as the structural response parameter for comparison. Furthermore, in order to obtain a structural response distribution consistent with specified earthquake scenarios, the model developed by Hancock *et al.* (2008) is used here. This regression model is based on extensive analyses over an advanced structural model of the complete wall-frame structure. Finally, a target scenario of magnitude M_w 7, source to site distance of 5 km, strike-slip rupture mechanism and soft soil conditions was used.

3. SELECTION AND SCALING OF ACCELEROGRAMS BASED ON EQUIVALENT SDOF MODELS

3.1. Selection Based on Seismological Characteristics

The set of 25 records selected to match the seismological characteristics of the target scenario by Hancock *et al.* (2008) is used here as the reference set for comparison. A limit of ±0.2 magnitude units from the target magnitude and distances lower than 10.35 km from the surface projection of the fault had been specified for that suite.

3.2. Selection Based on an Equivalent SDOF Proxy

Additional sets of records were selected based on the peak roof drift they induced in an equivalent SDOF model. Firstly, a fairly large number of accelerograms were pre-selected based on their seismological characteristics, although in this case a wider range of magnitudes was included (6 to 7.9 M_w). The widening of the magnitude range reflects the fact that the duration of the motion (which is controlled by the magnitude of the event) has been shown to be relatively uncorrelated with peak displacement estimations (Hancock and Bommer, 2007).

Secondly, the nearly 200 accelerograms resulting from the previous step were imposed on a number of equivalent SDOF models and the 25 records that lead to roof drift estimates closest to the target value were finally selected. This target value was obtained by means of Eqn. 2.1 using the coefficients of Table 2.2. The mismatch was measured as the absolute value of the difference between the SDOF peak roof drift and the target regression estimate.

3.3. Selection Based on Two Equivalent SDOF Proxies

Alternatively, sets of records that induced peak roof drifts closest to the target value in not only one but two SDOF models were also tried. In this case, the square root of the summation of the squared differences of each SDOF prediction with respect to the target values was used as the combined mismatch measure.

3.4. Selection Based on an Equivalent SDOF Proxy of Records Scaled to PGV and of Records Scaled to Average Spectral Acceleration over a Period Range

Previous researchers (Shome *et al.,* 1998) have demonstrated that the standard deviation in the response can be reduced by scaling the ground motion to the elastic spectral acceleration at the initial period of the structure. More recently, Hancock *et al.* (2008) showed that a further reduction can be obtained if the scaling is preformed with the aim of matching the spectral acceleration over a range of periods.

Previously scaling to peak values like PGA has been very common in practice. Nevertheless, it is recognized that while the structural response of short period buildings may be proportional to PGA, the response of structures of moderate periods would be more related to the peak ground velocity (PGV). Therefore, two scaling alternatives have been investigated here for their use in combination with the equivalent SDOF selecting procedure: a) scaling to PGV and b) scaling to the average spectral acceleration over a range of periods between T_n to $3T_n$ where T_n is the natural period of the structure (0.55 second). This range of periods is expected to influence the most the structural response of the wall-frame building (Málaga-Chuquitaype, 2007).

First, an initial and broad record selection was carried out with magnitudes between 6 and 7.9 and distances lower than 20 km. Note that, the careful selection based on spectral shape was not performed here with the aim of testing if the inclusion of the SDOF drifts predictions in the selection procedure would, by itself, cause some particular spectral characteristics to be selected; nevertheless it is recognized that such spectral shape selection would further improve the final outcome.

As a second step, scaling to PGV and to the average 5%-damped spectral acceleration value over a period range was performed. The scaling factor was defined as the one that led to the smallest root mean square difference over the period range considered (0.55 to 1.7 seconds). Finally, the 15 scaled records that led to peak roof drift predictions in the SDOF model nearest to the scenario expected value were selected.

3.5. Comparison of 5%-damped Response Spectra

Figure 2 presents the response spectra for the suite of records selected based on: a) seismological characteristics only; b) the peak roof drift they induced in an equivalent SDOF with bilinear hysteretic behaviour; c) selected based on the peak roof drift they caused on two SDOF with bilinear and trilinear hysteretic behaviour, and d) scaled to average 5%-damped spectral acceleration over a range of periods and selected based on the response of an equivalent bilinear SDOF.

From the comparison of average response spectra in Figure 2 the significant reduction in the response variability caused by the inclusion of the equivalent SDOF peak roof drift estimate as an additional criterion for record selection can be appreciated. Such reduction in the response spectra variability is slightly more apparent for periods equal to or greater than 0.5 seconds. On the other hand, when the response over two equivalent SDOF models is considered, the scatter is reduced even more. Nevertheless, such reduction comes at the expense of an overall underestimation of the target response spectra.

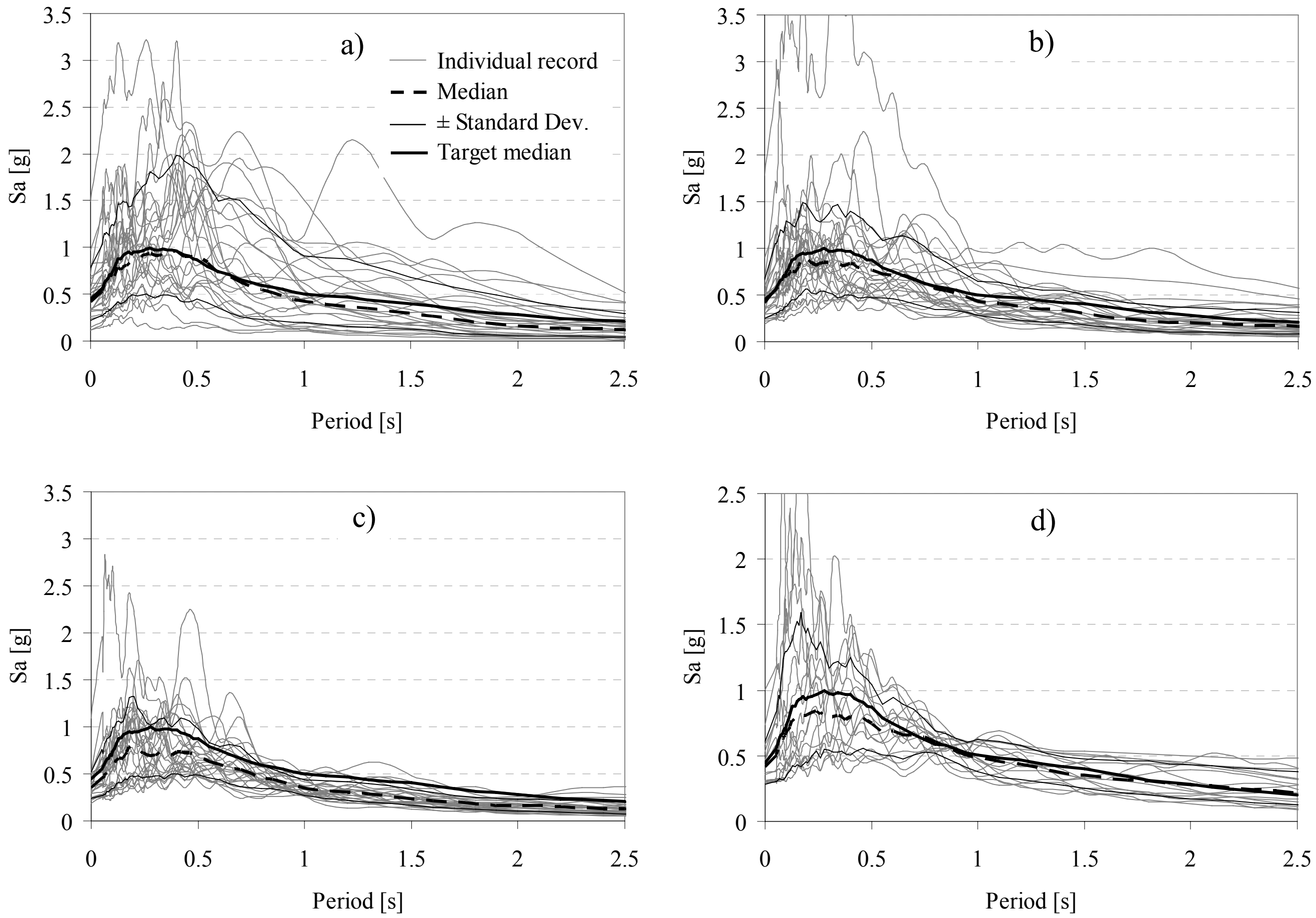

Figure 2 Response spectra for records selected based on: a) seismological characteristics only, b) the peak roof drift they induced in an equivalent SDOF with bilinear hysteretic behaviour, c) selected based on the peak roof drift they caused on two SDOF with bilinear and trilinear hysteretic behaviour and d) scaled to average 5%-damped spectral acceleration over a range of periods and selected based on the response of an equivalent bilinear SDOF. All backbones of SDOF models were obtained from pushover with triangular load distribution. Damping equal to 5% of the critical value.

4. STRUCTURAL RESPONSE TO SELECTED AND MODIFIED RECORDS

Nonlinear response-history analyses were performed subjecting the detailed MDOF model of the wall-frame building to each set of accelerograms previously described. A summary of the results is presented in Figures 3 and 4. Statistics of such results are also given in Table 4.1 where the standard error of the estimate (SEE) can be defined as

$$SEE = \frac{\sigma}{\sqrt{N_{obs}}} \tag{4.1}$$

with σ being the standard deviation and N_{obs} the number of records.

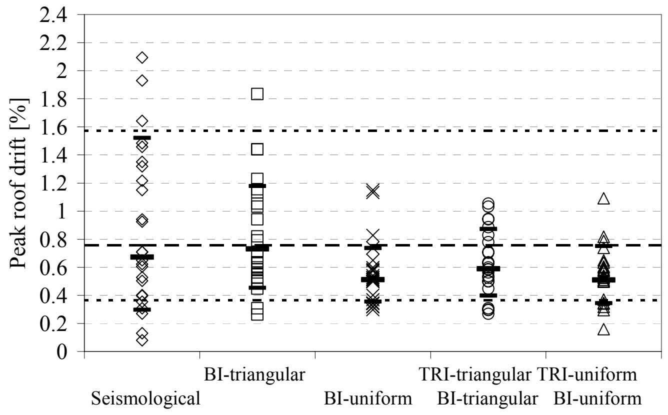

Figure 3 Peak roof drift [%] of suites of records selected according to seismological characteristics and based on equivalent SDOF representation based on different models (BI: bilinear, TRI: trilinear, triangular: triangular load distribution and uniform: uniform load distribution). Bars show median and median ±1 standard deviation. Dashed lines show predictions from regression analyses.

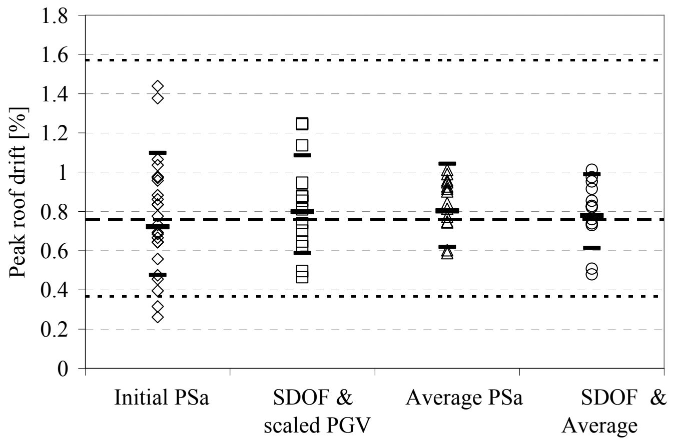

Figure 4 Peak roof drift [%] of suites of scaled records. From left to right: a) selected based on seismological characteristics and scaled to spectral acceleration at the structure's elastic period, b) scaled to PGV and selected based on an equivalent bilinear SDOF, c) selected based on seismological characteristics and scaled to average spectral acceleration and d) scaled to average spectral acceleration and selected based on an equivalent bilinear SDOF. Bars show median and median ±1 standard deviation. Dashed lines show predictions from regression analyses.

In general, reduction in the scatter of structural response is observed when including the equivalent SDOF peak roof drift in the selection process when compared with selection based on seismological characteristics only. This translates into a fewer number of records needed to estimate the structural demand parameter (in this case peak drift) with a certain confidence (say 5%). However, when two SDOF are used, this reduction happens at the expense of increasing the bias towards underestimation with respect to the value predicted by regression analyses.

All suites seem to provide peak roof drifts smaller than the median regression value but the underestimation has different levels of severity. In general, the suite of records selected based on a bilinear SDOF from pushover with triangular load distribution produces slightly better estimations. If scaling to spectral acceleration over a period range is additionally performed, the error in median estimation is minimal (3%) and only 4 records are required to give an estimate with 5% standard error at one standard deviation confidence level.

Table 4.1 Statistics of peak roof drift predicted from suites of selected and scaled records.

Selection Method	Median	1 St. Dev. (log)	Number of records	SEE at 1 St. Dev.	Number of records for 5% at 1 St. Dev.	Difference in median estimation (with regression)
Regression	0.759	0.316	834	1.1%	40	
Seismological characteristics	0.673	0.354	25	7.1%	50	-11%
Bilinear SDOF (triangular PO)	0.731	0.208	25	4.2%	17	-4%
Bilinear SDOF (uniform PO)	0.512	0.158	25	3.2%	10	-37%
Bilinear and trilinear (triangular PO)	0.590	0.170	25	3.4%	12	-22%
Bilinear and trilinear (uniform PO)	0.509	0.169	25	3.4%	11	-33%
Scaled to initial Sa	0.723	0.182	25	3.6%	13	-5%
Scaled to PGV bilinear SDOF	0.798	0.133	15	3.4%	7	5%
Scaled to average Sa	0.803	0.113	25	2.3%	5	6%
Scaled to average Sa bilinear SDOF	0.778	0.104	15	2.7%	4	3%

On the other hand, PGV scaling leads to higher variability in the response than scaling to the average spectral acceleration. However, by including the SDOF estimate in the selection criteria, scaling to PGV can still provide less dispersion in the response if compared to scaling to spectral acceleration at initial structural period (with careful spectral shape inspection).

The possibility of introducing bias by linearly scaling accelerograms was checked by plotting the peak roof drift estimations versus the corresponding scale factor in Figure 5. The linear trends show that there is no significant statistical bias introduced even when scaling factors of up to 8 are used.

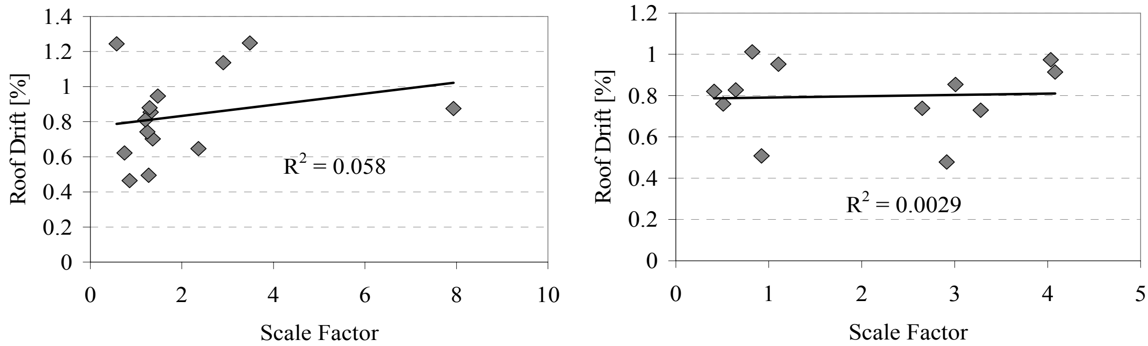

Figure 5 Scale factor versus peak roof drift for records selected based on equivalent SDOF displacement and scaled to: PGV (left) and to average spectral acceleration over a period range (right)

5. SUMMARY AND CONCLUSIONS

The possibility of using the outcome of equivalent SDOF models as an additional criterion for selecting and scaling real accelerograms for nonlinear time-domain analysis of structures has been studied. Indeed, the inclusion of peak roof drift estimations obtained by means of an equivalent SDOF model into the selection process has proven useful in both: a) reducing the variability of the median response (and hence the number of required records) and b) improving the median estimate for the design scenario studied here.

Given the findings of the present study, the following 5-step selection procedure can be suggested:

1. Construct an equivalent SDOF model of the structure under consideration.
2. Select the candidate ground-motion records for scaling which can be done in terms of magnitude and distance considerations (but with wider limits).
3. Scale the accelerograms to the design target scenario which may be defined by the corresponding seismic hazard assessment. Scaling to the average spectral acceleration over a period range is encouraged.
4. Run the scaled candidate ground-motions through the equivalent SDOF model defined in 1.
5. Select the records which give the structural demand parameter (peak roof drift in this case) nearest to the structural demand parameter defined by the target scenario.

Finally, it must be kept in mind that the results here presented, compelling and based on extensive analyses as they are, are strictly only for the structure studied. Further research is needed to verify the results in other structural configurations. In particular, the performance with regards to cumulative demand parameters like hysteretic energy and fatigue damage measures is currently undergoing. Also the possibility of accounting for the underestimation observed in some cases via a modification factor if such underestimation is shown to be consistent deserves further attention (particularly when two SDOF models are used which has proven to reduce the dispersion substantially).

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