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The seismic behaviour of steel frames with random material variability

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ABSTRACT: The influence of random yield strength variation of steel members on the seismic behaviour of framed structures is faced in this paper. The most important parameters affecting the random inelastic response are pointed out and a parametric analysis is carried out in order to grasp the importance of the role played by each parameter. Moreover, the stability of different definitions of the available global ductility is discussed from the statistical point of view, leading to results which should be used in order to provide a rational quantitative definition of the global ductility of frames.

1 INTRODUCTION

The traditional seismic design is based upon the ability of structures to sustain plastic deformations so that earthquake input energy is dissipated through hysterethic behaviour of the material. In any case, the damagement due to plastic deformations has to be limited in order to prevent collapse.

As this design criterium involves the plastic redistribution capacity of the structure, the safety of a structure against the severest design earthquake is guaranteed only if local and global ductility demands are compatible with the geometrical and mechanical properties of the structure and its members.

The available global ductility is strictly related to the collapse mechanism and it assumes a maximum value when global type mechanism occurs. For this reason, modern seismic codes such as ECCS Recommendations and EC8 provide simplified design criteria in order to control the failure mode.

Under this point of view, the randomness of yield strength of members plays a very important role affecting the plastic hinges formation process and, as a consequence, the ultimate behaviour of structures, leading to an energy dissipation capacity and to an available ductility different from the predicted ones. In fact, even if the structure is designed in order to obtain a mode of failure of global type on the base of the nominal yield strength, the increase of the coefficient of variation of the yield strength of members can be responsable of the formation of local failure modes which reduce ductility and energy dissipation capacity and therefore the structure is no longer able to resist the severest design earthquake.

The problem under examination is also strictly related to that of «structural regularity». Modern seismic codes classify structures into two categories, regular and irregular structures, on the basis of their geometrical configuration. In case of irregular structures, damage concentration is expected so that the worsening of the inelastic behaviour is taken into account by means of a reduction of the q-factor which has to be used in design. The concept of structural regularity has been probably introduced in order to define the structural situations for which the knowledge of the seismic response of SDOF (Simple Degree Of Freedom) systems can be extended to actual MDOF (Multi Degree Of Freedom) structures.

From the point of view of structural damage characterization, the main difference between SDOF systems and MDOF systems is due to the fact that only one parameter, i.e. the global ductility demand, can be sufficient to describe seismic damagement of SDOF systems, while the same parameter is unable to provide a sufficient characterization of the structural damage of MDOF systems, where different patterns of yieldings could correspond to a same value of such parameter. It is clear, therefore, that the use of the results regarding the seismic response of SDOF systems for predicting the seismic behaviour of actual MDOF structures requires the hypothesis of uniform damage distribution. Moreover, it can be stated that even if a structure is designed in such a way that this very ambitious pattern of damage is assured on the base of the nominal yield strength, the actual damage distribution can be very different due to the randomness of yield strength in structural members. Under this point of view the randomness of yield strength can be considered as a type of «irregularity».

The study of the inelastic behaviour of steel frames with random material variability has been already faced by different authors (Kuwamura and Kato, 1989 - Kuwamura and Sasaki, 1990 - Elnashai and Chryssanthopoulos, 1990 - Mazzolani, Mele and Piluso, 1991). All these studies have pointed out that random variability of material properties can produce the reduction of the available ductility and that the control of the failure mode can be undermined. However, the influence of the randomness of yield strength of members on the parameters characterizing the inelastic response of structures cannot be quantified by means of general rules, because the available results are referred to single structural schemes. In this paper a parametric analysis is presented in order to investigate in general way the influence of the main factors affecting the random inelastic response of steel framed structures with random variability of yield strength of their members.

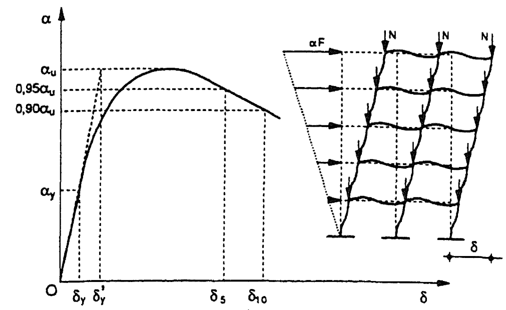

Figure 1. Multiplier of horizontal forces versus top displacement curve

2 SIMULATION METHOD

In the last 15 years the problem of the evaluation of structural safety by means of probabilistic approaches as an alternative to the traditional methods of limit state analysis, led to great theoretical developments due to the progress in the theory of structural reliability. The application of these procedures for evaluating structural reliability is in general very cumbersome so that, in spite of this theoretical progress, the use of the theory of structural reliability as a design tool is not widespread.

The analysis of structural system reliability can be based both on a static formulation or on a cinematic formulation (Nafday, Corotis and Cohon, 1987), but in both cases numerical procedures are difficult to apply also to very simple structural systems. In particular, the requested numerical procedures are inefficient or not even available in case of problems involving geometrical and/or mechanical non-linearity or in case of problems for which the evaluation of damage accumulation is requested. Therefore, the use of numerical simulation seems to be the best solution in case of complex structural systems (Faravelli and Casciati, 1980).

This investigation deals with a particular aspect of the structural safety, because in the evaluation of the probability of failure the randomness of loads is neglected and the attention is focused on the parameters which are usually considered intrinsic of the structural behaviour. Therefore, the used numerical procedure is the Monte Carlo Simulation method in which the generation of the random yield strength of the frame members and the inelastic structural analysis are carried out in sequence (Rubistein, 1981). The first step is represented by the generation of numbers uniformly distributed between 0 and 1. At each random number corresponds a random value of the yield strength, for a given probability distribution law of the structural steels (Mazzolani, Mele and Piluso, 1991). The trasformation of the random numbers in random values of the yield strength satisfying a given probability distribution law has been performed by means of the Box and Müller method (Rubistein, 1981).

In order to simplify the analysis, the variation of the yield strength along the shape of the section of structural members has been neglected by assuming an unique value depending upon the thickness of the flanges of the member section.

2.1 Investigated parameters

According to the modern seismic codes, the knowledge of the ability of a structure to sustain large deformations in plastic range plays a fundamental role in the design of seismic-resistant structures. Therefore, in order to completely characterize the inelastic perfomance of steel framed structures with random variability of the yield strength of members, the multiplier of horizontal forces versus top horizontal displacement curve has been derived for each frame by means of inelastic analyses (fig.1). As a consequence, the characterization of the inelastic structural behaviour has been based upon the following parameters:

- δ_y is the top horizontal displacement corresponding to the formation of the first plastic hinge;
- α_y is the multiplier of horizontal forces corresponding to the formation of the first plastic hinge;
- α_u is the ultimate multiplier of horizontal forces corresponding to the maximum load carrying capacity;
- δ'_y is the top horizontal displacement which the frame presents under horizontal forces corresponding to the ultimate multiplier α_u when the material is considered indefinitely elastic;
- α_u/α_y is the plastic redistribution coefficient;
- δ_{u5} is the ultimate top horizontal displacement computed by assuming as ultimate condition a 5% of strength degradation and, therefore, corresponding to the multiplier 0.95 α_u;
- δ_{u10} is the ultimate top horizontal displacement computed by assuming as ultimate condition a 10% of strength degradation and, therefore, corresponding to the multiplier 0.90 α_u;
- μ_5 is the global ductility corresponding to a 5% of strength degradation ($\mu_5 = \delta_{u5}/\delta_y$) ;
- μ_{10} is the global ductility corresponding to a 10% of strength degradation ($\mu_{10} = \delta_{u10}/\delta_y$);
- $\bar{\mu}_{10}$ is the global ductility corresponding to a 10% of

strength degradation and by considering δ_y' as the parameter characterizing the departure from the elastic behaviour: $\bar{\mu}_{10} = \delta_{u,10} / \delta_y'$.

The use of different formulations for evaluating the global ductility of structures is justified by the consideration that an universally recognized definition for its quantitative evaluation doesn't exist yet (Powell and Allahabadi, 1988). Moreover it is possible to clarify the sensitivity of the ductility ratio definition to the effects due to random variability of yield strength.

Furthermore it is useful to note that collapse is often defined as the reaching of maximum deformation capacity at the yielded end of a member; therefore the ultimate displacement should be computed as the one corresponding to the attainment of this condition and the available ductility should be evaluated as the ratio between this ultimate displacement and the one at the departure from the elastic behaviour. This consideration leads to the consequent admission of the necessity to take into account the effect of randomness in yield stress on the rotational capacity of members. In fact, by considering the experimental relations available in literature, i.e. Mitani and Makino (1980), for the evaluation of the rotational capacity of beam-columns, we can observe that when the actual yield stress is less than the nominal one the formation of the first plastic hinge can develop for a displacement less than the predicted one. Due to the modification of the rotational capacity higher values of the ultimate displacement can be reached and therefore the ductility ratio should change being influenced by these two phenomena.

However due to the approximation of the available relations for computing the rotational capacity of steel members, this kind of phenomena are not easy to be taken into account. In any case, it has been controlled that the values of the ductility ratios, previously defined as derived by the inelastic analyses, are compatible with the available rotational capacity which has been estimated by means of the relations provided by Mitani and Makino (1980). In such a way, it is possible to state that the obtained values for the ductility ratios are not only conventional values but also values that the structure is able to really withstand.

The main problem in the design of seismic-resistant structures is represented by the evaluation of an IDRS (Inelastic Design Response Spectrum) starting from a LEDRS (Linear Elastic Design Response Spectrum) by means of a coefficient, namely q-factor, which takes into account plastic redistribution capacity, available global ductility and energy absorption capacity of the structure. The evaluation of the q-factor requires a great number of dynamic inelastic analyses which have to be repeated for different ground motions; from the computational point of view the requested procedure is very cumbersome so that the use of simplified method is usually accepted (Guerra, Mazzolani and Piluso, 1990).

As the q-factor represents the most synthetic parameter for evaluating the seismic inelastic performance of structures, its evaluation by means of an energy approach (Como and Lanni, 1983) has been included in the analyses. Therefore, the following two parameters have to be added to the list of parameters given above:

- q_5 is the q-factor value computed by assuming the attainment of the displacement δ_{u5} as collapse condition;
- q_{10} is the q-factor value computed by assuming the attainment of the displacement δ_{u10} as collapse condition.

The evaluation of these parameters provide new informations which are not given in the previous works (Kuwamura and Kato, 1989 - Kuwamura and Sasaki, 1990 - Elnashai and Chryssantopoulos, 1990).

3 PARAMETRIC ANALYSIS

It has been already pointed out that the random inelastic response of steel framed structures, due to random material variability, can be studied by means of the hybrid Monte Carlo simulation, in which numerical simulation is used in order to obtain a statistical sample of the structural response and, successively, the mathematical models are investigated in order to define the probability distribution functions. This approach has been already applied (Mazzolani, Mele and Piluso, 1991), leading to interesting results, but general conclusions cannot be obtained because the previous study was referred to a single structural scheme.

In order to grasp all parameters affecting the random inelastic response of steel framed structures, a parametric analysis is requested. However, the computational effort is very cumbersome so that a compromise has to be found between the necessity to point out the influence of the main parameters and the one to reduce the high computational time required by Monte Carlo simulation.

The following aspects have been chosen to be investigated:

- the influence of the coefficient of variation COV of the yield strength of the structural steel;
- the dependence of the yield strength upon the flange thickness of the frame members;
- the influence of the column-to-beam strength ratio;
- the influence of the structural scheme.

With reference to the coefficient of variation COV of the material yield strength, the following values have been considered: 0.025, 0.050, 0.075, 0.100, 0.150. They cover the whole variability range of standard structural steels.

The dependence of the yield strength upon the flange thickness of frame members has been introduced by means of a linear variation law:

$$f_{y,m}(t_f) = -\alpha\, t_f + \beta$$

dove $f_{y,m}$ is the medium value of the yield stress and t_f is the flange thickness.

From the inelastic behaviour point of view, the importance of the dependence of the yield stress upon the thickness increases as far as the value of α increases. The values $\alpha = 2.987$ N/mm^3 and $\beta = 444.2$ N/mm^2, which correspond to the Fe510 grade of steel, have been adopted in order to consider the case of maximum influence (Mazzolani, Mele and Piluso, 1990). The ratio between the flexural strength of columns and the one of beams has been introduced by means of the parameter:

$$\rho = \frac{Z_{p,c}}{Z_{p,b}}$$

being:

- $Z_{p,c}$ the plastic section modulus of columns;
- $Z_{p,b}$ the plastic section modulus of beams;

This parameter presents, obviously, a decisive influence on the collapse mechanism which is expected on the base of the nominal yield strength of members. The influence of this parameter should be investigated until the value corresponding to the attainment of a global type collapse mechanism. However, we have to take into account that the random response of steel frames, which are designed on the base of the nominal yield strength in order to obtain a global failure mode, has been already analysed (Mazzolani, Mele and Piluso, 1991) and, therefore, for reducing the computational efforts it was decided to limit initially the analysis to the values $\rho = 1$ and $\rho = 1.64$. These values, for beams of HE200B shape, correspond to columns of HE200B and HE240B shapes respectively.

Finally, with reference to the possible influence of the structural scheme, two different types of frames, both with two bays, have been considered: a three storey frame (FRAME 1) and a six storey frame (FRAME 2). In both cases the length of the bays is 450 cm while the interstorey height is 300 cm.

It can be assessed that each case of the parametric analysis is defined by the values assumed by COV and ρ for a given structural scheme, leading to a total of 20 studied cases. For each value of the coefficient of variation COV of the material yield strength, 100 frames with a random gaussian distribution of member yield strength have been generated by means of the procedure previously described. Therefore, the entire parametric analysis (Piluso, 1992) has requested 2000 numerical simulations of the structural inelastic response.

The random frames generation, the inelastic analysis and the evaluation of the parameters characterizing the structural inelastic response have been carried out by means of three peculiar computer programs which work in sequence.

For all cases of the parametric analysis, the representation of the random response by means of the normal probability graph paper has been performed for all the parameters describing the structural inelastic behaviour. Moreover the confidence interval with a 5% confidence level has been derived (Piluso, 1992). The use of the confidence interval allows to estimate fractiles including the scatters due to the fact that, unavoidably, we can provide only an estimate of the values of the average and the standard deviation.

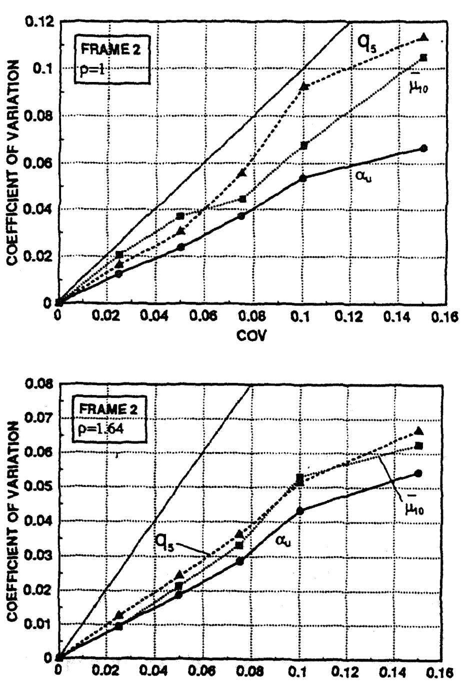

Figure 2. Stability of different definitions of available global ductility

3.1 Discussion of the results

The first result which particularly merits to be emphasized comes from the comparison among the different definitions of available global ductility in terms of stability, from the statistical point of view, expressed by means of the corresponding coefficient of variation.

In fig.2, with reference to FRAME 2 and for $\rho = 1$ and $\rho = 1.64$, the values of the coefficient of variation corresponding to the different definitions of the available global ductility are given as a function of the coefficient of variation COV of the material yield strength. From this representation, it can be observed that global ductility defined as $\bar{\mu}_{10}$ is, from the statistical point of view, the most stable parameter leading to little values of the corresponding coefficient of variation. Moreover, it is more stable than the material yield strength itself. Due to the fact that the available global ductility defined as $\bar{\mu}_{10}$ is the most stable parameter, it can be concluded that this parameter is more suitable than the others in order to provide a measure of the available global ductility. For this reason, in the following reference is made to the available global ductility defined as $\bar{\mu}_{10}$ while the results corresponding to the alternative definitions of global ductility are provided in another work (Piluso, 1992).

This kind of result, which is very important in order to define significant parameters for the structural inelastic response characterization, has been obtained also with reference to the q-factor computed by means of energy

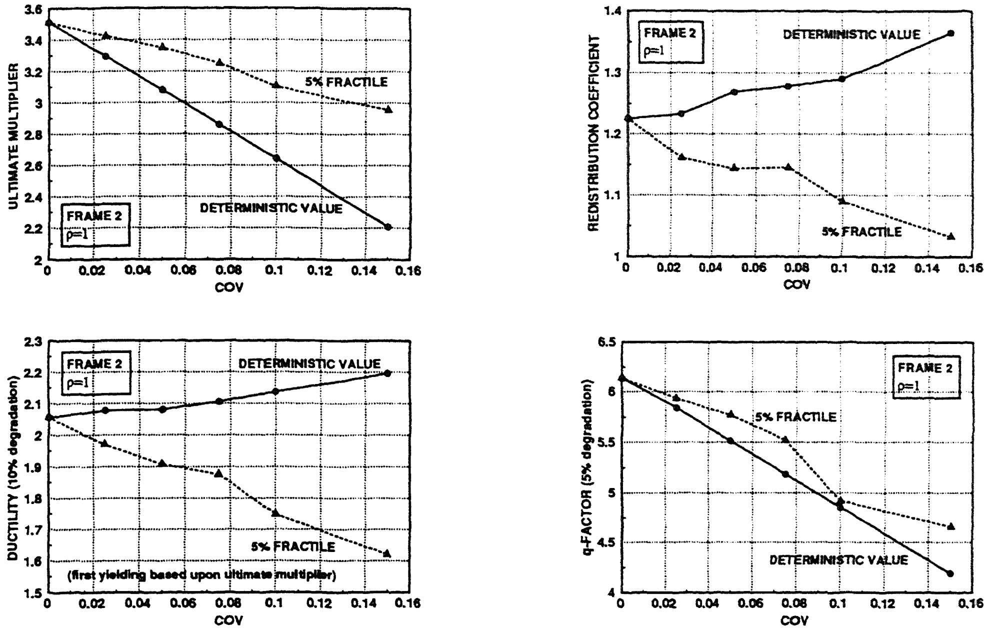

Figure 3. Comparison between probabilistic and deterministic analyses.

approach.
The comparison between the results of the probabilistic analysis, obtained through the Monte Carlo simulation, and the ones obtained through the deterministic analysis seems to be particularly interesting.
The results of probabilistic analyses are mainly represented by the values of the 5% fractiles, while the results of the deterministic analysis correspond to the values obtained through the inelastic response simulation performed by assuming a constant value (for members with the same flange thickness) of the yield stress given by:

$$f_y(t_f) = f_{y,k}(t_f) = f_{y,m}(t_f)\,(1 - 2\,COV)$$

The comparison between the results of the probabilistic analysis and the ones obtained through the deterministic approach is presented in figure 3 for all the parameters characterizing the inelastic structural response. As an example reference is made here only to the six-storey frame (FRAME 2) with $\rho = 1$.
It can be observed that the prediction of the ultimate multiplier of horizontal forces and of the q-factor, by means of the deterministic approach, is always on the safe side, leading to values which are less than the corresponding 5% fractiles computed by using the probabilistic approach. On the contrary the use of deterministic analysis is not safe when the plastic redistribution capacity and global ductility have to be computed. In fact, the above comparison shows, in particular, that the deterministic value of the available global ductility, independently upon the adopted definition (Piluso, 1992), is greater than the corresponding 5% fractile computed by the probabilistic approach.
Finally, it is useful to point out that the scatters between the results of deterministic and probabilistic analyses increase as far as the coefficient of variation COV of the material yield strength increases.
The same results have been obtained also in the case $\rho = 1.64$ for the FRAME 2 and in the cases $\rho = 1$ and $\rho = 1.64$ for the FRAME 1(Piluso,1992).
An useful representation of the 5% fractiles of the parameters characterizing the inelastic response can be obtained by normalizing them with the corresponding value copmputed for COV=0. For each parameter the ratio φ between the 5% fractile calculated for $COV \neq 0$ and the value computed for COV=0 has been considered. In figure 4, the relation φ versus COV is represented for the most stable and, therefore, significant parameters. It can be observed that the column-to-beam strength ratio ρ plays always an important role, while the influence of the structural scheme is not so significant and, in any case, it is dependent upon the column-to-beam strength ratio.
Moreover, the ratio φ can be interpreted as a correction factor which can be used in order to predict the 5% fractiles starting from the values computed by means of a deterministic analysis (COV=0). Furthermore, they are able to point out the influence of the randomness in material yield strength on the inelastic behaviour of steel framed structures. In particular, it is very important to point out, for the seismic point of view, that the increase of the coefficient of variation of the members yield strength can produce a significant reduction of the available global ductility. This reduction can reach about 40%, which seems a value not at all negligible.

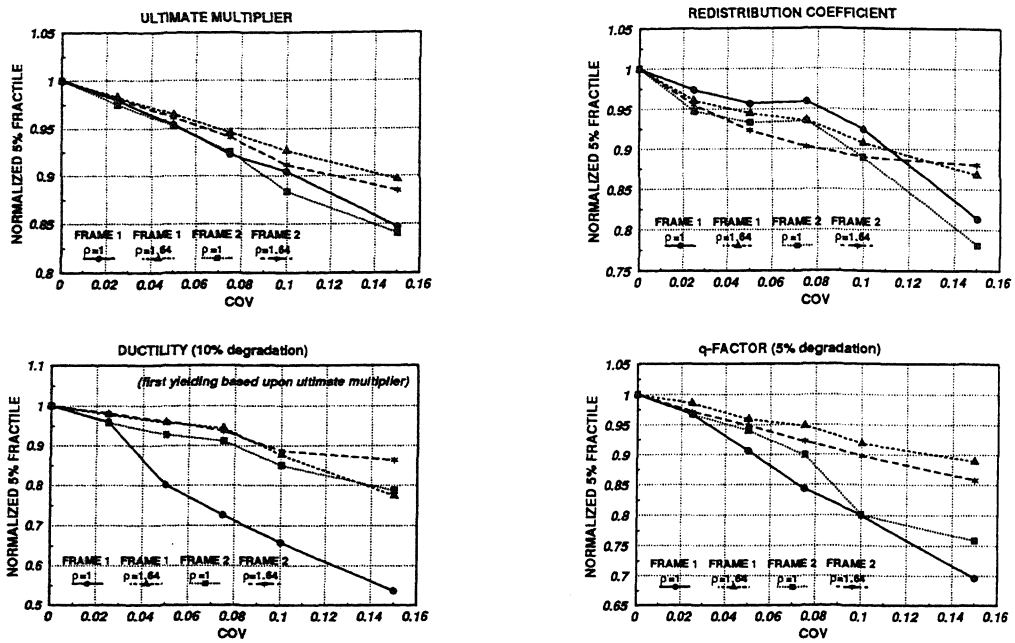

Figure 4. Influence of the column-to-beam strength ratio and of the structural scheme.

4 CONCLUSIONS

The results of this paper have confirmed that the effects of randomness in material yield strength increase as far as the coefficient of variation COV increases. In addition they have pointed out that the importance of this influence depends also upon both the column-to-beam strength ratio and the structural scheme.

The increase of the coefficient of variation of material yield strength can produce a significant reduction of the available global ductility which is not negligible from the seismic design point of view.

Finally, it has pointed out that the use of the probabilistic approach represents an useful tool in order to define stable parameters for characterizing the inelastic structural response of seismic-resistant steel frames.

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