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COLLAPSE ASSESSMENT OF IRREGULAR STRUCTURES BASED ON DAMAGE FUNCTIONALS

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ABSTRACT

The objective of this study is the simulation of the non-linear behaviour and assessment of structural collapse of reinforced concrete structures under seismic loading. For this purpose, a new non-linear element implemented in DRAIN-2D is presented and the criterion for the assessment of different control parameters is discussed. A methodology based on damage functionals is used, allowing for the establishment of behaviour factors (*q* factors) for seismic design. The methodology allows the assessment of the variation of the damage functionals with the acceleration level and the identification of the critical elements and zones over the structure. Relations between different damage functionals and the seismic severity are estimated for four different reinforced concrete structures - one regular and three with different types of irregularities.

KEYWORDS

Damage functional; collapse; behaviour factor; non-linear behaviour; fibre model.

INTRODUCTION

The analysis of reinforced concrete structures under static or dynamic loading based on linear elastic models is a common procedure in seismic design. These methods, may conduct to safe designs, but do not provide a correct information about the response of structures subjected to strong ground motions. There is, hence, the need for computer programs which are able to accurately account for the inelastic behaviour of reinforced concrete elements, if not directly for design procedures, at least to comparatively calibrate the use of the linear models. This is specially true for the assessment of *q* factors for seismic design.

The simulation of the appropriate non-linear behaviour of reinforced concrete structures under seismic loading is essential to correctly model the capacity of the structural elements to deform beyond the elastic limits, and to take into account the distribution of non-linear excursions among the various members.

There are numerous factors that affect the *q* factors evaluation. Upon others, they depend on the seismic load itself, on the structural materials adopted, on the its capacity to withstand non-linear deformations and on its geometrical and structural regularity.

In this study, a methodology for the quantification of q factors based on damage functionals is suggested, with special emphasis on the influence of the structural irregularities. Four damage functionals; peak and cyclic ductility, Park and Ang and Miner indexes, are used for the assessment of structural collapse. Special emphasis is given to the Park and Ang and Miner damage functionals' results.

The methodology is applied to four representative reinforced concrete frame structures, one regular and three with different kinds of irregularities, showing that the admissible q factors can be evaluated based on the damage functionals and that they should be dependent on the structural regularity.

METHODOLOGY OF ANALYSIS

To analyse the applicability of the methodology of collapse assessment herein presented and observe the evolution of the damage functionals with the q factors and the seismic load, some parametric studies were carried out for one regular and three irregular reinforced concrete frame structures (figure 1). The definition of the dimensions of the irregular structures was based on the regular configuration. The three examples of irregular structures are two setbacks (Irr. 1 and Irr. 2) and a soft story with the consideration of the effects of infill masonry walls (Irr. 3). Table 1 and figure 1 present the structural geometry and members dimensions.

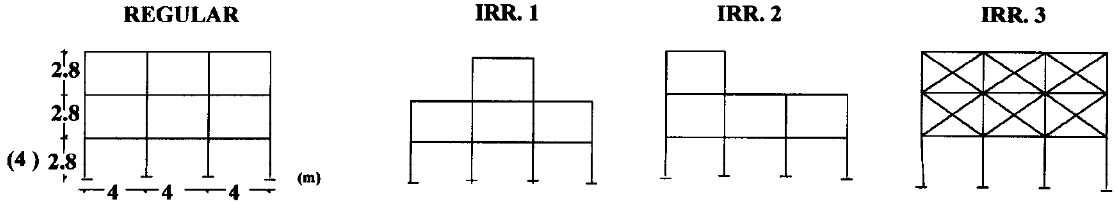

Fig. 1 - Geometrical configuration of the analysed structures.

Table 1. Element cross sections

Exterior Columns (b x h)	Interior Columns (b x h)	Beams (b x h)
0.4 x 0.2	0.2 x 0.4	0.2 x 0.4

(Dimensions in meters)

All structural elements were modelled based on a R. C. non-linear model implemented as an additional element in DRAIN2D (Kannan & Powell, 1975) which accounts for important characteristics of the behaviour of R.C. elements under cyclic loading, such as pinching, strength degradation and the effect of axial load variation. In this element, the dissipative hinge behaviour is analysed by means of a fibre model (Gomes, 1992), where the section is considered to be composed of a number of element fibres describing separately concrete and steel behaviour. The non-linear stress-strain laws of these two materials, including the effect of stress reversals, are essential features of the model. The fibre model allows for the use of complex material behaviour models under cyclic loading and reflects the section behaviour rather accurately. To model the reinforcing steel a relationship proposed by Giuffrè-Menegoto-Pinto and modified by Popov-Bertero is used. The confined concrete is modelled using a modified version of the model proposed by Thompson and Park. Figure 2 displays a typical moment-curvature diagram obtained for the bottom central column of the soft story structure where some of the above mentioned behaviour features can be observed.

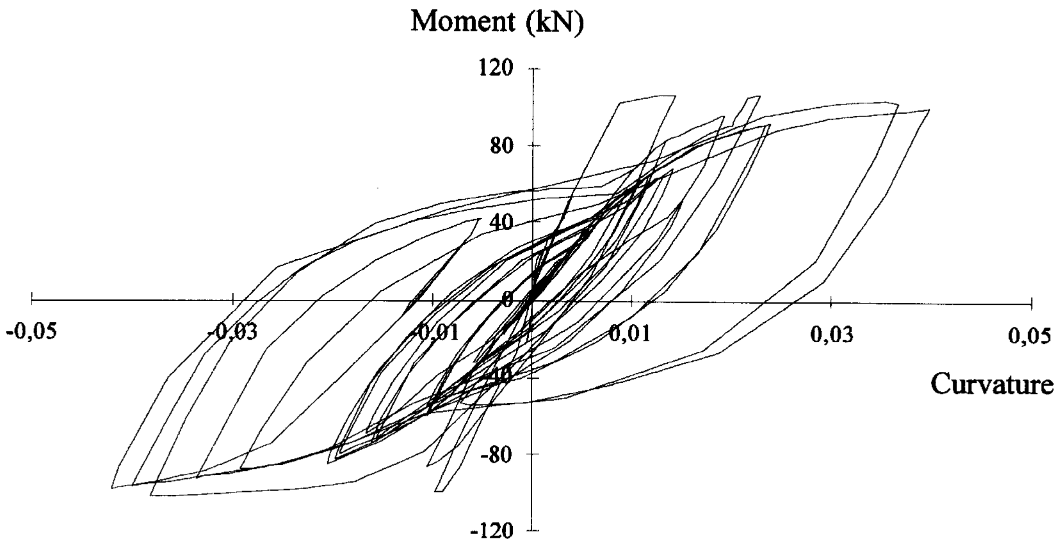

Fig. 2 - Typical moment-curvature diagram for the R. C. model

The used damage functionals to assess structural behaviour include peak and cyclic ductility, Park and Ang and Miner indexes. The peak and cyclic ductility were evaluated to allow comparison with the other damage functionals. The Park and Ang damage index (D_{PA}) (Park et al., 1987) takes into account not only the maximum observed curvature (X_{max}), and thus in some sense a measure of ductility, but also the dissipated hysteretic energy (HE).

$$D_{PA} = (X_{max} / X_{u,mon}) + \beta \, HE / (F_y \, X_{u,mon})$$

Calibration of the parameters used to evaluate the index, such as the β values, was made by comparison with the results obtained for the Miner damage index. The Miner damage index, was evaluated simulating constant amplitude tests in a cross section of typical reinforced concrete columns and beams, using the described fibre model. For each amplitude, the number of cycles up to collapse was determined. For that purpose, collapse was considered to occur when the accumulated plastic deformations in the reinforcing steel exceeded a certain limit or when there was a total loss in the concrete resistance. In these results, the influence of shear forces has not been taken into account, which anyway could be easily be done, at least in an approximate way. Figure. 3 shows, for a typical beam and column elements, a log-log representation of the number of cycles up to collapse for different imposed constant amplitude curvatures. It can be seen that there is a very good agreement with similar laws (S-N curves) for high and low cycle fatigue tests of materials or structural elements. For the column, two different laws can be observed according to the type of failure (in the steel, before or after the outer concrete spalling, or in the concrete).

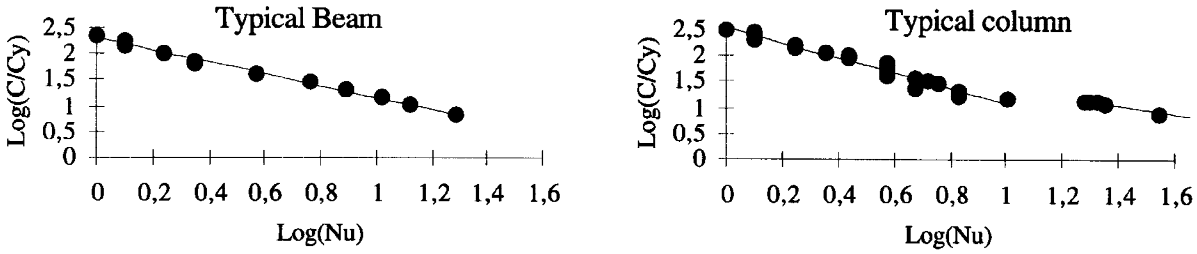

Fig. 3 - Number of cycles up to collapse as a function of constant amplitude curvature

Once the collapse criteria for the structural elements is defined, different global collapse criteria for the whole structure can be implemented. One, could be that local collapse occurs in a sufficient number of end sections to cause a structural mechanism. Other, more conservative, would be to admit that global collapse has occurred once collapse occurs in a specific end section, specially if this section corresponds to a load carrying element such as a column. Other criteria that could be implemented are related to maximum top displacements and/or interstory drift. Discussion of these criteria is nevertheless not in this work's scope.

RESULTS

The purpose of analysing the four different structures was twofold. On one hand to evaluate the sensitivity of the damage functionals to the seismic demand and to the structural characteristics, thus testing their usefulness to characterise the seismic behaviour. On the other hand, assuming that the first objective could be achieved, it was also intended to show that damage functionals can be used to determine *q* factors to be applied in seismic design and that, to different buildings with different regularity characteristics, should correspond different *q* factors.

Previous studies made in terms of required ductility (Bento & Guerreiro, 1994), and in terms of Park and Ang and Miner damage functionals (Bento & Azevedo, 1995), have shown that there is indeed an influence of the building regularity on its seismic performance. This is recognised by different earthquake design codes (Eurocode 8, 1993), although only in terms of qualitative rules.

To analyse the sensitivity of the damage functionals to the level of structural resistance as related to the seismic load, the structures under study were designed for a seismic action according to Eurocode 8, assuming a soft soil, a peak ground acceleration equal to 0.3g and different *q* factors ranging between 3 and 6. Permanent and quasi-permanent loads were also considered in the design. In all cases, design rules like the existence of minimum steel reinforcement and the observance of the strong column-weak beam principle were followed. Then, those structures, designed according to the Eurocode rules, were subjected to different increasing peak ground acceleration levels to study the variation of the damage functionals with the seismic severity. The acceleration levels vary according to an amplification factor A_F corresponding to 1 to 4 times the design earthquake and so to values between 0.3 and 1.2g.

Figure 4 presents some of those results, namely the values of the different damage functionals, as a function of the adopted *q* factors and for two different levels of the seismic action (the original level used for design and that level multiplied by 1.5). The presented results correspond to the base of the bottom central column of all analysed buildings.

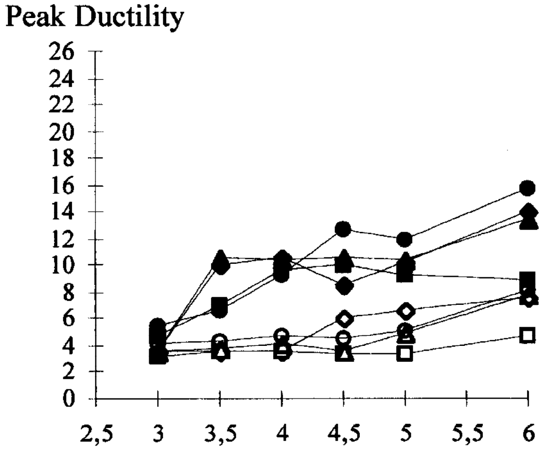

Fig. 4 a -Peak ductility vs *q* factor

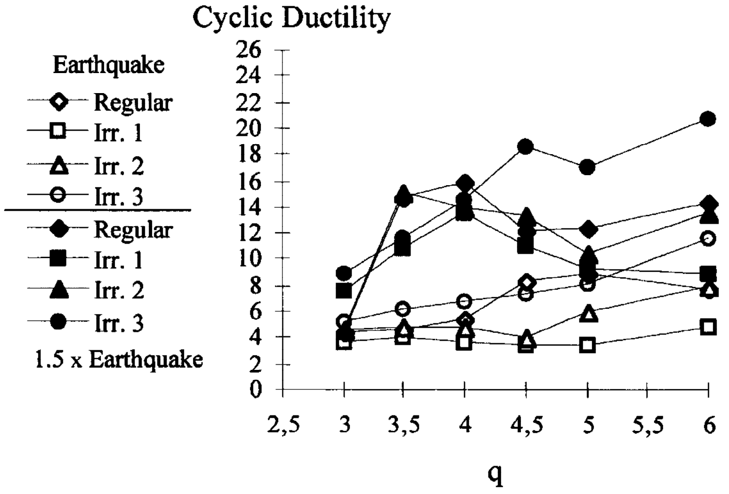

Fig. 4 b - Cyclic ductility vs *q* factor

In figure 4 *a* and *b* are displayed the results in terms of required peak and cyclic ductility. It can be seen that, generally, for all analysed structures, the required ductility significantly increases with the increase of the seismic action level. Ductility values also increase with *q*, although the compliance with minimum reinforcement rules causes the ductility values to remain stable for higher *q* factors, given that the increase in *q* does not decrease the resistance of the sections. It can also be seen that the higher ductility demands occur in the soft story structure. For the other irregular buildings the ductility requirements are comparable to the regular structure ones. In other zones of the structure, like the beams closer to the set-backs, the difference between the ductility requirements for the irregular set-back and the regular buildings becomes more evident.

Figures 4 *c* and *d* show similar results obtained respectively for the Park and Ang and Miner damage indexes. The results are qualitatively comparable to the ones obtained in terms of ductility, although the Miner indexes seem to be quite insensitive to the increase of q.

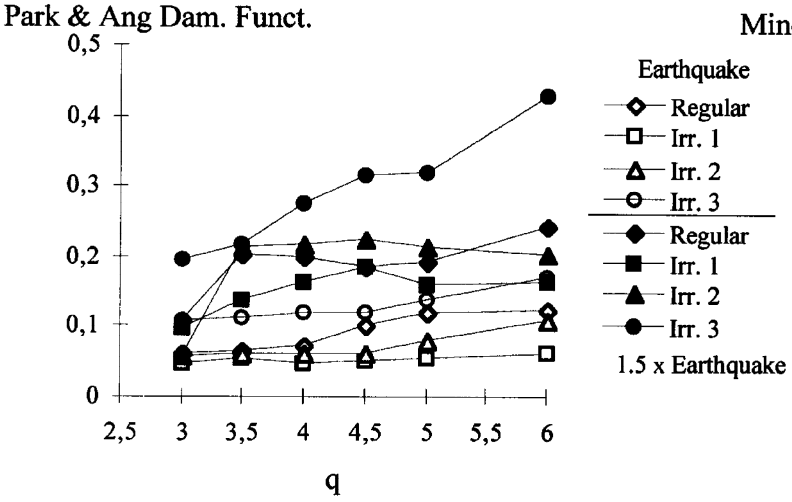

Fig. 4 c -Park and Ang index vs *q* factor

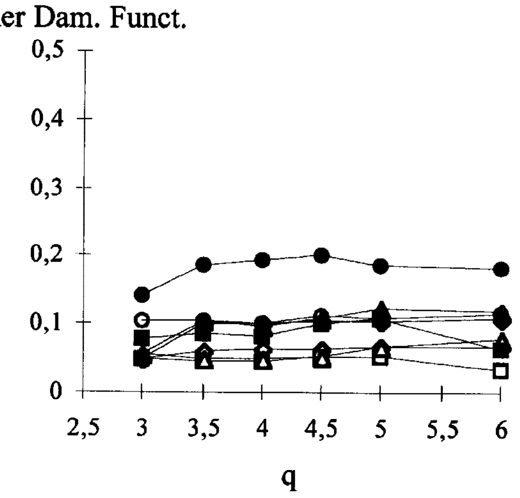

Fig. 4 d - Miner index vs *q* factor

To understand some of the effects of the irregularities, the distribution of the Miner damage indexes for the all columns and beams is displayed in figures 5 and 6, respectively. In that figure are shown, for both ends of each element and for each structure, the obtained damage indexes when the structure is designed assuming *q* equal to 4 and an amplification of the seismic action by 1.5. The columns are numbered from bottom to top and from left to right and the beams from left to right and then from bottom to top.

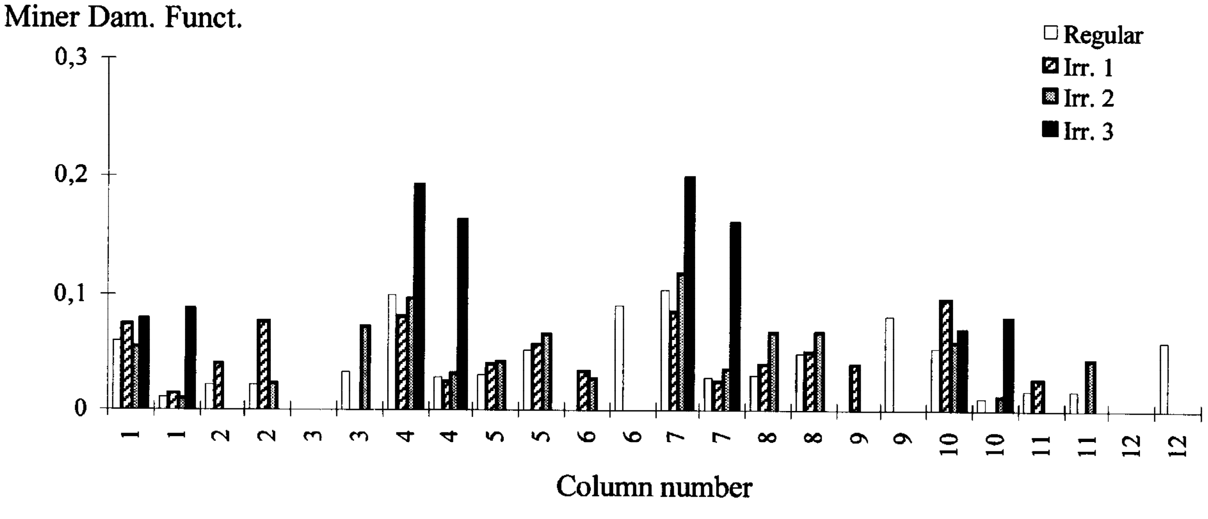

Fig. 5 - Miner damage functional in column elements

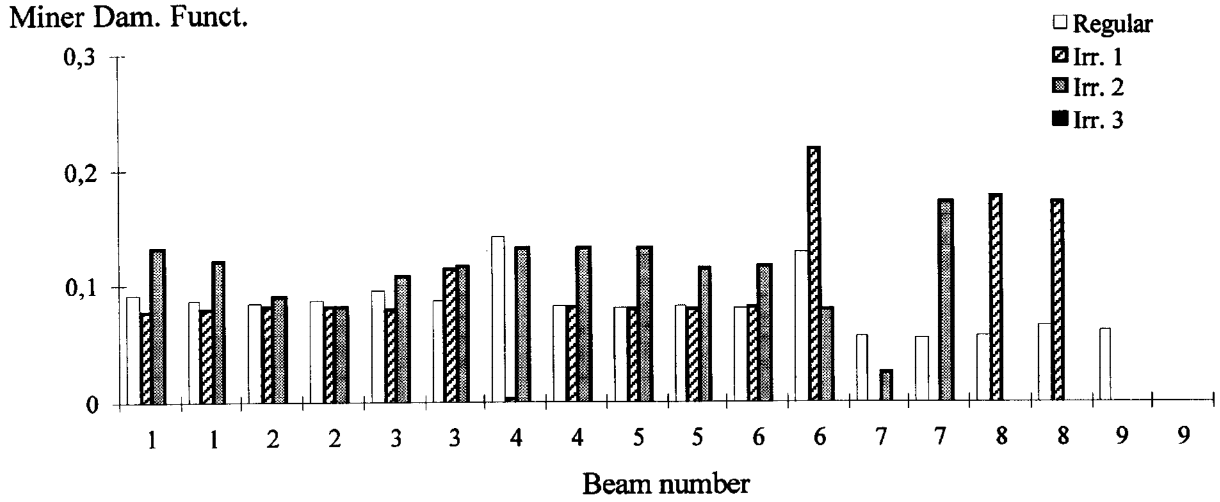

Fig. 6 - Miner damage functional in beam elements

It can be observed that in the regular structure there is a quite homogeneous distribution of the damage indexes all over the structure, with the beams having slightly higher values given the adoption of the weak beam - strong column design concept. On the contrary, in the irregular structures the distribution is more heterogeneous, with zones of the structure showing very high damage indexes, as is the case of the soft story bottom columns or the beams located near the setbacks, while other zones exhibit very low values or even zero values.

These results show that, in irregular structures, even if appropriate seismic design procedures are applied, there is a tendency to concentrate damage, and thus energy dissipation, in just a few elements. This fact is against a basic principle of good seismic performance, that energy dissipation should occur in the largest possible number of zones of the structure and most certainly not just in elements such as the bottom columns of a building as is the case of the soft story.

Given the uncertainty in the seismic load as well as in the structural resistance, admissible q factors for seismic design have to be evaluated by means of non deterministic procedures. All previously presented results correspond to analyses where the mean values of the materials properties were assumed and where the variability of the seismic load was not considered. This explains why, even for large q factors, the obtained damage indexes do not exhibit very high values. Thus, even if acceptable damage indexes are obtained in the above results, as the assumed properties of the materials have a 50% probability of being smaller and the assumed seismic load value also has a large probability of being exceeded, the probability of collapse is clearly much higher than the commonly admitted limits.

It is thus necessary to study the influence of the materials properties and of the seismic action on the obtained damage indexes, and then perform a probabilistic assessment of the structural performance, adopting the q factors which lead to acceptable probabilities of collapse. Although this procedure was out of the scope of the present study, a sensitivity of the damage indexes to the seismic load level was carried out. In figure 7 *a* and *b* is shown, also for the bottom central columns, the influence of the seismic load level respectively on the peak and cyclic ductility and on the Park and Ang and Miner damage indexes, for an assumed q factor equal to 3.

As could be expected, it can be seen that both, the ductility demands and the damage indexes, increase with the amplification of the seismic action. One worth mentioning detail about this increase is that the rate of increase is larger for the higher values of seismic load, and for amplification values approximately equal to 3 the damage indexes are already close to 1. It should be stated that if a safety factor would be applied to the materials properties, to use characteristic values instead of mean values, the amplification factor needed to get damage indexes close to 1 would of course be reduced and most probably be within the range of safety factors also usually applied to transform mean seismic load values into characteristic ones.

Apart from these aspects it can also be noticed, both in figure 7 *a* and 7 *b*, that the column of the soft story building always exhibits the highest values both in terms of ductility and damage indexes. For this structure and for A_F values larger than 3, failure of elements leading to collapse occurs and hence no damage values are displayed. The symmetric set-back building (Irr. 1) shows values slightly lower than the ones exhibited by the regular one. This would not be the case if, as could be seen in figure 6, the comparison would be made for other elements, like the beams located near the set-back, where the damage indexes for the regular structure would be the lowest ones.

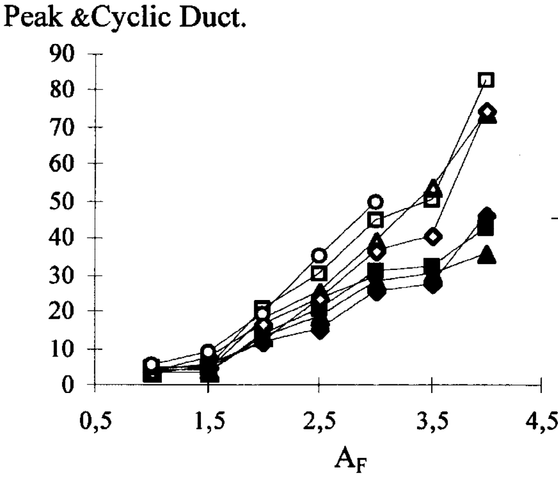

Fig. 7 a - Peak and cyclic ductility vs A_F

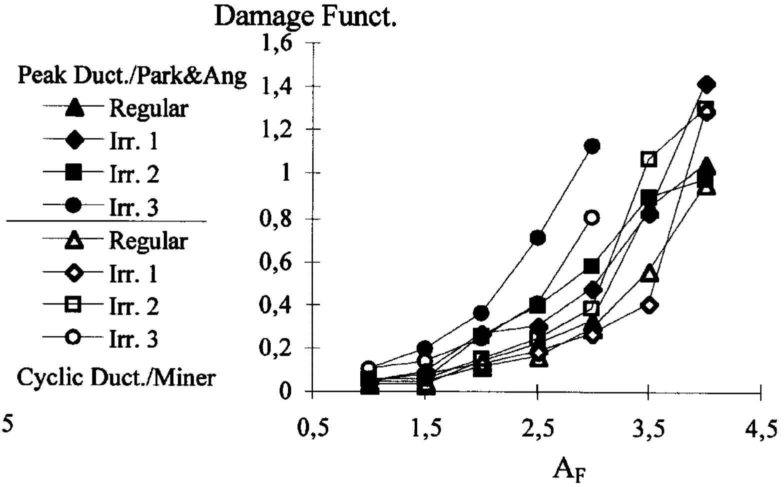

Fig. 7 b - Park & Ang and Miner indexes vs A_F

Still another observation that can be made regarding figure 7 *b* is that there is a quite good agreement between the evolution of the two damage indexes, although there is a consistent tendency for lower Miner index values.

CONCLUSIONS

The proposed model used to assess the seismic behaviour of R.C. structures seems to adequately simulate the most important features of the response of such structures.

The methodology to define damage indexes, which is based on the use of such model, also seems to yield good results, and, specifically in what regards the proposed Miner damage index, relationships between number of cycles to failure and cycle amplitude have a very good agreement with typical S/N curves for low or high cycle fatigue laws.

Damage functionals, such as peak or cyclic ductility and also Miner and Park and Ang damage indexes have been shown to be dependent on parameters like the used design *q* factor and the severity of the seismic action.

Furthermore, it can be shown that the admissible *q* factors can be evaluated based on damage functionals such as the above mentioned ones and that they are highly dependent, among other factors, on the structural regularity.

Irregular structures not only exhibit higher values of the damage functionals than regular structures do, but also show a very large heterogeneity in terms of the distribution of such values all over the structure. For some simple irregular structures, the zones where those higher values occur were identified and the characteristics of their location are in agreement with findings from past earthquake occurrences.

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