Relevant and minor criteria in real record selection procedures based on spectral compatibility

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ABSTRACT
In this paper, the nonlinear response of a code-conforming four-story reinforced concrete (RC) frame building is analyzed with the aim of comparing different options to obtain sets of code-compliant (Italian seismic code is considered) accelerograms. All the considered sets of records are selected to represent the seismic hazard and other seismological characteristics (e.g., magnitude, distance, site conditions) relevant for the site and the structure under consideration. Moreover, records sets match, on average (in a range of periods) or individually (at the fundamental vibration period of the structure), the same target (i.e., design) spectrum for a case-study site in southern Italy. Results confirm that, if the target spectrum is matched, it may be not strictly necessary to also select records carefully matching design earthquakes in terms of magnitude and distance and soil conditions. Moreover, the linearly scaled records do not show significant differences with respect to the unscaled records, in terms of both peak and cyclic structural response parameters. Finally, it is discussed how, when selecting and scaling ground motion records, one can improve the estimation of structural cyclic response taking into account integral ground motion intensity measures, other than spectral accelerations.

1 INTRODUCTION

Modern seismic codes, such as Eurocode 8, or EC8 (CEN, 2003), and the new Italian Building Code, or NIBC (CS.LL.PP., 2008), enable the practitioner to use different analysis methods for both earthquake-resistant design and seismic assessment of existing structures. While buildings are usually designed for seismic resistance using elastic analysis method implicitly expecting them to experience significant inelastic deformations under large earthquakes, nonlinear dynamic analysis (NLDA) represents the tool for explicitly calculating structural response beyond the elastic range.

Seismic assessment of structures via NLDA requires a suitable set of ground motion accelerograms to represent the seismic excitation. Codes suggest different procedures to select ground motions signals, most of those assuming spectral compatibility to the elastic design spectrum, as the main criterion, this also applies to both EC8 and NIBC. More specifically, practitioners have several options to get input signals for their structural analysis: e.g., real or real manipulated records and various types of synthetic and artificial accelerograms. All these options are usually acknowledged by codes which may provide additional criteria or limitations.

Concerning real accelerograms, according to NIBC, selection should match the seismogenetic features of the source and the soil conditions appropriate to the site. This choice may be guided by the disaggregation of seismic hazard for the site of interest (Convertito et al., 2009, Iervolino et al., 2011). NIBC, then requires to render the spectra of the records *similar* to that of the design spectrum. Alternatively, if information about the *seismogenetic features of the source* is not available, it is possible to only match the elastic design spectrum (CS.LL.PP., 2009).

If the latter approach is chosen, the main condition to be satisfied is that the average elastic spectrum (of the chosen set) does not underestimate the 5% damping elastic code spectrum, with a 10% lower bound tolerance (see next section), in the larger range of periods between [0.15s, 2s] and [0.15s, $2T_1$] for safety verifications at ultimate limit state (T_1 is the fundamental period of the structure in the direction where the accelerograms will be applied).

It was discussed (e.g., Iervolino et al., 2008 and 2009) that it may be difficult for practitioners to get code-compliant real record sets if adequate tools are not available. Moreover, inherent limitations of the existing earthquake ground motion databases and conservativism of design spectra lead often to scaling of records to obtain accelerograms consistent with a ground motion target for structural design and evaluation.

This study tries to discuss some of the issues regarding code-based methods for real record selection and scaling. The approach followed is (1) to select (and eventually scale) real ground motions sets using a variety of code-allowable options; (2) to use these ground motions sets as inputs to NLDA of a code-conforming case-study structure and then (3) to analyze the differences in the resulting structural response.

The ground motions selection and modification methods considered in this study are those that rely on selecting natural ground motions and (possibly) linearly scaling their amplitude. (Other methods that use modification of the frequency content of records or simulate ground motion are not included here but are analyzed in other studies; e.g., Iervolino et al., 2010b.)

Herein, 18 sets of seven Italian and European records are considered. Preliminary search of records in terms of earthquake magnitude, source-to-site distance and site classification is considered according to code. Moreover, each record set matches, via the average of the spectral ordinates (in a range of periods), or individually record-by-record (at the fundamental vibration period of the considered structure), the same design spectrum for a case-study site in southern Italy.

The seismic response of an Italian code-conforming four-story reinforced concrete frame building is analyzed. As structural response measures, or engineering demand parameters (EDPs), maximum inter-story drift ratio (MIDR), roof drift (RD) and the total (cumulative) hysteretic energy (E_H) are considered.

Analyses here aim at comparing statistically the differences in the EDPs associated to each set of records.

1.1 Background on record selection for NLDA

Lack of knowledge about the influence of seismological parameters on the structural response had driven general prudence to assume that several earthquake features matter to structural response. In particular, earthquake magnitude (M) and distance (R) of the rupture zone from the site of interest are the most common parameters related to a seismic event. Then, it is evident that the simplest selection procedure involves identifying and matching these parameters during selection. However, a structural designer who attempts to select records based on some seismological features, not being a seismology expert and knowing little detail about the probabilistic seismic hazard analysis (PSHA) framework (McGuire, 1995), may face a difficult task. Moreover, given the limited availability of recorded ground motions within relatively narrow magnitude and distance range appropriate to a given site condition, there is often a need to relax these constraints.

Based on all these considerations, recent studies have questioned the effectiveness of the basic selection procedure. More specifically, in Iervolino and Cornell (2005) the dependence of structural response on (M,R) pair was studied. Conclusions based on investigating the non-linear response (i.e., maximum drift and inter-story drift) of a series of structures to sets of records selected by matching a specific moderate-magnitude and distance scenario (thus simulating the case of a carefully chosen scenario) and other records selected arbitrarily (without limitation, other than being scaled to the median, first-mode spectral acceleration period of the first class of target sets), showed no evidence that the first, site-specific (M,R) record pair selection process was different in terms of predicted structural response. Although the cases made in that study were admittedly limited.

Before of that, Shome et al. (1998) pointed out the insensitivity of some post-elastic damage indices (i.e., three local and three global measures) to (M,R) for a five-story steel structure subjected to real records. This conclusion, however, may be not quite valid in the case of cumulative damage measures (i.e., energy-based indices), since these damage measures shown some form of dependency on record duration and then, indirectly, on (M, R).

In Iervolino et al. (2006) this conclusion was confirmed by examining the non-linear response of a number of SDOF systems and considering six different demand indices. By selecting real accelerograms (three sets characterized by short, moderate and large duration) as representative of specific duration scenarios, it was concluded that duration is insignificant for displacement-based demand indices while it influences energy-based measures such as hysteretic ductility and equivalent number of cycles.

Other studies (e.g., Luco and Bazzurro, 2007; Iervolino et al., 2010b) demonstrated that scaling is not only legitimate (i.e. no bias is induced in median response) but also useful for assessing structural response, reducing the record-to-record spectral variability within a set, which is a desirable feature if one has to estimate the seismic demand on the basis of a limited number of analyses only, as discussed below. In general, the belief that scaling procedures bias non-linear structural response is mostly based on unquestionable differences in ground motion characteristics (e.g., response spectral shape, duration, etc.) and much less on their effects on structures.

The study presented here is aimed at consolidating all the concepts illustrated and at improving the knowledge on the topic from a structural engineering perspective; moreover, here, code-based procedures in record selection are specifically considered and code-based criteria are taken into account; special attention is given also to additional selection criteria in terms, for example, of local geological conditions.

2 CASE-STUDY STRUCTURE

A modern, NIBC-conforming, four-story RC frame building is selected as case-study.

2.1 *Design principles and structural features*

The geometry of the case-study structure is reported in Figure 1; it is regular (both in plan and in elevation) with 4-bay by 2 bay and with total dimensions in plan equal to 19 x 10 m^2.

The bottom interstory height is equal to 4 m, while at other levels it is equal to 3.2 m; at the first story the dimensions of the cross-sections (all the beams and columns have rectangular cross-sections) of all the columns are 30 x 55 cm^2, of all the beams are 30 x 50 cm^2, at the second story such dimensions are respectively 30 x 50 cm^2 and 30 x 45 cm^2, at the third 30 x 45 cm^2 and 30 x 40 cm^2, while at the top level they are 30 x 40 cm^2 both for column and beam sections.

A small variation of the element dimensions between adjacent floors is assigned in order to favor the structure’s vertical regularity, while columns dimensions are kept larger than beam ones in order to take into account the capacity design.

The dimensions of the frame elements are assigned in order to satisfy the *damage limitation requirement* (the limitation for buildings having non-structural elements of brittle materials attached to the structure), resulting in warranting MIDR less than 0.5% for a 50-years exceedance return period of seismic action.

In order to take the cracking into account, the elements’ stiffness is assigned to be one-half of the corresponding uncracked one. The first two modes are translational, whereas the thirds mode is torsional; the periods of the first three modes of such models are 1.06s (X dir), 0.98s (Y dir) and 0.74s (rot.).

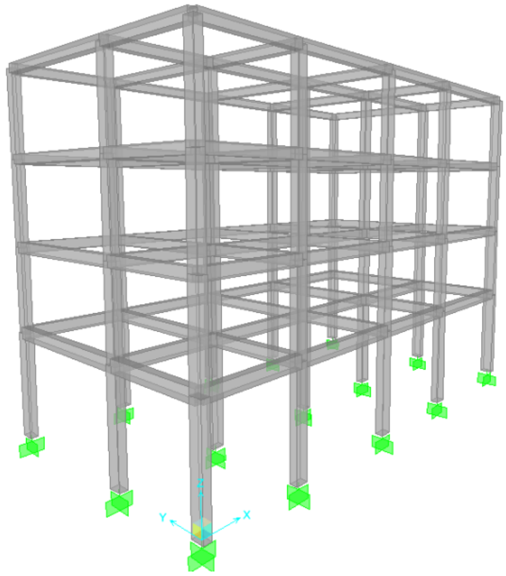

Figure 1. Geometry of the case-study building.

The building is designed according to NIBC by modal response spectrum analysis; gravity load is represented by the uniformly distributed load on the beams and live load of 2 kN/m^2 is adopted in the design of the structure.

As imposed by the code, a 5% accidental eccentricity is considered and then four models are analyzed whit the centre of mass place in four different positions.

A design spectrum according to NIBC is considered for a case-study site in Ponticelli, Naples (latitude: 40.8516°, longitude: 14.3446°), southern Italy. The elastic spectrum considered is that corresponding to the life-safety limit state of an ordinary construction with a nominal life of 50 years (i.e., corresponding to a return period of exceedance of seismic action equal to 475 years) on B-type (stiff soil) site class, according to EC8 classification.

The building is designed to meet the provisions of high ductility class (DCH) as specified in NIBC (that are perfectly consistent with EC8 rules) and assuming a behavior factor equal to 5.85, also considering that the frame is regular in elevation.

Concrete characteristic cylindrical strength equal to f_{ck} = 25 MPa (medium value, according to NIBC, equal to 33 MPa) and steel characteristic yielding strength equal to f_{yk} = 450 MPa are adopted. The concrete Young modulus and its maximum tensile strength are also computed according to NIBC; steel Young modulus is assumed equal to 200000 MPa.

The columns are reinforced by 16 mm bars, with a total reinforcement ratio equal to 1% (minimum longitudinal reinforcement in columns according to both NIBC and EC8) for all cross-sections. Beams are reinforced by 14 mm bars with symmetric reinforcement. Stirrups with a 8 mm diameter each 10 cm have been defined in all the *critical regions* (where plastic hinges are expected to form) of the columns based on codes’ limitations.

Design always meets the code requirements as close as possible to minima.

2.2 Nonlinear model

Because the structure meets fully the regularity requirements, the main central frame in the structure is extracted (fundamental period T_1 = 1s) and used as the structural model for NLDA; only one direction of seismic action is analyzed.

Analyses are performed by means of the Open System for Earthquake Engineering Simulation (OpenSees, http://opensees.berkeley.edu/index.php/) software. Both beams and columns are modelled by one-component lumped plasticity located at element's ends. Nonlinearity regards only flexural rotations.

Plastic hinges are characterized by a tri-linear backbone curve (Figure 2), defined by yielding (M_y) and maximum (M_{max}) moment and the corresponding rotations. Codes do not provide an expression for rotation at maximum moment, θ_{max} and this rotation value is therefore assumed arbitrarily equal to $0.5\theta_u$, where θ_u is the ultimate rotation.

The moment-rotation relationship in the positive and negative direction is symmetrical for both beams and columns due to symmetrical reinforcement. Such moments and the corresponding curvatures are computed by analyzing the cross-sections of the elements considering the Mander-Priestly (1988) constitutive relationship for confined concrete under compression.

An elastic-perfectly plastic steel stress-strain diagram is considered, characterized by a maximum strength equal to 548 MPa, computed as mean of tests on more than 200 bars made by steel 430 MPa grade (Galasso et al., 2010).

The yielding (θ_y) and ultimate (θ_u) chord rotations and the plastic hinge length are evaluated using semi-empirical equations provided by the instructions for the implementation of the NIBC (CS.LL.PP., 2009), where the already cited average values are assigned to concrete maximum (f_c) and steel yielding (f_y) strength. Zero axial force and the axial load due to gravity loads are taken into account when determining the moment-rotation relationship for beams and columns, respectively. The possibility of shear failure is not taken into account; this should be allowed by the capacity-design approach implicit in the code.

As it regards the post-peak behavior, it is assumed that the section resistance drops to a value of the moment equal to one tenth of the maximum moment (M_{max}) corresponding to a rotation value equal to the ultimate value (θ_u). These post-peak values for moment and rotation are chosen rather arbitrarily in order to avoid numerical in-convergence problem. The structural damping is modeled based on the Rayleigh (Chopra, 2007) model and it is assumed to be equal to 5% for the first two modes.

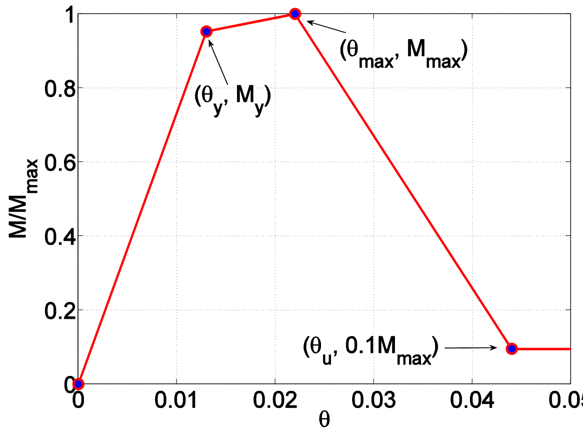

Figure 2. Sketch diagram of the monotonic behavior of plastic hinge model used in this study.

2.3 Nonlinear demand measures

EDPs chosen are selected to investigate both the peak and the cyclic seismic response and then to be representative of the most common building damage measures. The displacement-based parameters are MIDR and RD. These parameters are of interest for both code-based design checks as well as performance-based assessment, and there is much research experience in prediction these response parameters.

The cyclic response-related parameter is E_H, evaluated as the sum, for all the plastic hinges in the model, of the areas of the hysteretic cycles (Manfredi, 2001).

3 RECORD SETS

As discussed above, code-consistent ground motion selection requires the definition of a benchmark to compare selection procedures and, eventually, manipulation of records to match this target.

In an effort to analyze structural response when it goes severely in the nonlinear range, a strong target ground motion level (in terms of elastic response spectrum) was defined for Ponticelli; i.e., corresponding to an exceedance return period of 2475 years.

The considered sets were selected using REXEL 3.1 (beta), software that is freely available at http://www.reluis.it/, which allows users to select combinations of seven or thirty multi-component natural records contained in the European Strong Motion Database (ESD) and the ITalian ACcelerometric Archive (ITACA), which on average match a code-based or user-defined elastic spectrum in a desired period range and with specified upper and lower bound tolerances (Iervolino et al. 2010a). Record may be pre-selected by magnitude, distance, site classification and by peak and integral ground motion intensity measures accounting for *disaggregation* and *conditional hazard*, which are options also available in REXEL.

A all the sets averages are selected to be within [-10%,+30%] tolerance range (few exceptions were allowed) with respect to the target spectrum and in most of the compatibility interval they approximated the target spectral shape very well (see next section).

Each set is comprised of seven records, this is based on code requirements. In fact, seven is the minimum set size for which it is possible to consider the average structural response as design value.

3.1 Selection strategies

In Figure 3, disaggregation of seismic hazard in terms of Sa(T=1s) (i.e., spectral acceleration at period equal to 1s) for the site (for a return period of 2475 years) is shown as obtained by REXEL. The joint distribution of M and R for Ponticelli has a bimodal shape; i.e., two relative maxima exist (Iervolino et al., 2011). As a consequence, for the spectral ordinate at the fundamental period of the structure, at least two design earthquakes (DEs) exist: one representing closer, smaller magnitudes, i.e., (5.8,6.5km), and the other more distant, larger magnitudes, i.e., (7.3,55km).

Two groups of spectrum-compatible sets are identified: the first group comprises 8 sets of seven records carefully chosen to reflect, within tolerable limits (i.e. bins centered on modal values), the specific magnitude and distance scenario corresponding to the first DE; the second group comprises 6 sets of seven records chosen to reflect the specific magnitude and distance scenario corresponding to the second DE. All the considered combinations belonging to the two first groups of accelerograms are compatible, via their averages, with the target spectrum, in the range of periods 0.15 s – 2 s. Within each group, the sets differ for the source database (ESD or ITACA), for the soil class of the records (*same as target spectrum* or *any site class*) and finally, for being not manipulated (original records) or linearly scaled in amplitude, controlling the mean (for set) scale factors (SF_{mean}) between 5 and 15.

Two additional groups, each of those including 2 seven-record sets (one for each database) are considered: these records are selected to match the values of magnitude and distance corresponding to the DEs and the soil condition of the site (i.e., B-type soil class) and are then scaled to match precisely the target spectrum level at the fundamental period of the structure (T_1 = 1s).

Information on selected set, divided by groups, is summarized in Table 1 – Table 4 (detailed information can be found in Mezza, 2010, and Sica, 2010). Figure 4 and Figure 5 show the average spectra of each ground motions set, divided by groups, together with the target spectrum and the range of periods of interest. In order to have statistically independent results, it is desirable to select combinations having no accelerograms in common. This requirement conflicts with limitations of the reference ground motion databases, especially if one looks at the relatively large target spectrum and the corresponding controlling earthquake scenarios. Then, some exceptions (sets sharing more than 2 records) are allowed because complete avoidance of overlap was not feasible.

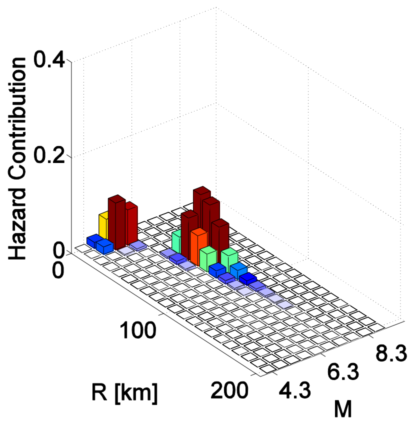

Figure 3. Disaggregation results for Ponticelli for Sa(T=1s) and for a 2475-years return period (from REXEL 3.1 beta).

Table 1. Summary of records sets of Group I

Matching target spectrum on average, with [-10%, +30%] tolerance, over the periods' range 0.15 s – 2 s				
Set no.	*Database*	*Site class*	*Scaling/SF_{mean}*	*Design earthquake (M,R)*
1	ESD	B	Yes/5	Disag. 1st mode
2*	ESD	B	No	Disag. 1st mode
3	ESD	Any site class	Yes/5	Disag. 1st mode
4	ESD	Any site class	No	Disag. 1st mode
5**	ITACA	B	Yes/10	Disag. 1st mode
6**	ITACA	B	No	Disag. 1st mode
7**	ITACA	Any site class	Yes/5	Disag. 1st mode
8**	ITACA	Any site class	No	Disag. 1st mode

* Lower tolerance is increased to 15% to find a compatible set.

** Lower tolerance is increased to 20% to find a compatible set.

Table 2. Summary of records sets of Group II

Matching target spectrum on average, with [-10%, +30%] tolerance, over the periods' range 0.15 s – 2 s				
Set no.	*Database*	*Site class*	*Scaling/SF$_{mean}$*	*Design earthquake (M,R)*
9**	ESD	B	Yes/5	Disag. 2st mode
10***	ESD	B	No	Disag. 2st mode
11	ESD	Any site class	Yes/5	Disag. 2st mode
12	ESD	Any site class	No	Disag. 2st mode
13**	ITACA	B	Yes/15	Disag. 2st mode
14	ITACA	Any site class	Yes/15	Disag. 2st mode

** Lower tolerance is increased to 20% to find a compatible set.

*** Lower tolerance is increased to 25% to find a compatible set.

Table 3. Summary of records sets of Group III

Matching target spectrum precisely at T = 1s				
Set no.	*Database*	*Site class*	*Scaling/SF$_{mean}$*	*Design earthquake (M,R)*
15	ESD	B	Sa(T$_1$) scaling	Disag. 1st mode
16	ITACA	B	Sa(T$_1$) scaling	Disag. 1st mode

Table 4. Summary of records sets of Group IV

Matching target spectrum precisely at T = 1s				
Set no.	*Database*	*Site class*	*Scaling/SF$_{mean}$*	*Design earthquake (M,R)*
17	ESD	B	Sa(T$_1$) scaling	Disag. 2st mode
18	ITACA	B	Sa(T$_1$) scaling	Disag. 2st mode

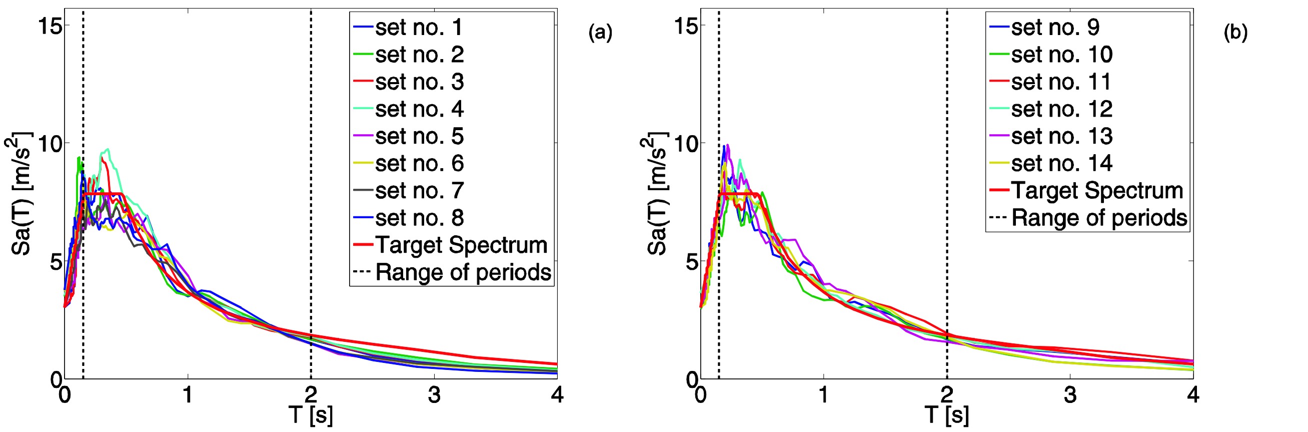

Figure 4. Average spectra for each individual set of seven records selected (and eventually scaled) to be compatible (on average) with the target spectrum in [0.15-2s] periods range (a) Group I (1st disaggregation mode-based); (b) Group II (2nd disaggregation mode-based).

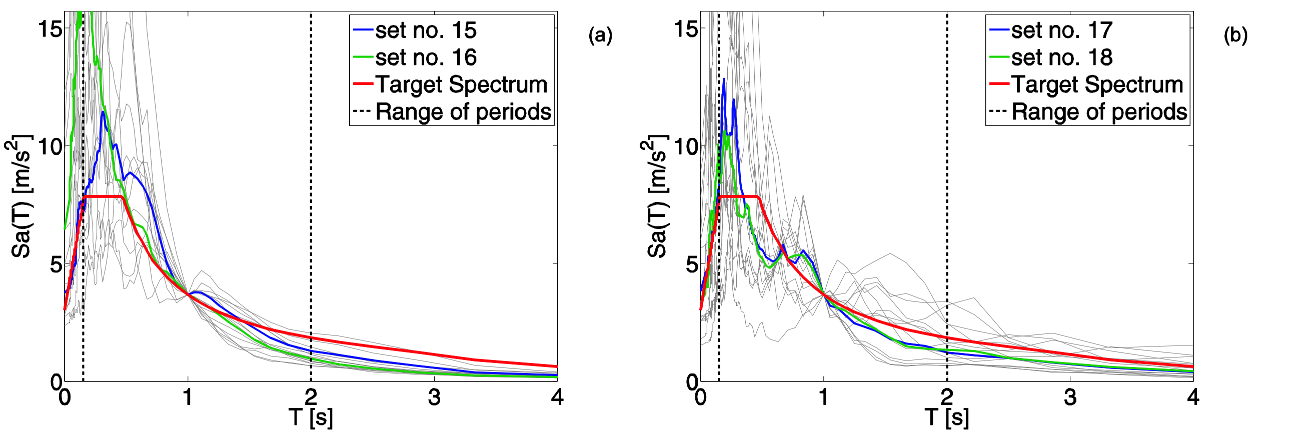

Figure 5. Individual and average spectra for each individual set of seven records selected and scaled to match precisely T=1s-ordinate of the target spectrum (a) Group III (1st disaggregation mode-based); (b) Group IV (2nd disaggregation mode-based).

4 RESULTS AND DISCUSSIONS

All records selected for each group are used as input for NLDA applied to case-study structure. The MIDR for all the records sets is shown in Figure 6, which shows the individual response predictions from each record (x-markers) as well as the mean values for each set (squares) and the mean plus (and minus) one standard deviation (crosses).

There is large scatter in the MIDR values predicted for each record of a single ground motion suite, as expected, while there is not large variability in the median predicted values from each set. This visual evidence will be confirmed by the hypothesis tests described in the following section.

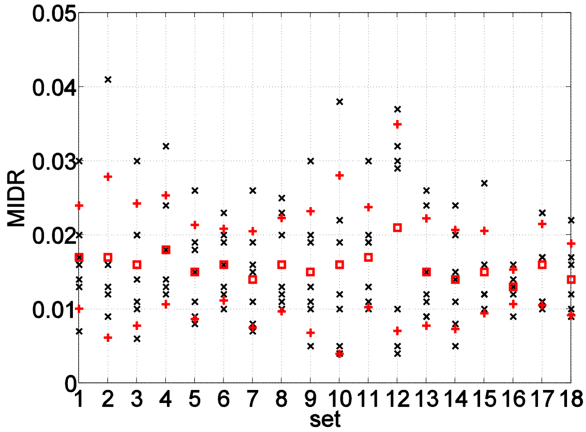

Figure 6. Summary of MIDR responses for the selected ground motions sets.

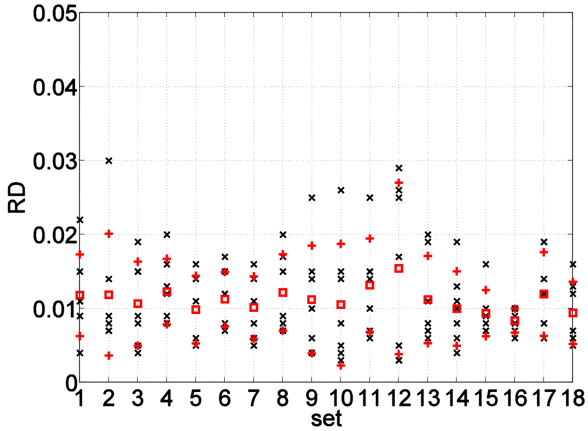

Figure 7. Summary of RD responses for the selected ground motions sets.

The RD ratios and the total hysteretic energy for all the records sets are shown in Figure 7 and in Figure 8 respectively with the same representation style of Figure 6. Large variability in the median predicted values of E_H may be observed (Figure 8). The median of the response parameters, for each pair of sets, are compared statistically for equality (or not) in the next section.

Note that Figure 6-Figure 8 show, obviously (Shome et al., 1998), how standard deviation of response for scaled combinations are generally smaller if compared with the values of standard deviation in the case of unscaled sets, while medians appear unaltered.

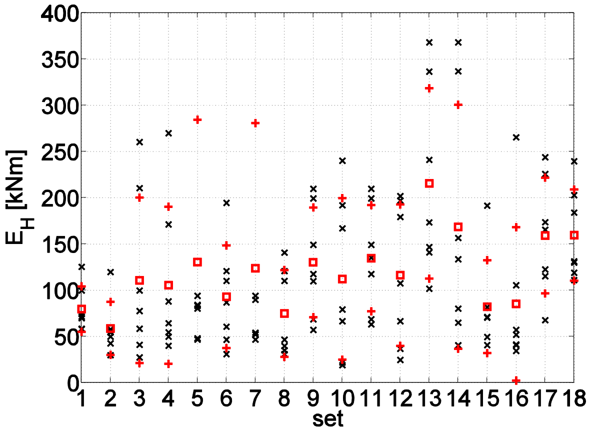

Figure 8. Summary of E_H responses for the selected ground motions sets.

4.1 *Hypothesis tests*

Parametric hypothesis tests (Benjamin and Cornell, 1970) are performed to assess to what significance the median values of the response, from a given set of records, may be considered equal to that from another class. Hypothesis tests are performed for both peak and cyclic EDPs, assuming a lognormal distribution for all the response parameters of interest. This distribution assumption are checked with the Shapiro-Wilk (1965) test and could not be rejected at the 95% significance level in all cases.

The null hypothesis to check is whether the median EDPs for a given set (i) are equal (null hypothesis) or not (alternate hypothesis) to that from the set (j). To this aim, a two tail Aspin-Welch (Welch, 1938) test is preferred with respect to the standard Student t-test as the former does not require the assumption of equal, yet still unknown, variances of populations originating the samples.

The test statistic employed is reported in Eq. (1), in which z_i and z_j are the sample means, s_i and s_j are the sample standard deviations and n and m are the samples sizes (in this case always equal to 7). The test statistic, under the null hypothesis, has a Student-t distribution with the number of degrees of freedom given by Satterthwaites's approximation (Satterthwaites, 1949).

$$t = \frac{z_x - z_y}{\sqrt{\frac{s_x^2}{n}} + \sqrt{\frac{s_y^2}{m}}} \tag{1}$$

As an example of the results, not reported here for the sake of brevity, in Table 5 the *p*-values for Aspin-Welch test in terms of E_H are reported. Bold font values are the rejection cases assuming a 95% significance level; i.e., choosing I-type risk (α) equal to 0.05. The hypothesis tests show that there are 2/153 (i.e., < 2%), 3/153 (i.e., ≅ 2%) and 7/153 (i.e., ≅ 5%) for MIDR, RD and E_H respectively.

For both MIDR and RD, the results found show that there are no rejections in comparing the seismic response to accelerograms selected accurately based on magnitude, distance and soil condition and then scaled to match precisely the target spectrum (sets 15-18, Groups III-IV) and to accelerograms matching the target spectrum on average in a broader range of period (sets 1-14), independently from the (M,R) pair of interest (Groups I and II), suggesting that there is no reason to not consider these groups equivalent with respect to these structural responses. In fact, spectrum matching in a broad range of periods, without a careful site-specific process of record selection by magnitude, distance and soil class may be considered equivalent to exact spectrum matching at the fundamental period, with an accurate site-specific process of record selection based on disaggregation and site conditions.

Looking only at spectrum matching sets over the periods range 0.15 s – 2 s (Groups I and II) and looking at pairs of sets that significantly differ in terms of average (M, R) pairs characterizing the set (first DE versus second DE) with all other conditions (database, site class and scaling) being equal, e.g. 1-9, 2-10, 3-11, 4-12, 5-13, 7-14, no rejections are observed for all the EDPs considered, including E_H.

Similarly, looking only at spectrum matching sets precisely at T = 1s (Groups III and IV) and looking at pairs of sets 15-17 and 16-18 (first DE versus second DE) no rejections are observed for all the EDPs considered, including E_H.

In fact, the nonlinear response of case-study structure are, at least in these examples, independent on M and R beyond the dependence through the intensity (i.e., spectral ordinates) level. (This may appear not expected for what discussed above about dependency of E_H on magnitude and distance. In fact, looking at integral parameters of ground motions compared sets are not significantly different; see next section).

Similarly, considering pairs of sets that differ only for soil condition (same as target spectrum versus any site class), i.e., 1-3, 2-4, 5-7, 6-8, 9-11, 10-12, 13-14 with all other conditions being equal (design earthquake, database and scaling), no rejections are observed for both peak and cyclic response parameters. It is suggested that if the spectral shape is assigned in a large period range, the site class of real records may be of secondary importance.

Finally, note that the sets of linearly scaled records (i.e., 1, 3, 5, 7, 9, 11) do not show systematic difference with respect to those unscaled with all other conditions being equal (i.e., 2, 4, 6, 8, 10, 12) for all the types of response considered, which seems to confirm that amplitude scaling is a legitimate practice, as many studies point out, if the spectral shape is controlled.

4.2 Including additional record selection criteria: Cosenza and Manfredi index

If the cyclic response is considered, some sets show significant differences in the prediction of seismic demand (i.e., *p*-values lower than 0.05). These differences were a predictable result looking at the integral intensity measures, characterizing each record (see also Iervolino et al., 2006). To this aim, each record of the 18 sets is processed to evaluate its characteristics other than spectral shape; for each set, average values of the *Arias intensity*, I_A, Eq. (2), and of the *Cosenza and Manfredi index*, I_D (Manfredi, 2001), Eq. (2), taken as the average on the sample of seven records, are computed. In Eq. (2), $a(t)$ is the signal's accelerometric time-history, whose duration is equal to t_E and PGA and PGV represent the peak ground acceleration and velocity respectively.

$$I_A = \frac{\pi}{2g}\int_0^{t_E} a^2(t)\,dt; \qquad I_D = \frac{2g}{\pi}\frac{I_A}{PGA \cdot PGV} \tag{2}$$

Figure 9a shows the I_A versus E_H plot; similarly, Figure 9b shows the I_D versus E_H plot; the estimated linear regressions (for both I_A and E_H and I_D and E_H) are reported in the legend of the two panels of Figure 9. In this plots, to increase the significance of the results, 15 additional sets of records are selected for another case-study site in southern Italy; i.e., S. Angelo dei Lombardi (15.1784°, 40.8931°).

It is possible to note a fairly good correlation in both cases, confirmed by statistical tests on coefficients of regressions and on the estimate correlation coefficients (these results are not reported for the sake of brevity).

In Table 6, the values of Cosenza and Manfredi index for rejection cases of Table 5 are reported: generally, significant differences in I_D values characterizing the records, imply significant differences in the consequent E_H response.

Table 5. Aspin-Welch test results in terms of E_H; *p*-values lower than 0.05 are reported in bold.

	Group I								*Group II*						*Group III*		*Group IV*	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	1.00	0.36	0.67	0.48	0.21	0.44	0.54	0.26	0.59	0.40	0.97	0.13	0.10	0.63	0.56	0.56	0.66	0.33
2		1.00	0.07	0.89	**0.03**	0.16	0.10	**0.02**	0.83	0.07	0.43	**0.01**	**0.01**	0.07	0.69	0.16	0.18	0.07
3			1.00	0.31	0.22	0.56	0.72	0.21	0.32	0.51	0.67	0.10	0.06	0.91	0.18	0.72	0.88	0.41
4				1.00	0.12	0.23	0.26	0.16	0.79	0.21	0.51	0.10	0.08	0.29	0.72	0.27	0.32	0.17
5					1.00	0.87	0.45	0.66	0.10	0.65	0.23	0.86	0.75	0.25	0.05	0.59	0.43	0.84
6						1.00	0.72	0.92	0.26	0.87	0.45	0.78	0.71	0.59	0.23	0.79	0.67	0.99
7							1.00	0.63	0.27	0.79	0.55	0.32	0.25	0.78	0.19	0.93	0.90	0.64
8								1.00	0.13	0.89	0.30	0.46	0.33	0.27	**0.04**	0.79	0.58	0.88
9									1.00	0.20	0.63	0.07	0.05	0.30	0.93	0.31	0.36	0.17
10										1.00	0.42	0.50	0.41	0.56	0.13	0.89	0.71	0.83
11											1.00	0.16	0.13	0.64	0.62	0.56	0.66	0.34
12												1.00	0.88	0.13	**0.02**	0.48	0.32	0.72
13													1.00	0.08	**0.01**	0.41	0.26	0.62
14														1.00	0.17	0.76	0.93	0.45
15															1.00	0.26	0.30	0.12
16																1.00	0.85	0.74
17																	1.00	0.59
18																		1.00

Table 6. I_D values for rejection cases of Table 5.

Set	I_D	*Set*	I_D	*p-value*
2	5.2	5	10.7	**0.03**
2	5.2	12	10.6	**0.01**
2	5.2	13	23	**0.01**
15	6.3	8	5.3	**0.04**
15	6.3	12	10.6	**0.02**
15	6.3	13	23	**0.01**

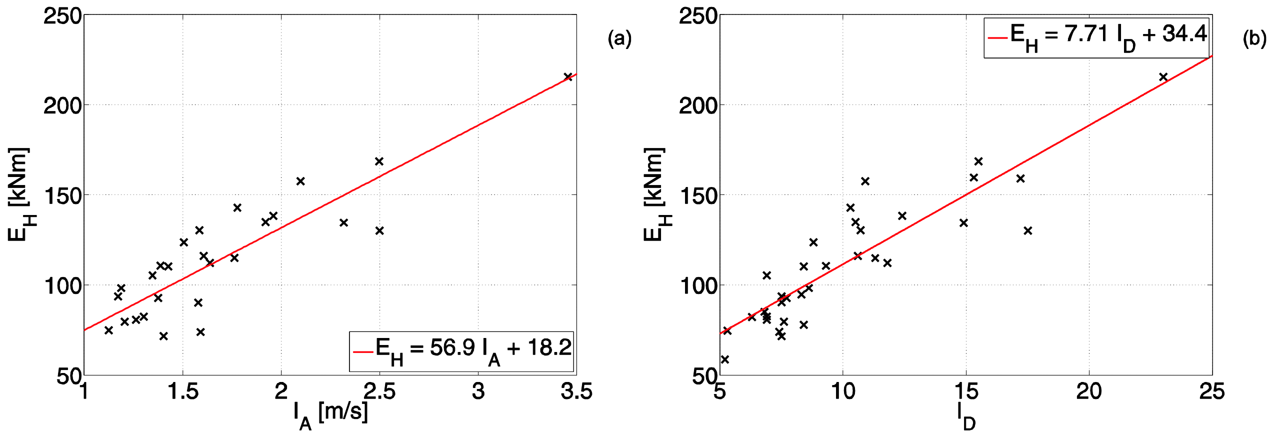

Figure 9. E_H versus (a) I_A and (b) I_D for the considered sets.

In fact, I_D (or other parameters of the same kind) can be suggested as an additional criterion in selection (or generation) procedures for accelerograms when the cyclic response represents a critical performance parameter for the structure to be analyzed.

In Iervolino et al. (2010c) an easy yet hazard-consistent way of including secondary intensity measures in record selection was presented. The proposed methodology requires mainly the manipulation of result from the scalar PSHA and allows to develop conditional hazard maps, i.e., maps of secondary ground motion intensity measures (e.g. integral parameters) conditional, in a probabilistic sense, to the design hazard for a primary parameter (e.g., a spectral ordinate).

5 CONCLUSIONS

In this study, different ways to obtain code-conforming real records sets are compared in terms of post elastic seismic peaks and cyclic structural response. This was pursued by considering a modern code-conforming (Italian code is considered) four-story RC frame building at high nonlinearity levels and a ground motion scenario (a reference site and the corresponding code-response spectra) as a case study.

Maximum inter-story drift ratio, roof drift and the total (cumulative) hysteretic energy were analyzed with respect to 18 sets of natural ground motion accelerograms obtained exploring all the possible options allowed by modern building codes in real record selection and modification.

Hypothesis tests were carried out with the aim of assessing quantitatively how significant there results are. Tests results confirm literature results, that is, no particular care is required in selecting records with respect to magnitude, distance and soil condition if accelerograms match, individually or in an average sense, a target spectral shape. Moreover, results indicate that the linearly scaled records do not show any systematic trend with respect to the unscaled record results independently of response parameters if the spectral shape is controlled in selection.

Finally, it worth noting that, as it is well known, some differences in cyclic response may be observed. These differences may be predicted by some integral parameter of ground motion, which, if an appropriate hazard analysis tool is available, could be used as an additional criterion for record selection, especially in those cases when cyclic behavior has an important role in determining the seismic performances.

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