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Seismic Damage Assessment and Performance Levels of Reinforced Concrete Members

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Abstract

It has been well accepted that performance-based seismic design (PBSD) principles and procedures will be at the core of the next generation of seismic design codes. PBSD necessitates the quantitative assessment of seismic damage suffered by structures. Quantitative assessment of damage has been proved effective in controlling the earthquake-induced damage of structures, and is feasible by the use of damage models. So far the Park-Ang damage model has remained to be the most widely used one. This model, however, has the inherent deficiency referring to its convergence at upper and lower limits.
In this study, a modification is proposed for the original Park-Ang damage model, eliminating its non-convergence problem at upper and lower limits. The combination coefficient of the modified model is calculated using the cyclic test results of flexure-dominant RC members from the database provided by the Pacific Earthquake Engineering Research Center and the author's own tests. An empirical formula is derived through multivariable nonlinear regression analysis to relate the coefficient with three design parameters. The comparison between the modified and original model indicates that the damage index of flexure-dominant RC members can be determined by the modified model with higher precision and smaller scatter. The damage indices at principal damage states are calculated by the modified damage model for each member in the above mentioned database. Accordingly, the performance levels of RC structural members are quantified by setting the individual limit value of damage index.

Keywords: Reinforced concrete structures, seismic damage model, performance level.

1. INTRODUCTION

The performance-based seismic design (PBSD) makes it possible for designers to intentionally control the damage levels of structures within acceptable range during earthquakes of different intensities, which

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necessitates the quantitative assessment of seismic damage. Damage models have bridged the gap between structural performance and damage levels. Over the past decades, significant advancement has been achieved in the method of building damage model (Rodriguez and Padilla 2009; Teran-Gilmore and Jirsa 2005). The earliest and simplest measures of damage were either ductility-based or with introducing the parameter of the inter-story drift. These models which are classified in non-cumulative damage models fail to reflect the effects of repeated cycling that occurs under seismic loading, but remain quite widely used because of their simplicity.

It is now generally accepted that the energy dissipated by structures during earthquakes has an effect on the level of the structural damage. Experiments on structural members and structures indicate that the excessive deformation and hysteretic energy are both the most important factors contributing to seismic damage. Hence, the damage models combining deformation ductility and hysteretic energy appear to be more reasonable. One of the best known and most widely used cumulative damage models is the Park-Ang model (Park and Ang 1985) as follows:

$$D_{PA} = \frac{\delta_m}{\delta_u} + \beta_{PA} \frac{\int dE}{F_y \delta_u} \tag{1}$$

where δ_m = maximum deformation; δ_u = ultimate deformation under monotonic loading; F_y = yield strength; dE = incremental absorbed hysteretic energy; β_{PA} = non-negative combination coefficient. This model has been well accepted owing to its simplicity and the fact that it has been calibrated against a significant amount of observed seismic damage. Although being the most widely used, the Park-Ang damage model does not converge at its upper and lower limits, that is: (1) damage index is greater than 0 when structures are loaded within elastic range, thus sustain no damage; (2) damage index is greater than 1.0 while structures are loaded monotonically to failure.

The objective of this paper is to develop a modified Park-Ang damage model in which the non-convergence problem at upper and lower limits does not exist. Furthermore, based on this new damage model, the performance levels of RC members are quantified through setting the individual limit value of damage index for each critical damage state by using the cyclic test results of RC columns from the database provided by the Pacific Earthquake Engineering Research Center (PEER 2004) and the author's own tests (Chen et al. 2009).

2. MODIFIED DAMAGE MODEL

The new modified form for the Park-Ang damage model is given as below:

$$D = (1-\beta) \cdot \frac{\delta_m - \delta_c}{\delta_u - \delta_c} + \beta \cdot \frac{\int dE}{F_y(\delta_u - \delta_y)} \tag{2}$$

where δ_c = deformation at initial cracking of outer concrete; δ_y = yielding deformation; β = combination coefficient which is different from β_{PA}. In this model structures are considered as non-damage before the cracking of concrete. Therefore, δ_m is determined by the following equation:

$$\delta_m = \begin{cases} \delta_c, & \delta_m \le \delta_c \\ \delta_m, & \delta_m > \delta_c \end{cases} \tag{3}$$

The load-deformation relationship of typical RC members under monotonic loading can be well represented by a bilinear curve with zero slope for post-yielding section (Park 1989). In this case, the plastic strain energy can be written as:

$$E_{hm} = F_y(\delta_u - \delta_y) \tag{4}$$

The new damage model converges at its upper and lower limits, which lays the basis for evaluating precisely the damage level of RC members. Since the deformation of RC members at initial cracking, δ_c, is significantly smaller than the deformation at yielding, the modified damage model can be further simplified as:

$$D = (1-\beta)\cdot\frac{\mu_m}{\mu_u} + \beta\cdot\frac{\int dE}{F_y\delta_y(\mu_u - 1)} \tag{5}$$

where $\mu_m = \delta_m/\delta_y$, $\mu_u = \delta_u/\delta_y$. The analysis of comparing Equation (5) with Equation (2) shows that the error due to this simplification of damage model is very small and can be neglected in the view of engineering practice.

3. CALIBRATION OF COMBINATION COEFFICIENT

The direct application of the new damage model in the seismic design of RC structures necessitates the quantitative determination of the combination coefficient which is closely related to the seismic performance of structures. In this paper the combination coefficient is calibrated upon the database of a large number of tests.

3.1. Experimental Database

The experimental database for calibrating the combination coefficient consists of two parts of tests. One is the chosen from PEER structural database (PEER 2004). The criteria for being chosen as the calibrating database are that: (1) the specimen is RC rectangular column; (2) the specimen was loaded cyclically until failure; and (3) the failure was judged as dominantly flexural type. This part of the calibrating database consists of 115 RC column tests. The other part of the database comes from the authors' own tests (Chen et al. 2009), which comprises 13 tests of flexure-dominant RC rectangular columns and beams under cyclic loading. The main design parameters such as the axial load ratio, the shear span ratio, and the total longitudinal reinforcement ratio vary in a wide range, indicating the well representativeness of specimens in the calibrating database.

3.2. Derivation of Combination Coefficient

The combination coefficient β is derived on the assumption that the damage index equals to 1 at the ultimate limit state:

$$\beta = \frac{f_y(\delta_u - \delta_{u,c})(\delta_u - \delta_y)}{\delta_u\int dE - \delta_{u,c} f_y(\delta_u - \delta_y)} \tag{6}$$

where $\delta_{u,c}$ is the ultimate deformation under cyclic loading. The yielding deformation δ_y is determined by the first yielding of longitudinal reinforcement, or by the intersection of the horizontal line at maximum force with the straight line passing through the origin and the 75% maximum force point on the envelope curve, whichever is less. The ultimate state is defined by the point on the descending section of envelope curve with a 15% force drop, or when the longitudinal rebar buckles or fractures, whichever is less. The yielding force of specimen is averaged between the maximum moments at column bottom in two loading directions. The hysteretic energy $\int dE$ is obtained by integrating the force with deformation up to the ultimate state. The ultimate chord rotation under monotonic loading is calculated by the following formula (FIB 2003) which was derived through regression analysis of 1282 tests of RC members and also adopted by Eurocode 8:

$$\theta_u = \alpha_{st}(1-0.38a_{cyc})\left(1+\frac{a_{sl}}{1.7}\right)\cdot 0.3^{n_0}\cdot\left(\frac{\max(0.01,\omega')}{\max(0.01,\omega)}\cdot f_c'\right)^{0.2}\left(\frac{L}{h}\right)^{0.425} 25^{\alpha\rho_{sx}\frac{f_{yw}}{f_c'}} \tag{7}$$

where α_{st} is the coefficient for the type of steel, equal to 0.016 for ductile hot-rolled or heat-treated steel and to 0.0105 for cold-worked steel; a_{cyc} is the zero-one variable for cyclic loading, equal to 0 for monotonic loading and to 1 for cyclic loading; a_{sl} is the zero-one variable for slip, equal to 1 if there is slippage of the longitudinal bars from their anchorage beyond the section of the maximum moment, or to 0 if there is not; ω and ω' are the mechanical ratios of the tension and compression longitudinal reinforcement, respectively; $\rho_{sx}=A_{sh}/bs_h$ is the ratio of transverse steel parallel to the direction of loading, in which A_{sh} is the section area of transverse steel parallel to the direction of loading, b is the section width, and s_h is the spacing of stirrups; f_{yw} is the yield stress of transverse steel; α is the confinement effectiveness factor; f_c' is the concrete compressive strength.

The mean value of the combination coefficient derived by Equation (6) for the 128 tests is 0.17, and the standard deviation is 0.15. The coefficient of variation (COV) reaches 88%, which implies the complexity of the damage behavior of RC structures.

3.3. Regression Analysis

Based on the calculated values of β for each test, a negative correlation was observed between β and the confinement index $\alpha\rho_{sx}f_{yw}/f_c'$, and weak positive correlations were observed between β and the shear span ratio L/h, and the axial load ratio n_0. The nonlinear multi-variable regression analysis was conducted for the values of β in order to establish the analytical relation between the combination coefficient with these three parameters. The result of regression analysis is as follows:

$$\beta = \left(0.023\frac{L}{h}+3.352n_0^{2.35}\right)\cdot 0.818^{\alpha\rho_{sx}\frac{f_{yw}}{f_c'}}+0.039 \tag{8}$$

where $\alpha\rho_{sx}f_{yw}/f_c'$ is in percentage. Figure 1 compares the experimental and regressed β values. It can be seen that the experimental values of β are well regressed by Equation (8).

The modified damage model is evaluated through comparison with the original Park-Ang damage model. The damage indices at ultimate limit state of all the tests in the calibrating database were calculated by the modified and original Park-Ang damage model, respectively. As illustrated in Figure 2, the mean of damage indices at ultimate state calculated by the modified model is much closer to 1, and the scatter is significantly reduced compared to the results of the original model.

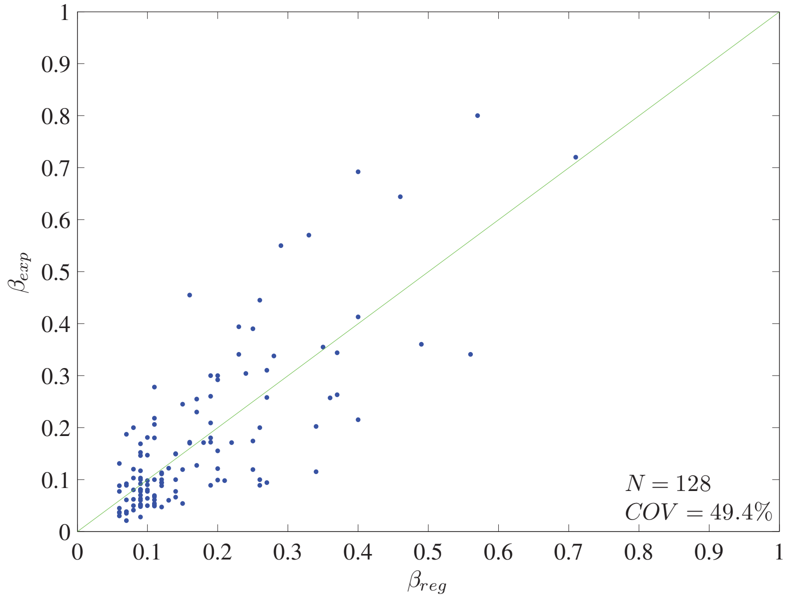
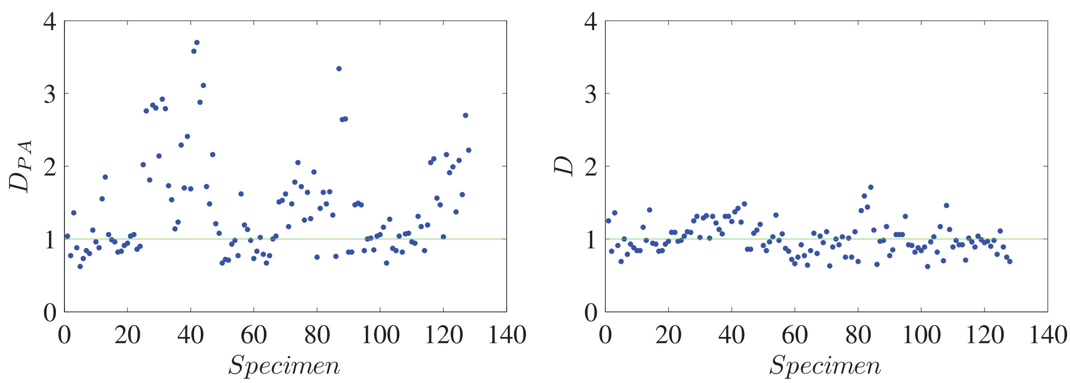


Figure 1: Comparison of combination coefficient determined by experiment and regression.

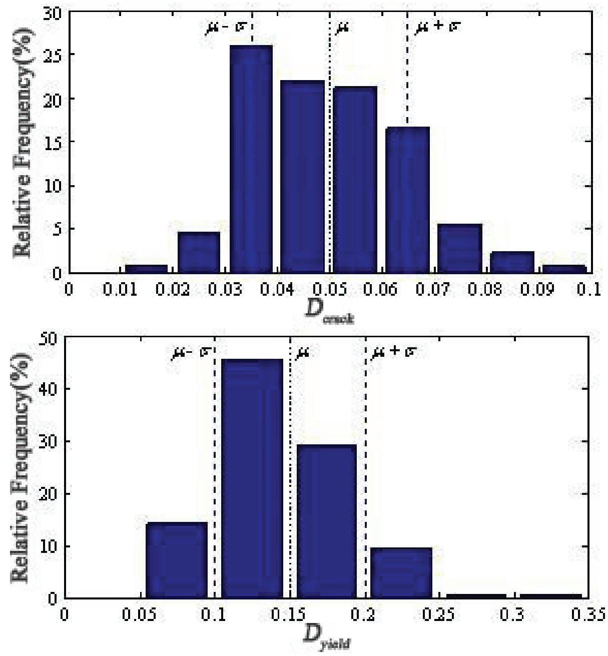


(a) By Park-Ang model (b) By modified model

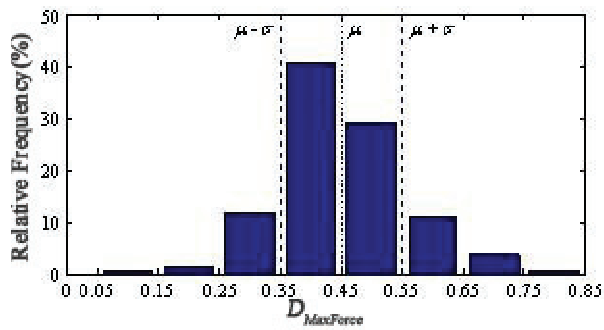
Figure 2: Comparison of damage indices at ultimate limit state.

4. LIMIT VALUE OF DAMAGE INDEX FOR PERFORMANCE LEVELS

By means of the modified damage model, the damage indices corresponding to principal damage states, nominal cracking, yielding, maximum load point and ultimate limit state were calculated for each member in the above experimental database. The nominal crack point is defined as the first obvious inflexion point of the experimental load-displacement envelope curve. As mentioned above, the combination coefficient is derived by the assumption that the damage index equals to 1. Figure 3 shows the histogram of distribution of the damage index at nominal cracking, yielding, and maximum load point respectively.



(a) At cracking **(b) At Yielding**



(c) At maximum load point

Figure 3: Distribution of damage index.

The mean, standard deviation and COV of damage index at nominal cracking are 0.0495, 0.015 and 30% respectively. As for the state of yielding, they are 0.147, 0.044 and 30%. And for maximum load point, they are 0.447, 0.108 and 24%. The probability of cumulative distribution within one time standard deviation deviated from mean at individual damage state is 68%, 72% and 76% respectively. The scatter of damage index at individual damage is not very significant. Four seismic performance levels, i.e., fully operational, immediate occupancy, life safety, and collapse prevention, have been extensively applied to describe the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur. These four performance levels could be specified by the above four damage states approximately. Therefore, the limit value of 0.05, 0.15, 0.45 and 1.0 could be used to classify the performance levels in quantitative term. That is, damage index less than 0.05 implies fully operational;

from 0.05 to 0.15, immediate occupancy; from 0.15 to 0.45, life safety; and from 0.45 to 1.0, collapse prevention.

5. CONCLUSIONS

Quantification of seismic damage is essential in performance-based seismic design. A modified damage model eliminating the non-convergence problem existing in the Park-Ang model is presented in this study. The combination coefficient in the modified damage model is calibrated by using the experimental database from a large number of RC member tests. The damage level of RC members can be quantified by the modified model with higher precision and smaller scatter than by the original model. The damage indices at principal damage states of flexure-dominant RC members in the same database are calculated by using the modified damage model. The limit values of damage index classifying the seismic performance levels are proposed on the basis of the statistical analysis results.

6. ACKNOWLEDGMENTS

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