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THE INFLUENCE OF DESIGN CRITERIA ON THE SEISMIC RESPONSE OF MOMENT RESISTING STEEL FRAME

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SUMMARY

This paper is focused on the performance evaluation of Moment Resisting steel Frames (MRF), designed according to three different criteria, leading to Global-MRF (GMRF), Special-MRF (SMRF) and Ordinary-MRF (OMRF). To this scope, a complete probabilistic analysis has been developed considering the main sources of randomness concurring to the random seismic response of structures: mechanical properties of materials, vertical loads and earthquake action. The probabilistic analysis has been carried out by means of the Monte Carlo method which consists in generating geometrically equal structures, but structurally different in terms of distribution of vertical loads and material properties. The generated structural sample is successively subjected to nonlinear dynamic analyses considering a series of accelerograms, which have been generated to match the elastic design response spectrum to be representative of the zone seismicity. Successively, the seismic reliability of structures is determined by means of fragility curves, which provide the probability of occurrence of a pre-defined limit state, conditioned on a specific value of the Peak Ground Acceleration (PGA).

Finally, the influence of the randomness of material properties and vertical loads on the stochastic seismic response of steel frames is compared with the influence of the random variability of the seismic action. To this scope, from the statistical sample of the structural inelastic response, both the mean values and the standard deviation of damage control variables, Roof Displacement Angle (RDA) and Interstorey Drift Angle (ISDA), have been evaluated considering both all sources of randomness and by separating the seismic action variability from the material properties and vertical load randomnesses. From these results it can be observed that the evaluation of the structural reliability by means of probabilistic methods considering the effect of the randomness of the seismic action and ignoring the effects of randomnesses due to the vertical loads and the material properties is sufficiently accurate, provided that a strict control of the failure mode is carried out in the design process.

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INTRODUCTION

Structures designed according to the traditional seismic design philosophy should, in general, satisfy the following rules. First, resist minor level of earthquake ground motions without damage remaining in elastic range; second, resist moderate earthquakes without structural damage, but with some non-structural damage; and, finally, resist major earthquakes without collapse with some structural damage which has to be compatible with the local and global ductility supplies. Even though life safety is the most important objective, recent earthquakes have shown that buildings designed primarily for life safety according to the seismic provisions of current building codes could sustain too severe damage to structural elements, non structural elements and building contents. Although no loss of life occurred in modern buildings, economic losses from building damage have been staggering. This is clearly an unacceptable level of loss for frequent and moderate earthquakes; thus, the design professionals recognize that buildings should be designed not only for life safety of occupants but also for damage control.

Structures are allowed to undergo inelastic deformation when subjected to severe earthquake ground motion to dissipate energy. The inelastic behaviour is indirectly considered in traditional seismic design method through reduced level of seismic forces combined with elastic structural analysis. In fact, the usual design procedure for seismic-resistant structures is based on elastic analysis under seismic horizontal forces, which are defined scaling down the linear elastic design response spectrum (LEDRS) by means of a proper coefficient, namely q-factor. Under such reduced horizontal forces, structures have to possess sufficient strength and stiffness in order to satisfy the serviceability requirements. The safety against the ultimate limit state is considered automatically verified, provided that the detailing rules and the design procedures, suggested by seismic codes, are applied.

This procedure offers considerable simplification, but the limitations are evident. In fact, the suggested design procedures do not always lead to the foreseen failure mode and to the expected ductility supply, so that the energy dissipation capacity of the structures could be not sufficient to prevent collapse under destructive earthquakes; in addition, the designer is not aware about the collapse mechanism of the structure, the local and global ductility demands and the actual energy dissipation: such a method is inadeguate to have an exhaustive knowledge of the inelastic response of the structure and of the design measures to be adopted for its improvement.

Therefore, Performance-Based Seismic Design (PBSD) [1-7] with the consideration of both structural and non-structural damage [7], multiple performance objectives, specific quantification of performance criteria and explicit consideration of inelastic deformation of structures, has gained more and more attention within the international scientific community. The purpose of performance-based earthquake engineering is to ensure that the engineered facilities respond to the needs and objectives of the owners, users and society with well defined performance, under common and extreme earthquake ground motions. For this reason, it is essential that the design procedure is clear and transparent for the designers to understand the expected seismic performance and the inherent risks of structures under various levels of ground motions expected during their life cycles.

In this perspective, the SEAOC Vision 2000 Committee proposed the four well-known performance levels: Fully Operational, Operational, Life Safe, Near Collapse. In addition, four earthquake design levels are specified: Frequent, Occasional, Rare and Very Rare; the corresponding return periods are equal to 43, 72, 475 and 970 years [1, 2, 8].

However, it can be recognized that it is very difficult to predict with certainty how much damage a building will experience for a given level of ground motion. This is because there are many factors affecting the behaviour and the response of a building, such as the stiffness of nonstructural elements, the strength of individual building components and the quality of construction, which cannot be precisely defined. In addition, the analysis procedures used to predict building response are not completely accurate and the exact character of the ground motion that will actually affect a building is itself uncertain. Given these uncertainties, it is inappropriate to imply that performance can be predicted in an absolute sense, and correspondingly, that it is absolutely possible to produce designs that will achieve desired performance objectives. In order to evaluate the performance of a steel moment-frame building it is necessary to construct a mathematical model of the structure that represents its strength and deformation characteristics, and to conduct an analysis to predict the values of various design parameters when it is subjected to the design ground motion. The ability of the performance evaluation to estimate reliably the probable performance of the structure is dependent on the ability of the analysis procedure to predict the values of these response parameters within acceptable levels of confidence. In this context, it is clear that the linear dynamic procedure is able to provide relatively reliable estimates of the response parameters for structures that exhibit elastic, or near elastic, behaviour; while the linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it accounts less accurately for the dynamic characteristics of the structure. Instead, the nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behaviour, particularly if they are regular; but, if does not accurately account for higher mode effects. If appropriate modelling is performed, the non linear dynamic approach is the most capable of capturing the probable behaviour of the real structure in response to ground motion.

To this end, a more accurate knowledge of the seismic performances of a structure such as moment resisting steel frames requires sophisticated numerical procedures, because the quantitative evaluation of the structural damage for different earthquake design levels would require nonlinear dynamic analyses accounting also for the random nature of loads and resistances.

In particular, the prediction of the seismic response of structures, that suffered damage during earthquakes, by means of deterministic analyses can lead to an unsatisfactory agreement between predicted and surveyed damage, because inherent randomness and modeling uncertainties limit the quality of the agreement that is possible from a single deterministic analysis [9]. Therefore, the evaluation of the seismic reliability of structures requires stochastic response analyses.

The use of fragility curves providing the probability of occurrence of a pre-defined limit state, conditioned on a PGA or other control variable that is consistent with the specification of the seismic hazard, has been proposed [10-11]. As soon as such fragility curves are derived, the limit state probability can be calculated from the convolution of the derivative building fragility and seismic hazard [12]. In fact, generally the probability that a building may experience damage more severe than that defined for a given performance level is a function of two principal factors. The first of these is the structure's vulnerability, that is, the probability that it will experience certain levels of damage given that it experiences ground motion of certain intensity. The second of these factors is the site hazard, that is, the probability that ground shaking of varying intensities may occur in a given time period. The probability that damage exceeding a given Performance Level may occur in a period of time is calculated as the integral over time of the probability that damage will exceed that permitted within a performance level.

Structural response parameters that may be used to measure capacity include the structure's ability to undergo global building drift, maximum tolerable member forces and maximum tolerable inelastic deformations. However, the process of predicting the capacity of a structure to resist ground shaking demands as well as the process of predicting the severity of demands that will actually be experienced entail significant uncertainties. Generally, uncertainty can be reduced by obtaining better knowledge or using better procedures [13].

Therefore, in this paper, a probabilistic approach has been applied with reference to a four bay-four storey MRF designed according to different criteria. In particular, regarding the dimensioning of the structural elements, three design criteria are examined:

- GMRF: frames designed to assure a global failure mode [14, 15];
- SMRF: frames designed according to member hierarchy criterion [16];
- OMRF: frames designed without any requirement aimed at the control of the failure mode [17].

The aim of this paper is to focus on the performances of these criteria from a probabilistic point of view. The reliability analysis has been carried out considering all the randomnesses concurring to the seismic response of structures. In particular, mechanical properties of materials, vertical loads and earthquake

action are considered as random variables. Therefore, a complete probabilistic analysis is proposed to evaluate the structure ability to dissipate the seismic energy. This approach includes both the random nature of the seismic loading condition and the random location of plastic hinges, whose formation depends on the random values of material properties. The probabilistic analysis is developed by means of the Monte Carlo method which consists in generating geometrically equal structures, but structurally different in terms of distribution of vertical loads and material properties. The generated structural sample is successively subjected to nonlinear dynamic analyses. Finally, the seismic reliability of structures is determined by means of fragility curves, which provide the probability of occurrence of a pre-defined limit state (one of the four proposed by PBSD), conditioned on a PGA value.
Although the electronics took a step forward in the last years, the burdensome calculation remains the most overhanging limit for the application in everyday design experience of a probabilistic procedure. Therefore, it is very interesting to develop relatively simple methodologies for the evaluation of the seismic hazard for the use in the design practice, both as design tool for new buildings and for evaluating the conditions of existing buildings. The tools for reliability analysis are enough sophisticated and they can be used for special structures, but they absolutely are not economically flexible and thinkable for the design and evaluation of the most part of buildings.
Therefore, in order to reduce the number of dynamic analyses required by a probabilistic approach, in this work the influence of the randomness of material properties and vertical loads on the stochastic seismic response of steel frames is compared with the influence of the random variability of the seismic action. The aim is the identification of the most important sources of randomness to be included in the structural reliability evaluation with probabilistic methods. In particular, from the statistical sample of the structural inelastic response, both the mean values and the standard deviation of damage control variables, RDA and ISDA, have been evaluated considering both all sources of randomness and by separating the seismic action variability from the material properties and vertical load randomnesses.

STOCHASTIC RESPONSE ANALYSIS

The variability in the response of structures to earthquake ground motions depends on many sources of randomness: material properties, vertical loads, seismic event, structural geometry, structural analysis, welding process, quality of workmanship, base metal properties and connection design.
In this paper the structural parameters treated as random variables are the following:

- Yield strength: two different yield strength distributions have been considered: the first one for the flanges and the second one for the web of the cross section. The random values of the yield strength of these two elements are independently generated, to this end a preliminary statistical analysis of available experimental data has been performed. This analysis suggested the use of a log-normal distribution [12], whose mean value ($E[\ln f_y]$) is dependent on the plate thickness (t):

  $$E[\ln f_y] = c_1 - c_2 t \qquad (1)$$

  Conversely, the variance ($VAR[\ln f_y]$) can be assumed constant.
- Vertical loads: the considered random variables are the dead load due to concrete slab and the live load. The normal distribution has been assumed for these random variables with a coefficient of variation (Cov) equal to 0.10 [18]. The mean value of the dead load due to the concrete slab is equal to the product between the concrete cross section area and its specific weight. The mean value (F_m) of the live load is given by:

  $$F_m = \frac{F_k}{1 + k \cdot Cov} \qquad (2)$$

  where F_k = characteristic value (code specified value) and k = coefficient equal to 1.64 for normal distribution.

- Seismic event: the dominant source of randomness in response is the ground motion; in fact, basic seismic hazard at a site, phasing, amplitude and frequency content are random. In this work the randomness of the seismic action is considered by means of an ensemble of ten simulated ground motions, generated by means of the SIMQKE program, to match the LEDRS of Eurocode 8, for a soil type A and for high seismicity zone. In addition, each ground motion is scaled to obtain increasing values of PGA.

Several structural response parameters are used to evaluate structural performance. In this paper, the building response statistics considered are the mean and the standard deviation of the maximum roof displacement angle (RDA) and of the maximum interstorey drift angle (ISDA).
RDA is defined as the maximum roof displacement (δ) normalized by the building height (H):

$$RDA = \frac{\delta}{H} \tag{3}$$

The maximum ISDA is defined as:

$$ISDA = \max_{i=1}^{n_p}\left(\frac{\delta_i}{h_i}\right) \tag{4}$$

where δ_i = maximum interstorey drift of the i-th storey, n_p = number of storeys and h_i = storey height. ISDA is more significant than RDA and is usually adopted in seismic codes as a numerical measure of damage. In fact, damage is stochastically averaged over the height of the frame in determining the RDA; conversely, the ISDA provides a measure of damage focusing on the storey in which the concentration of the seismic energy occurs. In addition, interstorey drift is an excellent parameter for judging the ability of a structure to resist P-Δ instability and collapse. It is also closely related to plastic rotation demand or drift angle demand on individual beam-column connection assemblies, and it is therefore a good predictor of the performance of beams, columns and connections.

Table 1. Limit states

Limit States	RDA (%)	ISDA (%)
Fully Operational	0.5	0.5
Operational	1.0	1.0
Life Safe	2.0	2.0
Near Collapse	5.0	5.0

FRAGILITY MODEL

The seismic fragility of a structure $F_r(x)$ is defined as its Limit-State (LS) probability, conditioned on a specific PGA, spectral acceleration, spectral velocity or other control variable that is consistent with the specification of the seismic hazard:

$$F_r(x) = P[x = LS|PGA] \tag{5}$$

where LS represents the corresponding limit state. $F_r(x)$ often is modeled with a log-normal probability distribution [10].
The fragility for any given limit state is obtained from the cumulative distribution function (CDF) of ISDA or RDA. For example, if the limit state is ISDA = 1%, then:

$$F_r(1\%) = P[ISDA \geq 1\%|PGA] = 1 - [ISDA < 1\%|PGA] \tag{6}$$

The aim of this paper is the analysis of the inelastic performances of structures, designed according to different criteria, by evaluating the fragility curves corresponding to the three limit states (Operational, Life Safe and Near Collapse) engaging the structures in plastic range. The numerical values of RDA and ISDA corresponding to the limit states, suggested by SEAOC, are given in the Table 1 [10].

The numerical procedure used to determine the fragility curves is a Hybrid Monte Carlo simulation [19], which consists in the following phases:

1. Deterministic dimensioning of frames (three design criteria - OMRF, SMF and GMRF - are examined in this paper);
2. Random generation of geometrically equal structures, but structurally different, in terms of distribution of vertical loads and material properties. To make this generation, the first step is represented by the generation of numbers uniformly distributed between 0 and 1. Each random number corresponds to a random value of the random variable (yield strength or vertical loads), for a given probability distribution law. The transformation of random numbers into random values of involved variables, for a given probability distribution law, has been performed by means of the Box and Muller method [19];
3. Structural analysis: the structures previously generated are subjected to nonlinear dynamic analyses obtaining a statistical sample of the inelastic seismic response;
4. Statistical interpretation of results which consist in the following steps:
    - Selection of the control variables (x) to evaluate the seismic structural response (in this paper the adopted control variables are RDA and ISDA);
    - Definition of the limit values of the control variables (x_{lim}), corresponding to the different performance levels;
    - For any given PGA, the values of the control variables obtained from nonlinear dynamic analyses are rank-ordered ($x_1 \leq x_2 \leq .. \leq x_{n-1} \leq x_n$ where n is the sample size, i.e. the number of the analysed frames);
    - Determination of the CDF value corresponding to the i-th value (x_i) of the control variable by means of Gringorten formula:

      $$F_x(x_i) = \frac{i-\alpha}{n-2\alpha+1} \cong \frac{1}{n} \tag{7}$$

      where α is a constant equal to 3/8.
    - Estimate of the mean value and the standard deviation of rank-ordered series:

      $$\bar{x} = \frac{\sum x_i}{n} \qquad \sigma = \sqrt{\frac{\sum \left(x_i - \bar{x}\right)^2}{n-1}} \tag{8}$$

    - For any given PGA, data are plotted in log-normal probability graph paper, as depicted in Figure 1;
    - For any fixed limit value of the control variable (x_{lim}), the corresponding cumulative probability is evaluated, as denoted in Figure 1;
    - For any given PGA and for any fixed limit value of the control variable (x_{lim}), the fragility is evaluated as follows:

      $$F_r\left(x_{\lim}\middle|PGA\right) = 1 - F_x\left[x_{\lim}\middle|PGA\right] \tag{9}$$

    - Determination of the corresponding standard normal variable (u_r) for a fixed limit state (x_{lim}) and for any value of $F_r(x_{lim}|PGA)$ previously computed;
    - For any fixed limit state x_{lim}, the points (PGA, u_r) are represented on log-normal probability graph paper and the regression curve, representing the fragility curve corresponding to x_{lim}, is evaluated (Fig. 2a);
    - The last step is the representation of the fragility curve (Fig. 2b).

    For any fixed limit state, Figures 2a, 2b can be used to determine both the fragility corresponding to a given PGA and, conversely, the maximum PGA that the structure is able to sustain without violating the design requirements corresponding to the fixed limit state.

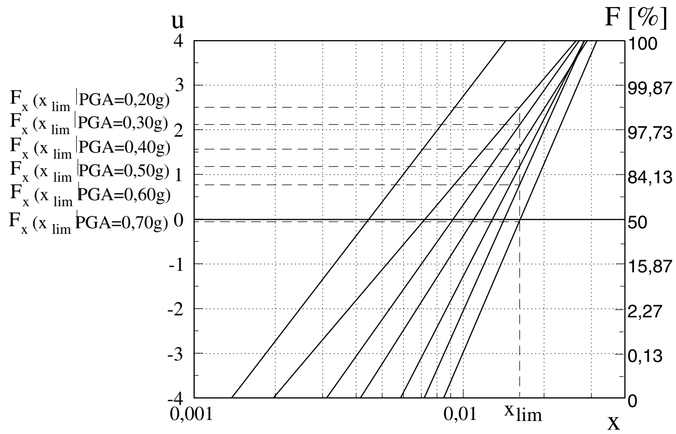

Figure 1. CDF in log-normal probability graph paper.

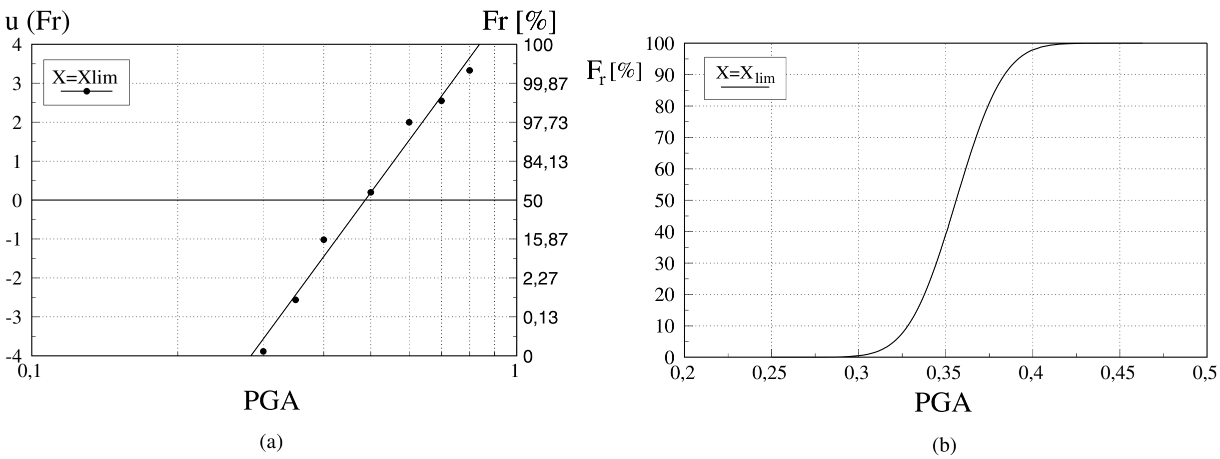

Figure 2. Fragility curve.

EXAMINED FRAMES

A four bay-four storey frame has been analyzed (Fig. 3). Steel grade Fe430 is used. Regarding the deterministic design loads, a uniform dead load of 18.0 kN/m and a uniform live load of 12.0 kN/m are applied. The results of the dimensioning according to the three design criteria are presented in Table 2 and in Figure 3. In particular, Table 2 shows the influence of the design criteria on the column sections.

In order to generate geometrically equal structures, but structurally different, the yield strength is considered log-normal with a mean value defined by Equation 1 with $c_1 = 5.7779$ and $c_2 = 0.0030$, while the variance is constant ($VAR[\ln f_y] = 0.0038$); the uniform dead load is constituted by a deterministic part and a random part due to the concrete slab, whose weight has been assumed normally distributed with mean value equal to 2kN/m and coefficient of variation equal to 0.10.

Successively, nonlinear dynamic analyses have been performed using DRAIN-2DX. Preliminarily, in order to establish the size of the statistical sample of the structural response, 5000 frames have been generated and subjected to a single ground motion having a PGA equal to 0.75g.

Table 2. Column sections.

Storey	Type	OMRF	SMRF	GMRF
1	A	HEB 160	HEB 200	HEB 340
1	B	HEB 220	HEB 240	HEB 340
1	C	HEB 200	HEB 240	HEB 340
2	A	HEB 160	HEB 200	HEB 300
2	B	HEB 180	HEB 240	HEB 300
2	C	HEB 180	HEB 240	HEB 300
3	A	HEB 140	HEB 180	HEB 300
3	B	HEB 180	HEB 220	HEB 300
3	C	HEB 160	HEB 220	HEB 300
4	A	HEB 140	HEB 180	HEB 260
4	B	HEB 120	HEB 220	HEB 260
4	C	HEB 120	HEB 220	HEB 260

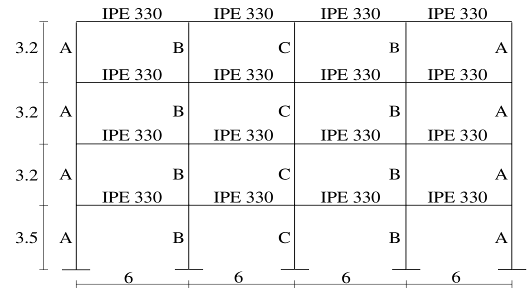

Figure 3. Examined frame.

From these analyses, the sample size to be used for the complete procedure for evaluating the fragility curves has been selected as the sample size assuring sufficiently stable values of average and standard deviation of RDA and ISDA. The choice to limit the size to 1000 frames appeared to be an appropriate compromise between accuracy of computation and its time effort.

Therefore, for any selected design criterion, 1000 frames have been generated and successively subjected to ten simulated ground motions. Each ground motion has been scaled considering the following PGA values:

- OMRF: 14 cases (0.05, 0.075, 0.1, 0.15, 0.18, 0.2, 0.22, 0.3, 0.4, 0.45, 0.55, 0.6, 0.8, 1, 1.1)g;
- SMRF: 17 cases (0.05, 0.075, 0.1, 0.15, 0.2, 0.3, 0.35, 0.4, 0.45, 0.5, 0.6, 0.7, 0.8, 0.9, 1, 1.05, 1.2)g;
- GMRF: 15 cases (0.1, 0.2, 0.3, 0.35, 0.4, 0.5, 0.55, 0.6, 0.7, 0.8, 0.85, 0.9, 1, 1.1, 1.2)g.

The number of cases to be analysed and the values of PGA have been selected to have a sufficient number of points to determine the fragility curves.

FRAGILITY CURVES

Figures 4-6 show, for any fixed limit state, the fragility curves for the analyzed design criteria. The following observations can be made:

- Considering the limit state ISDA = 1% (Fig. 4), corresponding to the beginning of damage to non structural elements, for the same value of PGA, GMRF provides the smallest fragility and, therefore, the greatest reliability. It can be noted that for PGA equal to 0.20g OMRF provides 100% fragility while GMRF and SMRF provide null fragility;
- Regarding the limit state ISDA = 2% (Fig. 5), corresponding to Life Safe, for a given value of PGA, GMRF still provides the smallest fragility and, therefore, the greatest reliability. In particular, the fragility curves are significantly spaced pointing out that the adopted design criterion plays a very important role for a limit state significantly engaging the structure in plastic range, even though collapse is relatively far. For example, for PGA = 0.5g, the fragilities corresponding to OMRF, SMRF and GMRF are equal, respectively, to about 100%, 10% and 0%; while for PGA = 0.80g the fragilities corresponding to OMRF and SMRF are equal to 100% while GMRF is almost completely reliable;
- Concerning the limit state ISDA = 5% (Fig. 6), corresponding to the incipient collapse, for a given value of PGA, GMRF is still the most reliable; but in this case the fragility curves are more closely spaced.

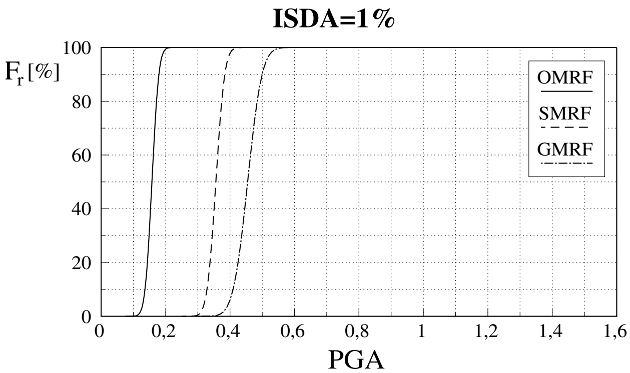

Figure 4. Fragility curves for limit state ISDA = 1%.

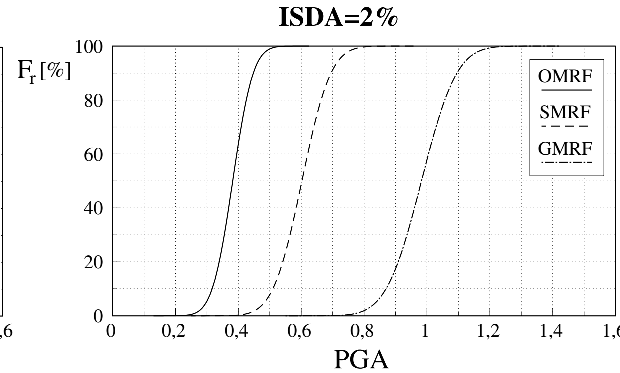

Figure 5. Fragility curves for limit state ISDA = 2%.

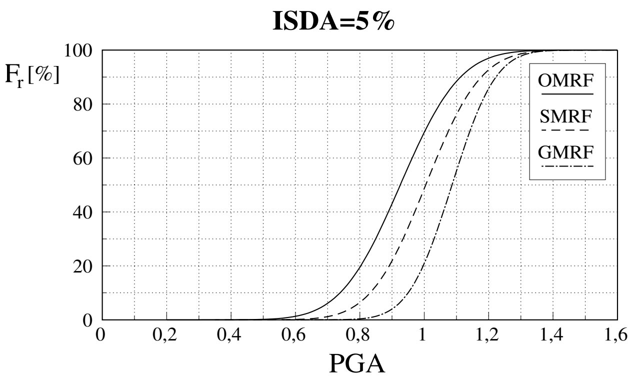

Figure 6. Fragility curves for limit state ISDA = 5%.

ANALYSIS OF THE INFLUENCE OF THE DIFFERENT SOURCES OF RANDOMNESS

Considering all the random variables, 460.000 dynamic nonlinear analyses have been carried out. From these analyses the mean value and the standard deviation (ST.DEV.) of RDA and ISDA have been computed and are presented in Figures 7-9.

Based on the above mentioned results, several conclusions can be drawn. For a fixed design criterion, increasing PGA, the mean value of ISDA increases. The gradient of ISDA versus PGA points decreases as the design criterion becomes more severe. In addition, increasing PGA, also the standard deviation of ISDA increases; this is due to the fact that increasing the plastic deformations the effects coming from the randomness of mechanical properties of materials are more significant.

With reference to RDA, similar observations can be made. In both cases, the design criterion aimed to assure a global failure mode, involving an increase of the column sections, leads to the formation of a significant number of plastic hinges leading to a better plastic redistribution. Instead, the other design criteria, involving a concentration of plasticity in few sections, determine an higher rotation of plastic hinges which lead locally to a bigger plastic engagement. This is clear observing the reduction of the difference of the mean value of ISDA and the mean value of RDA as the design criterion becomes more severe: the value of this difference becomes maximum for OMRF and minimum for GMRF. Moreover, the mean value of RDA is always less than the mean value of ISDA. This is because, in determining RDA, damage is stochastically averaged over the height of the frame.

Therefore, in this paper, aiming to compare the performances of the designed structures in terms of fragility curves, only ISDA has been considered.

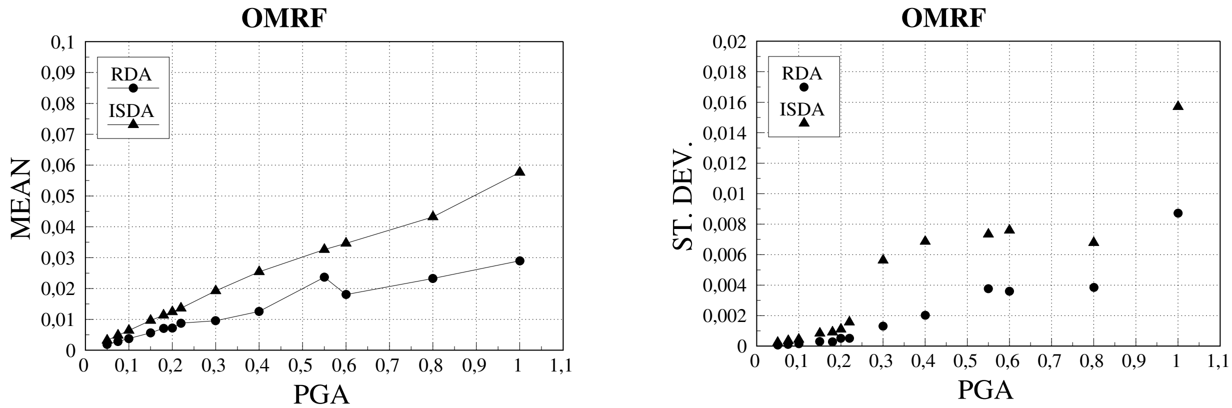

Figure 7. Mean values and Standard Deviation in terms of RDA and ISDA for OMRF.

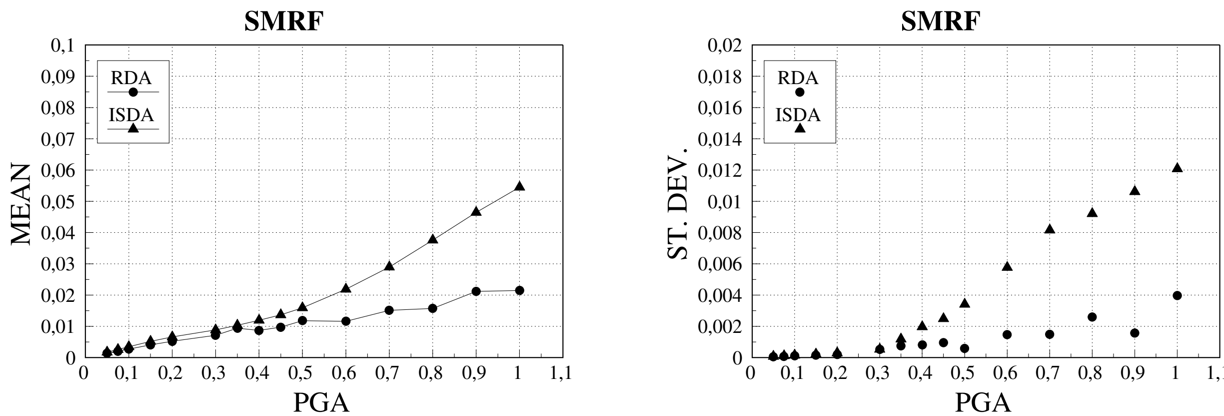

Figure 8. Mean values and Standard Deviation in terms of RDA and ISDA for SMRF.

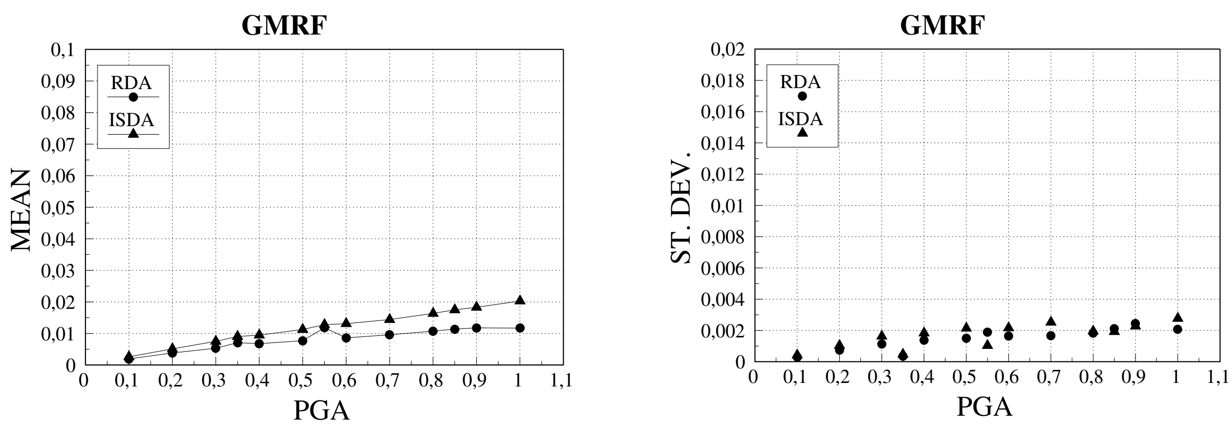

Figure 9. Mean values and Standard Deviation in terms of RDA and ISDA for GMRF.

Figures 10-12 show, instead, the mean values and the standard deviation in terms of ISDA for any analyzed design criterion. These values have been obtained, given the ground motion, by considering the values of the considered control variable (ISDA) for the 1000 random generated frames. It is clear that this representation of the results in terms of both mean value and standard deviation provides, for any PGA value, a measure of the effects of the seismic action randomness conditioned to the selected random set of frames. In fact, for a given PGA value, the obtained fork of values corresponds to the ten considered accelerograms. The extent of the scatter becomes more and more evident as PGA increases and, as a consequence, the plastic engagement increases too. In addition, this scatter decreases increasing the severity of the design criterion, i.e. passing from OMRF to GMRF.

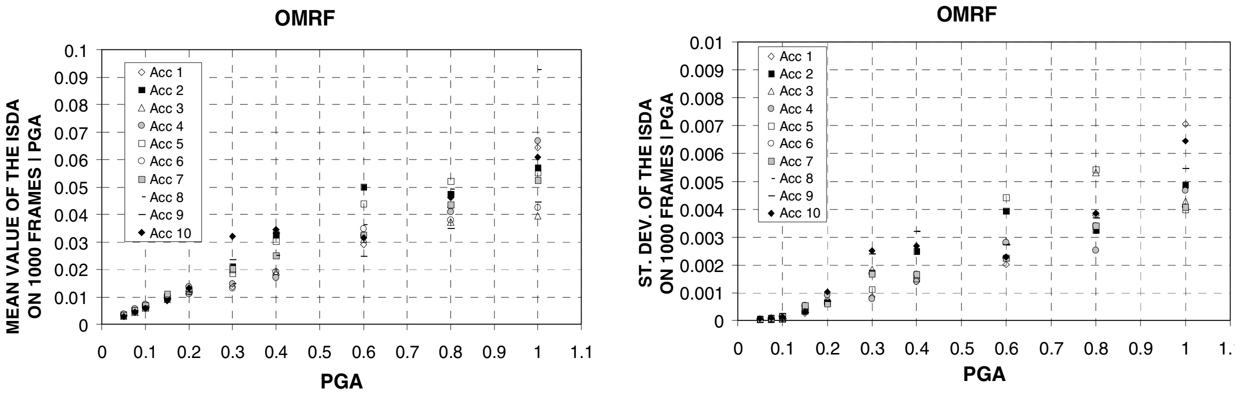

Figure 10. Mean values and Standard Deviation in terms of ISDA obtained for a given ground motion and by considering ISDA values for the 1000 random generated frames (OMRF).

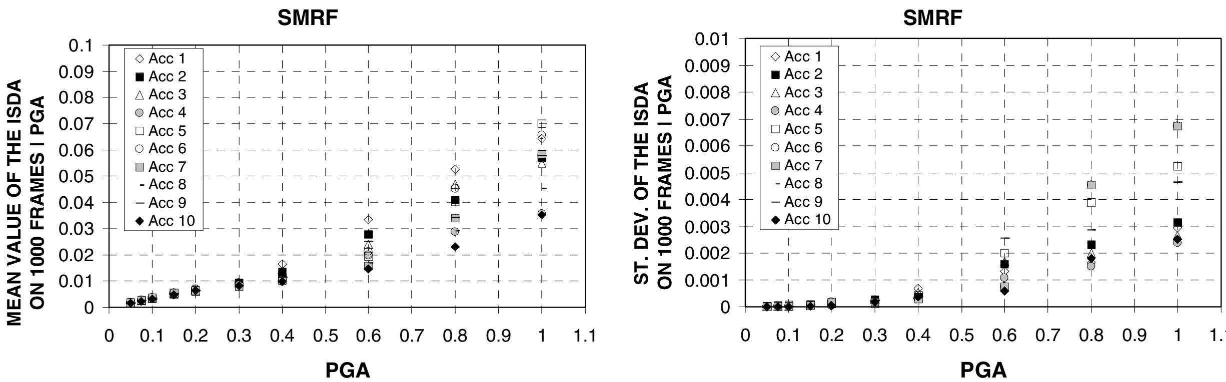

Figure 11. Mean values and Standard Deviation in terms of ISDA obtained for a given ground motion and by considering ISDA values for the 1000 random generated frames (SMRF).

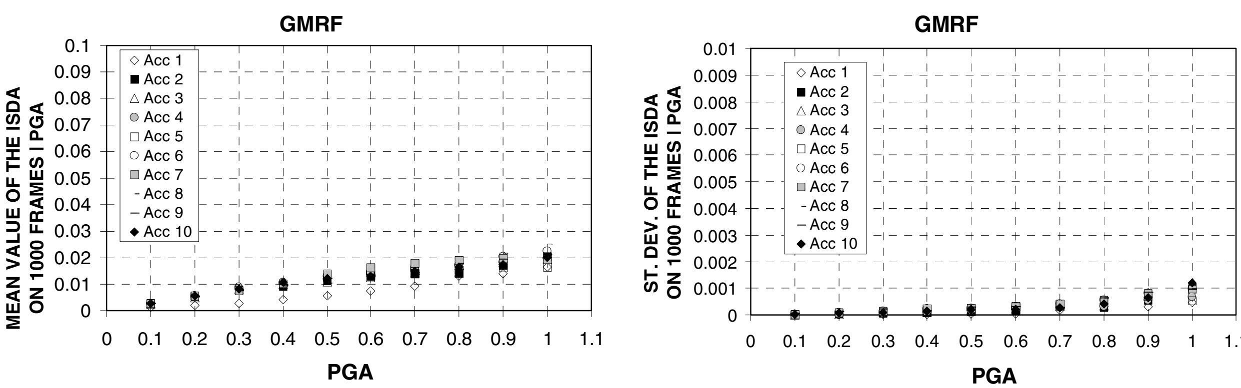

Figure 12. Mean values and Standard Deviation in terms of ISDA obtained for a given ground motion and by considering ISDA values for the 1000 random generated frames (GMRF).

In the figures 13-15 the mean and the standard deviation in terms of RDA has been presented for any analyzed design criterion. They have been obtained, given the ground motion, by considering the values of the control variable (RDA) for the 1000 random generated frames. In this case, observations similar to those made with reference to ISDA can be outlined, but the effects are smoothed with respect to ISDA.

Aiming to show in synthetic form both the effects of randomness due to seismic actions and those due to the other two considered random variables (vertical loads and material properties), for any PGA value, the mean values of ISDA and of RDA are shown in the figures 16-18 where have been depicted only the accelerograms providing the minimum and the maximum value of the above parameters for any analyzed design criterion. In the same figures the corresponding 16% and 84% fractiles have been represented (mean value ± standard deviation). From these figures the influence of the randomness of vertical loads and materials mechanical properties can be evaluated by measuring, for any PGA value, the segment connecting the point corresponding to 16% fractile to that corresponding to 84% fractile.

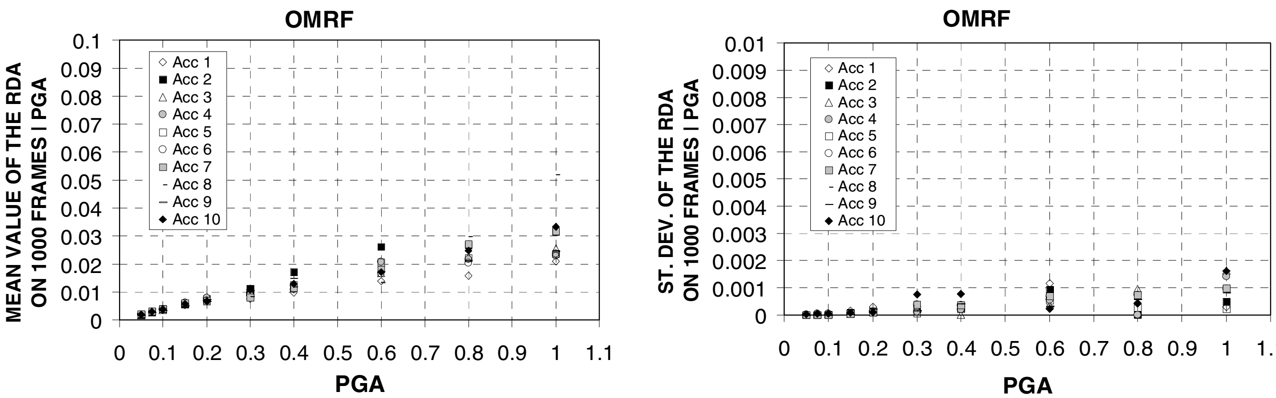

Figure 13. Mean values and Standard Deviation in terms of RDA obtained for a given ground motion and by considering RDA values for the 1000 random generated frames (OMRF).

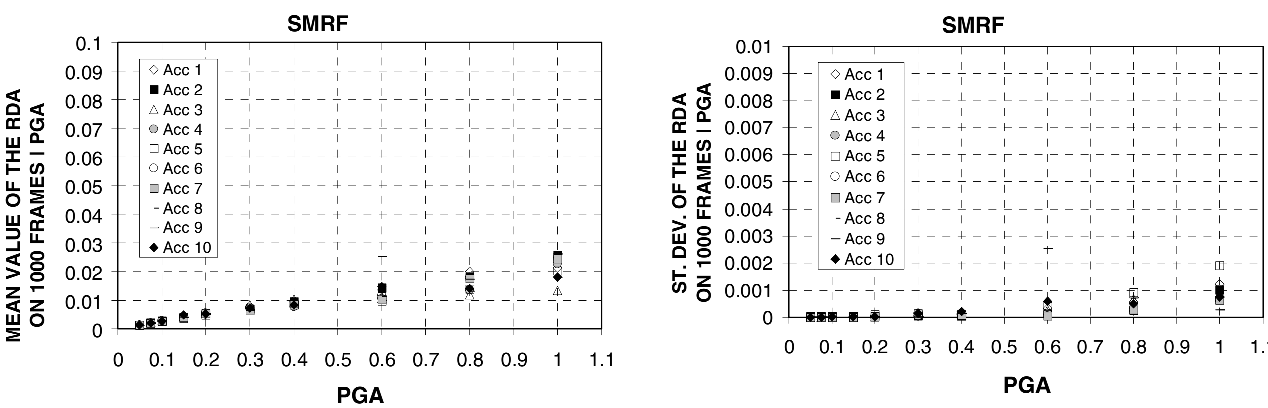

Figure 14. Mean values and Standard Deviation in terms of RDA obtained for a given ground motion and by considering RDA values for the 1000 random generated frames (SMRF).

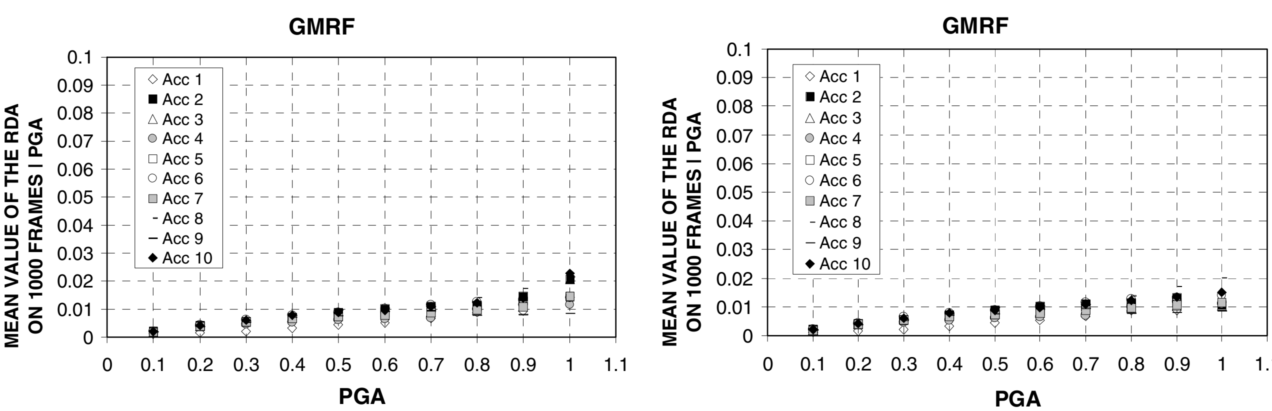

Figure 15. Mean values and Standard Deviation in terms of RDA obtained for a given ground motion and by considering RDA values for the 1000 random generated frames (GMRF).

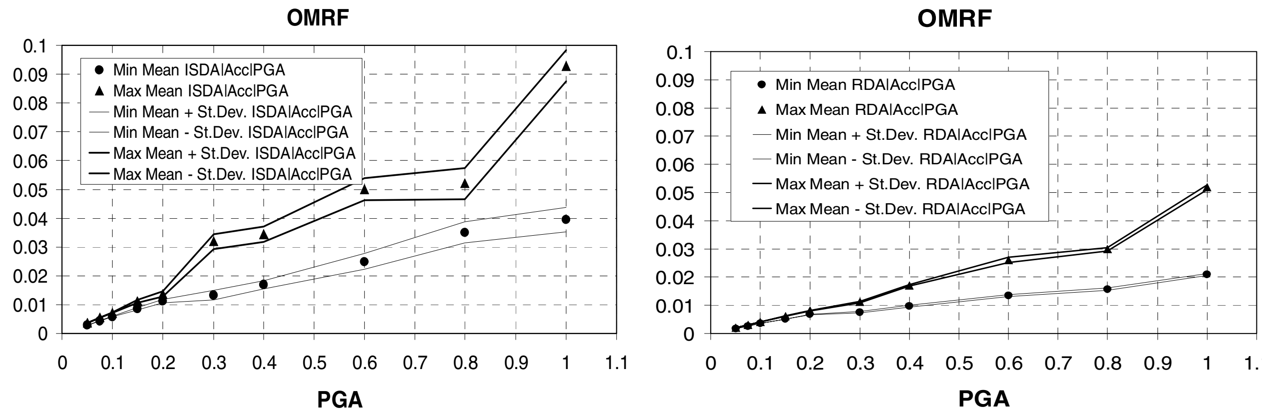

Figure 16. Minimum and Maximum Mean values in terms of both ISDA and RDA and corresponding 16% and 84% fractiles for OMRF.

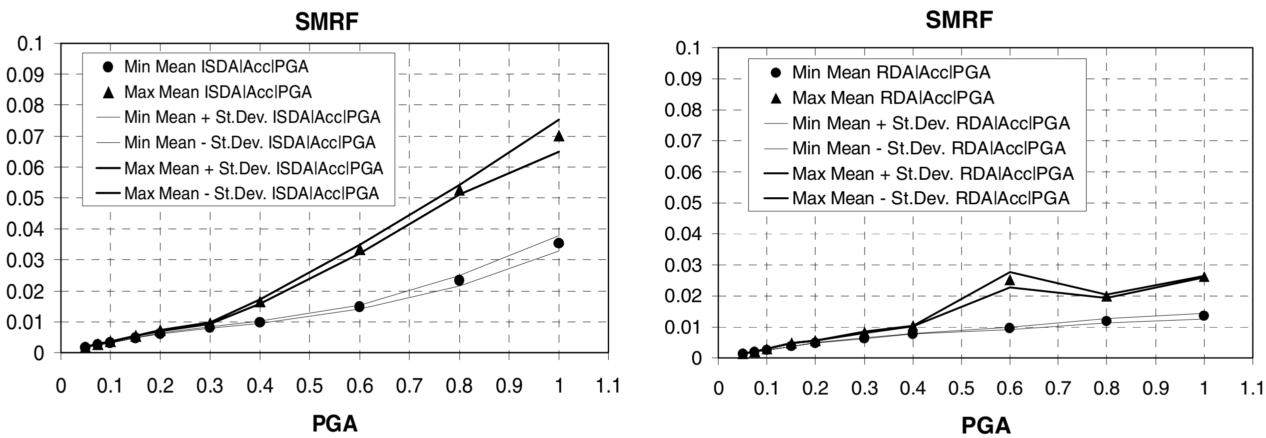

Figure 17. Minimum and Maximum Mean values in terms of both ISDA and RDA and corresponding 16% and 84% fractiles for SMRF.

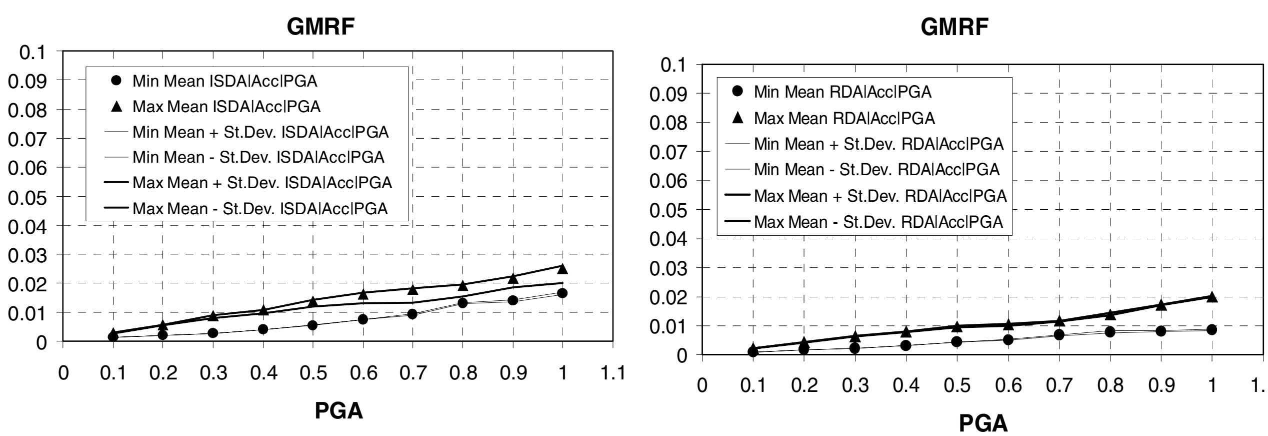

Figure 18. Minimum and Maximum Mean values in terms of both ISDA and RDA and corresponding 16% and 84% fractiles for GMRF.

Moreover, for any PGA value, the segment obtained by connecting the points corresponding to the maximum mean value (solid triangle) and to the minimum mean value (solid circle) provides a measure of the scatter of the inelastic structural response due to the earthquake action, with reference to the examined

set of accelerograms. It can be recognized that the seismic stochastic response of steel structures is more significantly affected by the randomness of the seismic action rather than by the other two considered random parameters (mechanical properties and vertical loads). This is more evident as the design criterion becomes less severe.
As an example, with reference to a seismic event with PGA equal to 0.60g and considering only the most significant control variable (ISDA), from these figures it can be observed that the scatter due to randomness of vertical loads and material properties is, respectively for OMRF, SMRF and GMRF, equal to 31.31%, 14.19% and 6.88% of the scatter due to the randomness of seismic action. Moreover, with reference to the considered set of accelerograms and to the examined range of the peak ground acceleration values, the magnitude of this scatter is, respectively for OMRF, SMRF and GMRF, on average equal to 33.49%, 19.50% and 10.22% of that due to randomness of the earthquake.
Therefore, with reference to frames designed to assure a collapse mechanism of global type (GMRF), the effects of randomnesses are significantly reduced. For this reason, in this case the use of probabilistic methods for evaluating the structural reliability could be promoted by considering only one source of randomness, i.e. the seismic action, and ignoring the randomness due to vertical loads and material properties.

CONCLUSIONS

A procedure, based on Monte Carlo simulation, aiming at the evaluation of fragility curves corresponding to predefined limit states has been illustrated in this paper. The fragility curves include the randomnesses due to material mechanical properties, vertical loads and seismic action. In other words, they represent the result of an almost complete probabilistic analysis and, consequently, they represent an accurate tool for predicting the nonlinear seismic response of structures.
The application of such methodology to MRFs designed with different criteria (OMRF, SMRF and GMRF) has pointed out that frames designed to assure a collapse mechanism of global type are the most reliable for any considered limit state leading to a 100% reliability until 0.32g for ISDA = 1%, which corresponds to the Operational limit state, and until 0.70g for ISDA = 5%, which corresponds to the Near Collapse limit state. However, with reference to earthquakes having a relatively moderate magnitude (PGA_{max} = 0.35g), such as those occurring in European countries, also SMRFs provide good results for any considered limit state. Conversely, the lack of any requirement aiming at the failure mode control, leads to unacceptable structures, because of their poor energy dissipation capacity and, as a consequence, poor reliability independently of the magnitude of the earthquake action.
In addition, the seismic stochastic response of steel structures, both in terms of RDA and ISDA, is clearly more prone to the effects of the seismic action randomness rather than to the effects due to the other two considered random parameters (mechanical properties and vertical loads). This is more significant as the design criterion becomes less severe. With reference to frames designed to assure a collapse mechanism of global type (GMRF), the effects of randomnesses are significantly reduced, so that for such frames the use of probabilistic methods considering only the effects of seismic action randomness could be promoted.

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