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Assessment of Rehabilitation Strategies for RC Columns

Ghobarah, A.[1] and El-Attar, M.[2]

ABSTRACT

The effectiveness of different column retrofit schemes in improving the seismic performance of existing reinforced concrete buildings is evaluated. The selective retrofit schemes may include increasing the column strength, ductility and /or stiffness. A probabilistic analysis is performed to assess the performance of existing as well as retrofitted frames. The performance is evaluated in terms of global or interstory drift and potential damage to the building. The approach is applied to the case of a thirty-years old low-rise, three-story reinforced concrete office building. The building columns are retrofitted using different strategies targeted at increasing the column strength, ductility or stiffness. The effect of the different retrofit schemes is examined using the nonlinear static pushover analysis. A dynamic response of the original as well as the retrofitted structures is evaluated in terms of damage to the frames and drift limits. The results show that increasing the strength or strength with stiffness of the reinforced concrete columns is the most appropriate technique to retrofit the structure under consideration due to the reduced drift and damage levels.

INTRODUCTION

Many of the existing buildings are gravity load designed with little attention paid to the lateral load resistance. During recent earthquakes (the 1994 Northridge and the 1989 Loma Prieta earthquakes), the behaviour of reinforced concrete buildings designed according to current seismic codes was satisfactory from life safety point of view. However, structures designed to earlier codes or prior to the seismic design requirements did not behave satisfactorily due to insufficient lateral capacity and limited ductility due to nonductile detailing.

It is not the normal practice in retrofitting to attempt to make the existing structure comply with the current code provisions. The retrofitting objectives for the structure should rather depend on a performance based criteria to ensure a defined level of damage or to prevent collapse of the building during a specified level of ground motion (SEAOC 1995; and Ghobarah et al. 1997). The selection of the retrofit system and the level of protection to the structure are the important decisions in the design process. To provide the basis for these design decisions, it is necessary to evaluate the effect of various retrofit systems on the performance of the structure and to quantify the upgrade to the seismic performance. This approach will provide effective targeted rehabilitation schemes that are cost effective.

Column retrofitting is one of the widely used techniques for upgrading the lateral load carrying capacity of reinforced concrete buildings. Several schemes are available for retrofitting reinforced concrete columns. Upgrading the column performance normally involves increasing its strength, ductility, stiffness or in most cases a combination of two or the three parameters. Available test results provide information on the performance of the column. However, it is not clear what would be the effect of each strategy on the overall behaviour of the structure in terms of its drift and damage potential when subjected to earthquake ground motion. Although in most cases it is difficult to change the stiffness without affecting the strength and ductility of a column, the effect of the change in each aspect of behaviour will be examined separately in order to investigate the selective and targeted rehabilitation strategies for columns.

The objective of this study is to assess the seismic performance of existing reinforced concrete buildings and to evaluate the effectiveness of different selective rehabilitation strategies for the concrete columns. The analysis will provide the basis for decisions concerning the rehabilitation strategy for nonductile RC frames and the design of selective and cost effective rehabilitation techniques.

[1]Professor, Department of Civil Engineering, McMaster University, Hamilton, Ontario, Canada L8S 4L7.

[2]Lecturer, Structural Engineering Department, Cairo University, Cairo, Egypt.

METHODOLOGY

The performance of the original and retrofitted structures is evaluated in terms of drift and damage potential. A selected damage index relates the peak ground acceleration (PGA) to the probability of exceedance of different damage and drift levels. The load-displacement performance curve for the structure is determined using the nonlinear static pushover analysis. To eliminate the effect of randomness in ground motion characteristics, the material strength and the dimensions of the structure, on the seismic demand for the original and retrofitted frames, a probabilistic analysis is carried out using Monte Carlo Simulation where a random set of ground motion records is generated and scaled to different levels of PGA. In the probabilistic analysis, an artificially generated ground motion is used to evaluate the response of the structure. Using the appropriate probability distribution parameters for material strength and dimensions of the structure, a random set of properties of the structure is generated to be used in the analysis. This approach will provide a basis for comparative evaluation of the performance of various frames.

The state of damage in the structure is predicted using a damage index procedure. The selected damage index for use in the analysis is that proposed by Park et al. (1984). The described methodology is applied to the analysis of a three story reinforced concrete frame office building. Fig. 1 shows the layout, elevation and plan of the building. The frame is thirty years old and was gravity load designed. The design live load was taken equal to 2.4 kN/m^2. The steel reinforcement yield strength and the concrete compressive strength are 300 MPa and 21 MPa, respectively. The reinforcement detailing includes light shear reinforcement in columns and beams. Dimensions of the columns and beams and their reinforcement details are also shown in Fig. 1.

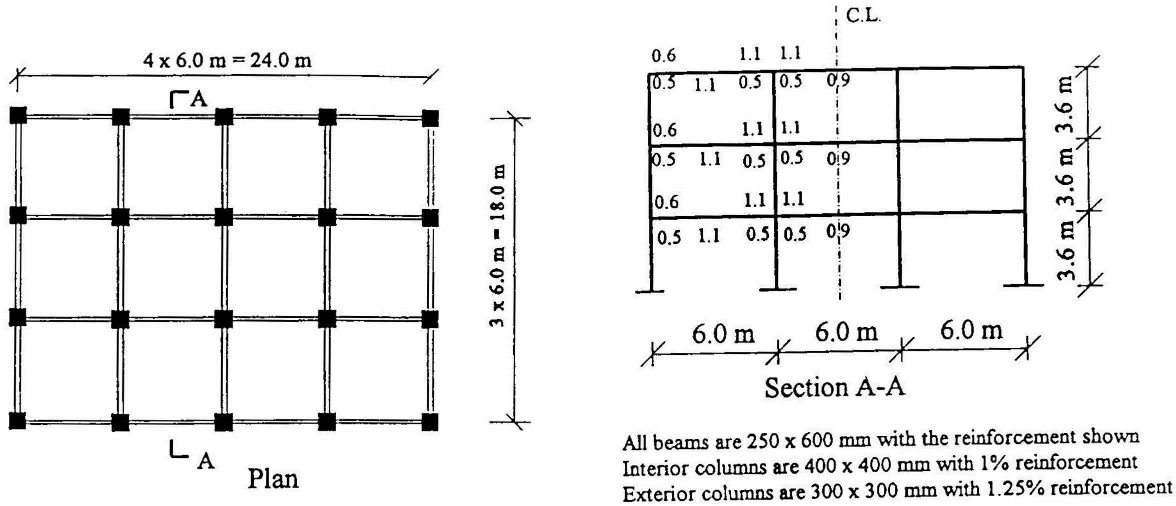

Figure 1 Layout of the reinforced concrete office building

Selective Column Retrofit Schemes

Four different schemes for retrofitting the columns of the frame are adopted as illustrated in Fig. 2 for an interior column. In the first scheme, the strength of all columns is increased by 30% (Frame F1). The second retrofit scheme is designed to increase the ductility of all columns by 100% (Frame F2). In the third scheme the stiffness of the columns is increased by 500% (Frame F3). Since the stiffness increase is normally associated with a corresponding increase in strength, in the fourth retrofit scheme the stiffness is increased by 500% and the strength is increased by 50% at the same time (Frame F4). The existing frame is denoted Frame F.

Ground Motion Modelling

Artificial stationary ground motion time histories are generated in the form (Shinozuka, 1974):

$$\ddot{u}_g(t) = \sum_{k=1}^{n} \sqrt{2\, S_g(\omega_k)\, \Delta\omega}\, \sin(\omega_k t + \phi_k) \tag{1}$$

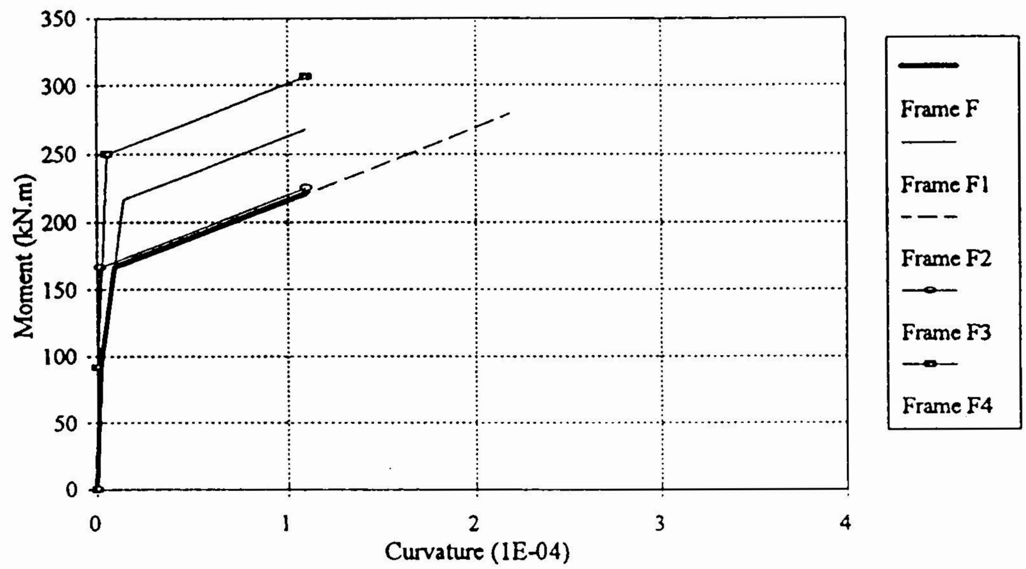

Figure 2 Moment-curvature relationship for different retrofitting schemes

where, $S_g(\omega_k)$ is the value of the power spectral function for the ground acceleration calculated at a frequency ω_k, n is the number of frequencies considered in the analysis. Typically this is taken equal to 200; $\Delta\omega = \omega_{k+1} - \omega_k$; ϕ_k is a random phase angle uniformly distributed between 0 and 2π; and $S_g(\omega_k)$ is the value of the power spectral function for the ground acceleration calculated at ω_k. The Clough-Penzien (1975) power spectral density function for the ground acceleration is used to generate the ground motion. It is defined as:

$$S_g(\omega) = S_0 \frac{1 + 4\zeta_g^2 \left(\frac{\omega}{\omega_g}\right)^2}{\left[1 - \left(\frac{\omega}{\omega_g}\right)^2\right]^2 + 4\zeta_g^2 \left(\frac{\omega}{\omega_g}\right)^2} \frac{\left(\frac{\omega}{\omega_f}\right)^4}{\left[1 - \left(\frac{\omega}{\omega_f}\right)^2\right]^2 + 4\zeta_f^2 \left(\frac{\omega}{\omega_f}\right)^2} \qquad (2)$$

where S_0 is the amplitude of the white noise bedrock excitation; ω_g and ω_f are the frequencies of the first and second filters, respectively; and ζ_g and ζ_f are the damping of the first and second filters, respectively. The frequencies and damping of the filters can be estimated using a collection of actual records.

Based on the study of 118 soil-site records, Lai (1982) proposed a mean value for ω_g and ζ_g equal to 3.04 Hz and 0.32, respectively. The coefficients of variation of ω_g and ζ_g were suggested to be 0.425 and 0.426, respectively. The value of ω_f is taken equal to 0.1 ω_g and ζ_f is considered equal to ζ_g. The probability distribution of the filter parameters is assumed to be normal distribution. Creating the artificial ground motion is based on generating random numbers representing ω_g and ζ_g using the mean, standard deviation and probability distribution type for each parameter. Using these sets of random numbers, different power spectral functions are generated using Eq. (2) and different ground motion records are generated using Eq. (1). To model the time varying intensity of a typical earthquake, the stationary acceleration time history generated by Eq. (1) is typically multiplied by a suitable envelope nonstationary function. A simple and realistic form for this envelope function $\psi(t)$, is defined by Sues et al. (1985) as:

$$
\begin{aligned}
\psi(t) &= \left(\frac{t}{t_1}\right)^2 \quad ; \quad 0 \le t \le t_1 \\
&= 1 \quad ; \quad t_1 \le t \le t_2 \\
&= e^{-c(t - t_2)} \quad ; \quad t_2 \le t
\end{aligned}
\tag{3}
$$

where t_1 and t_2 are the rise and decay times of the ground motion, t_2-t_1 is the strong shaking duration and c is a decay parameter. Sues et al. (1985) used values of t_1 = 1.5 s and c = 0.18. They also recommended a mean duration of the strong ground shaking of 10.0 s for soil sites.

PERFORMANCE EVALUATION

The performance of the frames is evaluated in terms of the maximum storey drift, the damage index and the force-displacement results from the static pushover analysis for the original existing frame as well as the retrofitted frames. The results of the pushover analysis are presented first, followed by selected results from the probabilistic analysis of the seismic response for some of the retrofitted frames.

Pushover analysis

The results from the pushover analysis presented in Fig. 3 show that the original frame (Frame F) sustained a lateral yield load of approximately 0.14 W and an ultimate lateral load of 0.177 W. The total weight of the structure is denoted by W. The results also show that increasing the strength by 30% (Frame F1) increased the yield load to 0.15 W and the ultimate lateral load to about 0.195 W. The drift corresponding to various values of lateral load is reduced as compared to the drift of the original frame. For example, for a lateral load of 0.15 W, the roof drift equal to 1.4 % and 0.4% for Frame F and Frame F1, respectively.

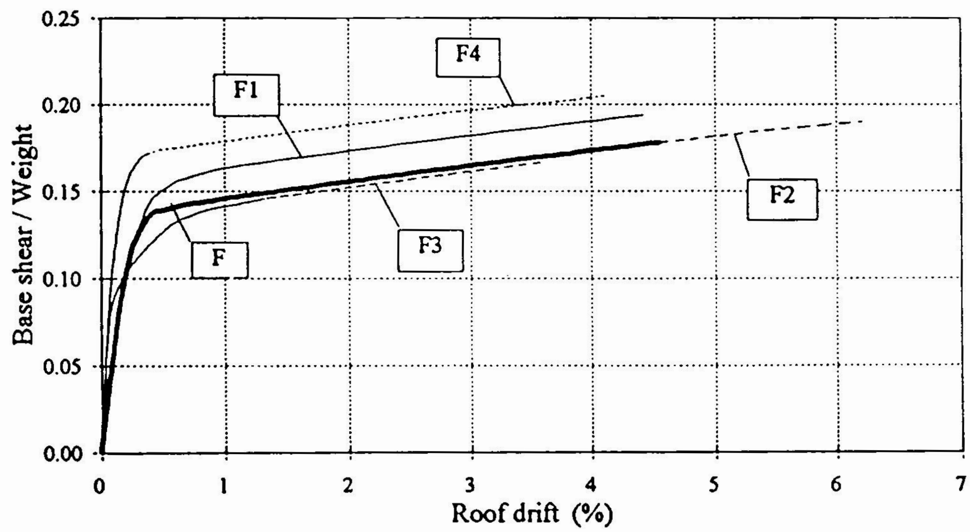

Figure 3 Performance curve from nonlinear pushover analysis

As would be expected, increasing the column ductility by 100% (Frame F2), had no effect on the yield load. However, the maximum roof drift increased from 4.5% to about 6.2%. The ultimate sustained load was approximately equal to 0.19 W. Increasing the stiffness of the columns without increasing the ductility or strength (Frame F3) reduced the yield load, the ultimate load and the maximum drift of the frame. However, increasing the stiffness of the columns with their strength (Frame F4), which is the practical case, increased the lateral yield load to 0.17 W and the ultimate lateral load to about 0.21 W. Results obtained from the nonlinear pushover analysis suggest that upgrading the lateral resistance of structures by increasing strength or strength and stiffness (Frames F1 and F4) are the most effective techniques to retrofit the frame discussed in this application as both approaches result in higher lateral load carrying capacity.

Seismic analysis

Frames F, F1, F2 and F3 are selected to undergo a probabilistic seismic analysis. The frames are subjected to the generated set of ground motion and a probabilistic analysis is carried out to analyze the effects of increasing columns strength, ductility or stiffness on the damage index and story drift. Fig. 4 shows that increasing the strength of the columns (Frame F1) reduces the story drift for all values of PGA. At PGA of 0.35 g, the story drift is 2.08% and 1.94% for Frames F and F1, respectively. This represents a reduction of about 7%. By increasing column ductility (Frame F2), the frame tends to experience high values of story drift at PGA levels greater than 0.2 g. For example, at PGA of 0.35 g, the story drift is 2.43% for Frame F2 which represents an increase of 16% in the story drift over Frame F. By increasing column stiffness (Frame F3), the story drift increases for high PGA levels. For example, at PGA of 0.35g, the story drift increases to 2.25% by increasing column stiffness which corresponds to an increase of 8% over the drift of Frame F. The poor performance of the frame associated with the stiffness increase is due to the increased demand corresponding to stiffness change.

The relationship between the damage index and the PGA for different frames is shown in Fig. 5. The figure indicates that the mean value of the damage index of the existing frame is reduced for all levels of PGA by either increasing the strength or the ductility. For example, a PGA equal to 0.35 g results in a mean value of the damage index of 0.373, 0.306 and 0.281, 0.45 for Frames F, F1, F2 and F3, respectively. This indicates a reduction in the damage index of 18% and 24.6% for Frames F1 and F2, respectively as compared to the damage index of the existing frame F. The results also indicate an increase in the damage index of 20.6% for Frame F3. As expected, the increase in strength decreases the damage. Similarly, the increase of ductility decreases the damage in spite of increased drift values due to the improved energy dissipation capacity. However, due to the increased demand created by the increase in column stiffness in the case of increased stiffness without increasing the strength, damage is increased.

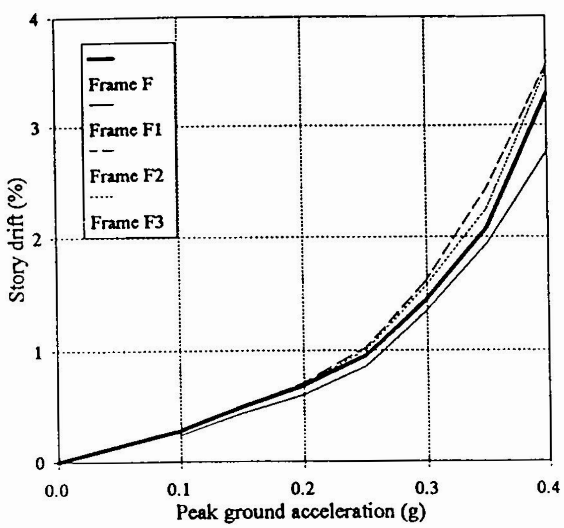

Figure 4 Mean value of Story drift

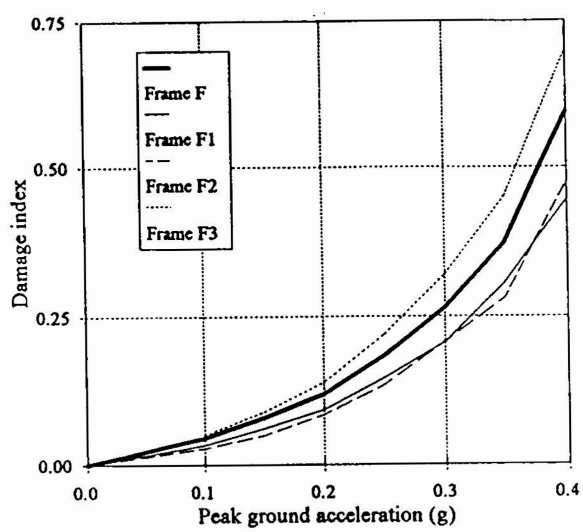

Figure 5 Mean value of damage index

The probability of exceedance curves for the story drift for Frames F, F1 and F2 for PGA of 0.35 g are shown in Fig. 6. For a PGA of 0.35 g, the probabilities of exceeding 2% story drift are 28%, 22% and 45% for Frames F, F1 and F2, respectively. This indicates a reduction in story drift by 21.5% for Frame F1 and an increase by 78.5% for Frame F2. Fig. 7 shows the probability of exceedance curves for the damage index for PGA of 0.35 g. The probabilities of exceeding 0.5 damage index are 13%, 9% and 9% for Frames F, F1 and F2, respectively. This indicates a reduction in the damage index of 31% for both Frames F1 and F2. The two retrofitting schemes result in a significant decrease in the probability of exceeding high values of the damage index.

The results of the probabilistic analysis show that increasing either the strength or the ductility of the columns resulted in a significant reduction in the damage index. As would be expected, the analysis shows that increasing the strength of the reinforced concrete columns resulted in a lower values for the story drift, while increasing the ductility of the columns resulted in a higher story drift. This indicates that in the case of the three story office building considered in this study, the increase in strength has a more desirable effect on the seismic performance of the structure.

CONCLUSIONS

Increasing the strength of reinforced concrete columns in the low-rise building considered is a more effective retrofitting technique for reducing the drift and damage. Increasing the ductility is associated with high drift and the potential for lower damage level. In this case, early collapse may be prevented and the damage level is reduced due to increased energy dissipation. Increasing the stiffness of columns only was detrimental as the frame attracted higher seismic forces. An important outcome of the analysis is that special care has to be taken in the selection and design of a specific retrofitting technique. It is necessary to evaluate the implications of the retrofit schemes on the performance of the structure. The prediction of drift and damage level provided a simple and effective criteria for the selection and design of the retrofit scheme.

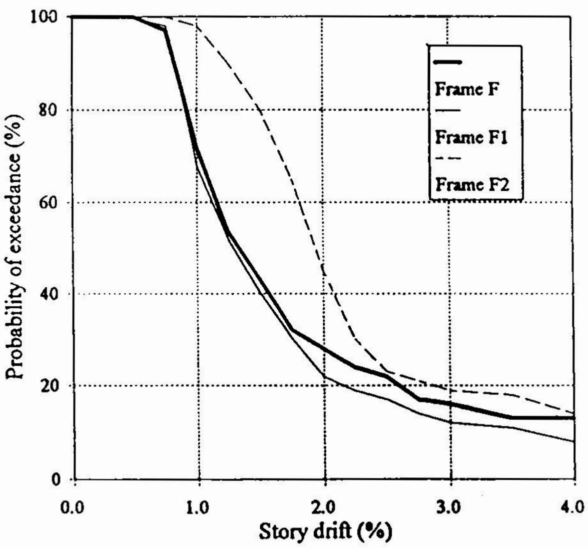

Figure 5 Probability of exceedance of story drift

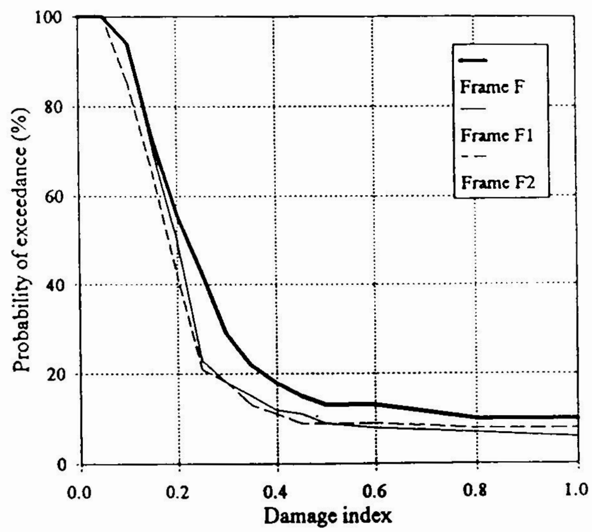

Figure 6 Probability of exceedance of damage level

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