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Numerical calibration of damage indices

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ABSTRACT

In this study a numerical calibration procedure is proposed; while its application in some of the most widely accepted damage indices (DIs) used for quantifying the extent of damage in reinforced concrete structures is presented. In particular, without loss of generality of the applicability of the proposed procedure, the Park and Ang local damage index, its modified variant presented by Kunnath, Reinhorn and Lobo; the Chung, Meyer and Shinozuka local damage index; along with the maximum and final softening damage indices proposed by DiPasquale and Çakmak, are calibrated on the basis of the width of crack openings. The estimation of the crack width is performed by means of detailed modelling with hexahedral finite elements for the concrete and rod elements for the steel reinforcement; while due to the computing demands the databank of values for the damage indices under investigation is defined based on coarse models with beam–column elements. These two steps of the proposed procedure are based on the incremental dynamic analysis. Next, the statistical characteristics of the DIs are computed by means of horizontal statistics in conjunction with the maximum likelihood function method and an optimization algorithm.

1. Introduction

Performance-Based Design (PBD) in earthquake engineering is the state-of-the-art framework for future developments in structural design. PBD in engineering practice requires the definition of clearly determined levels of damage achieved for different seismic intensity levels. In order to implement this design framework, appropriate models for assessing the structural damage, within the context of a random seismic environment, are required. The idea of describing the state of damage of the structure by one number on a defined scale in the form of a damage index (DI) is attractive because of its simplicity. So far, a number of researchers have studied various DIs for reinforced concrete or steel structures [1–4].

Damage indices can be broadly divided into two classes [5]: (a) strength-based DIs and (b) response-based DIs. Strength-based DIs do not require finite element (FE) analysis [6,7]; however, they must be calibrated against observed damages using a large experimental database. Although the seismic performance of the structures is commonly related to their capacity to undergo inelastic deformations, experimental studies have shown that ductility as well as alternative measures of the structural performance in case of seismic loading, based on the theory of low-cycle fatigue, do not seem to provide a satisfactory index of the seismic damage [8]. These test results are consistent with the notion that failure of brittle systems is caused by excessive deformation, while the failure of ideal ductile systems is initiated by repeated inelastic deformations. The damage indices, used for structural systems that are neither ideal brittle nor ideal ductile, need to account for the damage effect of both excessive and repeated inelastic deformations [9]. Thus, there is a need for more general and reliable indices able to characterize the performance of the structures. The response-based DIs can be divided into three groups according to the quantity that the index accounts for [2]: (a) maximum deformation indices [10–14]; (b) cumulative damage indices [8,15]; and (c) combination of maximum deformation and cumulative damage indices. In this work some of the most widely accepted damage indices accounting for both maximum deformation and cumulative damage are considered. In particular, the Park and Ang [9] local damage index, its modified variant proposed by Kunnath et al. [16], the Chung et al. [17,18] local damage index, along with the maximum and final softening damage indices proposed by DiPasquale and Çakmak [19,20] are studied in this work.

In this study a numerical calibration procedure is proposed and it is applied to the above mentioned damage indices that are used to quantify the extent of damage in reinforced concrete structures. Furthermore, the width of the crack openings is used as the basis for implementing the calibration procedure. The estimation of the crack width is performed using detailed numerical modelling with hexahedral finite elements for the concrete and rod elements for the steel reinforcement; while due to the computing demands

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the databank of values for the damage indices under investigation is defined based on coarse models with beam–column elements. Both steps of the proposed procedure are based on the incremental dynamic analysis (IDA) [21]. Next, the statistical characteristics of the DIs are computed by means of horizontal statistics in conjunction with the maximum likelihood function method and an optimization algorithm. The proposed numerical calibration procedure is not limited neither to the DIs considered, nor to the quantity used as a basis (crack width), or the material (reinforced concrete). For the purposes of the current study two test examples are considered, one plane frame test example and one 3D framed structure.

2. Damage indices

In this section a short description of the five DIs considered in this study is provided.

2.1. Park and Ang local damage index

The Park and Ang [9] damage model accounts for the damage due to maximum inelastic deformation, as well as due to the cyclic history of the deformations. This damage index, which was modified later by Park et al. [22] and Kunnath et al. [16], is composed by two parts, namely the scaled values of the ductility and the dissipated energy of the structural element during the seismic shaking. Therefore, according to this damage index, the structural damage is expressed as a linear combination of the damage caused by excessive deformation and the damage attributed to the repeated cyclic loading effect:

$$DI_{PA} = \frac{\delta_M}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \tag{1}$$

where δ_M is the maximum deformation obtained under the earthquake loading; δ_u is the ultimate deformation achieved under a monotonic loading; Q_y is the calculated yield strength; E is the incremental absorbed hysteretic energy (all calculated at the element level); while β is a non-negative parameter calibrated from experiments ($\beta = 0.25$, as suggested by Park [23]). In Eq. (1) the scaled ductility part is defined as the ratio of the maximum experienced deformation demand to the ultimate deformation, while the dissipated energy part is defined as the scaled ratio of the absorbed hysteretic energy with respect to $\beta/(Q_y \delta_u)$. The Park and Ang damage index has been calibrated with observed structural damages on nine RC buildings [22], where it was noted that values in the range of 0.20–0.30 correspond to slight damage, 0.50–0.60 correspond to minor-severe damage, while values greater than 1.0 signify complete collapse. Although the value of DI_{PA} may exceed unity, the structural failure is assumed to occur when the value of DI_{PA} ranges from 0.8 to 1.0. Under elastic response the value of DI_{PA} should theoretically be zero; however, in practice the values of DI_{PA} in the elastic range are usually close to zero and not necessarily equal to zero.

2.2. Kunnath, Reinhorn and Lobo local damage index

This damage index as proposed by Kunnath et al. [16] is a modification of DI_{PA}. For the case of damage of the structural element end-section, the following modification to the Park and Ang model was introduced:

$$DI_{KRL} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} \int dE \tag{2}$$

where deformation was replaced by the rotation; while θ_m is the maximum rotation attained during the loading history; θ_u is the ultimate rotation capacity of the critical region; θ_r is the recoverable rotation after unloading; M_y is the yield moment; and E is the dissipated energy in the critical region. The element damage is selected as the largest damage index of the end critical region.

2.3. Chung, Meyer and Shinozuka local damage index

Chung et al. [17,24] proposed a damage index that reflects the effect of the loading history, and considers the difference of the flexural response of the members to positive and negative moments. The effect of the loading history is taken into account via a parameter which includes the change in stiffness and sustained bending moment up to the calculation cycle. The damage index is evaluated at the section level and relates to the flexural response, therefore it is based on the evaluation of the curvature φ, while it takes into account the fact that reinforced concrete members typically respond differently to positive and negative loadings:

$$DI_{CMS} = \sum_{i=1}^{n_{st}} \left(\alpha_i^+ \frac{n_i^+}{N_i^+} + \alpha_i^- \frac{n_i^-}{N_i^-} \right) \tag{3}$$

where N_i is the number of cycles causing failure at curvature φ_i, n_i is the number of actually applied loading cycles at curvature φ_i, α_i is the damage modifier while +/− depicts the loading direction. The damage modifiers α_i are defined as a function of the number of loading cycles and the previous loading history. Calibration for DI_{CMS} was performed by Chung et al. [17] comparing the visible damages of one-bay one-storey frames with the computed DI_{CMS} values at the corresponding loading steps. According to the study by Chung et al. [17], values in the range of 0.0–0.2 correspond to invisible cracking, 0.2–0.5 correspond to visible cracking, and 0.5–1.0 correspond to concrete spalling while values greater than 1.0 indicate concrete crushing. According to Chung et al. [17], from the engineering point of view, in order to prevent total collapse, DI_{CMS} should be limited in all members to the maximum value of 0.5; while, for frequent seismic excitations DI_{CMS} should be limited to 0.2 which is the maximum acceptable damage to maintain the structure's serviceability with minor repair needs.

2.4. Maximum and final softening global damage indices

According to Saiidi and Sozen [12] the seismic damage to reinforced concrete structures depends mostly on the maximum strain that is observed during the seismic event, while the particular sequence (or path) of the loading is not very important in determining the damage level. Thus, a rational notation is that the maximum softening damage index, which depends on the combined effects of the stiffness degradation and the local nonlinearities, can be used as a damage index for reinforced concrete structures. To this end, DiPasquale and Çakmak [20] developed two damage indices based on: (i) the evolution of the natural period of a time-varying linear system equivalent to the actual nonlinear system for a series of non-overlapping time windows (maximum softening) and (ii) the final (post-earthquake) state of the building (final softening). These two global damage indices depend on the combined effect of stiffness degradation and plastic deformation. DiPasquale and Çakmak [20] used the change in the fundamental period of the structure as a measure of the stiffness degradation caused by the seismic event. However, the instantaneous fundamental period depends also on the damping and inertia forces. The advantage of final softening damage index is that it can be evaluated from the initial natural period and the final natural period. A shortcoming of the damage measurements defined based on the final softening is that local element and storey damage, as well as the data related to the structural response during the seismic event, are not available; as a result the maximum softening was proposed. The two parameter-based global damage