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SEISMIC BEHAVIOR OF HIGH STRENGTH RC COLUMNS

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SUMMARY

This paper deals with the assessment of the structural damage of normal and high strength RC columns subjected to earthquake excitations. On the basis of a large number of data from tests of normal and high strength RC columns, an extensive study was developed in order to investigate the assessment of earthquake damage in isolated structural elements.

INTRODUCTION

The use of high strength concrete in building constructions has many advantages such as: increased strength of structures, reduced cross-sections, more durable material and therefore substantial savings. However, due to the limited amount of experimental research and to the uncertainty inherent in the prediction of failure of the structural elements under cyclic loading, the use of high strength concrete structures in seismic risk areas needs extra caution in order to ensure adequate ductile behavior. In the present study the experimental results of a range of normal and high strength RC columns tested by the writers and other authors ([Atalay and Penzien, 1975], [Azizinamini et al, 1994], [Bayrak and Sheikh, 1996], [Bett et al, 1985], [Galeota et al, 1996], [Gill et al, 1979], [Imai and Yamamoto, 1986], [Kanda et al, 1988], [Kimura et al, 1996], [Legeron and Paultre, 1996], [Muguruma et al, 1989], [Nagasaka, 1982], [Ohno and Nishioka, 1984], [Ouhe et al, 1985], [Saatcioglu and Ozcebe, 1989], [Sakay et al, 1990], [Sheikh and Khoury, 1993], [Sheikh et al, 1994], [Soesianawati et al, 1986], [Tanaka and Park, 1990], [Tanaka et al, 1994], [Umehara and Jirsa, 1982], [Watson and Park, 1994], [Wight and Sozen, 1973], [Zahn et al, 1986]) were collectively analysed. The data include test results of RC columns with square cross-sections and with different concrete compressive strength (f_{c0}), longitudinal (ρ_l) and transverse (ρ_t) reinforcement percentage, shear span ratio (l/d), level of axial load ($N/(f_{c0}A_c)$) and loading conditions. The range covered by f_{c0} was 21÷55MPa for the normal strength columns and 59÷118MPa for the high strength columns. On the basis of the available experimental data an analytical model, capable of reproducing the hysteretic behavior of RC columns, was calibrated. The model was used in an extensive testing program, involving the nonlinear dynamic analysis of single degree of freedom structures (isolated elements), subjected to earthquake excitations. The information obtained in the force-deformation responses (maximum response deformation, resistance at yield and cumulative dissipated energy) was used in the damage function [Park and Ang, 1985] in order to predict the measure of damage of the normal and high strength RC columns examined.

ANALYTICAL MODEL

The analytical model [Beolchini et al, 1998] used to describe the cyclic moment-curvature relationships of RC column sections is made up of a series of straight lines (Fig. 1). The basic parameters defining the analytical model are the yield moment (M_y), the yield curvature (ϕ_y), the stiffness of the inelastic loading branch (K_p) and three parameters regarding the hysteretic behavior (unloading stiffness, strength degradation, and shear effect). The unloading stiffness and strength degradation depend on ductility.

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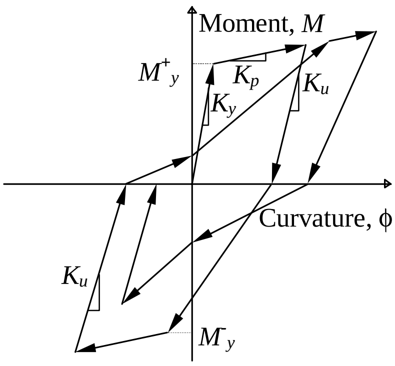

$K_u = K_y(\phi_i/\phi_y)^{\alpha}$
K_u = unloading stiffness in the i-th cycle
K_y = stiffness corresponding to the yield curvature
ϕ_i = curvature in the i-th cycle
ϕ_y = yield curvature
α = power coefficient to be adjusted on the basis of experimental results.

Figure 1: Analytical moment-curvature relationship.

IDENTIFICATION OF THE MODEL

The same rules describing the moment-curvature relationships were used to reproduce the load-displacement curves of the RC columns analysed. The previously mentioned basic parameters of the analytical model were adjusted by using an identification technique, in order to obtain the best possible match between the experimental curves of each column analysed and those predicted. The experimental and best theoretical load-displacement curves for some columns with different test parameters are shown in Fig. 2.

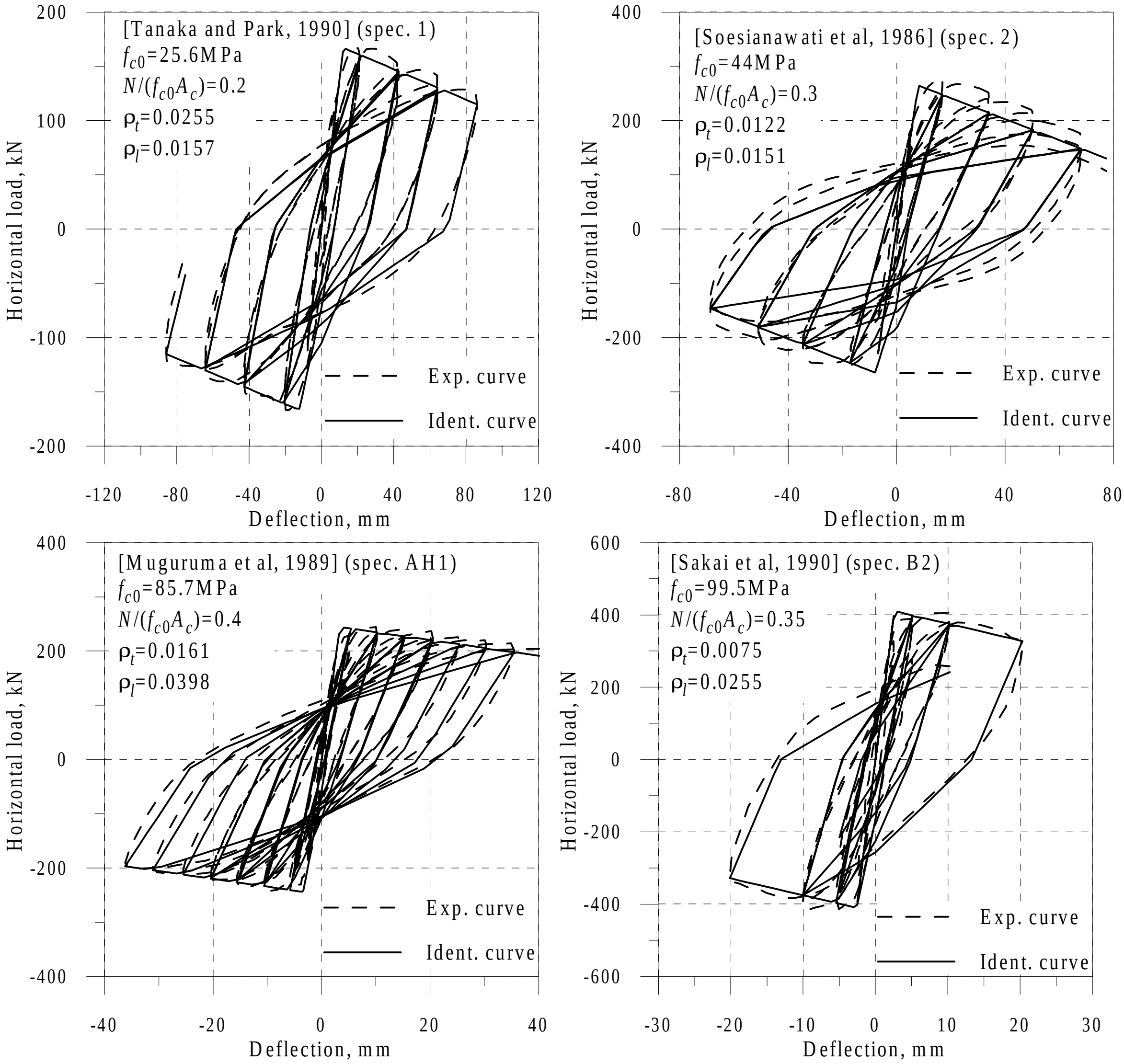

Figure 2: Comparison of experimental and identified load-displacement curves.

LOCAL DAMAGE INDEX

Structural damage prediction under cyclic loading has been extensively studied and various models have been proposed [Williams and Sexsmith, 1995]. In this paper the damage index proposed by Park and Ang was adopted. Park and Ang introduced a local damage index for the design of reinforced concrete structures, ranging from 0 (undamaged structures) to 1 (failure). This index includes both the maximum displacement ductility and cumulative dissipated energy. The local damage index is represented as follows:

$$D = \frac{x_m}{x_u} + \frac{\beta}{F_y x_u} \int dE \qquad (1)$$

where x_m = maximum deformation under cyclic loading; x_u = ultimate deformation under monotonic loading; F_y = calculated yield strength; dE = incremental dissipated hysteretic energy; β = dimensionless parameter to be determined on the basis of the experimental data.

In this study x_u in eq. (1) was calculated, for each column analysed, by using a computer program describing the monotonic load-displacement relationship of RC column. The program was based on the model proposed for compressive high strength concrete by Cusson and Paultre [1995]. The value of the ultimate compressive strain of the confined concrete was limited to the value producing the buckling of the longitudinal compressive reinforcement [Papia and Russo, 1989]. The parameter β was determined, according to the procedure described by Park and Ang, on the basis of the available cyclic test data of the columns.

The calculated β values showed a negative correlation with the transverse reinforcement percentage (ρ_t), a positive correlation with the longitudinal reinforcement percentage (ρ_l), a weak correlation with both the shear span ratio (l/d) and the level of axial load ($N/(f_{c0}A_c)$). No correlation was found between the calculated β values and the compressive strength of concrete f_{c0}. By using a nonlinear regression analysis program, the coefficients of the following eq. (2) were calibrated through the experimental values of β:

$$\beta = 0.66^{\rho_t} \left(-0.28 + 0.19\rho_l + 0.06 \frac{l}{d} + 0.47 \frac{N}{f_{c0} A_c} \right) \qquad (2)$$

Equation (2) is similar to that proposed by Park and Ang. Figure 3 shows the comparison between the calculated and experimental β values.

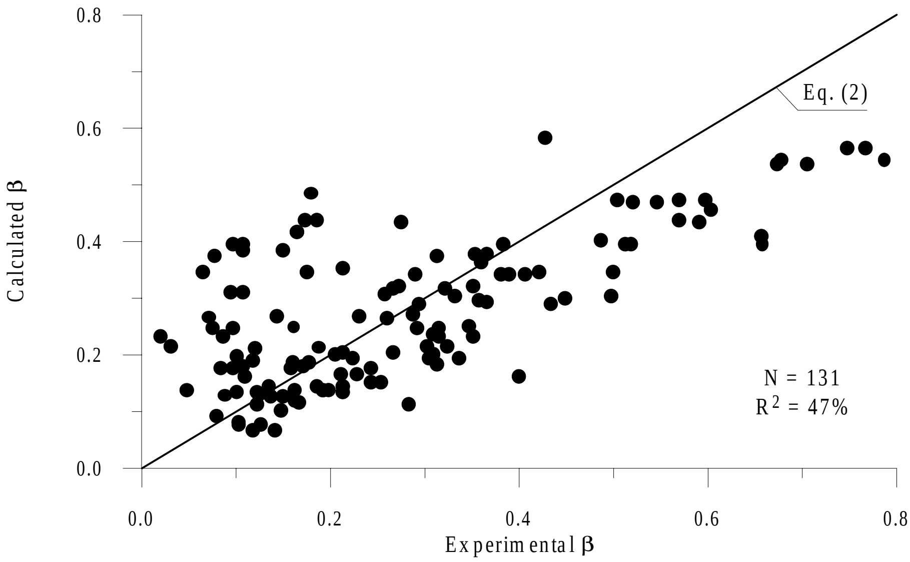

Figure 3: Calculated β vs. experimental β.

DYNAMIC ANALYSIS

Single non-linear d.o.f. models, having the characteristics of specimens above (f_{c0}, ρ_l, ρ_t, N, cross-section, height,…), were used to perform the dynamic analysis. Three earthquake records were selected, on the basis of their elastic response spectra shown in Fig. 4, as input ground motion. The records were intended to cover three types of ground motion: the first (Qk1) having significant peaks in the low period range, the second (Qk2) in the middle range; while the peak accelerations of the third spectra (Qk3) are distributed over a larger interval of natural period and decrease significantly at long structural periods. The corresponding accelerograms were scaled to compare the responses of models with very different characteristics: the peak ground acceleration were set equal to the ratio $a_y/g = (F_y/m)/g = F_y/N$, where $a_y = F_y/m$ is the acceleration which produces yielding in the specimen (F_y and m are respectively yield force and mass of the model), g is the acceleration of gravity, and N is the axial load in the specimen.

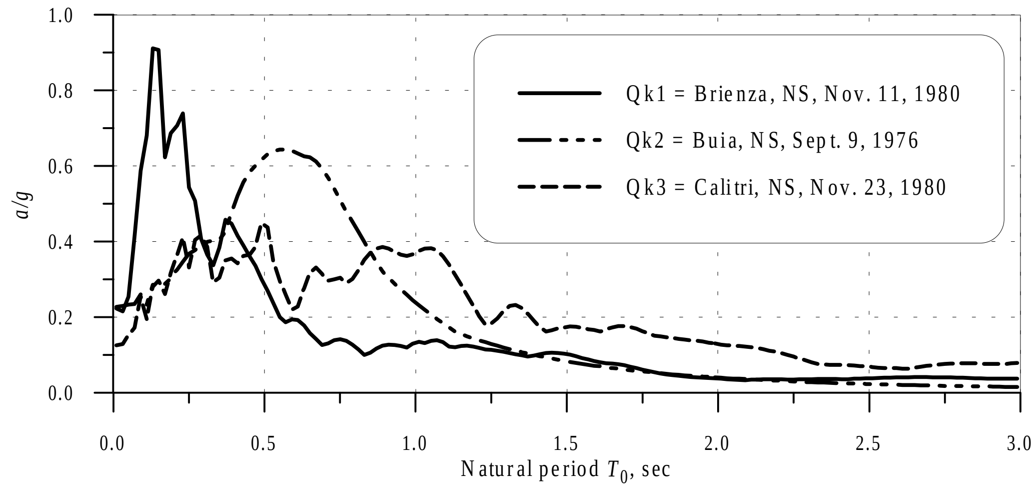

Figure 4: Elastic acceleration response spectra for some Italian accelerograms (ζ = 0.05).

A total of 79+52 models corresponding, respectively, to normal and high strength columns were analysed. Many response parameters were computed, some having been proposed in literature as damage predictors. Special attention was paid to the local damage index (1) and to the models where D was less or equal to 0.4, which, according to the damage classification suggested by Park and Ang, defining the threshold value between repairable and irreparable damage, is commonly considered a limit. The results can be summarised as follows.

Those parameters which are not able to reflect the cumulative damage effect on failure, such as ductility factor, or stiffness reduction, or, generally speaking, parameters related simply to a displacement, were at all not correlated to the local damage index (1). Other parameters based on the energy dissipated in the time history responses were strictly dependent on the earthquake spectra and on the period of specimen. Figure 5 shows two examples referring to the reduced set of high strength columns with $D \leq 0.5$. The parameters in this figure are: dissipated energy E_h normalized to the maximum elastic energy $F_y x_y/2$, and energetic equivalent ductility factor $\mu_{eq} = x'_m/x_y$, where x'_m is the displacement that, being reached monotonically, gives the same energy dissipated in the actual process, and x_y is the yield displacement.

The relationships between the parameters above and the natural period shown in Fig. 5 are evident: the curves in the diagrams help to recognise the trend of the numerical responses. However their dependence on earthquake characteristics do not make them very useful. Furthermore, the correlation with the damage index was definitely absent. This behavior was similar both in the normal and in the high strength models.

The diagrams plotted in Fig. 6 are of interest as they report only the high strength column results, where two parameters are defined:

$$\xi = \frac{x_m}{x_u}; \qquad \eta = \frac{E_h}{E_u} \tag{3}$$

the first is the ratio of maximum deformation under earthquake over the ultimate deformation under monotonic loading; the second is the ratio of energy dissipated under ground motion over the area under the load-displacement curve for the column monotonically loaded until x_u.

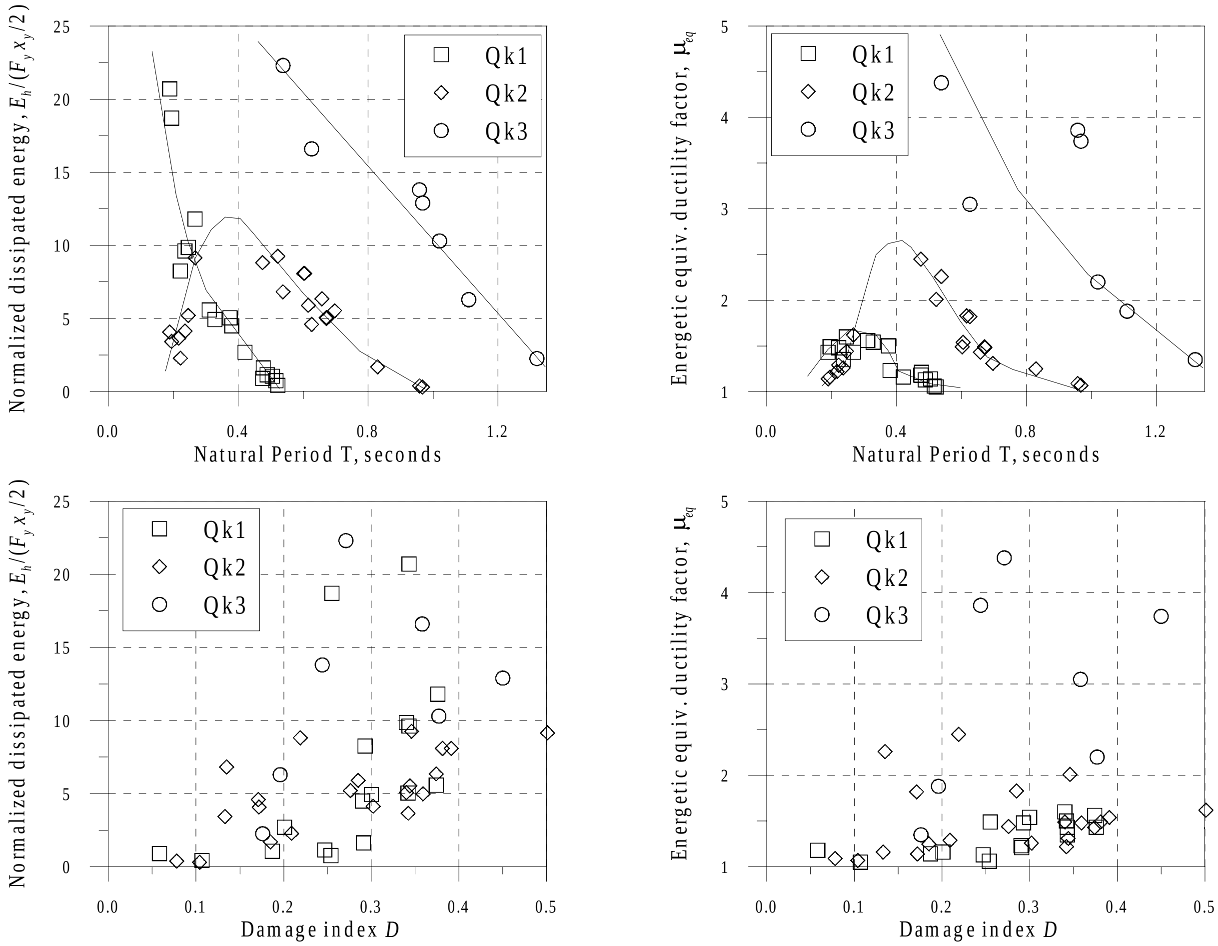

Figure 5: Normalized dissipated energy and equivalent ductility factor μ_{eq} vs. period and damage index.

There is an accurate correlation between D and ξ, especially in the range of $D < 0.3$. The values of D versus η are more scattered, although a weak correlation can still be seen. It is interesting to observe that the data plotted in this figure refer to models having very different characteristics: the periods vary from 0.2 up to 1.2 seconds, the reinforced concrete resistance is in the range of 59÷118MPa; initial and softening branch stiffness, and axial loads are scattered in an analogous way, and three different earthquakes records were used. The first plot in Fig. 6 suggests that the damage level could be simply limited by bounding the parameter ξ. ξ depends on x_u,, and on x_m; x_u is easy to calculate on the basis of characteristics of the section and x_m is the maximum displacement of the column under earthquake excitation. The coefficient ξ can be considered a kind of ductility, where the maximum displacement is compared to x_u instead of yielding displacement x_y. Furthermore, if equation (1) is re-written as

$$D = \frac{x_m}{x_u}\left(1 + \frac{\beta x_m}{F_y}\int dE\right) = \xi\alpha; \qquad \alpha = 1 + \frac{\beta x_m}{F_y}\int dE \tag{4}$$

the first plot in Fig. 6 suggests that α might not depend on a single parameter but only on their combination, at least when $D < 0.3$.

This aspect is confirmed if normal strength and high strength columns are collectively considered (80 numerical results). For these structures the relationship D-η becomes clearly confused, whereas the comment concerning D-ξ is still valid, as shown in Fig. 7, where normal strength column data are added. It can be seen that the slope of the best fitting line is slightly altered.

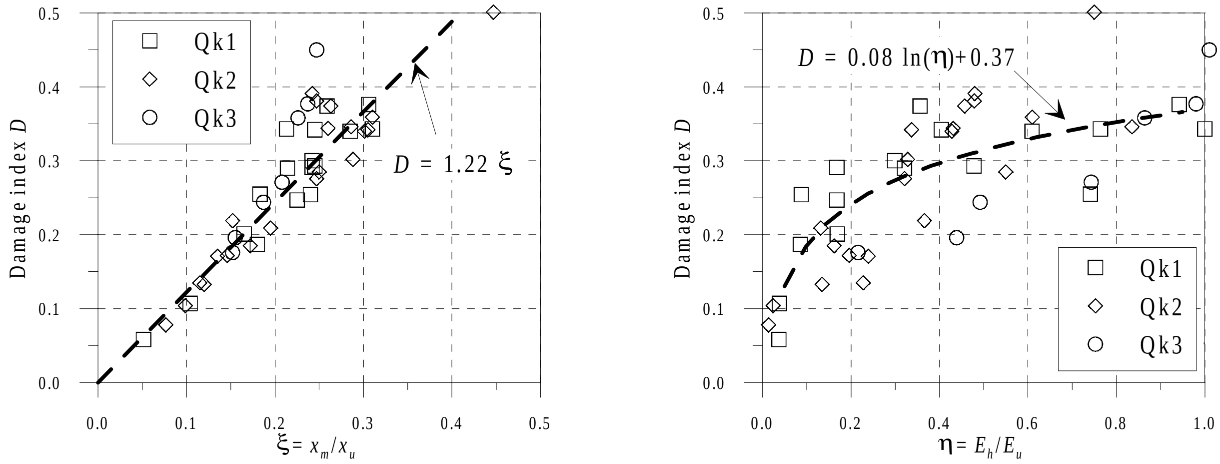

Figure 6: Damage index vs. ξ and η (high resistance column models only).

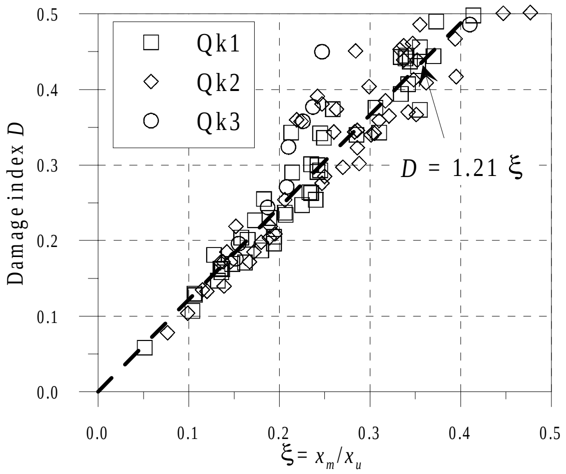

Figure 7: Damage index versus ξ (normal and high resistance column models).

CONCLUSIONS

After reviewing a large number of results on normal and high strength RC columns tested by the writers and other authors, the following conclusions can be drawn:

The experimental results were used to calibrate the basic parameters of an analytical model able to describe the cyclic load-displacement relationships of RC columns. The same experimental results were used to calibrate the basic parameters of the local damage index proposed by Park and Ang.

Extensive dynamic analyses have confirmed that only the parameters able to reflect the cumulative damage effect on failure are correlated to the local damage index (1). Furthermore, the damage index D seems to be linearly related to the ratio of maximum displacement of the time-history over the maximum displacement reached monotonically: the agreement with linear relationship, evaluated by more than 80 numerical results corresponding to very different high and normal strength columns and different ground motions, is highly satisfying if $D < 0.3$, whereas numerical data are slightly scattered beyond this value. The ratio η gives another criterion to evaluate the damage index, even if its validity is restricted only to the high strength columns and the numerical data are more scattered than in the previous case.

ACKNOWLEDGEMENTS

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