AN OBJECTIVE SEISMIC DAMAGE INDEX FOR THE EVALUATION OF THE PERFORMACE OF RC BUILDINGS

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ABSTRACT :

In modern earthquake-resistant design codes, structural elements have a non-linear behavior during an earthquake, similar to what is considered in the design process, which implies that elements are damaged and it is interesting for designers to be able to estimate structure expected global damage and correlate it to design ductility, as well as to ductility demand. Damage indexes calculated when applying the finite element method have values not reflecting deterioration in case of buildings designed for low ductility; this feature being contrary to damage indexes calculated for ductile buildings. Therefore, an objective damage index is proposed, based on ductility and elastic and ultimate stiffness values, independent to selected structural typology. The procedure is illustrated by means of an index assessment from damage caused to three buildings, two designed for low ductility (buildings with waffle slabs and framed buildings with wide beams) and a framed building, designed for high ductility. For the three buildings static non-linear response has been determined by means of a force-based procedure. Results obtained demonstrate that the proposed objective damage index provides values which describe properly the damage suffered by the buildings at the instant of collapse.

KEYWORDS: Damage Index, Seismic Damage, Pushover Analysis, Limit States, RC Buildings.

1. INTRODUCTION

In current earthquake-resistant design elastic procedures are applying response-reduction factors to reduce elastic response and convert it into equivalent elastic-plastic response. This approach implicitly accepts the fact that structures have a plastic deformation capacity without losing their stability. But the concept of ductility also implies that structures reach a certain state of damage when subject to earthquakes. It is useful for the structural designer to assess the magnitude of this damage, and co-relate it to structural ductility and ductility demands. Vielma *et al*. (2007).

Damage Indexes have received special attention during the past two decades, mainly based in the possibility of correlating Damage Indexes to Limit States of performance-based design. For Kunnath (2006), in performance-based design procedures, the process of transforming calculated demands into demands which suitably quantify the behavior of buildings is a questionable part of the global procedure. For this reason, it is necessary to consider indexes which account for an objective way seismic damage in buildings.

Global seismic damage indexes provide a measure of structural deterioration. They are calculated from numerical simulation of structures with lateral static or dynamic forces representing seismic forces. Depending on load type, various damage indexes have been formulated. These damage indexes include some of the main characteristics of non-linear response (static or dynamic) of structures.

Some indexes measure overall seismic damage of a structure from its local damage, i.e., contribution of cumulative damage in structural elements in a given instant to the structure being subject to a seismic demand. Among indexes which have served as baseline for many researches, the one proposed by Park and Ang (1985) can be mentioned, which can determine damage in an element, based on non-linear dynamic response by the following expression:

$$DI_e = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y}\int dE_h \tag{1.1}$$

Where δ_m is the maximum displacement, δ_u is the ultimate displacement, β is a parameter adjusted depending on materials and structural type, P_y is the yield strength and $\int dE_h$ is dissipated hysteretic energy. This damage index is valid at a local level, for an element; however, it is possible to apply this index in calculation of values for a specific structural level, or for the whole structure.

For non-linear analysis, due to static horizontal loads, to consider damage indexes which incorporate stiffness degradation is useful. Skærbæk *et al*.(1998) propose the following damage index:

$$DI_e = 1 - \sqrt{\frac{K_i}{K_{i-1}}} \tag{1.2}$$

Where DI_e is the damage index for beam or column, K_i is the current tangent stiffness and K_{i-1} is the initial tangent stiffness.

Period degradation provides a measure of stiffness degradation. For this reason, Hori and Inoue (2002) have formulated an expression to calculate period degradation based on design ductility as follows:

$$T_\mu = 2\pi \sqrt{\frac{\mu}{\alpha_y}} T_0 \tag{1.3}$$

Where T_μ is the period on collapse state, μ is the design ductility, α_y is a stiffness degradation dependent coefficient and T_0 is the structure elastic period. Another damage index based on stiffness degradation is proposed by Gupta *et al*. (2001). They have formulated an expression based on the relationship between ultimate and yielding displacements, equivalent to ultimate and yielding stiffness. This formulation also includes a design ductility value according to:

$$DI = \frac{{x_{max}}/{z_{00}} - 1}{\mu - 1} \tag{1.4}$$

Damage indexes, especially those calculated with stiffness relationship, have the shortcoming of producing consistent results in case of ductile behavior structures. However, in the case of structures designed to be less ductile, i.e. framed buildings with wide beams or waffle slab buildings, damage indexes do not describe objectively the overall state of damage when response is close to collapse threshold. In order to overcome this drawback, this article develops an objective seismic damage index, independent of structural typology, formulated as a function depending on stiffness relationship and maximum ductility values, calculated directly from capacity curve of buildings. Numerical examples in the application of this index are presented, which consist in three RC buildings designed for different values of ductility, typified on the NCSE-02 Spanish seismic code, and characterized by the corresponding performance point calculated by the N2 method Fajfar (2002).

2 FORMULATION

The above mentioned indexes have been developed in order to quantify global damage in ductile structures. However, at the moment of studying non-linear response of restricted-ductility structures, it can be possible to observe that damage index values corresponding to the collapse threshold are lower than those corresponding to ductile structures, Vielma *et al*. (2007). This shortcoming does not allow the use of the above referred indexes in order to carry out an objective characterization of damage in restricted-ductility buildings.

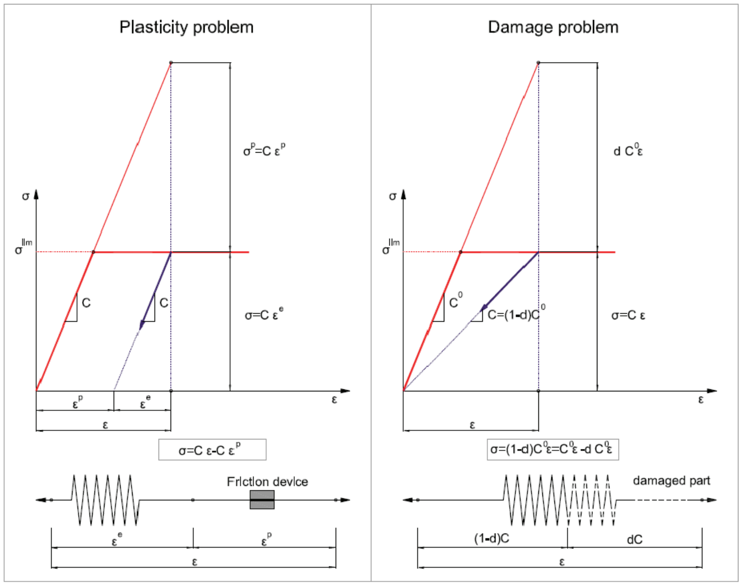

Fig. 1. Schemes of damage and plastic behaviors

Starting from the assumed hypothesis that non-linear behavior of structure follows the principles of Mechanical Damage Theory from Oliver *et al.* (1990), the following analysis is made based on continuum mechanics; this theory fulfills fundamental thermodynamic principles. Not all materials used for structural purposes follow a behavior which can be assimilated by damage (degradation/loss of stiffness); instead, their behavior follows the Plasticity Theory (development of irreversible deformations), see Figure 1. Other materials combine both behaviors and have stiffness loss with irreversible deformations, which is the case of RC structures.

It is necessary to observe unload branch in Figure 1, to determine whether damage or plastic behavior has occurred. It is accepted that damage has occurred if an unload branch passes through the origin; on the other hand, if the unload branch is parallel to the load branch, behavior corresponds to plasticity. Reinforced concrete has a combined behavior (plasticity and damage) but its main feature corresponds to degradation, Oller (1991). This last affirmation can be validated by laboratory tests or by numerical simulations using the Simple Substances Mixing Theory, Car *et al.* (2000, 2001).

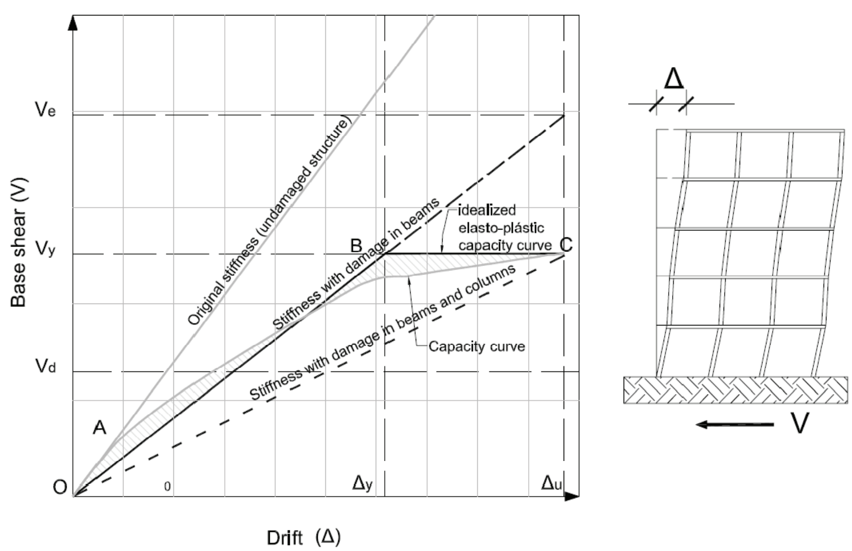

Fig. 2. Determination of initial stiffness from capacity curve

With the objective of describing structural degradation under seismic loads starting from a few non-linear characteristics, the following procedure has been proposed. This feature allows the procedure in a simple way and its application is quick and efficient when evaluating seismic behavior. A static non-linear analysis (push-over analysis) is applied to a structure. Calculated roof displacements Δ are plotted vs. base shear V. The resultant curve, has an initial slope which corresponds to initial stiffness K_0, see Figure 2.

For a specific ductility value postulated in seismic code, and if yield base shear is known, damage at point C, where maximum damage is reached, can be evaluated according to mechanical continuum-damage:

$$D_c = 1 - \frac{K_c}{K_0} = 1 - \frac{V_y/\Delta_u}{V_y/\Delta_y} = 1 - \frac{1}{\mu} = \frac{1-\mu}{\mu} \tag{2.1}$$

According to Equation 2.1, maximum damage is developed at collapse point C; implying that damage depends only on adopted structural ductility; therefore, it is possible to affirm that:

$$\begin{gathered} Ductile\ structure\ \mu = 4; D_c = 0{,}75 \\ Fragile\ structure\ \mu = 2; D_c = 0{,}50 \end{gathered}$$

In other words, ductile structures have a damage value greater than the damage value reached by fragile structures at collapse point. However, this procedure used for calculating damage values allows for some misunderstandings: the structural designer might interpret that ductile structures suffer greater damage than those corresponding to fragile ones at Collapse Limit State. This shortcoming implies that damage index must be reformulated in order to avoid dependence on structure fragility. This is possible if damage index is normalized with respect to maximum damage which can occur in the structure. Thus, objective damage index $0 \leq D^P_{obj} \leq 1$ achieved by a structure at any point P is defined as:

$$D^P_{obj} = \frac{D_P}{D_C} = D_P \frac{1-\mu}{\mu} = \frac{(1 - K_P/K_0)\mu}{\mu - 1} \tag{2.2}$$

For example, P might be the performance point resulting from intersection between inelastic spectrum (demand) and capacity curve (obtained from pushover analysis). Under these conditions, Equation 2.2 provides the maximum damage that the structure would reach subject to earthquake prescribed by code.

3. NUMERICAL EXAMPLES

In this section, objective damage index is applied when evaluating non-linear behavior of three buildings designed according to NCSE-02 Spanish seismic code for different ductility levels. First building is designed with a ductility value of two; it is a waffle slab building. Second one is a framed building with wide beams designed with a ductility of two; and third one is a framed moment-resisting building, designed with a ductility value of four, see Figure 3. In order to calculate non-linear response of these buildings, a force-based procedure is applied. Selected pattern of forces correspond to an inverted triangle; this shape is recommended only when buildings have plan and elevation regularities.

Buildings are modeled as 2D frames and equivalent frame in the case of the waffle slab building. Discretization was performed according to different applied confinements, thus it is necessary to define elements for the confined zone near nodes and in the middle of beams and columns. Confinement zone longitudes depend on dimensions of beam or column sections, span longitude, and height between floors or reinforcement steel diameter. Sectional discretization is also applied. This consists in splitting sections in strips parallel to the main flexure axis. Reinforcement characteristics are incorporated by means of applying Mixing Theory, Mata et al. (2007). The effect of different confinements provided by longitudinal and transversal reinforcement is incorporated with modification of concrete strength according to the Mander *et al.* (1988) procedure.

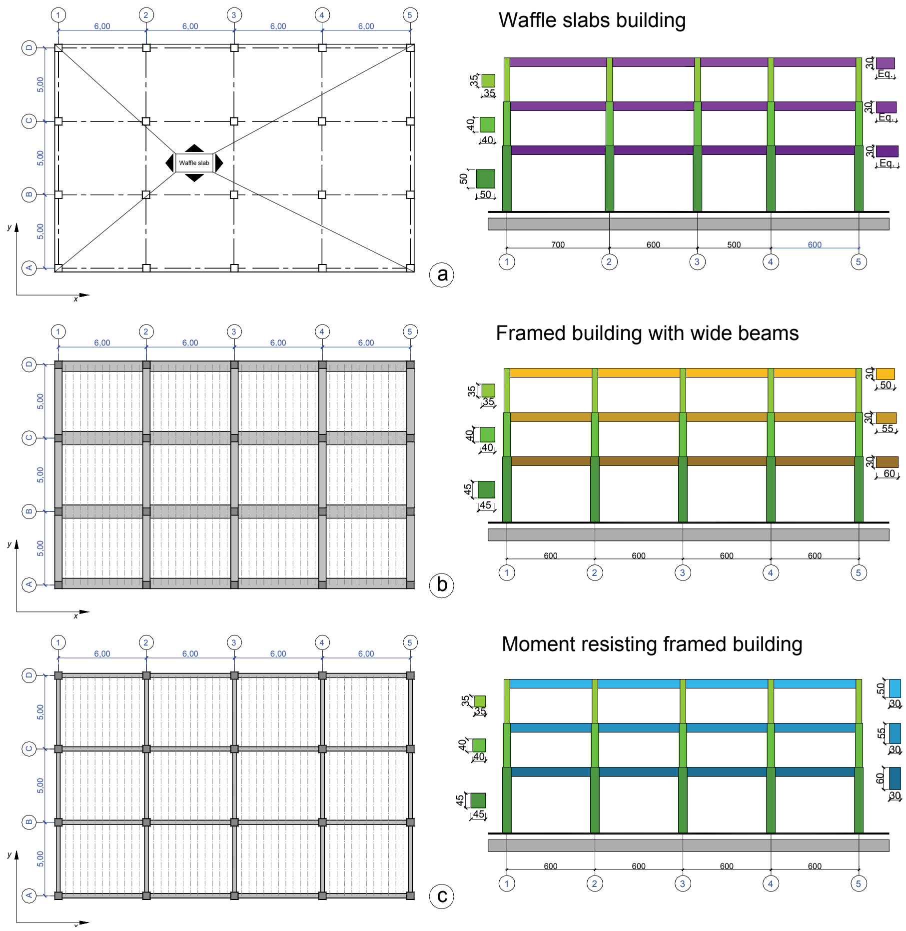

Fig. 3. Plan and elevation views of the three studied buildings

Capacity curves are the result of static non-linear analysis of frames or equivalent frame, depending on the studied case. These capacity curve plots normalized base shear (V/W) vs. normalized roof displacement $(\Delta/H)$of the whole structure as a model of a multiple degree of freedom (MDOF) system. In order to determine its performance point, it is necessary to intersect capacity curve with demand, typified on seismic codes by inelastic spectrum. Therefore, it is necessary to convert non-linear MDOF into response of equivalent SDOF by means of dynamic characteristics of first mode. Roof displacements are converted into pseudo-displacements according to Equation 3.1:

$$S_d = \frac{\delta_c}{MPF} \tag{3.1}$$

Where S_d is pseudo-displacement, δ_c is roof displacement and MPF is the modal participation factor, obtained from dynamic characteristics of frames.

Performance point represents the point with maximum lateral displacement of equivalent SDOF system, produced for seismic demand. In this article performance points are calculated through the N2 procedure (Fajfar, 2002)

The intersection of the projection of the elastic branch with elastic demand spectrum provides displacement of performance point. An alternative procedure is to determine performance point by means of intersection of inelastic branch of idealized capacity spectrum with inelastic demand spectrum, calculated from elastic demand spectrum reduced by a response reduction factor Rμ, defined as:

$$R_\mu = \begin{cases} (\mu - 1)\dfrac{T}{T_C} + 1 & when\ T \leq T_c \\ \mu & when\ T > T_c \end{cases} \tag{3.2}$$

In Equation 3.2 T is the first mode period, μ is the design ductility and TC is the corner period of elastic design spectrum, which limits branch of constant acceleration with the decreasing one. The performance points of the three cases studied are shown in Figure 4.

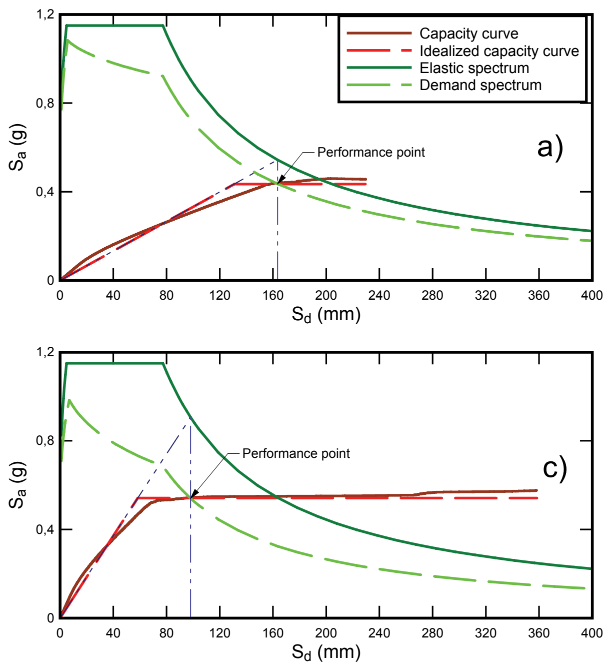

Fig. 4. Performance point of the a) waffle slabs building, b) framed building with wide beams and c) framed moment-resisting building

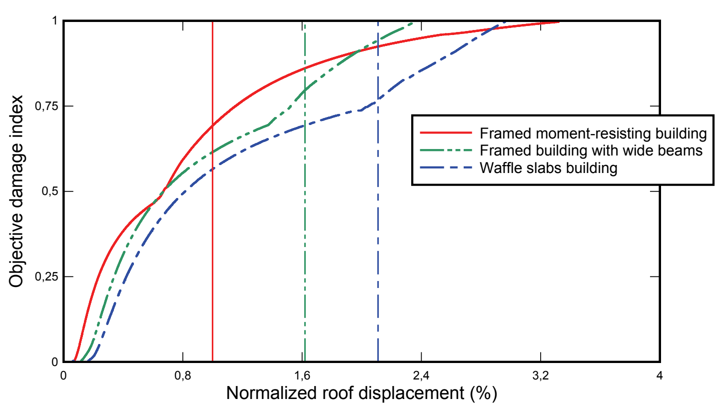

Fig. 5. Evolution of the objective damage index of the case studied

Objective damage index is calculated by applying Equation 2.2 and it is plotted vs. normalized roof drift. Figure 5 shows the evolution of the damage index of buildings under study; a special characteristic of these curves is a smooth approximation of ductile building curve to the collapse threshold; on the other hand, curves of two restricted-ductility buildings have a strong slope near to collapse threshold. This feature highlights that restricted-ductility has an abrupt collapse in contrast to ductile behavior of moment-resisting framed buildings.

Finally, the values of damage indexes calculated for performance point displacements are shown in Table 1.

Table 1. Values of the objective damage index of the case studied

Building	Objective damage index
Waffle slabs building	0,79
Framed building with wide beams	0,80
Framed moment-resisting building	0,69

4. CONCLUSIONS

According to calculated non-linear response of reinforced concrete buildings, conventional damage index values depend on structural typology. Thus, for RC restricted ductility buildings, conventional damage indexes do not provide results comparable to those calculated by applying finite element method.

Previously performed structural analysis allows for an objective assessment of structural damage in a simple manner. Specifically, the use of equation 2.2 allows for obtaining index values very close to those resulting from more expensive calculation procedures. Thus, it is possible to know the level of global structural damage in a specific point, for example, the performance point obtained by means of intersection of demand curve, or demand spectrum, with capacity curve of structure.

Objective damage index, which incorporates stiffness degradation and maximum value of structural ductility, enables achievement of appropriate values of global structural damage, regardless of analyzed structure typology.

Moment-resisting framed buildings have an acceptable value of damage at performance point and their behavior remain ductile; this non-linear response feature exceeded expected design values.

Among the three cases studied, it is possible to affirm that in the case of framed buildings with wide beams and waffle slab buildings, exists the possibility of anticipating a high value of damage index corresponding to performance point. Also, these buildings have insufficient structural ductility when compared to Spanish seismic code requirements.

A new procedure for calculating response of non-linear static-controlled forces is proposed. This solves the problem of singularity at the collapse threshold by implementing a calculation iterative process which considers obtaining a certain damage index as a convergence standard.

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