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USE OF DAMAGE MECHANICS IN PERFORMANCE BASED DESIGN (PBD)

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ABSTRACT

Performance Based Design (PBD) is becoming the preferred method for the seismic design of structures. PBD is based on reaching performance objectives which are related to the seismic hazard and to the performance levels associated with the damage condition. Hence, reliable tools for capturing the evolution of this damage condition, to measure it and to locate it are required. Moreover, it is essential to establish, accurately, the relation between the predicted damage and the already known performance levels. Damage mechanics based finite element programs offer such possibilities. EFiCoS, a layered damage mechanics based finite element program is presented. This paper focuses on prediction of the response of normal strength (NSC) and high-strength (HSC) concrete columns subjected to cyclic flexural loading and various axial load levels. The damage predicted by EFiCoS was used to elaborate several damage charts expressed as a function of the relative displacement (drift) and the ductility level. This in turn made possible to establish relationship between the damage predicted by EFiCoS and the known performance levels. Important results obtained with the program such as extent of cracking, spalling of concrete, yielding of steel underline its capacity to predict in a refined way the complete behavior of the specimens studied in this research, making EFiCoS an important tool for PBD.

Introduction

Performance Based Design (PBD) is considered over the last few years as a clear and rational methodology for seismic design. In a deterministic approach, this design philosophy consists in carrying out a seismic design according to a performance objective previously selected. This performance objective is defined as a function of a desired performance level for a stated level of seismic hazard. In turn, the performance level is defined in terms of allowable damage, which can vary from simple cracking to damage state close to collapse of the structure.

However, the success of the PBD depends on the capacity to obtain a reliable

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prediction of damage and to develop realistic quantitative relationships between the damage and the different performance levels described by FEMA 273-306, SEAOC, ATC 40 or other equivalent committee. It is obvious that if the measurement of the performance is directly related to a level of damage, a PBD methodology should be supported on computer tools able to analyze, predict and quantify accurately the damage as well as its distribution. This is needed in order to reach a suitable and optimized design of elements according to the performance level previously selected.

In recent years, computer tools have been developed in order to make PBD a feasible design methodology. Normally, these programs use traditional inelastic time history analyses or pushover analyses in order to evaluate the progressive damage of the concrete structures during an earthquake by predicting the global response of the structure. However, local damage is usually not predicted. In general, it is possible to estimate total and local deformations, drift and total strength capacity of the structure. But for the expected damage, actually, it is only possible to obtain a qualitative estimation through relationships between some of these control variables mentioned previously and the levels of performance. Moreover, some calibration studies with experimental tests have made it possible to detect shortcomings in the predictions, particularly, for ductility with computed values that are very far from the measured values. Problems related with the prediction of the maximum resisting load have also been detected.

The objective of this research is to show how damage mechanics can be applied to solve some of the points of discussion associated to the deterministic approach of performance-based design and how it is possible to relate directly the predicted damage to the several performance levels described by PBD. Only structures controlled by flexure are considered in this paper. Based on damage mechanics principles, the finite elements software EFiCoS used in this research is presented hereafter. Material constitutive relationships used in EFiCoS are also presented. These include two new cyclic behavioral laws for steel implemented in the program, one of them developed in during the course of this study. A review of modeling techniques and appropriate meshing configurations are also presented. Comparison between predictions using EFiCoS and experimental data are illustrated. Some commentaries related to the localization and damage evolution are included. Finally, damage charts elaborated with the predicted damage by EFiCoS and expressed in terms of drift and deformation ductility are presented. These charts will permit to relate directly damage to several performance levels described by PBD methodology.

EFiCoS (Finite Elements by Superposed Layers)

EFiCoS is a damage mechanics-based finite-element program. A Bernouilli-type multilayered 2D beam element (Fig. 1) is used by default in the original version of the program (La Borderie, 1991). Using the uniaxial damage mechanical constitutive law proposed also by La Borderie (1991), EFiCoS tracks the progressive deterioration of concrete through the damage variables. These variables are evaluated at the center of each layer, at mid-span of the element (by default). Plane sections are assumed to remain plane and the strain at the point of evaluation in each layer is found by interpolation.

Nodal forces and displacements are evaluated by solving the traditional system of equations formed by the direct stiffness method. The analysis in EFiCoS can be conducted by force or by displacement. Analysis under monotonic, cyclic and seismic loads can be performed.

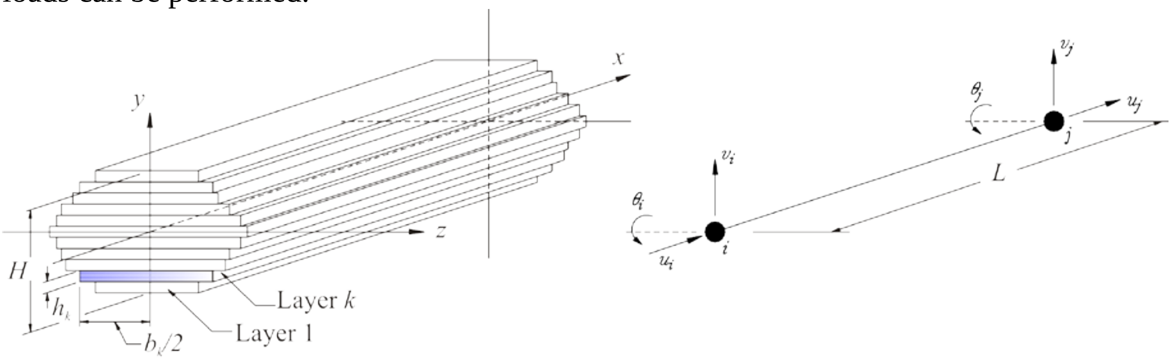

Figure 1. Bernoulli layered beam element in EFiCoS.

EFiCoS allows the step by step evaluation of the stiffness level in the structural system. This is possible by the local quantification of the progressive concrete damage, which causes the stiffness decrease and formation of residual deformations. The consideration of the unilateral effects related to the crack closure is also possible in EFiCoS. Thus, the program can consider the stiffness restoration of the elements. EFiCoS is capable of describing in a particularly precise way the behavior of the structure until the collapse of the system, detailing the progressive damage of the structure and the internal damage of its members.

Material Constitutive Laws

Concrete – One Continuum Damage Law

The uniaxial damage mechanics material constitutive law proposed par La Borderie (1991) was used in this research. In this law, which is set by default in EFiCoS, damage is considered as not recoverable and has an isotropic character. The La Borderie law (Fig. 2) allows EFiCoS to accurately model the cracking, stiffness variation, crack closure mechanism, and cyclic behavior. Also, with additional considerations, this law can be adapted to take into account the confinement effect in compression provided by transverse reinforcement. The law considers two damage indexes to indicate the level of damage accumulated in the material. The scalar variable D_1 is used for damage in tension and variable D_2 is used for damage in compression. Both, D_1 and D_2 vary from 0 for an undamaged material to 1 for a totally damaged material ($0 \leq D_i \leq 1$). These damage variables represent how much of an undamaged material remains from an initial unit volume after a certain loading history. Total strain is given by:

$$\varepsilon = \varepsilon_e + \varepsilon_p \tag{1}$$

where ε_e and ε_p represent the elastic and inelastic strains, respectively, with

$$\varepsilon_e = \frac{\sigma^+}{E_0(1-D_1)} + \frac{\sigma^-}{E_0(1-D_2)} \quad \text{and} \quad \varepsilon_p = \frac{\beta_1 D_1}{E_0(1-D_1)} f'(\sigma) + \frac{\beta_2 D_2}{E_0(1-D_2)} \tag{2}$$

where E_0 is the modulus of elasticity; β_1 and β_2 are material constants; σ^+ and σ^- are the positive and negative parts of the stress tensor, respectively, which are evaluated as follows:

$$\text{For} \quad \sigma > 0 \rightarrow \sigma^+ = \sigma, \quad \sigma^- = 0 \qquad \text{and} \qquad \sigma < 0 \rightarrow \sigma^+ = 0, \quad \sigma^- = \sigma \tag{3}$$

In Eq. 2, $f'(\sigma)$ represents the crack closure function, which provides a stiffness recovery procedure associated to the crack closure mechanism (Fig. 1) and is defined as:

$$\begin{cases} \sigma \geq 0 & f'(\sigma) = 1 \\ 0 \geq \sigma \geq -\sigma_f & f'(\sigma) = 1 + \sigma/\sigma_f \\ \sigma \leq -\sigma_f & f'(\sigma) = 0 \end{cases} \qquad \therefore \quad \sigma_f = \text{the crack closure stress} \tag{4}$$

The damage evolution is controlled by the energy release rate (Y_i). This variable is expressed for the damage evolution en tension (Y_1) and in compression (Y_2) as:

$$Y_1 = \frac{(\sigma^+)^2}{2E_o(1-D_1)^2} + \frac{\beta_1 \ f'(\sigma)}{E_o(1-D_1)^2} \qquad \text{and} \qquad Y_2 = \frac{(\sigma^-)^2}{2E_o(1-D_2)^2} + \frac{\beta_2 \ \sigma}{E_o(1-D_2)^2} \tag{5}$$

Finally, the damages variables in tension, D_1, and in compression, D_2, are defined as:

$$D_i = 1 - \frac{1}{1 + [A_i(Y_i - Y_{0i})]^{B_i}} \qquad if \ \ Y_i > Z_i, \quad Z_i = \max(Y_i, Y_{0i}) \tag{6}$$

where A_i, B_i, β_i and Y_{0i} are the parameters that control the monotonic or cyclic behaviour for this law. Légeron et al. (2005) proposed some parameter selection criteria in order to avoid the execution of experimental test required to evaluate them. One part of these criteria was revised in this research (Cardona, 2008) allowing improved prediction quality. This process consists in analyzing one cubic numerical element with EFiCoS and trying to fit the stress-strain curve obtained with a reference curve (Fig. 3). Thus, the parameters are found by trial and error. The Légeron and Paultre (2003) model was used to obtain the parameters that control the response in compression (A_2, B_2, β_2) for confined and unconfined concrete.

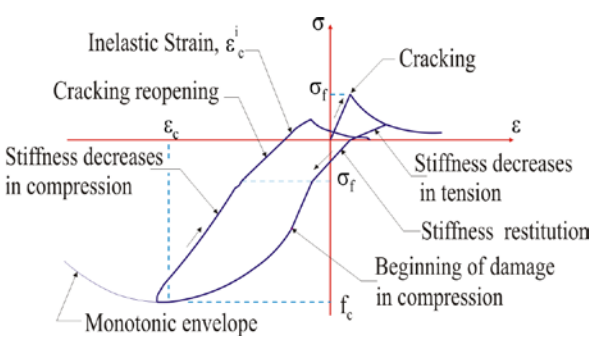

Figure 2. Concrete stress-strain (La Borderie ,1991).

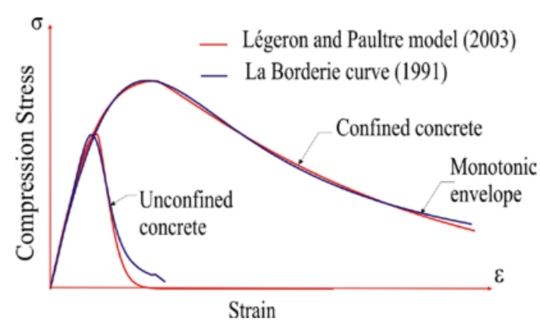

Figure 3. Compressive stress-strain curves. One example of fitting.

A review of the criterion developed by Légeron et al. (2005) was made concerning the preliminary selection of parameter β_2, which controls the evolution of plastic strain during cyclic test in compression. For the confined concrete, β_2 was usually found to be close to f_c''. For the unconfined concrete, where the post-peak behavior describes an important slope in the stress-strain curve, some new criteria were found more appropriate. It consists in taking $\beta_2 \approx 0.1 f'_c$ for the high performance concrete and $\beta_2 \approx 0.22 f'_c$ for the normal strength concrete. Others suggestions for the remaining concrete parameters are made by Cardona (2008).

Steel

EFiCoS uses a traditional cyclic behavioral law with linear kinematic hardening by default. In order to improve the representation of Bauschinger effect, two new steel laws were integrated in EFiCoS (Fig. 4). In this way, the Dodd and Restrepo-Posada (1995) law (Fig. 4a) and a simplification of this law made by Cardona (2008) (Fig. 4b) were included. This simplified law follows one bilinear envelope response. Also, considerations related to Bauschinger effects and reversal criteria, among others, were kept from Dodd and Restrepo-Posada (1995) formulation.

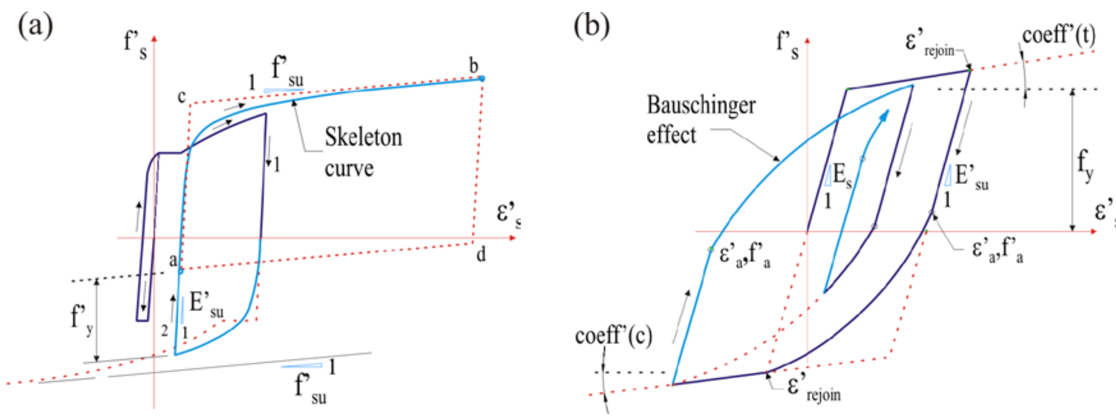

Figure 4. New cyclic behaviour laws for steel, integrated in EFiCoS: (a) Dodd and Restrepo-Posada (1985), (b) Cardona (2008).

Mesh Configuration for Modeling

For softening structures, it is well known that element size impacts the quality of response, specially the displacement capacity (Légeron et al., 2005). This is why finite elements mesh definition is a fundamental stage for EFiCoS. Some of the criteria concerning this subject, such as those exposed by La Borderie (1991) and Légeron et al. (2005) were revised to obtain the best performance possible of EFiCoS. The criterion adopted in this research for the finite elements mesh selection consists in dividing the column into five parts. Each one of these parts is supposed to be composed by two elements which share the same nodes at the ends. One of these elements is used to represent the unconfined area of the section while the other simulates the confined concrete (core). Moreover, the elements will not always have the same length and some restrictions related to layer thickness were removed. Additionally, the base of the tested columns was also modeled.

Given the type of structural elements studied in this research, which are those controlled by flexion, the damage concentration is expected to be close to the column base where the larger dissipation of energy is expected to take place associated to the development of a plastic hinge region. Hence, the new criterion proposed for the mesh in this research is to consider only the length of the first column element (L_a) as equal to the equivalent plastic hinge length (l_p). All the other elements were considered to have the same length equal to $L_b=(L-L_a)/4$, where L is the length between the end of the column and its contraflexure point equal to 2 m. However, for comparison purposes, additional analyses of columns were performed with elements having a standard length of 400 mm according to Légeron et al. (2005).

Several formulations for the evaluation of the plastic hinge length region were studied given the important impact of localization of deformation in the response of the elements. This study allowed selection of the most suitable mesh configuration for the finite element models. More than 250 models for the final stage of calibration were developed. These calibrations were made by comparing the response predicted by EFiCoS with that obtained by a large number of the tests realized at the University of Sherbrooke.

The first formulation for l_p considered was the one proposed by Paulay and Priestley (1992):

$$l_p = 0.08L + 0.022 d_b f_y \leq 0.044 d_b f_y \tag{7}$$

where d_b and f_y are the diameter and yield strength of the longitudinal bars, respectively.

The second formulation is a variation of the Eq. 7 which consists in applying the correction factor γ_{ZAHN} proposed by Zahn et al. (1986), which is related to the axial load ratio ($P_f / f'_c A_g$):

$$l_p = \gamma_{ZAHN} \cdot l_{p\,(eq.7)} \quad \text{with} \tag{8}$$

$$\gamma_{ZAHN} = 0.5 + 1.67 \frac{P_f}{f'_c A_g} \quad if \quad \frac{P_f}{f'_c A_g} < 0.3 \qquad \text{and} \qquad \gamma_{ZAHN} = 1.0 \quad if \quad \frac{P_f}{f'_c A_g} \geq 0.3 \tag{9}$$

The third formulation considered was the one proposed by Priestley (2003), which is similar to the expression described by Eq. 7:

$$l_p = 0.054L + 0.022 d_b f_y \tag{10}$$

Two criteria were use to estimate the best formulation for l_p: the monotonic envelope of cyclic force-tip displacement response and the hysteretic energy dissipated. From this study, it was established that, for columns subjected to low axial load ($P_f / f'_c A_g \leq 0.15$), the more appropriate formulation corresponds to Eq. 8. Equation 7 tends to overestimate the plastic hinge length region in this case. For columns subjected axial load ratio $P_f / f'_c A_g > 0.15$, it was found that the Eq. 7 was more appropriate for square columns, giving l_p values close to the standard length proposed by Légeron et al. (2005).

Also, for circular columns, the more appropriate expression to evaluate l_p corresponds to Priestley's expression (Eq. 10), which gives l_p values close to the cross section diameter.

Comparison of Predictions with Experimental Tests Results

Test results obtained from 21 large-scale columns tested at the University of Sherbrooke were used in order to calibrate EFiCoS. Among the specimens, 18 columns were made of high-strength concrete (HSC): 12 with a rectangular cross section (Paultre et al., 2001)(Légeron et al., 2005) and 6 with a circular cross section (Paultre et al., 2007). The rest of specimens were made of normal-strength concrete (NSC) with a circular cross section (Osorio et al., 2008). All rectangular columns specimens had a 305x305 mm-cross section while the diameter for all circular columns was 305 mm. These columns represent a first floor column with the point of contra-flexure at approximately mid-height in a frame building. The specimens were subjected to cyclic flexural loading and various axial load ratios (P_f / $f'_c A_g$ =0.15 to 0.52). Different steel strengths as well as longitudinal and transverse reinforcement ratios were used. Concrete strengths vary from 30 to 120 MPa. Longitudinal steel strength varied between 425 and 533 MPa, and between 391 and 825 MPa for the transverse reinforcement. Spacing of transverse reinforcement varied from 37 to 130 mm resulting in volumetric ratio of confinement steel between 1.10% and 4.26%. More details concerning the description of the specimens as well as the material mechanical properties are shown in Cardona (2008).

Two examples of predictions made with EFiCoS are presented in Fig. 5. The prediction in Fig. 5(a) corresponds to a NSC column while the one shown in Fig. 5(b) corresponds to a HSC column. Excellent predictions concerning the force-tip displacement response as well as damage quantification, damage distribution and its evolution were obtained. As measured during the experimental test, the largest damage concentration predicted by EFiCoS was in the plastic hinge region close to the column base. Moreover, above the plastic hinge region, the predicted damage in compression decreases rapidly while the damage in tension (cracking) reduces more slowly. The onset of spalling and cracking (for the unconfined concrete) was predicted for an average strain level near to -0.003 and +0.0002, respectively, for HSC columns. For NSC column, the average strain corresponds to -0.002 and +0.0001, respectively. This average strain is related to a compression damage level as low as 0.1. Important cover deterioration was detected for a compression strain of about -0.005 which corresponds to an average compression damage level of 0.8. These results are similar to those found experimentally. First yielding of the specimens was also well predicted.

Damage Charts

Damage charts were elaborated for obtaining a direct link between the predicted damage by EFiCoS and the traditional performance levels. Theses charts were developed based on the monitored damage at the exterior layers of the unconfined (cover) and confined (core) concrete regions given that the elements are controlled by flexure. For theses charts, predicted damage by EFiCoS at the plastic hinge region was related with the drift ($\delta_\theta = \Delta/L$, Δ= tip displacement) or the displacement ductility (μ).

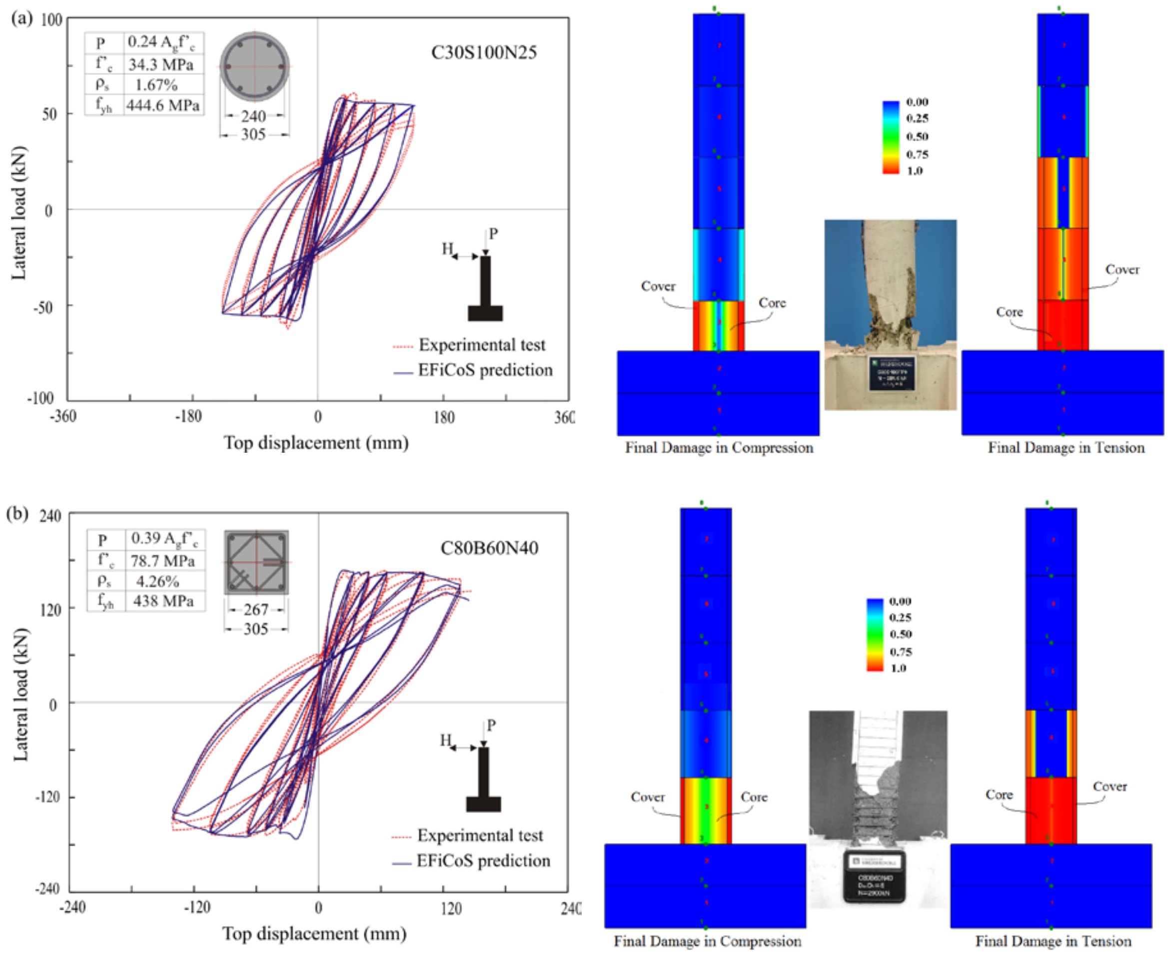

Figure 5. Force-displacement response and final damage prediction made by EFiCoS. (a) Column C30S100N25 (b) Column C80B60N40.

Only the compression damage was taken as reference for the elaboration of damage charts because the impact of damage in compression is more significant with respect to the capacity of the section compared to the damage in tension that increases rapidly. The onset of damage in tension is related to *Operational* level of PBD, with drift values as low as 0.2% (NSC) and 0.4% (HSC) with an average damage index of 0.6. Then, it jumps suddenly to 0.9 for drift values near to 0.4% (NSC) and near to 0.6% (HSC) which is associated with the *Life Safety* level. In order to establish the level of ductility, the idealized tip displacement was used as reference for yielding of the system.

For a better comprehension of these charts, graphic representations are shown in Fig. 6. Some conclusions can be obtained from these charts:

- Damage evolution in NSC is more gradual compared to the HSC. Also, for HSC columns, the damage presented sudden variations, especially for the unconfined concrete (UC) which deteriorates more rapidly and less gradually than the confined concrete (CC). In general, the damage associated to the imminent failure or total crushing (D ≈1.0) is predicted in the UC for a drift value of 2.0%

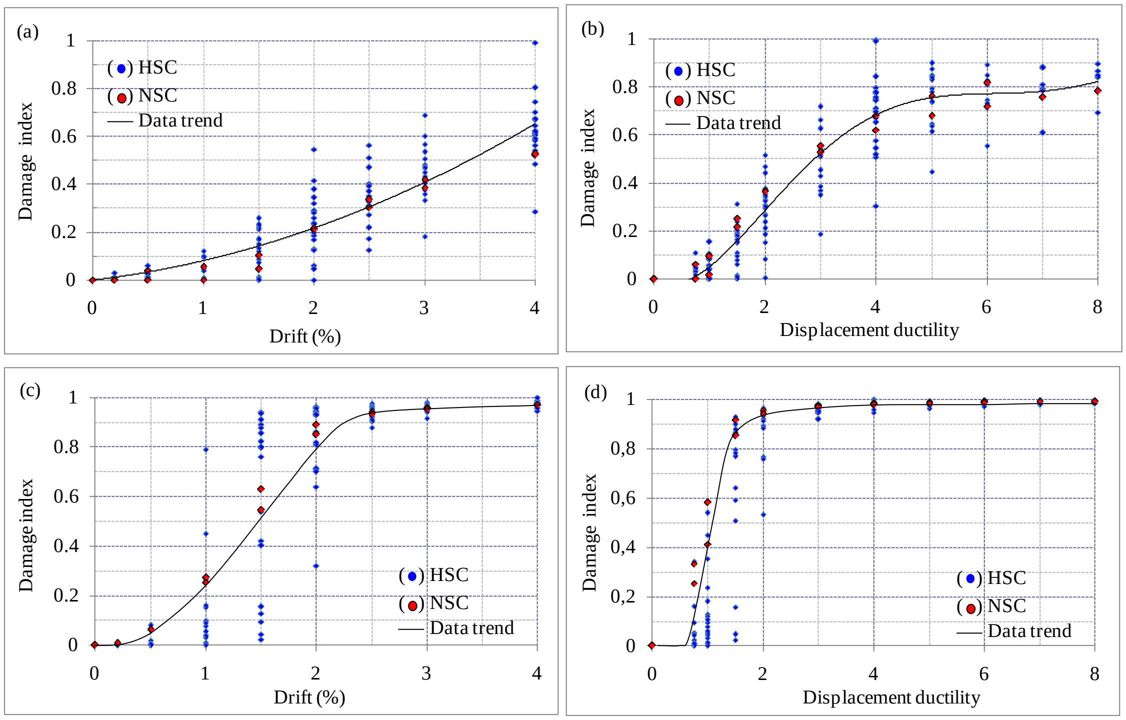

Figure 6. Graphic representation for the damage charts. (a) and (b) Confined concrete; (c) and (d) Unconfined concrete.

(*Collapse Prevention* level) and a ductility value of 2. Concerning the CC, an important deterioration for a drift of 4% (*Collapse* level) and a ductility value of 3 is obtained;

- Damage evolution and its intensity are also related to the axial load ratio. Thus, the onset of drastic damage variations and the damage increase are reported earlier in columns with higher axial load ratio. In those cases, damage in UC is important for drift values close to 1.0% (*Life Safety* level) or a ductility value of 1.0 for NSC and 1.5% (*Life Safety* level) for HSC. For the CC, damage is important for a drift value between 2% (*Collapse Prevention* level) and 3% (*Collapse* level) or a ductility of 3. For columns with lower load ratio, damage in UC becomes important for a drift value close to 2% and 3% for the CC. At the instant of the system yielding (μ = 1.0), the average damage in CC is very low (<0.1). Further information can be found in Cardona (2008).

Conclusions

A successful application of damage mechanics in PBD methods was presented. The software EFiCoS was demonstrated to be capable of evaluating accurately, the parameters associated to the performance such as strains, displacements, cyclic response, damage quantification, evolution and distribution. The introduction of new behavioral laws for steel in EFiCoS, the review of modeling criteria and the selection of parameters based on parametrical studies, among others, have allowed the improvement of

EFiCoS's performance. The elaboration of damage charts, as a function of drift and displacement ductility related to the damage predicted by EFiCoS, permitted to obtain a direct relationship between the damage and the different performance levels described for the PBD. As a result, it is possible to demonstrate that, for the elements studied in this research which are controlled by flexure, damage mechanics can contribute to the application of a reliable deterministic approach for PBD.

Acknowledgments

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