Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Influence of the cross-section of steel beam columns on the seismic design

L.Calado
Technical University of Lisbon, Portugal

ABSTRACT: This study deals with the numerical simulation for predicting the failure of steel beam columns under earthquake conditions. A model for the behaviour of beam columns subjected to cyclic loadings which considers the section shape, local buckling, material properties, fracture and low cyclic fatigue, is succinctly presented. A damage accumulation model expressed as a function of the inelastic strain and the dissipative hysteretic energy is proposed with a criterium to define failure based on the theory of metal fatigue and on the hypothesis of linear damage accumulation. With this model and criteria it is possible to show the influence of different parameters, namely the axial load, cross-section, steel grade, non-dimensional slenderness and arrange them in groups. These groups show the capacity of different cross-sections to dissipate energy by means of a ductile hysteretic process, and are useful for a logical and economical seismic design of beam columns.

1 INTRODUCTION

Beam columns are one of the most common steel structural elements and have been, in the past, exhaustively studied under static loadings. More recently, research centers and universities have started to develop experimental and numerical studies on this type of structural elements under cyclic loadings. But some doubts arise about their performance to resist seismic actions and to undergo large plastic deformation in the severe excursion in the inelastic range.

The cyclic behaviour of these types of structural elements is affected by some physical phenomena (local buckling, fracture, low-cycle-fatigue), geometric characteristics, level of axial load, boundary conditions and material properties.

For a better understanding of the complex behaviour of beam columns under seismic actions a research program was developed involving the Department of Structural Engineering of the Politecnico di Milano and the Department of Civil Engineering of the Instituto Superior Técnico. Experimental tests were performed in Italy and numerical models were developed in Portugal.

The purpose of this research was to develop a procedure for the assessment of behaviour coefficients of beam columns under earthquake actions. The beam column model, damage accumulation and failure criteria is summarized here as it was object of a detailed description in previous works: vide Ballio (1986), Calado and Azevedo (1989) and Calado (1989). The results of static and dynamic analyses performed on beam columns are presented and commented.

2 THE BEAM COLUMN MODEL

The numerical model of the beam column subjected to vertical load and horizontal displacements at the upper edge consists of a rigid bar which connection is made by means of an elasto-plastic cell in which all geometrical and mechanical properties are concentrated (Figure 1). The bar is assumed to be perfectly straight, and the horizontal displacement cause bending about a principal axis of the section. Shear deformation is disregarded. The equivalence between the model and the beam column is imposed in the elastic range by equating the moments and the horizontal displacement at the upper edge at the elastic limit.

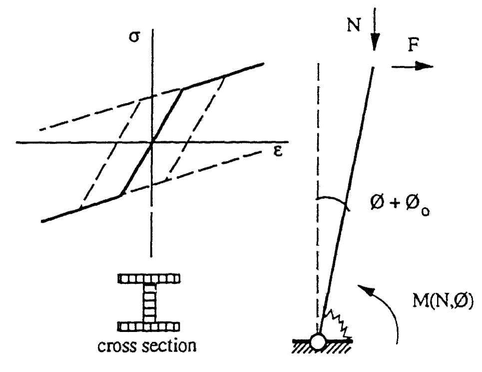

Figure 1. Numerical model of a beam column.

The constitutive stress-strain relation of the material was taken as elasto-plastic with kinematic strain-hardening. Recent work develop by Castiglioni (1987) on the influence of the constitutive relation on the cyclic behaviour of steel bent sections have shown that the elasto-plastic relation with kinematic strain-hardening give good results without excessive computing time when compared with other more complex constitutive stress-strain relations.

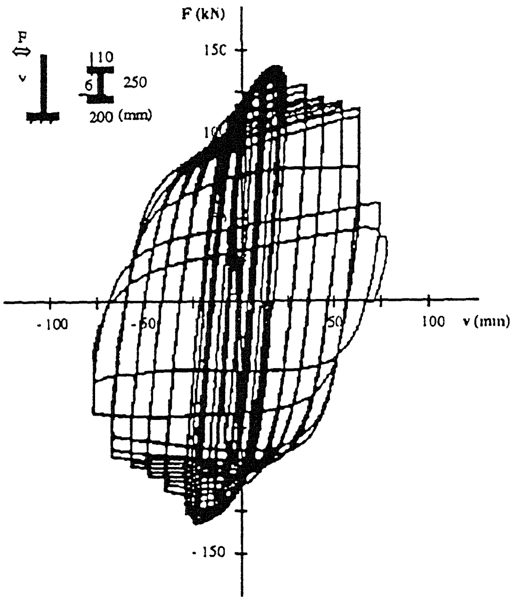

Figure 2. Experimental behaviour of a beam column.

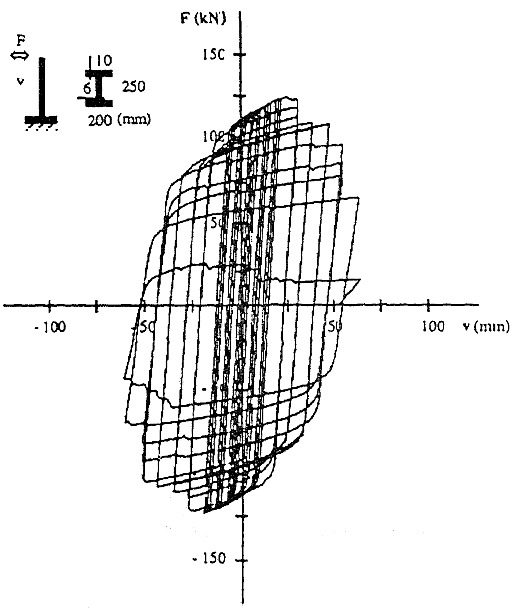

Figure 3. Numerical behaviour of a beam column.

It was also developed for the cell a numerical model that considers the local buckling, fracture and low cyclic fatigue. Local buckling was modeled using the elastic plate theory as a guideline whereas the method of the strain energy density criterium proposed by Sih and Madenci (1983) was used to simulate fracture. Low cycle fatigue was simulated using the Miner's rule.

The accuracy of the developed numerical model can be observed by the comparison of Figures 2 and 3.

Some small differences can be observed between the numerical and the experimental results due to the anticipation or delay of the simulation of the local buckling of flanges and web. Nevertheless, the numerical force-displacement diagram and the dissipated hysteretic energy are analogous to the ones observed during the experimental tests.

3 FAILURE CRITERIUM AND DAMAGE ACCUMULATION MODEL

When a material or structural element is subjected to cyclic loading, it can be assumed that the cycles exceeding a given amplitude will cause structural micro-modifications that lead to a degradation in the capacity to dissipate energy. Even if these modifications do not cause visible variations in the hysteretic response of the material or the structural element, they will cause structural damage that accumulates throughout the cycles sequence, until failure is reached.

The objective of this damage accumulation model was to find a simple formulation allowing the prediction of failure for steel structural elements. The failure in this context was defined as an unacceptable limit state associated with the reduction in the hysteretic energy dissipation capacity. The failure criterium can be expressed by the equation (1):

$$\eta_i = \frac{A_i}{A_{yi}} < \gamma \tag{1}$$

where η_i is the normalized hysteretic energy, A_i the hysteretic energy dissipated in the i^{th} cycle, A_{yi} the energy that the element would dissipate if it had an elasto-plastic behaviour and γ the percentage of the normalized hysteretic energy corresponding to failure. A_i and A_{yi} are obtained through the force-displacement diagram (Figure 4)

Due to the stochastic nature of all the time histories originated by earthquake, it is not advisable to admit that the failure occurs for extremely low values of the normalized hysteretic energy. The numerical model of the beam column can simulate the hysteretic behaviour up to very low values of the normalized hysteretic energy. Nevertheless, to ensure structural safety, it is advisable to admit that failure occurs for η_i values approximately equal to 50%.

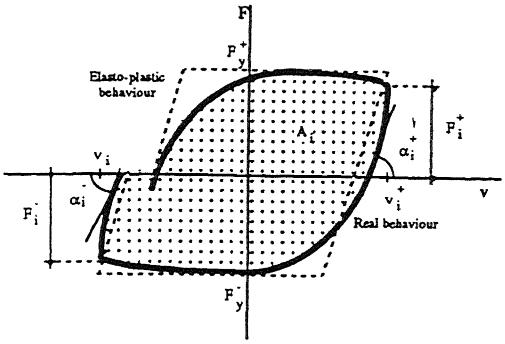

Figure 4. Force-displacement diagram in elasto-plastic range.

For the damage accumulation model a linear damage accumulation was assumed, expressed by the equation (2):

$$ID = a \sum_{i=1}^{N} (\Delta\zeta_{pi})^{c} \tag{2}$$

where ID is the damage index and represents the accumulated damage after N cycles of displacement, $\Delta\zeta_{pi}$ the average plastic deformation (in the cell of the beam column model) (Figure 5). a and c are parameters related to the structural behaviour. For beam column, and after several experimental tests and numerical simulations these parameters were set equal to 1.00.

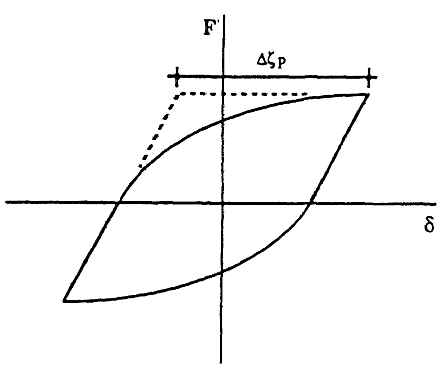

Figure 5. Plastic deformation amplitude $\Delta\zeta_{pi}$.

Structural elements under elastic cyclic loading histories have $\Delta\zeta_{pi}$ equal to zero, no damage accumulation and a null damage index ID. Ductile materials are characterized by large plastic deformations and so a great value of the damage index ID, whereas brittle materials have small deformation before failure and small values of the damage index ID.

4 PARAMETRIC STUDIES

The imposed histories used in this study were generated by means of a stochastic process and intend to re-create those corresponding to seismic loading. In Figure 6 is represented a typical time history.

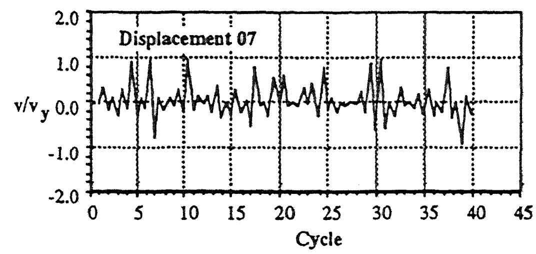

Figure 6. Typical time history.

These histories were normalized with respect to the elastic limit displacement (v_y) of the beam column and have a maximum value equal to one. To obtain the failure of the beam column using these time histories it is necessary to have them amplified.

The use of different time histories was intended to verify if the damage model presented was independent of the time history, and therefore if it was possible to predict the failure of a beam column based on any single time history. For that purpose several beam columns were studied, with different cross-sections, level of axial load, non-dimensional slenderness and steel grade.

4.1 Influence of the time history

The numerical results show that damage accumulation seems independent on the time history and dependent on the sum of the plastic deformations (Figure 7).

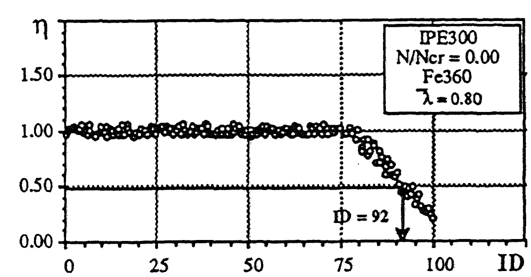

Figure 7. Typical evolution of the damage accumulation of a beam column up to failure.

The graphic in Figure 7 presents the damage index ID as a measure of the accumulated damage versus the normalized hysteretic energy (η). The numerical simulations were carried out until a 20% value of the normalized hysteretic energy was reached. At this stage of the research project the use of a very low value of η was regarded as convenient to enable the study of the model up to full failure. It should be remembered that from the point of view of limit state, failure should be related to a percentage value approximately equal to 50%.

4.2 Influence of the axial load level

Being the axial load one of the most important parameters relating with the stability of the beam column it is presumable that this parameter will have a great influence on the damage accumulation. As expectable, an increase in the level of axial load implies a reduction on the reserve of strength of the element and, simultaneously, smaller damage index ID values that show smaller ductile capacity (Figures 8 and 9).

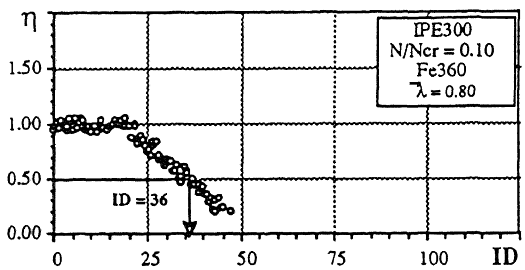

Figure 8. Influence of the level of axial load (N/Ncr = 0.10)

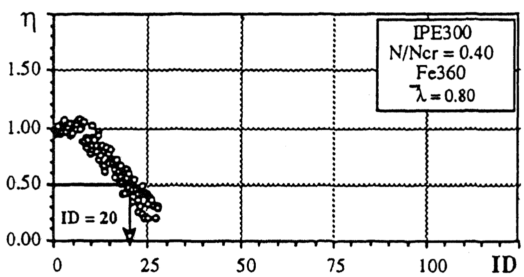

Figure 9. Influence of the level of axial load (N/Ncr = 0.40)

4.3 Influence of the b/t of the cross-section

Damage accumulation was sensitive to the b/t ratio of the cross-section (Figures 10 and 11). It was observed that the elements with more slender cross sections showed smaller ID values at failure due to local buckling.

Increasing the slenderness of the plates of the cross-section the ductility and the damage accumulation diminish. Because both profiles (IPE and HEA) belong to the same class of cross-sections of the Eurocode Nº 3 (1990) (class 1) if is guaranteed that the cross-section can form a plastic hinge with the rotation capacity required for plastic analysis.

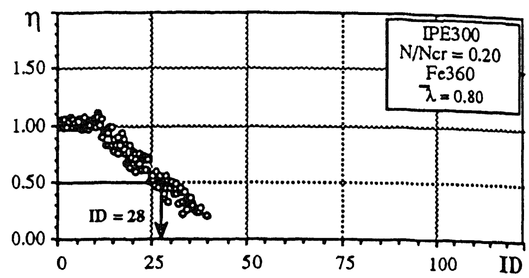

Figure 10. Influence of the slenderness of the cross-section (b/t = 7.0).

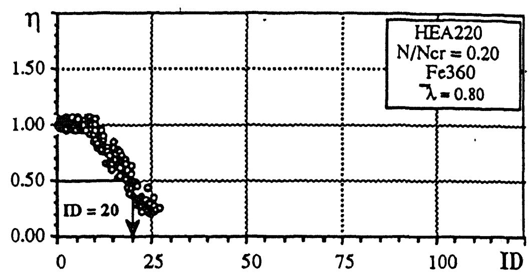

Figure 11. Influence of the slenderness of the cross-section (b/t = 10.0).

The values obtained for the damage index ID confirm that fact. The difference observed between the ID is related with the place occupied by the profiles inside the class. Whereas the IPE is clearly a plastic section, the HEA has the maximum slenderness allowed for that class, and therefore is more dependent on the local buckling phenomenon, resulting a lower damage index and a lower ductility.

4.4 Influence of the non-dimensional slenderness

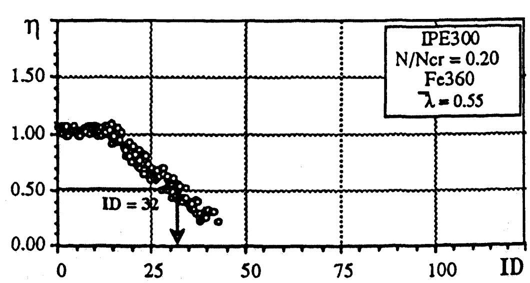

Figure 12. Influence of the non-dimensional slenderness ($\bar{\lambda}$ = 0.55).

The performed numerical simulations are not enough to achieve a conclusion, but it seems that damage accumulation is somewhat affected by non-dimensional slenderness. An increase of the non-dimensional slenderness implies a decrease of the damage accumulation and therefore a reduction of the ductile behaviour (Figures 12 and 13).

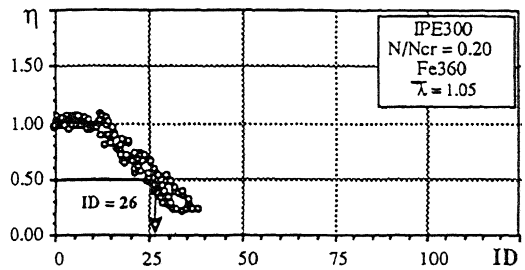

Figure 13. Influence of the non-dimensional slenderness ($\bar{\lambda}$ = 1.05).

4.5 Influence of the steel grade

The steel grade is directly related with the accumulation of plastic deformation. Elements with higher steel grades are less ductile and thus have smaller ID values when compared to the same elements with smaller steel grades (Figures 14 and 15).

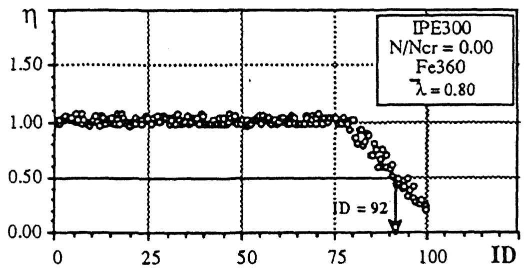

Figure 14. Influence of the steel grade (Fe360).

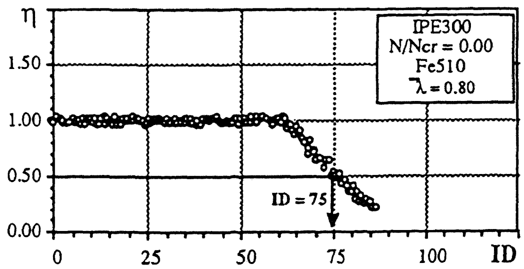

Figure 15. Influence of the steel grade (Fe510).

4.6 Interaction curves

The coefficients of variation achieved for all the studied cases are comprised between 5 and 9%, sustaining that the average damage index at failure can be considered "independent" of the time histories. The lower value obtained for that coefficient, allows that one numerical simulation or experimental test will be sufficient to obtain the average damage index at failure. For that purpose, the ECCS (1983) time history can be used because it is a standard loading history, easy to generate and to apply to numerical simulation or experimental tests. However it is preferable to obtain the damage index ID as an average of some indexes achieved in different numerical simulations.

Based on this numerical research, Figure 16 shows an example of a possible interaction diagram for beam columns.

The developed numerical studies suggest that the interaction diagrams should display curves in function of the type of cross-section. It will only be necessary to consider sections of class 1 and class 2 of Eurocode Nº 3 (1990) because they are the ones that can form a plastic hinge or develop a plastic moment and have a ductile behaviour. Cross sections belonging to class 3 and 4 are liable to local buckling phenomenon and for that reason cannot exhibit a ductile behaviour and large values of damage accumulation.

Damage index (ID) is then a measure of the ductile hysteretic capacity of the section. Beam columns built up of cross sections characterized by a large damage index exhibit certainly a better ductile hysteretic behaviour when compared with others built up of cross sections characterized by lower ID.

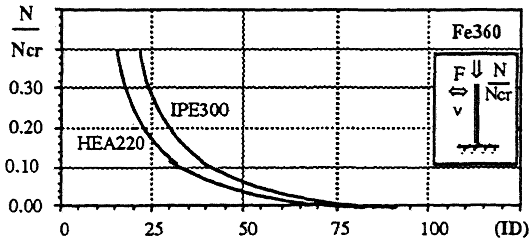

Figure 16. Interaction diagram for beam column.

Knowing the type of cross-section, the level of axial force and the steel grade it is possible to foresee the damage accumulation at the failure, and so choose the adequate cross-section.

5 DISCUSSION AND CONCLUSIONS

The model proposed for estimating the accumulated damage at failure through an average index of accumulated damage (ID), besides making possible the foresight of static or dynamic failure of a structural element based on the damage accumulated at failure, is also a good indicator of the ductility of the element. As ID is function of the sum of plastic deformation at failure, the curves shown in Figure 16 can be used whether for static or dynamic analysis.

Figures 17 and 18 show the results of the dynamic analysis of a beam column under seismic action until failure.

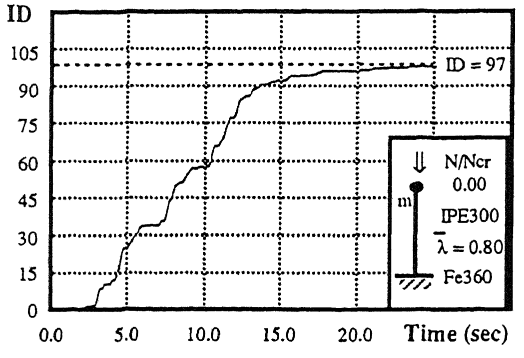

Figure 17. Evolution of the accumulated damage (dynamic analysis)

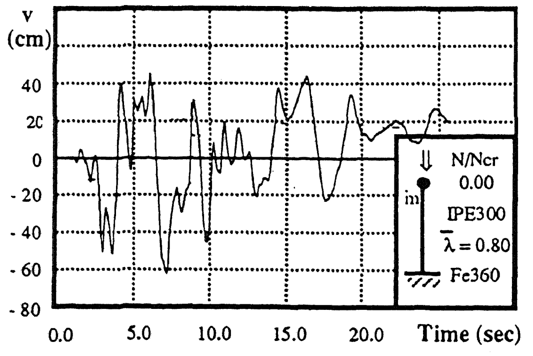

Figure 18. Evolution of the displacements (dynamic analysis)

The value reached for ID at failure is analogous to the value show in the interaction diagram. In fact, the larger the index that characterizes the failure, the larger will be the accumulated inelastic deformations and thus the larger the capacity of the beam column to dissipate energy by means of a ductile hysteretic process.

The elaboration of interaction diagrams analogous to the diagram of Figure 16 will be very useful for the choice of the cross-section to be used in a logical and economical seismic design of beam columns. These interaction diagrams allow the choice of the cross-section based in damage accumulation criteria and ductile behaviour of the beam column.

The proposed model can be advantageously used for the evaluation of behaviour coefficients (q factors) to be used in steel structural design.

ACKNOWLEDGEMENTS

The author would like to thank Prof. Giulio Ballio, Prof. João Azevedo and Prof. António Lamas for the guidance and help supplied during this research.

REFERENCES

Ballio, G. and Calado, L. 1986. Steel bent sections under cyclic loads - experimental and numerical approaches. *Costruzioni Metalliche*, nº 1.

Calado, L. 1989. *Caracterização do comportamento de estruturas metálicas sob acções sísmicas*. Phd thesis, Technical University of Lisbon, Portugal (in Portuguese)

Calado, L. and Azevedo, J. 1989. A model for predicting the failure of structural steel elements. *Journal of Constructional Steel Research*, vol.14 nº 1.

Castiglioni, C. 1987. Modellazione numerica di sezioni inflesse in acciaio soggette a carichi ciclici: influenza del legame costitutivo. *Costruzioni Metalliche*, nº 3.

ECCS - European Convention for Constructional Steelwork, 1983. Definition of a cyclic loading testing procedure for earthquake resistance evaluation of structural elements. Technical Committee 13 - Earthquake, *Document TC13-3-83*.

ECCS - European Convention for Constructional Steelwork, 1983. Recommended testing procedure for evaluating earthquake resistance of steel structural elements. Technical Committee 13 - Earthquake, *Document TC13-7-83*.

Eurocode Nº 3 1990. *Design of Steel Structures*. Part 1 General Rules and Rules for Building

Sih, G. and Madenci, E. 1983. Crack growth resistance characterized by the strain energy density function. *Engineering Fracture Mechanics*, vol.18.