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# Fiber-based damage analysis of reinforced concrete bridge piers

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## ABSTRACT

A fiber beam-column element is adopted to simulate the damage development process of reinforced concrete (RC) bridge piers under quasi-static and earthquake loadings considering global buckling and low-cycle fatigue of longitudinal bars. The tensile strain and low-cycle fatigue are used to represent the damage to longitudinal bars while the compression strain is adopted to calculate the damage to the cover concrete. A section damage index is proposed based on the material damage definition and bridge performance assessment. A set of circular RC bridge piers tested under different uniaxial quasi-static loading regimes are adopted to verify the reliability of the fiber beam-column element and the proposed damage model. Square RC columns subjected to different uniaxial and biaxial quasi-static loadings are used to verify the applicable scope of the fiber element and the damage index in biaxial quasi-static loading. In addition, a series of shaking table model tests on square, rectangular and circular piers subjected to bilateral earthquake ground motions are simulated to further verify the versatility of this model. The results show that, the fiber beam-column element can simulate RC columns/piers with different sections and loading regimes with good accuracy. The damage index proposed in this paper is compared against experimental results and other damage indices and it is found that the proposed index can reflect the damage state at any stage and the gradual accumulation of damage in RC columns/piers more convincingly than most other indices available in literature.

## 1. Introduction

The ability to predict damage states for the design of reinforced concrete (RC) bridge piers is fundamental to the performance-based seismic design (PBSD) of bridges [1]. Damage indices are used to quantify damage level of structures caused by an earthquake, which play a vital role in retrofit decision-making and disaster-planning in earthquake regions.

Several studies have proposed damage indices for RC members, including noncumulative and cumulative damage indices in general. Ductility is the most commonly used noncumulative damage indices [2–4] and still regarded as a critical design parameter by codes [5–10]. Stiffness and strength degradations [11–13] are also widely used noncumulative damage indices. The typical stiffness degradation-based damage index is the one proposed by Kunnath et al. [12], which is defined as:

$$D = \frac{k_m - k_0}{k_f - k_0} \tag{1}$$

where $k_m$ is the secant stiffness of the RC members at the maximum induced displacement, $k_f$ is the pre-established secant stiffness at failure under monotonic loading, and $k_0$ is the initial stiffness prior to loading. Noncumulative damage indices cannot consider the low-cycle fatigue damage caused by displacement reversals under earthquake loading.

Cumulative indices could be divided in low-cycle fatigue-based [14,15] and energy-based [16–19] formulations. One of the most practical approaches to modeling fatigue failure is a mechanics-based model proposed by Mander and Cheng [14]. Their final expression shown below in Eq. (2) is derived from plastic strain vs. fatigue life relationship obtained from actual testing of steel reinforcing bars [20] and the relationship between curvature and strain in a circular reinforced concrete section.

$$2\phi_p R = \frac{0.113}{1 - d/R} N_f^{-0.5} \tag{2}$$

In the above expression, $\phi_p$ is the plastic curvature, $R$ is the overall column radius, $d$ is the effective depth measured from the outermost compression concrete fiber to the center of tension reinforcement, and $N_f$ is the number of cycles to the appearance of the first fatigue crack in steel. The damage model based on fatigue-life expression accounts only for low-cycle fatigue in steel due to flexure and sometimes under-

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**Nomenclature**

| | |
|---|---|
| $D$ | Damage index of RC bridge piers |
| $D_{ss}$, $D_{sf}$ | Steel damage caused by tensile strain and low-cycle fatigue |
| $D_s$,$D_c$ | Damage values of steel and cover concrete |
| $D_{section}$ | Section damage index |
| $D^+$, $D^-$ | Damage caused by positive/negative deformations; |
| $k_m$ | Secant stiffness of the RC members at the maximum induced displacement |
| $k_f$ | Pre-established secant stiffness at failure under monotonic loading |
| $k_0$ | Initial stiffness prior to loading |
| $\phi_p$ | Plastic curvature |
| $d$ | Effective depth of RC bridge piers |
| $h$ | Effective height of RC bridge piers; |
| $N_f$ | Number of cycles to the appearance of the first fatigue crack in steel |
| $R$ | Overall column radius |
| $E_{p,i}$ | Energy in a primary half cycle |
| $E_i$ | Energy in the following half cycles |
| $E_f$ | Energy absorbed in a monotonic test to failure |
| $A_0$ | Total energy dissipated under a monotonic load-displacement curve up to failure |
| $A_n$ | Total energy under a monotonic load-displacement curve from the end of the last cycle $n$ (zero force point) to failure |
| $\delta_m$ | Maximum deformation under earthquake |
| $\delta_u$ | Ultimate deformation under monotonic loading |
| $Q_y$ | Yield strength of RC members |
| $\int dE$ | Cumulative hysteretic energy |
| $\beta$ | Non-negative parameter of Park-Ang model |
| $\varepsilon_y$ | Yield strain of longitudinal bar |
| $\varepsilon_{c1}$, $\varepsilon_{c2}$ | Strain of longitudinal bar corresponding to 1 mm and 2 mm crack widths |
| $\varepsilon_{bb}$ | Buckling strain of reinforcing steel |
| $\varepsilon_p$ | Plastic strain amplitude of steel |
| $f_y$, $f_{yh}$ | Yield stress of longitudinal bar and transverse reinforcement |
| $\rho_s$, $\rho_{sh}$ | Volumetric ratio of longitudinal bar and transverse reinforcement |
| $E_s$ | Young's modulus of reinforcement steel |
| $P$ | Axial load of RC bridge pier |
| $f'_c$ | Concrete strength |
| $C_f$ | Coffin-Manson constant |
| $\alpha$ | Cyclic strength reduction constant |
| $p$ | Percentage of cover concrete spalling |
| $A_g$ | Cross section area of RC bridge piers |
| $c$ | Thickness of cover concrete |
| $d_l$ | Longitudinal reinforcement diameter |
| $d_s$ | Transverse reinforcement diameter |

estimates the damage.

Kratzig and Meskouris [16] proposed a damage model based on energy dissipation, in which the damage caused by positive deformations is quantified as:

$$D^+ = \frac{\sum E^+_{p,i} + \sum E^+_i}{E^i_f + \sum E^+_i} \tag{3}$$

where $E_{p,i}$ is the energy in a primary half cycle, $E_i$ is the energy in the following half cycles and $E_f$ is the energy absorbed in a monotonic test to failure.

A similar expression is computed for negative deformations, and the two quantities are combined as follow:

$$D = D^+ + D^- - D^+D^- \tag{4}$$

However, this damage model cannot predict the damage condition under monotonic loading mode. Hindi and Sexsmith [17] proposed an energy-based damage index that was applicable to both monotonic and cyclic loadings. Their model uses the predicted hysteretic behavior of a concrete column, which is shown as follow:

$$D = \frac{A_0 - A_n}{A_0} \tag{5}$$

In Eq. (5), $A_0$ is the total energy dissipated under a monotonic load-displacement curve up to failure and $A_n$ is the total energy under a monotonic load-displacement curve from the end of the last cycle $n$ (zero force point) to failure.

Park and Ang [21] proposed a damage index combining deformation and energy dissipation, as shown in Eq. (6), which is perhaps one of the most popular damage indices because it has been verified by experimental data and proved to be able to predict damage conditions under different loading regimes.

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y\delta_y}\int dE \tag{6}$$

In Eq. (6), $\delta_m$ is the maximum deformation under earthquake, $\delta_u$ is the ultimate deformation under monotonic loading, $Q_y$ is the yield strength, $\int dE$ is the cumulative hysteretic energy and $\beta$ is a non-negative parameter.

A major problem of cumulative damage indices and Park and Ang combined index are that they include coefficients that must be

**Table 1**
Bridge performance assessment [32].

| Level | Performance level | Qualitative performance description | Quantitative performance description |
|---|---|---|---|
| I | Cracking | Onset of hairline cracks | Cracks barely visible |
| II | Yielding | Theoretical first yielding of longitudinal reinforcement | Crack widths <1 mm |
| III | Initiation of local mechanism | Initiation of inelastic deformation,<br>Onset of concrete spalling,<br>Development of diagonal cracks | Crack widths 1–2 mm,<br>Length of spalled region >1/10 cross-section depth |
| IV | Full development of local mechanism | Wide and extended cracks,<br>Significant spalling over local mechanism region | Crack widths >2 mm,<br>Diagonal cracks extend over 2/3 cross-section depth,<br>Length of spalled region >1/2 cross-section depth |
| V | Strength degradation | Buckling of main reinforcement,<br>Rupture of transverse reinforcement,<br>Crushing of core concrete | Crack widths >2 mm in core concrete |