INFLUENCE OF CODE-BASED RECORD SELECTION METHODS ON THE SEISMIC ASSESSMENT OF EXISTING STEEL BUILDINGS

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Abstract. *The recent concerns regarding the seismic safety of the existing building stock have placed the review of current seismic assessment procedures on the top of the agenda. Alongside with the development of more advanced commercial software tools and computational capacities, nonlinear dynamic analysis is becoming, more and more, a common and preferable procedure in the seismic assessment of existing buildings. Besides the complexity associated with the formulation of the mathematical model, major issues arise related with the definition of the seismic action, which can lead to different levels of uncertainty in terms of local and global building response. Aiming to address this issue, a comparative study of different code-based record selection methods proposed by EC8-3, ASC41-06 and NZSEE is presented herein. The various methods are employed in the seismic assessment of four steel buildings, designed according to different criteria, and the obtained results are compared and discussed. Special attention is devoted to the impact of the number of real ground motion records selected (three and seven) in the response of the buildings and the inclusion of additional selection criteria based on the control of the spectral mismatch of each individual record with respect to the reference response spectrum adopted in the seismic assessment. The results obtained indicate that the Eurocode selection procedure slightly underestimates both the local and global response of the buildings in comparison with the American and New Zealand procedures, which conversely, lead to similar results. It is also concluded that record sets incorporating the additional selection criteria significantly reduce the level of uncertainty in the response.*

1   INTRODUCTION

Nonlinear dynamic analysis is recognized as the most reliable tool to assess the seismic performance of structures, particularly in the case of irregular ones. The constant increase in computing power and the wide number of advanced structural analysis software tools available have set this type of analysis as the preferable procedure for the seismic assessment of structures, both for code-related proposes and probabilistic risk analysis. However, besides the complexity associated with the formulation of the mathematical model, major issues arise related with the definition of the ground-motion records, which can be artificial spectrum-compatible signals, simulated records or natural records [1].

From the various types of ground-motion data, natural records are known to be the ones that provide better unbiased estimates of structural seismic demands [2], although a number of issues related to the real ground motions selection and scaling procedures have been evoked in previous works [1, 3-5], which may lead to increasing levels of uncertainty in terms of local and global structural responses. It has demonstrated [6] that although material properties induce significant variability in the structural response, the latter is much smaller when compared to the ground motion variability.

The preferred current practice for record selection consists of carefully selecting the records that reflect the expected earthquake magnitude (M), source-site distance (R), rupture mechanism and soil profile of the source of the events that are in some sense most likely to attack the structure. Some previous studies have found no consistent evidence that suggested that it is required to take special care in the record selection with respect to M-R factors [1,7-9], albeit have not been shown completely conclusive [1]. In fact, [10] has demonstrated that scaling records randomly selected from an M-R bin can introduce bias in median nonlinear structural response, depending on the fundamental period of vibration of the structure, the overall strength of the structure and the sensitivity of the nonlinear structural response to higher modes of vibration.

In the context of probabilistic structural assessment, the frequency of the structure to reach a certain limit state (e.g., collapse) over a certain period of time provides an estimate of the failure probability conditional to a specific record set. As a result, the number of records in a set should be sufficient to capture with acceptable confidence the seismic response and the record-to-record variability. In other words, the computed median of structural demands is seen to be within one-sigma confidence band of 10% if $\sigma < 0.1\sqrt{N}$, where σ is the dispersion of the demand variable and N the number of records. As an example, 25 records should be necessary to ensure a standard error of the variable to measure of 10% admitting a dispersion of the variable of 0.5 [8, 11].

Generally speaking, the number of records falls in the range of seven to ten and the current practice of scaling records so that their mean ordinate matches or exceeds the target elastic spectrum is seen to be a reasonable approach. According to [1], the practice of using a small number of records, from three to six, and then taking the maximum inelastic response, should be abandoned. A more extensive study on the required number of records and the degree of bias may be found in [12], wherein it was concluded that both systematically decrease as one applies more constraint on the selection and scaling of the records.

Another source of bias induced in the response estimates is normally related to the ground motion scaling limits. Values varying from one to 10, or more, are considered acceptable within the engineering seismology community. Although [7] has found no compelling evidence that scaling up real records to match the strength of stronger records induces bias in the response estimation, [10] demonstrated otherwise and concluded that the bias is largely removed when records with spectral shape similar to the target spectrum are considered.

The existing code-based record selection methods, such as the one proposed by EC8-1 [13], have been shown to require significantly improvements regarding the selection of real records as an input for seismic performance assessment [14, 15], which may lead to large discrepancies in the structural response within a given set of records. As a result, the analyst may be discouraged to use natural records and eventually lean towards the use of artificial ones, despite their inherent disadvantages [1].

In the context of recent studies [16, 17] that have been carried out on the evaluation and calibration of the seismic safety assessment procedures proposed by EC8-3 [18] for existing steel buildings, a comparative study of different code-based record selection methods proposed in EC8-3, ASCE41-06 [19] and NZSEE [20] is presented herein. The various methods are employed in the seismic assessment of four steel buildings, designed according to different criteria, and the obtained results are compared and discussed, particularly in terms of the record-to-record variability of the response.

2 CODE-BASED RECORD SELECTION METHODS

In general, all main guidelines and codes set the ground motion records used for nonlinear dynamic analysis to groups of not less than three records, which may be obtained wither from real earthquake events or artificially simulated, although preference is given to the former. Specific conditions for spectral matching that these records should comply with are also specified in the above mentioned documents.

As referred above, most of the code-based record selection methods preconize that a first careful selection from a bin of random real signs should be carried out considering the expected magnitude, source-to-site distance, rupture mechanism and soil profile that are consistent with the seismic hazard scenario associated with the structure under analysis. When the ground motion records having these characteristics are not available, simulated data sets with equivalent duration and spectral content should be used to make up the total number required. Code prescriptions ask for matching of the average spectral ordinates of the chosen record set to the target code-based spectral shape. The set has to consist of at least seven records to consider mean response. Otherwise, if the set comprises three to six records, the maximum response obtained among all the records within the set should be considered.

The ground motion record selection and scaling methods proposed by the European, American and New Zealand standards and guidelines are presented next.

Eurocode 8

Based on EC8-1, EC8-3 establishes the following criteria for record selection:

- The mean of the zero period spectral response acceleration values calculated from the individual time histories should not be smaller than the value of $a_g S$ for the site in question, being a_g the design ground acceleration on rock ground and S the soil factor;

- In the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the record will be applied, no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

ASCE41

ASCE41-06 establishes, when performing bidirectional planar analysis, that the square root of the sum of the squares (SRSS) of the two principal horizontal components of the records

shall be scaled such that the average value of the set of constructed spectra does not fall below 1.3 times the 5% damping elastic spectrum for periods between $0.2T_1$ and $1.5T_1$. According to [14], matching the mean spectrum of the seven pairs of records to 90% of the target code spectrum or matching the mean spectrum of the SRSS spectra to the seven pairs of records to 1.3 times the code spectrum are numerically equivalent. For this reason, the recent version of ASCE 7-10 [21] defines the target spectrum as 1.0 times the design spectrum, recognizing that the design spectrum is now based on maximum response of the horizontal plane. The intent of ASCE 7-10 is to scale records such that the maximum responses in the horizontal plane of the scale records are, on average, equal to, but not less, the design spectrum over the defined period range [22].

New Zealand Guidelines

According to the NZSEE recommendations, based on NZS 1170.5:2004 [23], the ground motion records shall be scaled by a record scale factor k_1 and a family scaled factor k_2 applied in the time domain, i.e. the record ordinate shall be multiplied by the product $k_1 k_2$. The record scale factor k_1 is defined as the scale value which minimizes, in a least square sense, the function $log(k_1 SA_c / SA_t)$, where SA_c refers to the spectra constructed for each record component and SA_t the code target spectrum for the site given, over the period range of interest between $0.4T_1$ and $1.3T_1$, being recommended to ensure that $0.33 < k_1 < 3.0$ and that the record selected is of reasonable fit to the target spectra. The latter condition may be demonstrated by satisfying $D_1 < log(1.5)$, where:

$$D_1 = \sqrt{\frac{1}{n} \sum \left(log(SA_c) - log(SA_t) \right)^2} \tag{1}$$

The family scale factor, k_2, is defined as the maximum value of the ratio $SA_t / max(SA_c) > 1.0$ over the period range of interest for the direction under consideration, where $max(SA_c)$ is the maximum component of each record within a family at each period considered. It shall be verified that $1.0 < k_2 < 1.3$.

3 CASE STUDY DESCRIPTION

The study presented herein was conducted considering four 5-storey steel buildings composed by regular moment-resisting frames as illustrated in Figure 1. Each building was designed according to different criteria. The first building, denoted as GB, was designed according to Eurocode 3 [24] to resist gravity loads. The remaining three buildings were seismically designed according to EC8-1 assuming medium ductility (DCM class), with a behaviour factor q of 4.0, and to comply with different limits for the inter-storey drift sensitivity coefficient $\theta_{P-\Delta}$, which is defined in the code to address the treatment of second-order effects. Thus, the SB1 building was designed to comply with the criterion $0.2 < \theta_{P-\Delta} < 0.3$, according to which the second-order effects shall be directly included in the analysis. The SB2 building was designed assuming $0.1 < \theta_{P-\Delta} \leq 0.2$, being the second-order effects taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta_{P-\Delta})$, and finally, the SB3 building was designed in order to neglect second-order ef-

fects ($\theta_{P-\Delta} \leq 0.1$). The design seismic action was set for Zone 3 of the Portuguese territory and assuming a soil type B, defined by a soil factor S of 1.29 according to the Portuguese National Annex of Eurocode 8.

The main structural characteristics of the analysed buildings are displayed in Figure 1 and Figure 2. Grade S275 was considered for the structural steel. The analyses were carried out using the open source software OpenSees [25]. Regarding the models used for nonlinear analysis, force-based beam-column elements were adopted considering 10 Gauss-Lobatto integration points along its length, which offers a superior solution to the classical Gauss integration method when it is important to include in the integration the end points of the element. Also, a cross-section discretization solution by fibers was followed and a bilinear elasto-plastic material model with 0.5% hardening was adopted for structural steel. The effect of the panel zones was neglected in this study. Special attention was given to the modelling of the viscous Rayleigh damping, having the mass proportional damping been neglected, since it does not have a real physical meaning, and a tangent-stiffness proportional damping assumed, which greatly reduces the elastic damping force when the structural stiffness drops to the post-yield level. A fraction of critical damping equal to 2% was considered.
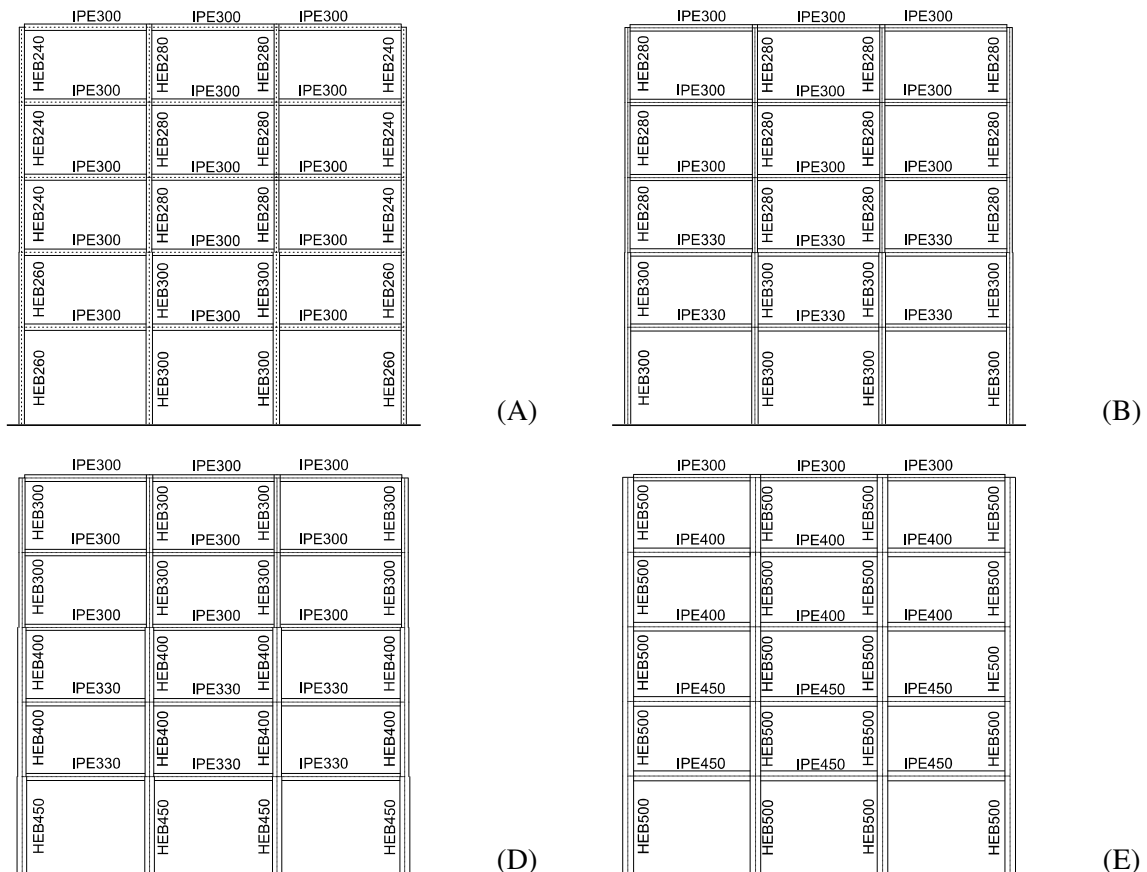

Figure 1: Plan view of the four 5-storey steel buildings studied: (A) GB building; (B) SB1 building; (C) SB2 Building; (D) and SB3 Building.
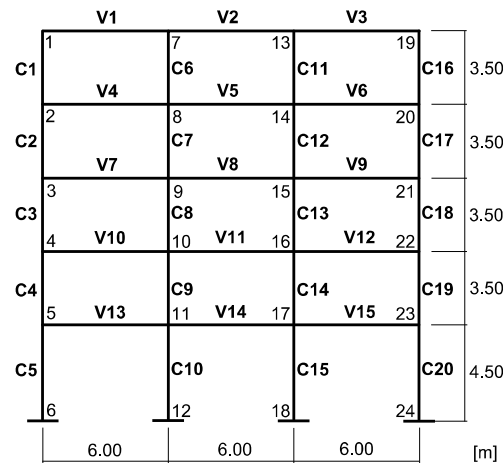
Figure 2: Frame dimensions and structural elements and nodes naming.

The gravity loading assumed in the analysis consisted of simple point loads applied at the mid-span of the beams and at each alignment of columns, which represent the vertical loads transmitted from secondary beams. On the top floor, both dead and live loads were set as $G_k = 85.5kN$ and $Q_k = 18kN$ at the inner alignment of columns and mid-span of beams and as $G_k = 42.75kN$ and $Q_k = 9kN$ at the outer alignment of columns. The dead and live loads on the remaining floors were defined as $G_k = 103.5kN$ and $Q_k = 36kN$ at the inner alignment of columns and mid-span of beams and as $G_k = 51.75kN$ and $Q_k = 18kN$ at the outer alignment of columns. The combination factor ψ_2 of the seismic combination of loads was defined as 0.0 at the top floor and 0.3 at the remaining floor levels. The masses were lumped at each node of the various floors and $P - \Delta$ effects were directly included in the numerical analysis.

Modal analysis was carried out for each frame to identify the dynamic characteristics of the buildings, which are listed in Table 1 for its first three modes of vibration.

Building	Periods of vibrations (s)			Mass Participation Ratios (%)		
	Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3
GB	1.63	0.50	0.26	85.49	9.24	3.40
SB1	1.50	0.48	0.25	83.86	10.41	3.58
SB2	1.20	0.39	0.20	78.73	12.06	5.34
SB3	0.90	0.28	0.13	80.56	13.22	4.33

Table 1: Dynamic characteristics of the buildings.

4 RECORD SETS CONSIDERED

As the Type 1 seismic action, which represents inter-plates earthquakes, was found to be the critical during the design, it served as the basis for the assessment study presented hereafter, being characterized by PGA values of 0.091g, 0.15g and 0.451g for the Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) limit states, respectively. For the sake of convenience and since a direct correspondence between the EC8-3 limit states and the ASCE41-06 performance levels may be established, the IO, LS and CP performance levels could be henceforth referred as DL, SD and NC limit states, respectively. The real ground-motion sets were selected using SelEQ [26], which consists of a web application developed at the Faculty of Engineering of the University of Porto (FEUP) which allows the selection of

real records or groups of real records. On the basis of the seismological characteristics of the events that define the Portuguese type 1 and Zone 3 seismic action, magnitudes and epicentral distances higher than 5.5 and 20km, respectively, were assumed. Also, an interval of values of the average shear wave velocity, $v_{s,30}$, between 360m/s and 800m/s was adopted in agreement with soil type B defined in EC8-1. An additional criterion was considered in the selection of each individual record, which consisted of imposing mismatch limits relative to the target spectrum of ±50%. The aim was to reduce the record-to-record variability of each set. 15 sets of three and seven records were considered and the scaling methods proposed by the EC8-1, ASCE 7-10 and NZS 1170.5:2004 G7 used (Figure 3 and Table 2).
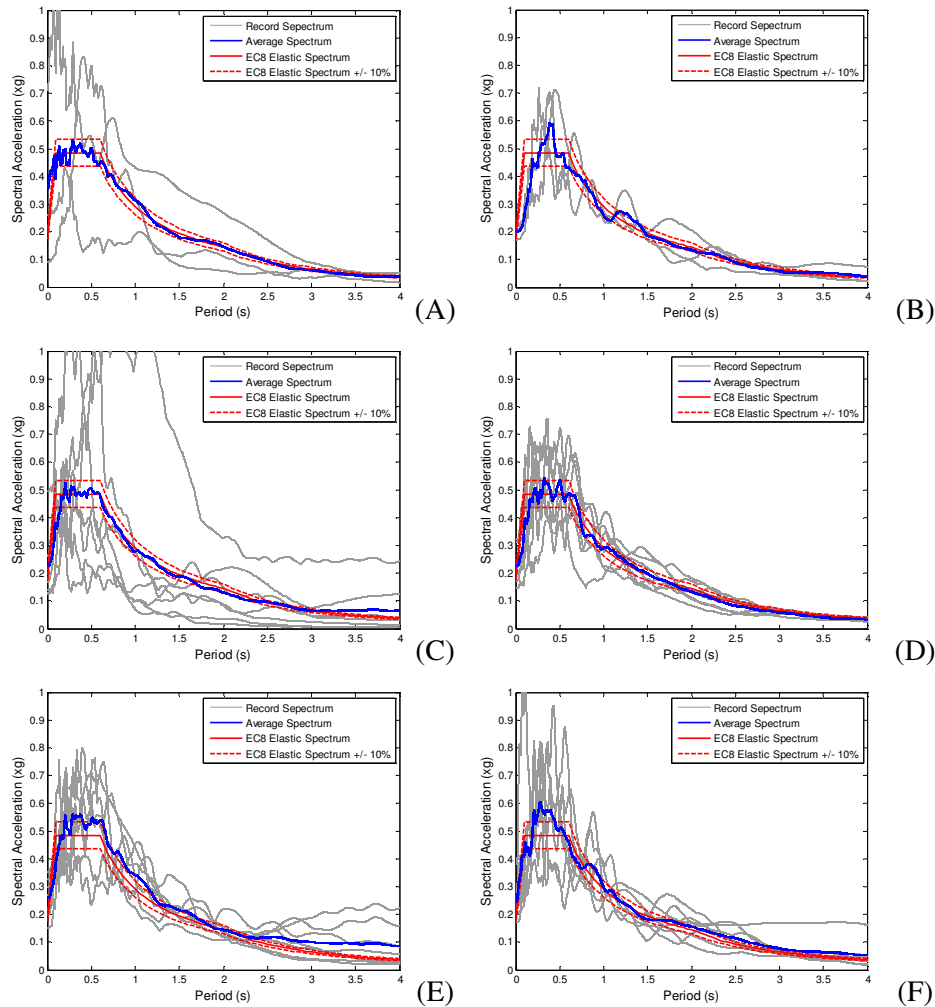

Figure 3: Matching of records with the design spectrum: (A) EC8-1 G3; (B) EC8-1 G3I; (C) EC8-1 G7; (D) EC8-1 G7I; (E) ASCE 7-10 G7I; (F) and NZS 1170.5:2004 G7.

Types of groups of records considered

Scaling Method	Number of records / Control of mismatch relative to target spectrum			
EC8-1	3/no (G3)	3/yes (G3I)	7/no (G7)	7/yes (G7I)
ASCE7-10	3/no (G3)	3/yes (G3I)	7/no (G7)	7/yes (G7I)
NZS1170.5	3 (G3I)		7 (G7I)	

Table 2: Identification of the groups of records considered.

Regarding the scaling factors adopted, it may be referred that the records were scaled to match the SD limit state elastic spectrum, being afterwards linearly scaled to match the DL and NC limit states elastic spectra. A maximum scaling factor of approximately nine was adopted. Also, it may be noted that since NZS1170.5 already considers criteria for scaling each record individually, no control of the mismatch relative to the target spectrum was adopted in this case.

5   RESULTS

Figures 4 to 8 depict both local and global structural responses obtained in terms of mean values and respective coefficient of variation (CoV) over the 15 groups considered for each group type. The global structural responses are defined in terms of global drift ratios (i.e. displacement recorded at the top of the building divided by its height), while the local structural responses are set in terms of plastic rotation demand-to-capacity ratios (DCR), being the EC8-3 capacity limits adopted and the demands calculated by numerical integration [17]. The results are presented individually for each building, limit state (LS) and record selection method. Also, on the basis of the code prescriptions, mG3 and mG3I refers to the maximum value of response when only three records are being considered within a group set and aG7 and aG7I the average value of the response of the sets with seven records.

Hence, as already mentioned above, it may be readily concluded that the technique of taking the maximum value of response when a small number of records, from three to six, is being considered should be abandoned. In fact, by looking at the GB building results, and particularly for the NC limit state (Figures 4 and 5 (A)) values of global drift ratios and local component DCRs two times higher than the reference ones (aG7I) are observed. Thus, in the context of seismic safety verifications, while both GB and SB1 buildings verify safety according to aG7I, it fails when considering mG3. Likewise, the level of variability of the response is significantly high and beyond the acceptable, with CoV values of 60% to 40% globally and approximately 100% locally. These limitations, however, can be significantly overcome by introducing additional selection criteria based on the control of mismatch relative to target spectrum (mG3I). Not only better estimates of both local and global responses were obtained, but also building GB and SB1 are now safe according to EC8-3 and ASCE41-06.

On the other hand, as previously expected, the increase on the number of records considered in each set (aG7) significantly reduced the mean estimates and the variability of the response, although leading to some unstable results, particularly at the SD limit state of buildings GB and SB1 using ASCE41-06 (Figure 4 and Figures 5 and 6 (B)). In fact, high CoV values were observed in building GB the at SD and DL limit states mainly due to the influence a single outlier set of records, which drove to significantly higher levels of structural demands comparing to the other sets. Yet again, the adoption of alternative criteria to control the mismatch relative to target spectrum (aG7I) improved the stability of the mean values of demand, reducing its CoV to values of the order and below 30%. Moreover, it is observed that the variability of the response decreases with an increase in the demand, characterized in terms of limit states and also by the increase in the structural stiffness of the buildings. In other words, as the structural stiffness of the buildings increases, the level of demand decreases, improving the quality of results. Additionally, the stiffer building exhibits large column-to-beam capacity ratios and hence tends to develop a more stable plastic mechanism, which also contributes to reduce the variability in the demand.
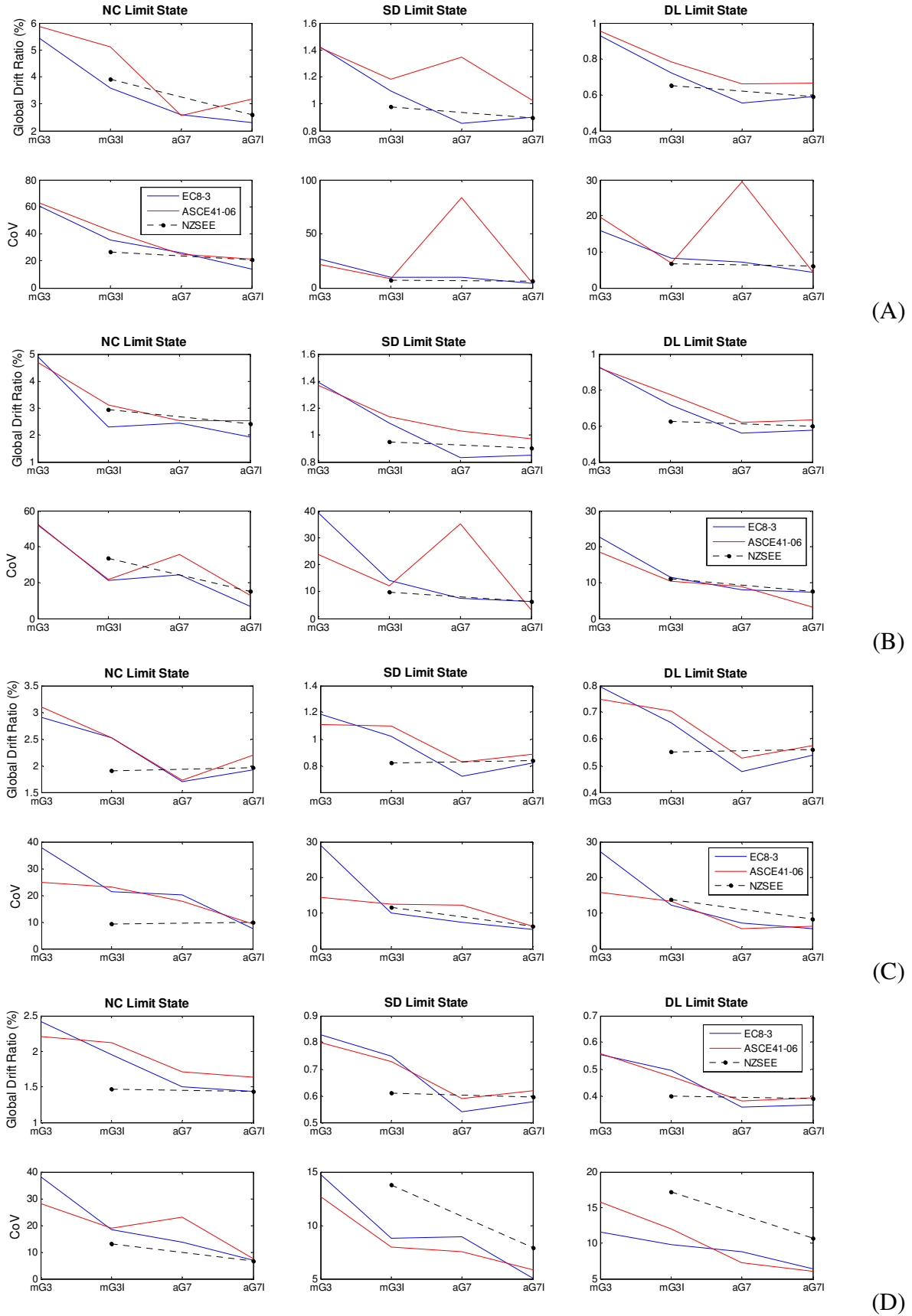
Figure 4: Mean global drift ratio results and respective CoV: (A) GB Building; (B) SB1 Building; (C) SB2 Building; (D) and SB3 Building.
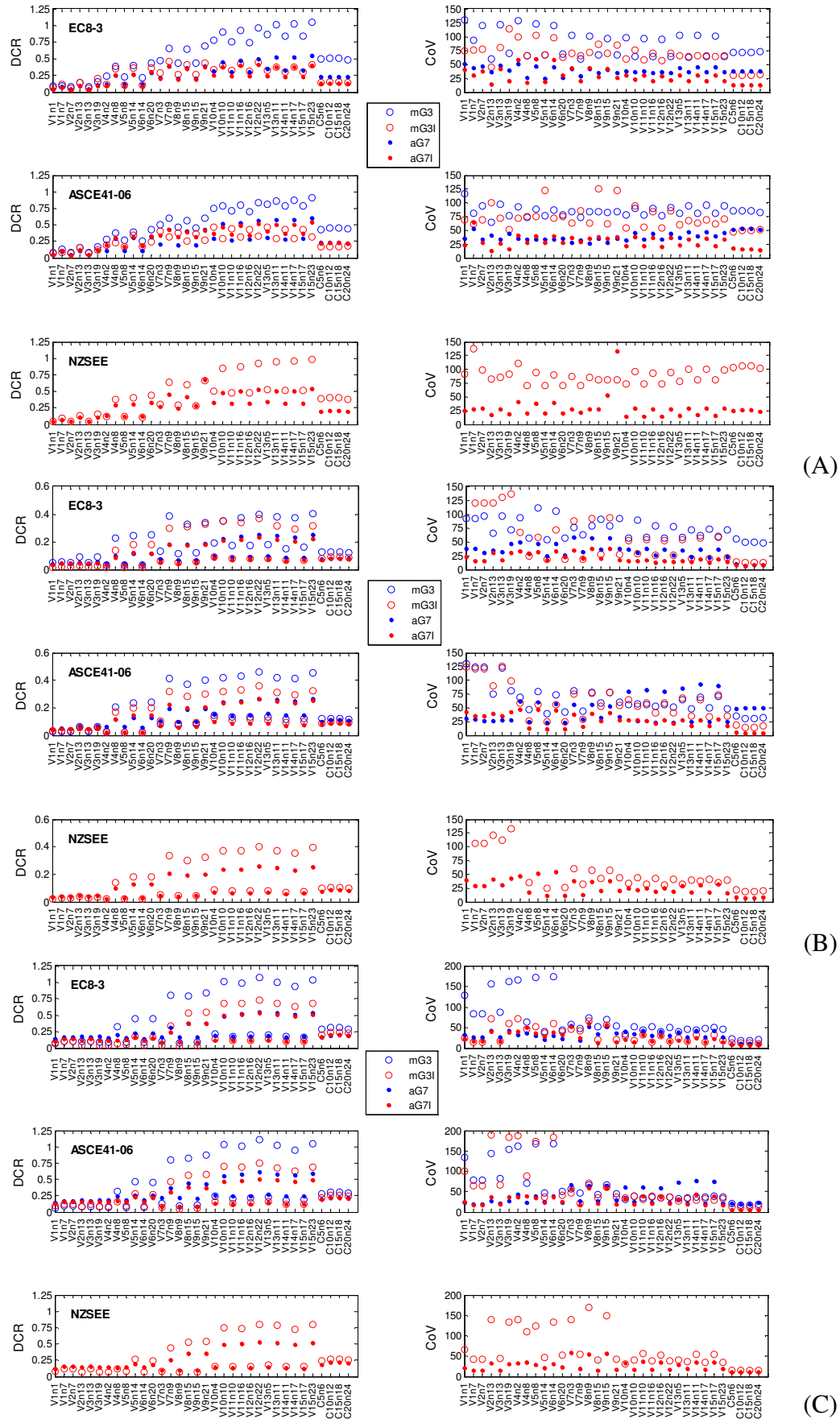
Figure 5: Mean DCR results for GB building and respective CoV: (A) NC LS; (B) SD LS; (C) and DL LS.

Figure 6: Mean DCR results for SB1 building and respective CoV: (A) NC LS; (B) SD LS; (C) and DL LS.

Figure 7: Mean DCR results for SB2 building and respective CoV: (A) NC LS; (B) SD LS; (C) and DL LS.

Figure 8: Mean DCR results for SB3 building and respective CoV: (A) NC LS; (B) SD LS; (C) and DL LS.

Finally, regarding the comparison between the EC8-1, ASCE 7-10 and NZS 1170.5:2004 selection and scaling methods, it may be seen that when considering the reference mG7I values EC8-3 continuously underestimates demands, particularly at the NC limit state, although its results tend to match the NZS 1170.5:2004 ones as the level of demands decreases and stiffer buildings are considered. On the contrary, ASCE 7-10 always defines an upper bound level of demand estimates, being the NZS 1170.5:2004 results in the interval defined by both European and American methods.

6 CONCLUSIONS

In this paper a comparison between the European, American and New Zealand methods for selecting and scaling real ground motion records is presented in the context of the seismic assessment of existing steel buildings. The selection techniques stipulated in the codes were discussed along with the impact of including an additional selection criterion based on the control of mismatch relative to target spectrum within a band of ±50%. Furthermore, the effects of using a minimum number of three records, as defined in all documents, in comparison to the most accepted number of seven records was discussed.

The application of the various methods to four steel buildings designed according to different criteria allowed identifying not only the limitations of using only three real records per set, and specially of having to compute the maximum value of response, but also the advantage of considering the additional selection criterion in the reduction of the level of variability of the response. Considering the comparison between the various code-based scaling methods, it was concluded that Eurocode tends to underestimate structural demands. On the other hand, the American standard always leads to higher values of seismic response.

It becomes clear from the study, as already concluded in previous works [14, 15], that further improvements should be made to the selection and scaling methods prescribed in seismic codes, particularly in terms of having to take the maximum value of response when the minimum number of three records per set is considered.

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