Seismic Evaluation of an 8-storey FRP Retrofitted RC Frame

A. Eslami & H.R. Ronagh
The University of Queensland, Australia

A. Dalalbashi
Yazd University, Iran

SUMMARY:
The results of a numerical investigation pertaining to retrofit an 8-storey intermediate RC frame using carbon fiber reinforced polymer are presented. Considering the fact that beam-column joints are often accounted as one of the most critical components of RC structures in terms of local and global performance, the strengthening design strategy focuses on the relocation of plastic-prone regions away from the column faces more towards the beams. In order to pursue this strategy, composite sheets are applied at the top and bottom sides of the plastic hinge regions of beams. The additional flexural stiffness generated by the composite sheets is calculated comparing the moment-rotation of the FRP retrofitted and the original joints obtained from the finite element analysis. Pushover results of the retrofitted frame indicate that a strengthening strategy which follows the strong column-weak beam design philosophy, could improve the seismic performance and load carrying capacity of the frame significantly.

Keywords: CFRP, beam-column joints, FE analysis, retrofitting, pushover analysis

1. INTRODUCTION

Recent earthquakes revealed the essential need of retrofitting for many existing reinforced concrete structures. In addition to pulse-type ground motions which are characterized by the existence of a large long period pulse in the velocity time history, changes in seismic hazard levels, design methods, and serviceability requirements, are other reasons for retrofitting a code-compliant structure subjected to an ordinary earthquake.

During the last couple of decades, application of composite materials for retrofitting/upgrading of reinforced concrete (RC) buildings has experienced a sharp increase. Compared to other building materials, FRPs offer several advantages; such as, possessing high tensile strength, low specific weight, high resistance to corrosion, and ease of application. In an experimental study, Balsamo *et al.* (2005) assessed the seismic performance of a full-scale RC structure repaired using carbon fibre reinforced (CFRP) laminates and wraps. Their results proved that a large displacement capacity exists in the repaired structure while no reduction of strength is seen after the application of FRP at beam-column joints and walls. In another experimental study, Di Ludovico *et al.* (2008) investigated seismic retrofitting of an under-designed, full-scale RC structure with FRP wrapping. Their research confirmed the effectiveness of FRP in confining the concrete and in turn improving the global performance of the structure in terms of ductility and energy dissipating capacity. Recently, Niroomandi *et al.* (2010) investigated the seismic performance of an ordinary RC frame. Their pushover analysis showed that relocating plastic hinges away from the column faces through web-bonded FRP retrofitting of joints in an 8-storey frame increased the lateral load carrying capacity and seismic behaviour factor by 40% and 100%, respectively. However, it is worth mentioning that the web-bonded technique comes with certain limitations in practical applications.

Among the methods of retrofitting, increasing the level of steel reinforcing in the critical regions of beams near the joint region, has been suggested as an effective method in relocating the plastic hinge away from the column faces (Paulay and Priestley, 1992). In addition to increasing the lateral load carrying capacity of the structure, this method can also prevent the undesirable failure mode of weak-column strong-beam. Installation of FRP sheets on the exterior surfaces of beams and columns provides an opportunity for strengthening RC joints through the relocation of plastic hinges towards the beams. This type of retrofitting was confirmed in a numerical study conducted by the authors (Dalalbashi *et al.*, 2012) using different configurations of CFRP sheets on the flanges of the beams. In order to investigate the overall behaviour of RC structures retrofitted at beam-column joints using the aforementioned technique; this study is focused on the seismic assessment and load carrying capacity of code-compliant RC structures. For this purpose, an 8-storey RC moment resisting frame designed by Maheri and Akbari (2003) was selected as a case study. This frame was designed based on intermediate (moderate) seismic provisions described in ACI 318-95 (1995). The moment-rotation curves of the retrofitted and original joints are determined using a detailed FE analysis. In order to consider the effect of CFRP retrofit in the joint properties of strengthened frame, the additional stiffness generated by the composite materials is imported into the RC frame model in a pushover analysis (Niroomandi *et al.*, 2010). Through the comparison of the pushover results of the FRP-retrofitted and the original frame, the effectiveness of this technique in improving the seismic performance of the RC frames is investigated.

2. FE ANALYSIS OF THE ORIGINAL AND RETROFITTED JOINTS

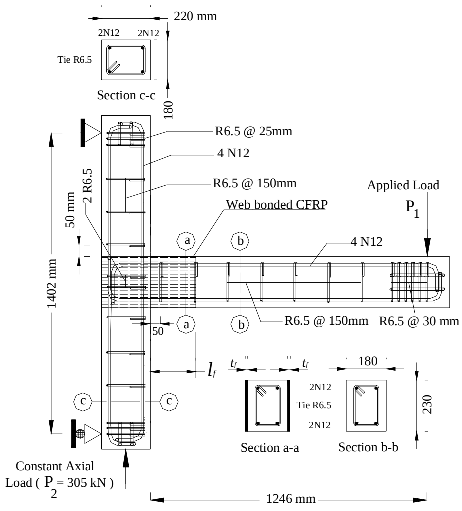
The retrofitting strategy aims at increasing the moment capacity of beam members at the both ends so that the formation of nonlinear plastic hinges occurs at a distance away from the joint core. Externally bonded CFRP sheets were used for this purpose. In order to calculate the additional stiffness provided by application of composite materials at the joints, a detailed linear and nonlinear finite element analysis of original and retrofitted joints was carried out using ANSYS (2009). The effect of the application of these CFRP sheets on the mechanical properties of RC frame was considered through the implementation of this additional stiffness into the nonlinear static analysis of the frame.

2.1. Experimental Calibration

It is common practice for the first step of every numerical study to be the verification of the analysis results through a comparison with an experimental investigation. In this study, an experimental study carried out by Mahini and Ronagh (2011) was selected in order to validate the finite element results and the analysis parameters. For this purpose, their specimen RSM2 was selected. In their study, the scaled-down beam-column joints were retrofitted using web-bonded FRPs in order to relocate plastic hinges away from the joint core of deficient exterior beam-column sub-assemblage. The details of CFRP strengthened beam-column joint tested in their study were demonstrated in Fig. 2.1.

The compressive strength of concrete was measured to be about $40.75\ MPa$. In addition, the yield steel strengths of longitudinal and transverse reinforcement in the beam-column joints were $500\ MPa$ and $382\ MPa$, respectively. In this study, the commonly used Hognestad’s model (Hognestad *et al.*, 1955) was used for the stress–strain curve of concrete in which the strain under uniaxial stress conditions corresponding to the concrete compressive strength was taken as 0.002. This value is recommended by Park and Paulay (1975) and many other researchers (Lam and Teng, 2003; Mander *et al.*, 1988) for normal concrete. The ultimate concrete strain was assumed to be 0.0038. The simplified bilinear model with strain hardening was also used to simulate the behaviour of longitudinal steels. For shear reinforcements, an elastic-perfectly plastic model was used, according to the test results reported by Mahini and Ronagh (2011).

ANSYS program (2009) was employed to perform the nonlinear FE analysis. All steel bars and stirrups were modelled using LINK8 truss element. In addition, SOLID65 element was employed to model concrete. This element, which is capable of modelling both cracking in tension and



Specimen	l_f (mm)	No. of ply	t_f (mm)
CSM0	-	-	-
RSM1	350	1	0.165
RSM2	200	3	0.495

Figure 2.1. The CFRP retrofitted beam-column joint tested by Mahini and Ronagh (2011)

crushing in compression, has been especially designed for modelling concrete in ANSYS. FRP composites were modelled using an eight-node 3D solid element called SOLID45. This multi-layer element is defined by eight nodes. This element, which is normally used to represent bilinear anisotropic materials, was reported as the most suitable element in ANSYS for modelling the behaviour of FRP (Mahini and Ronagh, 2011; Mostofinejad and Talaeitaba, 2006; Niroomandi *et al.*, 2010; Parvin and Granata, 2000). The above mentioned SOLID45 is also employed for the steel plates, which were added at the support locations of the column to provide a more even stress distribution over the support area. The behaviour of CFRP materials was modelled assuming anisotropic material behavior called ANISO (Mahini and Ronagh, 2011; Mostofinejad and Talaeitaba, 2006). This model allows introduction of the mechanical properties of FRPs in tension and compression in different directions. The mechanical properties of CFRP fibres used by Mahini and Ronagh (2011) are presented in Table 2.1. It is worth mentioning that these values satisfy the consistency equations necessary for an anisotropic material like ANISO in the nonlinear analysis, as described in ANSYS and stated by Kachlakev *et al.* (2001). The other assumptions for numerical modelling were the same as those implemented by Mahini and Ronagh (2011).

Table 2.1. Mechanical properties of CFRP fibres

Tensile strength, f_{fr} (MPa)	Ultimate tensile strain, ε_{fr}	Tensile modulus E_f (MPa)	Thickness t_f (mm)
3900	0.0155	240,000	0.165

Numerical analyses of the tested specimens were carried out according to aforementioned assumptions. In the nonlinear analyses, the loads were applied step by step using the modified Newton-Raphson method to arrive at the solution. A displacement control method was used for loading in order to avoid convergence problems. Fig. 2.2 compares the beam tip load-displacement curves obtained from the nonlinear FE analysis in the current study with that extracted from the experiments of Mahini and Ronagh (2011). Good agreement between the two curves proves reliability of the adopted FE analysis. This is particularly so when this agreement is measured in terms of the ultimate strength.

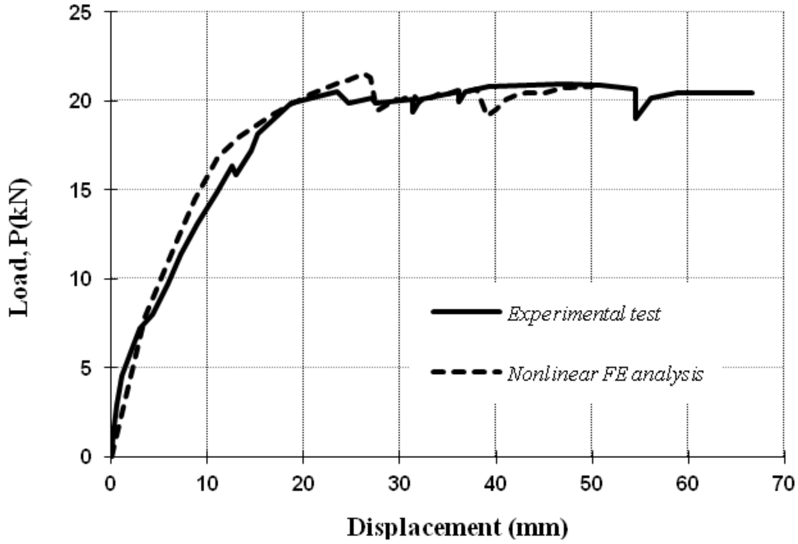
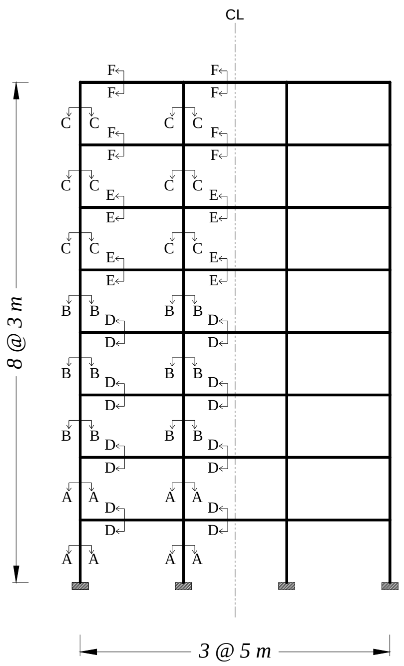
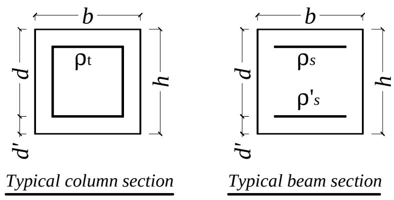


Figure 2.2. Comparison of load-displacement curves for specimen RSM2.

2.2 Numerical Analyses of Original and Retrofitted Joints

Following the validation of nonlinear FE parameters, the numerical analysis of the original and FRP retrofitted joints of the selected RC frame was carried out in order to calculate the additional stiffness generated by composite retrofit as well as thickness of CFRP sheets. The reinforcement details and dimensions of the frame members which were also used in FE analysis are given in Fig. 2.3. The compressive strength, f_c' and tensile strength, f_t of concrete were $27.46\ MPa$ and $3.67\ MPa$, respectively. Furthermore, the yield stress of steel bars was taken as $412\ MPa$ (Maheri and Akbari, 2003; Niroomandi *et al.*, 2010).





Section	b	h	D	ρ_t	ρ_s	ρ_s'
A-A	700	700	650	0.019	-	-
B-B	500	500	450	0.022	-	-
C-C	500	500	450	0.013	-	-
D-D	500	500	450	-	0.012	0.009
E-E	500	500	450	-	0.011	0.0071
F-F	500	500	450	-	0.0071	0.004

Figure 2.3. Reinforcement details and dimension of the selected moment resisting frame (Maheri and Akbari, 2003)

In numerical modelling, the joints were cut out from the inflection point to simulate the real performance of subassembly under seismic actions. For the sake of example, the FE model of a typical exterior joint is illustrated in Fig. 2.4. The boundary conditions at the column supports are similar to the actual behaviour of the structure during seismic loading. The elements and parameters employed for nonlinear FE analysis was similar to the one applied in the verified model. The LINK8 and SOLID65 were used to model concrete material and steel reinforcement, respectively. Steel plates were also modelled at the column support in order to eliminate stress concentration at that point. A bilinear model with strain hardening was used in the nonlinear analysis for modeling the steel reinforcement. The concrete nonlinear behaviour was simulated with the Hognestad model (Hognestad *et al.*, 1955).

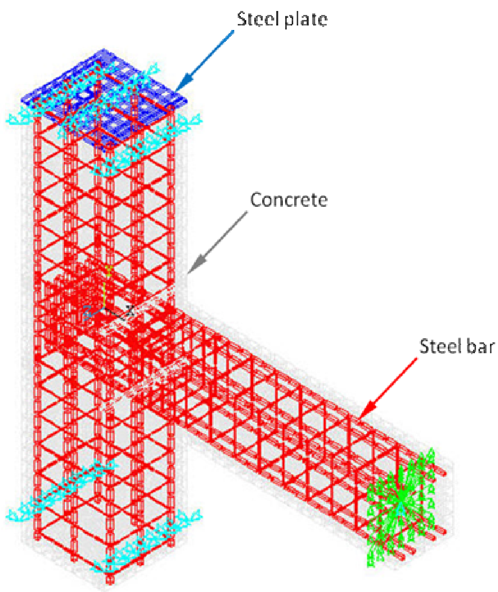


Figure 2.4. The FE model of a typical exterior beam-column sub-assemblage

In order to consider the effect of axial forces on the nonlinear analysis, the columns was subjected to a constant axial load equal to $0.2 f_c' A_g$ where f_c' is the concrete compressive strength and A_g represents the gross area of the column cross section (Ghobarah and Said, 2001). This load was applied in the form of surface pressure to the column. Reported experimental studies confirm that the application of column axial load increases the confinement effect of the beam-column joint area to a certain degree and results in increasing the shear strength of the joint (Prota *et al.*, 2003). As shown in Fig. 2.5. a monotonically increasing static load was applied at the beam tip up to the failure simulating the seismic load applied to the frame during a ground motion. The load was applied in the form of displacement to prevent convergence problems and the reaction was calculated (see Fig. 2.4.).

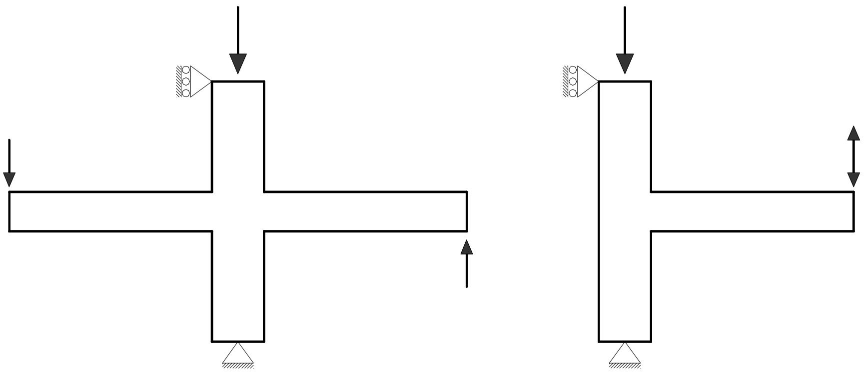


Figure 2.5. Schematic illustration of separated joints and applied loads in FE modelling

Composite sheets were applied based on a configuration suggested in an early research carried out by the authors (Dalalbashi *et al.*, 2012). Pursuing the aim of retrofitting and to achieve a good bond, they suggested a new novel design in which composite sheets are inserted into a groove created into the column concrete, as shown in Fig. 2.6. At the time of practical implementation, the groove was filled by injecting epoxy resin.

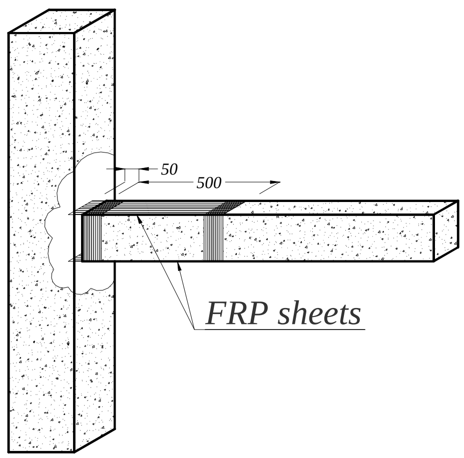


Figure 2.6. Retrofitting configuration in order to relocate plastic hinge away from column

Similar to the verified model, the multilayer SOLID45 element with ANISO material was used to model the behaviour of CFRP. The mechanical properties of composite materials adopted were assumed to be identical to those given in Table 2.1. Since the composite materials used were unidirectional, their properties were rotated for FRP wrapping in the numerical modelling.

The thickness of composite sheets should be sufficient in order to reassure the relocation of plastic hinges. This threshold thickness achieved by increasing the composite thickness in nonlinear analysis. Although not economical, the thicknesses of CFRP sheets were rounded to simplify the numerical analysis and were taken as 10 and 5 layers for the first four and the second four stories, respectively. It is worth mentioning that after the relocation of plastic hinges, a sensitivity analysis showed a negligible effect of composite thickness in the force-displacement curve. The additional stiffness generated by CFRP retrofits was calculated by subtraction of the moment values in moment-rotation curves of the original and the retrofitted joints at the same rotation values. Based on the concept of chord rotation adopted in the seismic rehabilitation codes such as FEMA-356 (American Society of Civil Engineering, 2000) and ATC-40 (1996), the rotation was calculated as the ratio of the beam tip vertical displacement to the horizontal distance of the beam tip load from the column face. As an example, the moment-rotation curves obtained from the FE analysis of the exterior joint at the first level after and before retrofitting together with the additional stiffness generated from the composite application are given in Fig. 2.7.

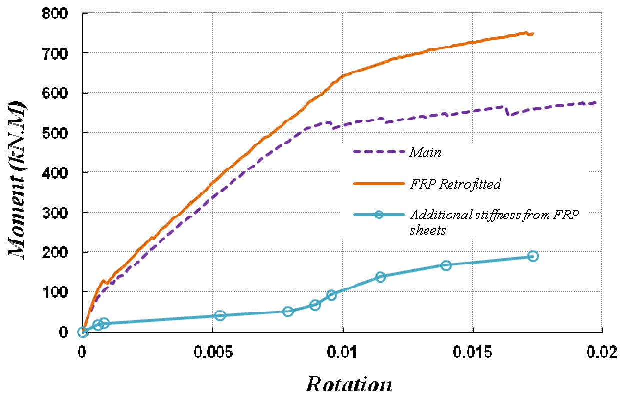


Figure 2.7. Moment-rotation curves of the original and the retrofitted exterior joint at the first story

3. NONLINEAR STATIC (PUSHOVER) ANALYSIS

For each considered structure, nonlinear static analysis was carried out in SAP 2000 (Computers and Structures Inc, 2009) and the base shear-roof displacement (the so called pushover) curve was determined for each analysis. In order to consider the nonlinear plastic behaviour of each component, the force-displacement properties provided in FEMA-356 have been implemented to the critical regions of beams and columns. Flexural moment and axial moment hinges were assigned to the end sections of beams and columns respectively, taking into consideration beam and column dimensions and plastic hinge length. In this study, the simple, yet accurate relation given in Eqn. 3.1 was assumed as the plastic hinge length. This relation is also recommended by ATC-40 and by other researchers (Zou *et al.*, 2007). It should be noted that according to Paulay and Priestley (1992), Eqn. 3.1 results in accurate values for the conventional beam and column dimensions.

$$L_p = H/2 \tag{3.1}$$

In the above equation, L_p and H are the plastic hinge length and the height of section, respectively.

Pushover analysis consists of a monotonically increasing lateral load applied to the structure up to the failure in the presence of a constant gravity load. In this study, the total dead load plus 20% of the live load based on the Iranian seismic code (Permanent Committee for Revising the Iranian Code for Seismic Resistant Design of Buildings, 2005), is applied to the frame studied. For the seismic evaluation of a building, the lateral force profile applied to the building should represent, albeit approximately, the likely distribution of inertia forces induced during an earthquake. In a comparative study, Mwafy and Elnashai (2001) concluded that the inverted triangular distribution of lateral load provides better estimates of the capacity curve and seismic responses in comparison to a uniform distribution. In addition based on their study, while inverted triangular distribution is more practical than the multi-modal distribution, it would yield similar results. Therefore, an inverted triangular distribution over the height is used as the lateral load pattern. It should be mentioned that this load pattern is similar to the lateral load distribution used for the seismic design of considered structures and has been suggested in the Iranian seismic code. Also, the effect of $P-\Delta$ has been considered in all nonlinear analyses. The initial effective stiffness values of the members have been assumed according to ACI 318-95 (1995) provisions.

3.1. Original Frame

The nonlinear results of the original frame confirm a column side-sway mechanism of the selected frame. Despite the fact that the frame was designed according to the weak-beam strong column design philosophy (Maheri and Akbari, 2003), this undesirable type of failure was anticipated from the dimension of columns in the top stories compared to the two bottom stories. Pushover curve of the original frame is shown in Fig. 3.1.

3.2. Retrofitted joint

In order to consider the effect of FRP retrofit in the nonlinear analysis of the selected frame, plastic hinges in the beams were relocated to a distance of $500\,mm$ away from the column faces corresponding to the FRP sheet length. In addition, the equivalent stiffness provided by the application of FRP on top and bottom sides of beams was considered using SAP2000 nonlinear link element. In the numerical model, these elements were positioned at the location of beams' plastic hinges. As observed in Fig. 3.1., the pushover curve obtained from nonlinear static analysis of the retrofitted frame (first retrofitting design) showed a sudden drop of loading at displacement around 187 mm after which the curve rises due to the redistribution of forces. Similar to the original frame, considering the hinge damage statues in the frame members, confirm the formation of plastic hinges in the exterior column of the third storey. Technically speaking, this type of failure was expected in the retrofitted frame due to the higher moment capacity of CFRP retrofitted beams compared to the original ones. In addition, the lateral displacement of the frame decreased by roughly around 20%.

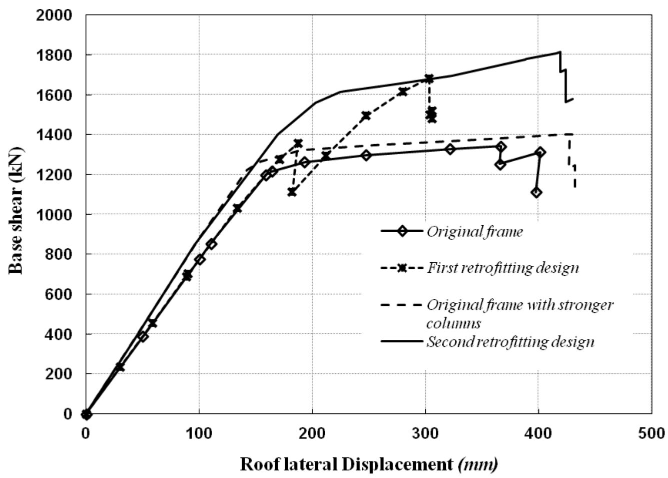


Figure 3.1. Comparison of pushover curves of the original and the retrofitted frames

To overcome the above drawback, in the second retrofitting design it was decided to strengthen the columns type B at the third to fifth stories in addition to the plastic hinge relocation of beams. For this reason, it was decided to increase the hinge properties of these columns to the level of column type A which was designed for the first two stories. This could easily be achieved by external attachment of composite materials or using concrete jacketing. The results of nonlinear analysis of the second retrofitting design indicated a substantial improvement in the seismic performance of the structure. The second retrofitting design resulted in an increase of 35% in the total lateral load with a slight increment in lateral displacement compared to the original frame. In addition, a desirable beam side-sway was substituted with devastating column side-sway failure. A comparison of all pushover curves obtained from nonlinear analysis could be made from Fig. 3.1.

In order to investigate the participation of column strengthening and plastic hinge relocation technique separately, in the increment of load carrying capacity, a numerical model of the original frame with modified column inelastic properties similar to the second design was created. The plastic hinges at beams were assumed to be located similar to the original frame. Although the nonlinear pushover analysis of this model resulted in a beam side-sway mechanism; the load carrying capacity increment was not considerable (see Fig. 3.1.).

Nonlinear pushover outcomes revealed that the plastic hinge relocation method accompanied by weak-beam strong-column design philosophy could be considered as an effective retrofitting technique in enhancing the load carrying capacity of structures.

4. CONCLUSIONS AND DISCUSSIONS

Seismic performance and load carrying capacity of an 8-storey RC structure retrofitted using composite materials in order to relocate plastic hinges away from the column faces into the beam was investigated numerically. The additional stiffness provided by CFRP sheets was calculated from the nonlinear FE analysis of beam-column connection sub-assemblage and then imported to the numerical model of frame. The results of pushover analysis showed a substantial increase in the load carrying capacity and presented a more favourable failure mechanism of the structure, in all cases but those in which s the strong-column weak-beam design philosophy was not adopted in the retrofitting design. Taking into consideration the advantages of plastic hinge relocation, it can be confidently stated that this method can be employed in order to improve the seismic performance of reinforced concrete structures. However, careful attention should be paid to the weak-column strong-beam design philosophy. If a column side-sway mechanism occurs, the structure could not benefit from all of the advantages that the plastic hinge relocation technique would offer. More research should be performed on the plastic hinge relocation technique using FRP as an alternative method of retrofitting RC structures in which the design has been based on the recent versions of seismic codes.

REFERENCES

ACI Committee 318 (1995). Building code requirements for structural concrete (ACI 318-95) and commentary (ACI 318R-95). Farmington Hills, Mich. : American Concrete Institute.

American Society of Civil Engineering (Asce) (2000). Prestandard and commentary for the seismic rehabilitation of buildings (FEMA-356). Washington, DC: Federal Emergency Management Agency.

Ansys Manual (2009). ANSYS, Inc. 12.1 ed.: Canonsburg, PA 15317, USA.

ATC (1996). Seismic evaluation and retrofit of concrete buildings(ATC-40). California: Applied Technology Council.

Balsamo, A., Colombo, A., Manfredi, G., Negro, P. and Prota, A. (2005). Seismic behavior of a full-scale RC frame repaired using CFRP laminates. *Engineering Structures.* 27:5**,** 769-780.

Computers and Structures Inc (2009). Static and dynamic finite element analysis of structures. *SAP 2000.* 14.1.0 ed. Berkeley (CA); .

Dalalbashi, A., Eslami, A. and Ronagh, H. R. (2012). Plastic hinge relocation in RC joints as an alternative method of retrofitting using FRP. *Composite Structures.*0.

Di Ludovico, M., Prota, A., Manfredi, G. and Cosenza, E. (2008). Seismic strengthening of an under-designed RC structure with FRP. *Earthquake Engineering & Structural Dynamics.* 37:1**,** 141-162.

Ghobarah, A. and Said, A. (2001). Seismic rehabilitation of beam-column joints using FRP laminates. *Journal of Earthquake Engineering.* 5:1**,** 113-129.

Hognestad, E., Hanson, N. W. and Mchenry, D. (1955). Concrete stress distribution in ultimate strength design. *American Concrete Institute Journal.* 27:Part 1, 4**,** 455-479.

Kachlakev, D., Miller, T., Yim, S., Chansawat, K. and Potisuk, T. (2001). Finite element modeling of reinforced concrete structures strengthened with FRP laminates. *SPR 316.* Oregon Department of Transportation Research Group and Federal Highway Administration.

Lam, L. and Teng, J. G. (2003). Design-oriented stress-strain model for FRP-confined concrete. *Construction and Building Materials.* 17:6-7**,** 471-489.

Maheri, M. R. and Akbari, R. (2003). Seismic behaviour factor, R, for steel X-braced and knee-braced RC buildings. *Engineering Structures.* 25:12**,** 1505-1513.

Mahini, S. S. and Ronagh, H. R. (2011). Web-bonded FRPs for relocation of plastic hinges away from the column face in exterior RC joints. *Composite Structures.* 93:10**,** 2460-2472.

Mander, J. B., Priestley, M. J. N. and Park, R. (1988). Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering.* 114**,** 1804-1826.

Mostofinejad, D. and Talaeitaba, S. B. (2006). Finite element modeling of RC connections strengthened with FRP laminates. *Iranian Journal of Science and Technology, Transaction B: Engineering.* 30**,** 21-30.

Mwafy, A. M. and Elnashai, A. S. (2001). Static pushover versus dynamic collapse analysis of RC buildings. *Engineering Structures.* 23:5**,** 407-424.

Niroomandi, A., Maheri, A., Maheri, M. R. and Mahini, S. S. (2010). Seismic performance of ordinary RC frames retrofitted at joints by FRP sheets. *Engineering Structures.* 32:8**,** 2326-2336.

Park, R. and Paulay, T. (1975). Reinforced concrete structures, New York, Wiley.

Parvin, A. and Granata, P. (2000). Investigation on the effects of fiber composites at concrete joints. *Composites Part B: Engineering.* 31:6-7**,** 499-509.

Paulay, T. and Priestley, M. J. N. (1992). Seismic design of reinforced concrete and masonry buildings, New York, Wiley.

Permanent Committee for Revising the Iranian Code for Seismic Resistant Design of Buildings (2005). Iranian code of practice for seismic resistant design of buildings (Standard No. 2800-05). 3rd ed. Tehran (Iran): Building and Housing Research Center (BHRC).

Prota, A., Nanni, A., Manfredi, G. and Cosenza, E. (2003). Capacity assessment of RC subassemblages upgraded with CFRP. *Journal of Reinforced Plastics and Composites.* 22**,** 1287-1304.

Zou, X. K., Teng, J. G., De Lorenzis, L. and Xia, S. H. (2007). Optimal performance-based design of FRP jackets for seismic retrofit of reinforced concrete frames. *Composites Part B: Engineering.* 38:5-6**,** 584-597.