NON LINEAR DYNAMIC RESPONSE VARIATION UNDER DIFFERENT SETS OF EARTHQUAKES

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ABSTRACT:

The paper deals with the effects of different seismic input on systems structural response, considering the variability of the input on dynamic non linear analyses performed according to Eurocode 8 (EC8) rules. A comparison between the systems response under six sets of accelerograms is reported in the paper. Every set is made of 7 earthquakes (both the horizontal components are considered), fully satisfying the EC8 provisions.
SDOF systems and a space frame (MDOF) are analysed. The results concerning the former systems are provided in terms of variation of strength reduction factors and ductility; they are compared to results obtained on corresponding elastic systems (elastic spectrum variation).
The analyses are extended to a reinforced concrete multi-storey space frame, which is designed according to Eurocode 2 (EC2) and EC8 provisions. Results in terms of frame top displacements, interstorey drifts and chord rotations at element ends, as well as in terms of variation coefficients of these parameters, are discussed.
It is shown that the EC8 provisions concerning the selection of the seismic input sets (the main condition to be satisfied by the set is that the average elastic spectrum does not underestimate the code's spectrum, with a 10% tolerance, in a broad range of periods depending on the structure's dynamic properties) are not completely effective, because they do not take into account the variability of the elastic spectra, which largely conditions the non linear response of structures.

KEYWORDS: Input selection, non linear analyses, natural records, response variation, Eurocode 8.

1. INTRODUCTION

The most of the papers concerning the effects of different seismic input on response of structures deal with the variability of the input in terms of magnitude, frequency content (Krawinkler et al., 2003; Wang et al. 2002), PGA (Peak Ground Acceleration), PGV (Peak Ground Velocity), PGD (Peak Ground Displacement), EPA (Effective Peak Acceleration), EPV (Effective Peak Velocity), Arias Intensity, Duration (Amiri and Dana, 2005; Van de Lindt and Goh, 2004), dissipated energy and damage indices (Cosenza and Manfredi, 2000).
In the present paper the effects of the variability of seismic input on systems structural response are discussed; the non linear response of SDOF systems and of a space frame (MDOF) is analysed. The originality of the presented study concerns the analysis of results obtained using as input sets of accelerograms fully satisfying the code provisions in terms of compatibility to an elastic design spectrum. This allows to set the topic of the effects of the variability of seismic input in a defined frame and also to analyse international code provisions.
The analyses are extended to an r/c multi-storey space frame, which is designed according to Eurocodes (CEN, 2004a; CEN, 2003) provisions. Elastic analyses are performed by the computer program SAP2000 (CSI Computer & Structures, 2004), while non linear ones by CANNY99 (Li, 1996); the average and the variation coefficient (CoV) of the results in terms of frame top displacements, interstorey drifts and chord rotations at element ends of seven non linear time history analyses are discussed.

2. SEISMIC INPUT

Both elastic and non linear response of SDOF systems and of a space frame are analysed, considering three sets of accelerograms, each of them made of 7 earthquakes; both the horizontal components of each earthquake are considered, consequently each set is characterised by 14 natural records. The number of earthquakes characterising each set is chosen taking into account the Eurocode 8 (EC8) (CEN, 2003) provision prescribing the possibility to use as design value of the action effect the average of the response quantities from 7 non linear time history analyses. The records are selected according to the procedure presented in Iervolino et al. (2008), in order to satisfy the compatibility with an elastic design spectrum. These provisions are: a) the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of $a_g \cdot S$ for the site in question; b) in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure, no value of the mean 5% damping elastic spectrum, calculated from all time histories, is less than 90% of the corresponding value of the 5% damping elastic design spectrum; the ordinate of the mean elastic spectrum at the period T_i is the average of the ordinates at the same period of the elastic spectra of the set natural records. The range of periods where the compatibility elastic design spectrum – natural records spectra is imposed is 0.04 s – 2 s; consequently, according to the EC8 provision b), the selected sets are suitable for analysis of structures with fundamental period belonging to the interval 0.2 s – 1 s, i.e. for many common buildings.
Figure 1 shows, for each of the three sets used, the elastic spectra of the 14 records, along with their average spectrum (black smooth line), the reference EC8 elastic design spectrum (black thick line) and the curve whose ordinates are equal to 90% of such spectrum ones (black thin line).

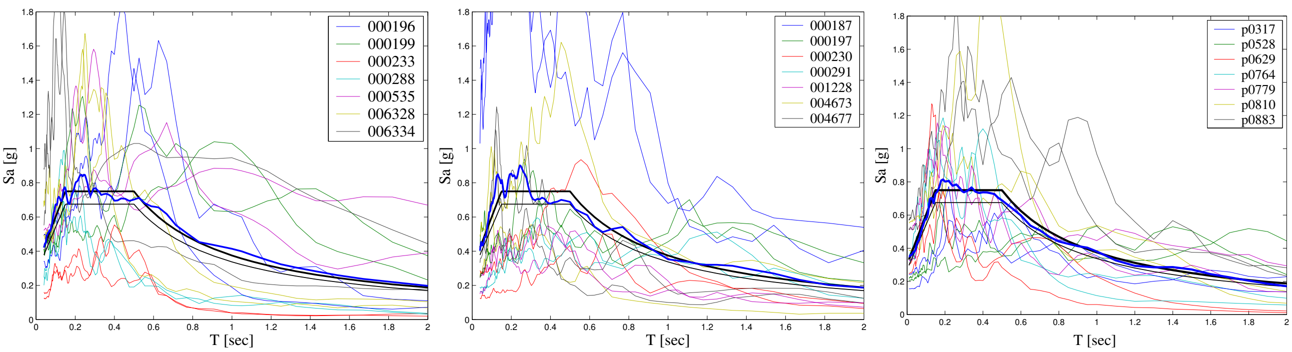

Figure 1 Elastic spectra of set 1 (left), set 2 (centre) and set 3 (right) from ESD and PEER (set 3) database (ESD, 2007; PEER, 2007)

The matched code spectrum is the one used for the design of the analysed building (MDOF system), assigning q=1; the parameters which characterise such spectrum are: the peak ground acceleration on stiff soil a_g equal to 0.25g, corresponding to second hazard level according to the Italian code (PCM, 2003) and the soil factor S equal to 1.20 (EC8 soil type B).
Set 1 and set 2 are selected from European Strong-motion Database (ESD, 2007), while set 3 from the Pacific Earthquake Engineering Research (PEER) Strong Motion Database (PEER, 2007). The details of records of set 1, set 2 and set 3 are listed in Table 1, 2 and 3 respectively.

Table 1 Set 1: records details

Code	Date	Earthquake name	Country	PGA NS [g]	PGA EW [g]
000196	15/04/1979	Montenegro	Yugoslavia	0.45	0.31
000199	15/04/1979	Montenegro	Yugoslavia	0.38	0.36
000233	24/05/1979	Montenegro (aftershock)	Yugoslavia	0.12	0.15
000288	23/11/1980	Campano Lucano	Italy	0.23	0.17
000535	13/03/1992	Erzincan	Turkey	0.39	0.51
006328	21/06/2000	South Iceland (aftershock)	Iceland	0.33	0.39
006334	21/06/2000	South Iceland (aftershock)	Iceland	0.45	0.72

Table 2 Set 2: records details

Code	Date	Earthquake name	Country	PGA NS [g]	PGA EW [g]
000187	16/09/1978	Tabas	Iran	0.93	1.10
000197	15/04/1979	Montenegro	Yugoslavia	0.29	0.24
000230	24/05/1979	Montenegro (aftershock)	Yugoslavia	0.12	0.27
000291	23/11/1980	Campano Lucano	Italy	0.16	0.18
001228	17/08/1999	Izmit	Turkey	0.24	0.14
004673	17/06/2000	South Iceland	Iceland	0.21	0.48
004677	17/06/2000	South Iceland	Iceland	0.28	0.23

Table 3 Set 3: records details

Code	Date	Earthquake name	Country	PGA NS [g]	PGA EW [g]
p00317	26/04/1981	Westmoreland	United States	0.24	0.16
p00528	08/07/1986	Palm Springs	United States	0.22	0.21
p00629	01/10/1987	Whittier Narrows	United States	0.30	0.25
p00764	18/10/1989	Loma Prieta	United States	0.36	0.33
p00779	18/10/1989	Loma Prieta	United States	0.51	0.32
p00810	25/04/1992	Cape Mendocino	United States	0.39	0.55
p00887	17/01/1994	Northridge	United States	0.57	0.51

In order to improve the fitting between the design elastic spectrum and the set mean elastic spectrum, their deviation δ is also considered:

$$\delta = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\frac{Sa_{mean}(T_i) - Sa_s(T_i)}{Sa_s(T_i)}\right)^2} \qquad (2.1)$$

where $Sa_{mean}(T_i)$ represents the ordinate of the set mean spectrum at the period T_i, while $Sa_s(T_i)$ is the ordinate of the code spectrum at the same period and N is an enough large number of periods belonging to the selected range (0.04 – 2 s): N = 116 is considered. A low δ value may prevent the overestimation of seismic demand. Set 1 is characterised by the lowest δ value among the sets of ESD database, while set 3 by the lowest δ value among the sets of PEER database; set 2 belongs ESD database and it is characterised by fourteen records different with respect the set 1 records (see Tables 1 and 2); this avoids that analyses performed by the two European sets could be conditioned by the same record.

3. RESPONSE OF SDOF SYSTEM

As already quoted, the elastic and non linear responses of SDOF systems subject to all the 14 records of each of the 3 selected sets are evaluated.

For all the records of each set, inelastic spectra are computed for assigned values both of ductility demand μ (μ=4 and μ=5) and of strength reduction factors q (q=4, q=4.68 and q=5).

For each set, for both elastic and inelastic spectra, the variation coefficient (CoV), i.e. the ratio between the sample standard deviation and sample mean (m), is computed. The CoV, for each of the three considered sets, is computed: a) at T_1 = 0.716 s, which is the fundamental period of the four-storey r/c frame building analysed in order to extend the results obtained for SDOF systems to MDOF ones and presented in the following; b) as average of CoV values computed at 100 periods within the range T_1-2T_1, which is relevant for the MDOF system; indeed, every system, during a strong earthquake, is affected by a stiffness reduction due to damage, which, in the case of r/c buildings designed according to High Ductility Class, can cause an increment of the fundamental period until twice the elastic value; c) as average of CoV values computed at 100 periods within the range 0-2 s, which is the range where the records sets satisfy the compatibility with the design elastic spectrum.

The results of analyses are also presented considering a fourth set (set 4), made of all the records of the other three sets; obviously, this set also fully satisfies the quoted EC8 provisions.

In Table 4, the CoV values computed for elastic and non linear spectra of the records of each set are presented. The large values reveal the large dispersion of results; the average CoV value among the four sets is about 0.7 both for elastic and non linear spectra, as confirmed by set 4. However, even though all the sets satisfy the same criteria in terms of selection, a not negligible variability among them of the CoV can be noted: the lowest values are provided by set 3, both considering them at the fixed period T_1 and evaluating their average on 100 periods in the ranges T_1-2T_1 and 0-2 s.

Table 4 CoV values for linear and non linear sets.

Type of spectra	Range of evaluation	Set 1	Set 2	Set 3	Set 4
Linear	T_1	0.71	0.86	0.47	0.69
	T_1-2 T_1	0.86	0.72	0.56	0.72
	0-2 s	0.58	0.91	0.46	0.69
Non linear (μ=4)	T_1	0.81	0.85	0.58	0.75
	T_1-2 T_1	0.96	0.67	0.61	0.76
	0-2 s	0.79	0.71	0.54	0.69
Non linear (μ=5)	T_1	0.84	0.61	0.60	0.69
	T_1-2 T_1	0.93	0.69	0.60	0.74
	0-2 s	0.78	0.72	0.53	0.68
Non linear (q=4)	T_1	0.70	0.85	0.45	0.68
	T_1-2 T_1	0.85	0.68	0.54	0.70
	0-2 s	0.75	0.71	0.53	0.68
Non linear (q=4.68)	T_1	0.69	0.84	0.45	0.67
	T_1-2 T_1	0.84	0.67	0.53	0.69
	0-2 s	0.75	0.70	0.53	0.67
Non linear (q=5)	T_1	0.69	0.83	0.45	0.67
	T_1-2 T_1	0.83	0.67	0.53	0.69
	0-2 s	0.74	0.70	0.52	0.66

In order to analyse the variability of non linear seismic response, strength reduction factors (q) and ductility demand (μ) are also considered; these parameters are important even because they are related to the structural design. For all the records of the four considered sets, two groups of inelastic spectra characterised by a constant ductility demand, i.e. μ=4 and μ=5, are computed; for each group and for each set, the average of the q factors at the reference period T_1=0.716 s is then evaluated: for μ=4 the q factors means are 4.02, 3.96, 4.04 and 4.00 for sets 1, 2, 3 and 4 respectively; for μ=5 the q factors means are 4.99, 4.93, 4.78 and 4.90 for sets 1, 2, 3 and 4

respectively. In Figure 4 (sx), each set corresponds to a square mark; this is characterised by an abscissa equal to the CoV of elastic spectra (of the set) ordinates at T_1=0.716 s and by an ordinate equal to the CoV of the q factors obtained at T_1 by inelastic spectra characterised by a constant ductility demand equal to 4. The triangular marks represent corresponding values of square ones, computed by inelastic spectra characterised by a constant ductility demand equal to 5. In the figure, the interpolation line of the results for both the inelastic spectra groups is also reported along with the square value of correlation coefficients (R^2=1 indicates the maximum correlation); it is evident the good correlation of results, which means that as the CoV of elastic spectra ordinates increases, the CoV of q factors of constant ductility inelastic spectra proportionally increases.
In Figure 4 (dx) as for Figure 4 (sx), each set corresponds to a square mark; this is characterised by an abscissa equal to the CoV of elastic spectra (of the set) ordinates at T_1=0.716 s and by an ordinate equal to the CoV of the ductility demand obtained at T_1 by inelastic spectra characterised by a constant q factor equal to 4. The triangular and circle marks represent corresponding values of square ones, computed by inelastic spectra characterised by a constant q factor equal to 4.68 and 5 respectively. In the figure, the interpolation line of the results for the three inelastic spectra groups is also reported along with the square value of correlation coefficients (R^2); it is evident the good correlation of results, as for those shown in Fig. 4, which means that as the CoV of elastic spectra ordinates increases, the CoV of ductility demand of constant q factor inelastic spectra proportionally increases.

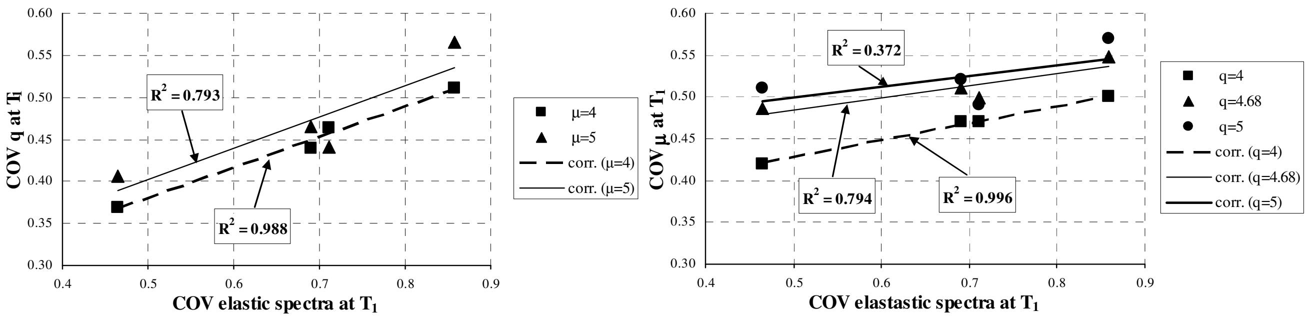

Figure 4 Correlation for each set at T_1 between the CoV of elastic spectra ordinates and: the CoV of q factors of inelastic spectra characterised by μ=4 and μ=5 (sx), the CoV of ductility demand of inelastic spectra characterised by q=4, q=4.68 and q=5 (dx).

4. EXTENSION OF ANALYSES TO R/C MULTI-STOREY SPACE FRAME

The analysed MDOF structure is a four-storey r/c frame building. The bottom interstorey height is equal to 4 m, while at the other levels it is equal to 3.2 m; at the first storey the dimensions of the sections of all the columns and beams are 30x55 cm^2, at the second storey such dimensions are 30x50 cm^2, at the third 30x45 cm^2, while at the top level they are 30x40 cm^2. Beams and columns are modelled as massless one dimension finite elements; the mass is concentrated at the floor levels, assumed rigid in their own planes, consequently the structure is characterised by 3 DOFs for each floor. The building is designed according to Eurocodes 0, 1, 2 and 8 (CEN, 2004a; CEN, 2003), by modal response spectrum analysis. A spectrum type 1 on ground type B with a design ground acceleration on type A ground, a_g, equal to 0.25g, taken from the Italian new seismic code, is considered. Concrete characteristic cylinder strength equal to f_{ck}=30 N/mm^2 and steel characteristic yielding strength equal to f_{yk}=450 N/mm^2 are adopted.
Non linear analyses are performed by means of the computer program CANNY99 (Li, 1996). Non linearity regards flexural rotations, while all the other deformations are assumed elastic. Both beams and columns are characterised by lumped plasticity models; in the latter case for each section two independent non linear springs are assigned, one for each orthogonal direction. No axial force-bending moment interaction is considered at plastic hinge. Bending moment springs are characterised by a tri-linear skeleton curve, defined by cracking and yielding moment and corresponding rotations; the post-yielding stiffness is assumed equal to zero. The cracking rotation is computed multiplying the corresponding curvature by L/6, where L is the span length of the beam. The yielding and the ultimate rotations are evaluated as provided by EC8 (CEN, 2004b) equations (A.10b) and (A.1) respectively, where the mean values are assigned to concrete maximum (f_c=38 N/mm^2) and steel yielding (f_y=530 N/mm^2) strength. The hysteretic model is Takeda type, even though in CANNY99 (Li, 1996) the

pinching effect is also taken into account; a small value of the unloading stiffness is assigned, i.e. in each cycle it is reduced by 50% with respect to the previous one.
The same sets of 7 earthquakes, adopted in order to investigate the response of SDOF systems, are the input of non linear time history analyses on the r/c four-storey space frame. The results in terms of maximum top centre of mass absolute displacements (D), interstorey drift angles (Δ/Hs) and demand/capacity ratios of maximum total chord rotations are considered.

5. COMPARISON SDOF VS MDOF SYSTEMS RESPONSE

In the following a comparison between the coefficients of variation (CoVs) of the results of non linear time history analyses on the space frame and those of the ordinates of elastic and non linear spectra is presented.
In Figure 5, each set corresponds to a square mark; this is characterised by an abscissa equal to the CoV average of elastic spectra (of the set) ordinates computed at 100 periods within the range T_1-$2T_1$ and by an ordinate equal to the CoV of the building maximum top centre of mass absolute displacements (D), in both horizontal directions, resulting by the different earthquakes of the set. The triangular marks represent corresponding values of square ones, but their ordinates are the CoVs of the building interstorey drift angles (Δ/Hs). In the described diagrams the interpolation line of the results for both the top centre of mass absolute displacements and interstorey drifts angles is also reported along with the square value of correlation coefficient (R^2). Figure 6 shows the same typology of results of Figure 5, but on vertical axis the CoV of the demand/capacity ratios of maximum total chord rotations are considered.

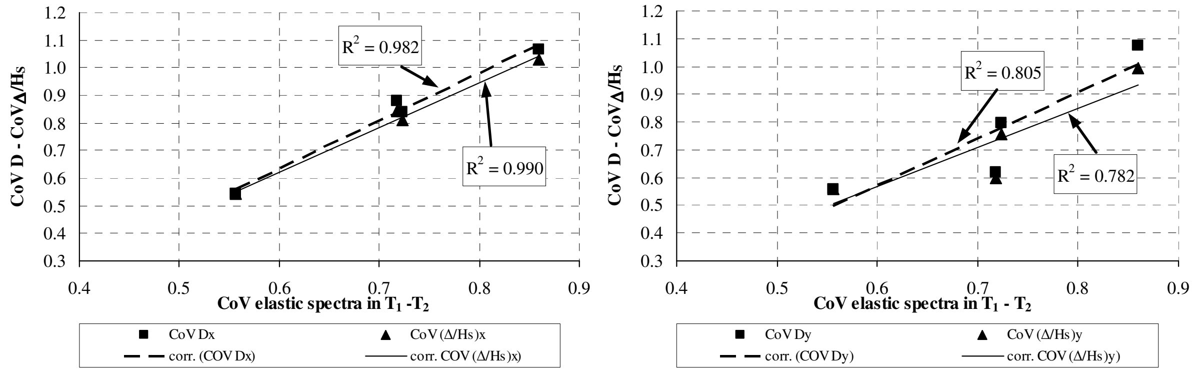

Figure 5 Correlation for each set between the CoV of elastic spectra ordinates and the CoV of maximum top centre of mass absolute displacements (D) and interstorey drift angles (Δ/Hs).

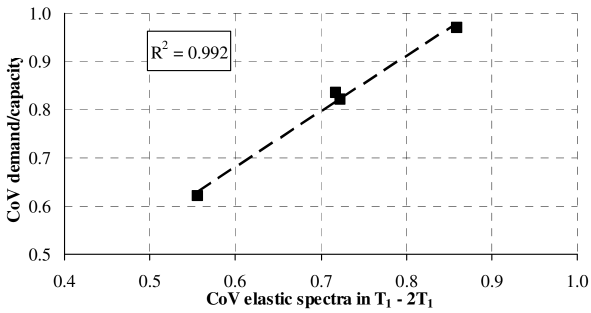

Figure 6 Correlation for each set between the CoV of elastic spectra ordinates and the CoV of the demand/capacity ratios of maximum total chord rotations.

Figure 5 and Figure 6 show that the CoVs of all the results presented, absolute displacements, interstorey drift angles and demand/capacity ratios, are well correlated to the average of the elastic spectra ordinates ones computed within the range T_1-$2T_1$; this means that as the CoV of elastic spectra ordinates increases, the CoVs of parameters characterising the non linear response of the space frame proportionally increase.
In order to confirm all the results presented in this paper and obtained considering the four sets of earthquakes

above presented, such results are also obtained using other 6 sets of seven earthquakes. Three of them are obtained, linearly scaling the records of sets 1, 2 and 3, in order to force all the spectra to assume at the period T_1=0.716 s the same ordinate of the reference EC8 elastic design spectrum. The other three sets are selected from European Strong-motion Database according to the procedure presented in Iervolino et al. (2008), as sets 1, 2 and 3, in order to satisfy the compatibility with the reference elastic design spectrum.
In Figure 7 the same typology of results presented in Figure 5 and Figure 6 are shown. The good linear correlation between the CoVs of parameters characterising the non linear response of the space frame and the CoVs of elastic spectra ordinates is fully confirmed.

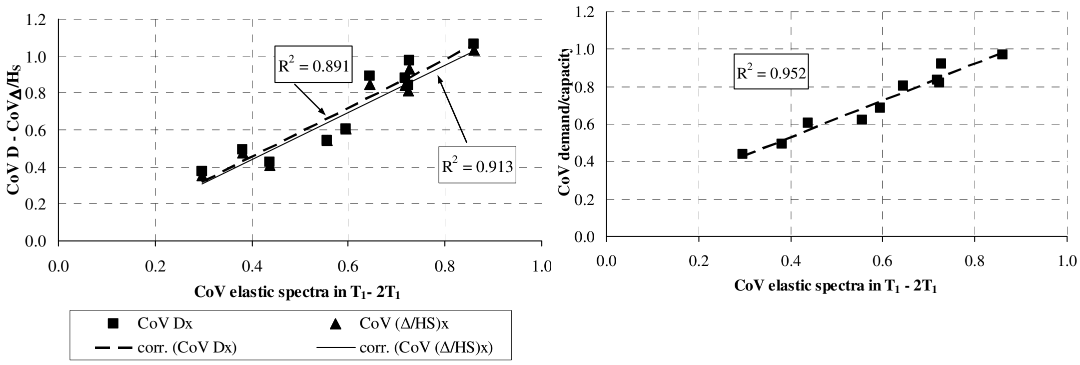

Figure 7 Correlation for each set between the CoV of elastic spectra ordinates and the CoV of parameters of building seismic response.

Analysing all the obtained results, it can be stated that, when non linear time history analyses are performed, the coefficient of variation should be also taken into account, both for input selection and for results evaluation. Concerning the input selection, a “natural” mean CoV of spectra ordinates has been found, which is about equal to 0.7; consequently, when synthetic records are selected for non linear time history analyses, they should be characterised by a CoV which is not much lower than the indicated “natural” value, because it also proportionally conditions the CoV of the parameters characterising the non linear response of structures.

6. CONCLUSIONS

In the present paper the effects of the variability of seismic input on systems structural response are discussed; the non linear response of SDOF systems and of a space frame (MDOF) is analysed. The originality of the presented study is that the results are obtained using as input sets of accelerograms fully satisfying the code provisions in terms of compatibility with an elastic design spectrum. This allows to set the topic of the effects of the variability of seismic input in a defined frame and also to analyse international code provisions. The provisions of Eurocode 8 (EC8) are considered for compatibility of input accelerograms to the elastic spectrum, but the paper conclusions have general validity.
The response of elastic SDOF systems is analysed in terms of spectral acceleration; the same parameter is investigated for corresponding non linear SDOF systems, as well as ductility demand and strength reduction factors. For each set, for both elastic and inelastic spectra, the variation coefficient (CoV), i.e. the ratio between the sample standard deviation and sample mean, is computed. The large CoV values reveal the large dispersion of results; the average CoV among the considered sets is about 0.7.
The analyses are extended to an r/c multi-storey space frame, which is designed according to Eurocodes provisions. The average and the variation coefficient (CoV) of the results in terms of frame top displacements, interstorey drifts and chord rotations at element ends of seven non linear time history analyses are discussed. The obtained results show that, as the CoV of elastic spectra ordinates computed within the range T_1-$2T_1$ increases, the CoVs of parameters characterising the non linear response of the space frame proportionally increase; the range of periods T_1-$2T_1$, where T_1 is the fundamental period of the space frame, is considered, because every system, during a strong earthquake, is affected by a stiffness reduction due to the damage, which, in the case of r/c buildings designed according to High Ductility Class, can cause an increment of the fundamental period until twice the elastic value.

Considering the above quoted conclusions, it can be stated that, when non linear time history analyses are performed, the coefficient of variation should be also taken into account, both for input selection and for results evaluation. Concerning the input selection, a "natural" mean CoV of spectra ordinates has been found; consequently, when synthetic records are selected for non linear time history analyses, they should be characterised by a CoV which is not much lower than the indicated "natural" value, because it also proportionally conditions the CoV of the parameters characterising the non linear response of structures.
Such conclusions are confirmed by non linear time history analyses performed using many sets made of seven earthquakes.

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