Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1661

SEISMIC STRENGTHENING OF RC BUILDINGS USING CFRP

R. García López[1], I. Hajirasouliha[2], K. Pilakoutas[3] and M. Guadagnini[4]

ABSTRACT

A full-scale RC frame with inadequate detailing in the beam-column joints was tested on a shake table as part of a European research project. After the initial tests which damaged the structure, the frame was strengthened using CFRP and further tests were carried out. In this paper, nonlinear time-history analyses of the bare and FRP-strengthened building are presented. Bond-slip and bond strength degradation models under cyclic loading were implemented to simulate deficient beam-column joints. Experimental data from the shake table tests were used to calibrate the analytical models. The numerical models are used to assess the efficiency of the CFRP intervention using a set of real earthquakes. It is shown that the CFRP strengthening resulted in substantial improvement of the seismic performance of the deficient building.

Introduction

Extensive human and economical losses in recent major earthquakes in developing countries (Bam, 2003; Kashmir, 2005; China, 2008; Indonesia, 2009) have highlighted the seismic vulnerability of existing reinforced concrete (RC) buildings. Many of these structures have been designed according to old standards and often suffer from inadequate construction practices and poor quality materials. Consequently, these buildings possess deficient lateral load resistance, insufficient energy dissipation and can rapidly lose their strength during earthquakes leading to collapse. In recent years, the use of externally bonded fiber reinforced polymers (FRPs) has offered engineers a variety of possible solutions for strengthening existing structures. Most of the research efforts have focused on developing innovative techniques to increase the strength capacity and/or ductility of existing frame structures using FRPs (Pinto et al. 2002; Balsamo et al. 2005a; Balsamo et al. 2005b), whereas less research has been conducted on the efficiency of FRP-strengthening considering different seismic excitations.

This study aims to investigate analytically the ability of FRPs to improve the seismic behavior of deficient RC buildings. This will be done by using data from an experimental project

[1] Research student, Dept. of Civil & Structural Engineering, University of Sheffield, UK. Sir Frederick Mappin Building, Mappin Street, Sheffield, S1 3JD, UK
[2] Marie Curie Fellow, Dept. of Civil & Structural Engineering, University of Sheffield, UK
[3] Professor of Construction Innovation, Dept. of Civil & Structural Engineering, University of Sheffield, UK
[4] Lecturer, Dept. of Civil & Structural Engineering, University of Sheffield, UK

that was undertaken as part of the EU-funded Ecoleader Project (Chaudat et al. 2005). A full-scale one-bay two-story RC frame with inadequate detailing in the beam-column joints was tested on a shaking table at the CEA Laboratory in Saclay, France. The structure was designed and built according to typical old pre-seismic construction practice of southern Europe; hence it is thought to be representative of substandard buildings typically found in developing countries. After the initial series of experiments, the damage was repaired and the building was strengthened using Carbon FRP (CFRP) materials for further tests. In this paper, the bare and strengthened buildings are modeled using DRAIN-3DX software (Prakash et al. 1994). To simulate deficient beam-column joints, models of steel concrete bond-slip and bond strength degradation under cyclic loading are adopted, and experimental data from the shake table tests are used to calibrate the analytical models. These models are then used to predict the behavior of the strengthened structures when subjected to a series of earthquakes. The results are used to calculate the ductility demand and ability of the FRP strengthening systems to supply it.

Experimental Program

Geometry, Material Properties and Set-up of Experiments

In order to study the performance of existing substandard buildings and FRP strengthening techniques, an RC frame was tested on the AZALEE shaking table of the CEA Laboratory in Saclay, France, as part of the Ecoleader Program. The one-bay two-story frame was regular in plan and elevation, and was designed so that columns and joints experienced damage during the initial shaking. The general geometry, element sections and corresponding reinforcement are depicted in Figure 1. The mechanical properties of the materials obtained from tests of bars and concrete cylinders were, for the reinforcement steel f_y=551 MPa and f_u=656 MPa, and for the concrete f_c=20 MPa and E_c=25545 MPa. An additional mass of 9 tonnes was added at each slab using steel plates.

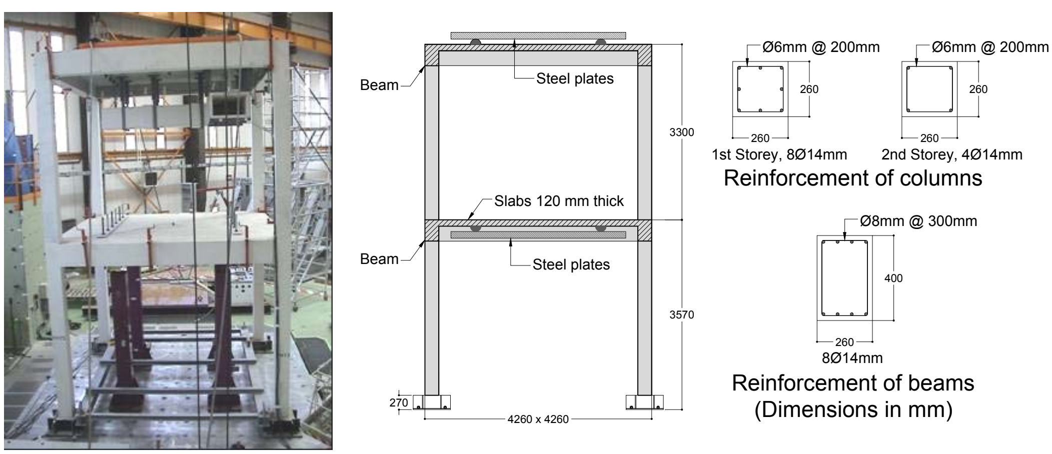

Figure 1. General view (left) and geometry (right) of the bare frame.

The structure was instrumented with displacement and acceleration transducers at each story to monitor the displacement history during the experiments. In this study, maximum drift was selected as an indicator of damage to structural and non-structural elements and is used for comparison with the results from nonlinear time-history analyses.

Tests on the Bare and FRP-strengthened Structure

Unidirectional horizontal input shaking table tests were carried out on the original building using increasing PGA accelerations ranging from 0.05g to 0.4g. A single artificial record was used, based on the Eurocode 8 soil type C spectrum (BSI, 2004). Natural frequencies of the structure were obtained using white noise as input signal before the start of each test. For this purpose, a low intensity excitation containing a frequency range of 0.5-50 Hz was used. The accelerations recorded at the base and at each story were then post-processed to identify the natural frequencies of the first two modes. As expected, significant damage was observed at the columns ends and beam-column joints after the initial tests (see Figure 2). Little damage was observed in the beams.

Figure 2. Damage in joints and columns after the test PGA=0.4g (notice bond-slip failure at the column-beam interface)

After the initial tests, the damaged structure was strengthened using FRPs (Figure 3 left). The main purpose of the rehabilitation was to produce a beam mechanism, which is in line with modern seismic design philosophy. Before the intervention, the damaged concrete was repaired and the main cracks filled by injecting epoxy resin. Concrete surfaces at the application zones were prepared to improve the adherence between the existing concrete and the fibers. It is well known that FRP strengthening can significantly increase the flexural capacity of columns (ACI 440 2008) As such, vertical strips of CFRP were attached at the interior and exterior faces of columns ends to enhance their flexure strength. Four transverse layers of CFRP fabrics (perpendicular to the column axis) were wrapped at the columns ends to increase their capacity and avoid possible buckling of the longitudinal sheets. In addition, to avoid a premature shear failure, beam-column joints at both stories were strengthened using vertical (parallel to the column axis) and horizontal CFRP sheets in the core zone (Figure 3 left). . Full details of the adopted strengthening strategy can be found in Chaudat et al. (2006).

After the strengthening, further shaking table tests were conducted with PGA accelerations ranging from 0.05g to 0.5g. Damage at the FRP strips was first detected at the 2nd story columns after the PGA=0.2g tests. No further damage was observed at the columns until after the final tests; however, significant cracking occurred at the beam ends (Figure 3 right). Therefore, the adopted strengthening strategy appeared to be effective in changing the plastic hinge mechanism from column-sway to beam-sway, which is in line with the strengthening goal.

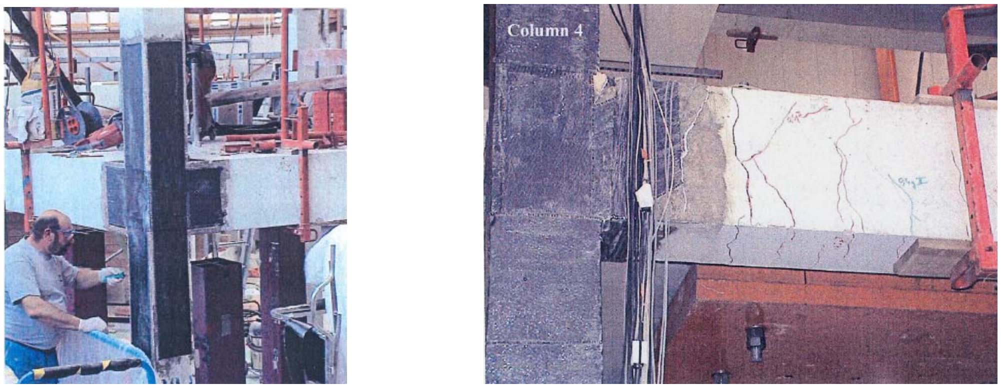

Figure 3. CFRP-strengthening of the frame (left), and corresponding damage at beam ends after the second series of tests (right) (from Chaudat et al., 2005)

Analytical Modeling

Modeling of the Bare and FRP-strengthened Frames

The experimental results on the bare and strengthened frames were used to calibrate numerical models developed in DRAIN-3DX software. Because of the symmetry in the structure, the frames were modeled in 2D for computational efficiency (Figure 4, left). Beams and columns were modeled using a fiber element (element 15) of distributed plasticity (Powell et al. 1994). The section comprises discrete steel and concrete fibers, which increases the accuracy of the analysis (Figure 4, right). Behavior of steel reinforcement and concrete was characterized by the constitutive models given in Eurocode 2 (BSI, 2004). The effect of accumulated stiffness degradation was included in the analyses by considering a stiffness degradation factor in the stress-strain relationship of concrete. Concentrated loads were assigned at intermediate nodes of the beams to simulate the distributed load from slabs and beams. Additional nodes added at the top and bottom of the outermost column elements simulated the actual geometry of columns and beam-column joints. The masses at each story were lumped at the two corresponding exterior nodes and calculated assuming a concrete density equal to 24 kN/m^3. Elastic damping was introduced using a stiffness and mass proportional Rayleigh damping model. Appropriate damping values (2 to 5%) were assigned to the first and second modes of vibration to calibrate the analytical models with the experimental results. Second order (P-Δ) effects were also included in the analysis.

Previous research (Kyriakides, 2008) showed the need of considering the additional deformations generated by bond-slip of steel bars. In the analytical models, deformations occurring at the joints were specified using zero-length connection hinges at column ends. The fiber properties used for the elements were chosen to model bond-slip within the beam-column joint and including degradation parameters. Partial degradation was initially assigned to both bond-stiffness and bond-strength. Gap properties were assigned at the connection face to simulate crack opening.

In order to model the strengthened frame, additional nodes were added to define the

FRP-confined zones of columns. The effect of the FRP confinement was introduced using the constitutive model for confined concrete proposed by Mortazavi (2003). The ultimate compression strength and ultimate strain of the confined concrete used for the analysis were f^*_{cc}= 30 MPa and ε^*_{cc}=0.010, respectively. Bond degradation parameters were adjusted to reflect the effect of the additional confinement provided by the FRPs.

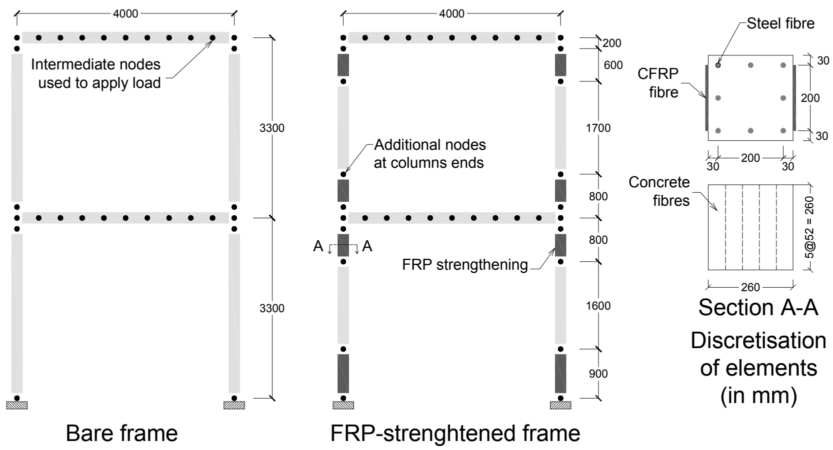

Figure 4. Frame models (left) and fiber elements (right) used in DRAIN-3DX

Calibration of the Analytical Models

The experimental data from the shake table tests were used to calibrate the analytical models developed in DRAIN-3DX. Natural frequencies obtained from white noise tests are compared with analytical results in Table 1. As can be seen, for the first two modes of vibration, the dynamic properties of the bare and strengthened frame are well captured by the analytical models.

The experimental and analytical displacement histories of the 1st and 2nd stories for the bare and FRP-strengthened buildings are compared in Figures 5 and 6. Only the results for PGA of 0.05g and 0.4g are presented in this paper, which are representative of the elastic and inelastic range of behavior. In spite of some differences, the illustrated results indicate that the predicted and measured displacements compare reasonably well along the entire time duration of the excitation. The test results show that the 2nd story displacements of the bare frame are larger than those of the 1st story. Conversely, for the FRP-strengthened frame, 2nd story displacements are smaller than the 1st story displacements. It should be mentioned that in Figure 5 the experimental response of the 2nd story becomes horizontal at PGA=0.4g due to the failure of the displacement transducer.

Table 1. Structural period of the frames obtained from tests and analysis (in s).

Mode No.	Bare frame		FRP strengthened	
	Tests	Analysis	Tests	Analysis
1	0.53	0.51	0.73	0.70
2	0.18	0.19	0.23	0.25

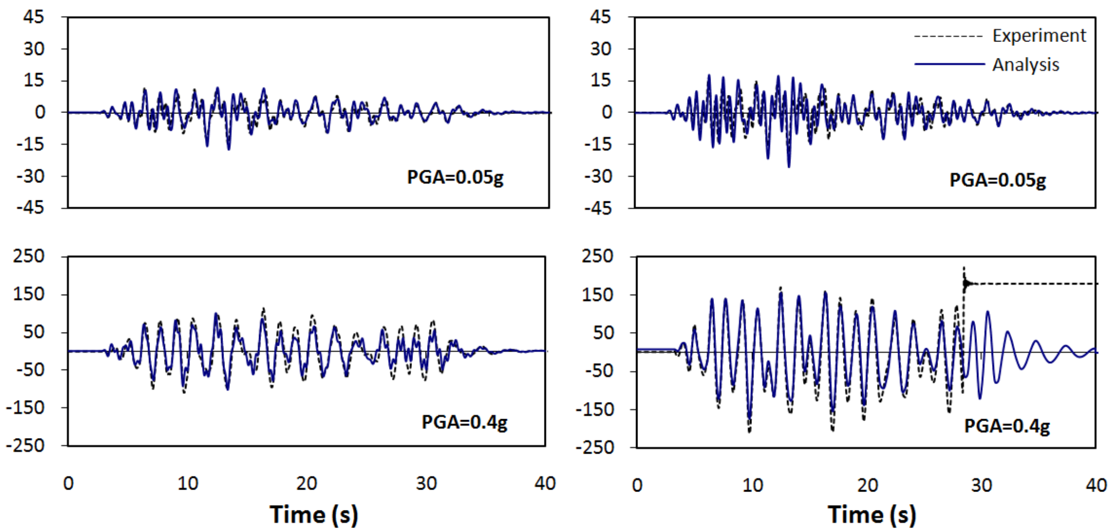

Figure 5. Results from nonlinear time history analysis for 1st (left) and 2nd (right) stories, bare frame

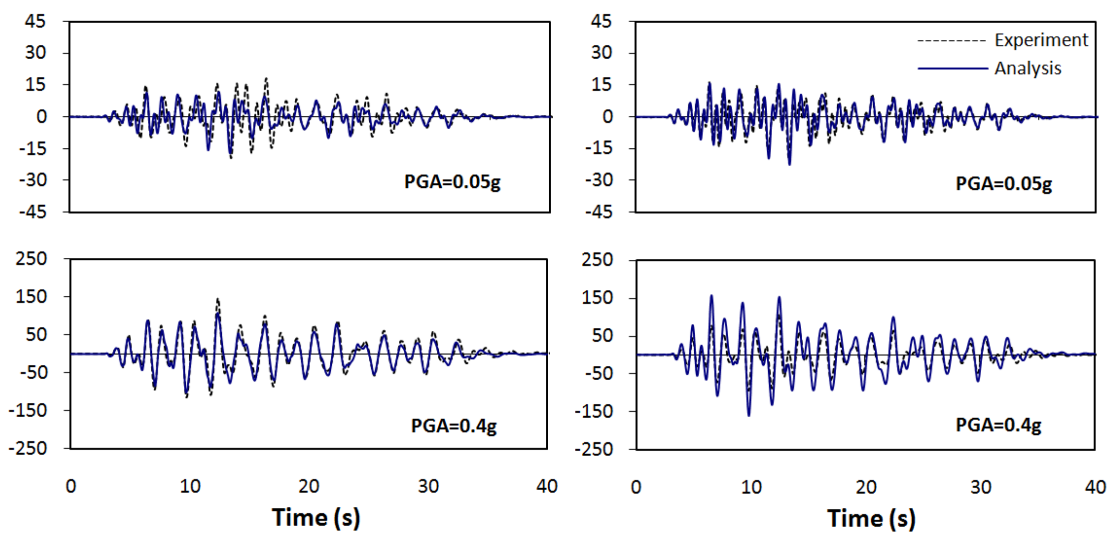

Figure 6. Results from nonlinear time history analysis for 1st (left) and 2nd (right) stories, CFRP-strengthened frame

Table 2 compares the maximum inter-story drifts obtained from the tests and analytical models for different PGA. In general terms, the analytical models for both bare and strengthened structures tends to slightly underestimate the story drift for the 1st story, while the drift response for the 2nd story is overestimated. Based on results, it can be concluded that the analytical models of both bare and strengthened frames provide a reasonable estimate of the maximum drifts for earthquake excitations with different PGA level. The results also indicate that the FRP application significantly decreased the 2nd story drift, while it increased the 1st story drifts to some extent. This may be attributed to the fact that the FRP strengthening changed the dynamic behavior of the bare frame by preventing the joint failure at the first story. It is worth mentioning that the calibration performed in this section does not represent verification of the analytical model or its results.

Table 2. Inter-story drift results from experiments and analysis (in %).

PGA	Floor	Bare frame		FRP-strengthened	
		Tests	Analysis	Tests	Analysis
0.05	2nd	0.2	0.2	0.1	0.1
	1st	0.2	0.3	0.4	0.3
0.20	2nd	1.2	1.5	0.6	0.8
	1st	1.4	1.3	1.4	1.3
0.40	2nd	3.9	3.9	1.3	2.2
	1st	1.9	1.6	2.5	2.2

Frame Performance using Real Earthquake Records

In order to investigate the efficiency of the FRP strengthening on the behavior of the structure under real seismic excitations, the calibrated analytical models were subjected to a set of six real seismic records as listed in Table 3 (PEER, 2009). These excitations correspond to sites having a soil profile similar to the Eurocode 8 soil type C; therefore, they are expected to have similar frequency content.

The efficiency of the rehabilitation strategy is investigated by exploring the expected structural damage experienced by the bare and FRP-strengthened models during the seismic excitation. To achieve this, demand to capacity ratios of curvature ductility, $(D/C)_{\mu k}$, are compared in Figure 7 for the bare and FRP-strengthened columns of the frames subjected to the six seismic excitations. It should be noted here that it is assumed that the structure will achieve its flexural capacity. This is not necessarily true for the bare frame, which has detailing errors. The results indicate that D/C ratios of the FRP-confined columns are significantly lower than those of the bare model by a factor ranging from 2 to 3. Therefore, it can be concluded that structural damage in the frame rehabilitated with the strengthening scheme can be expected to be considerably lower than in the original frame.

Table 1. Characteristics of the real earthquake records used in the evaluation.

ID	EQ name	M	Station	Dist.* (km)	PGA	Duration (s)
1	1989 Loma Prieta	6.9	Capitola	14.5	0.46	40.0
2	1989 Loma Prieta	6.9	Saratoga Aloha Av	12.4	0.51	40.0
3	1994 Northdridge	6.7	Canoga Pk	15.8	0.42	25.0
4	1994 Northdridge	6.7	N. Saticoy Street	13.3	0.42	30.0
5	1994 Northdridge	6.7	Castaic Old Ridge R	24.6	0.57	40.0
6	1987 Superstition Hill	6.7	El Centro Imp Co	13.9	0.35	40.0

*Closest distance to fault rupture

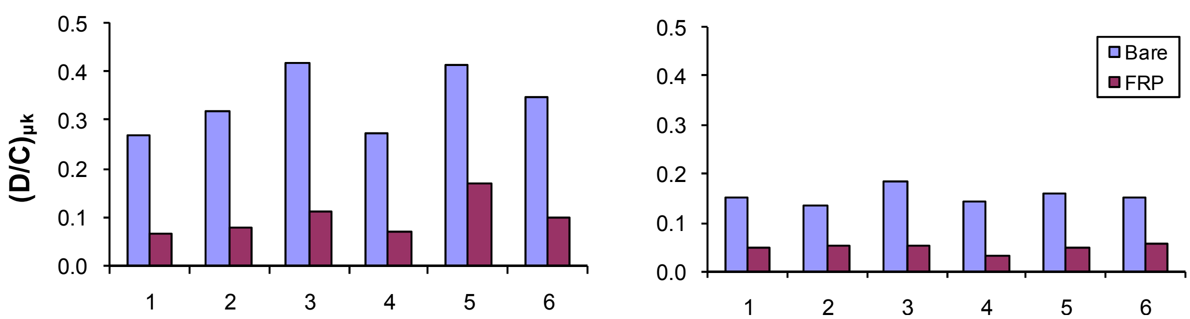

Figure 7. Demand to capacity ratios (D/C) for strain, curvature ductility and displacement ductility for 1st (left) and 2nd (right) stories columns, six earthquakes.

Recent design guidelines, such as FEMA 356 (2000) place limits on acceptable values of response parameters implying that exceeding these limits is a violation of a performance objective. Among various response parameters, inter-story drift is considered as a reliable indicator of damage to non-structural elements and is widely used as a failure criterion. According to FEMA 356, maximum transient drifts of 1%, 2% and 4% correspond to Immediate Occupancy, Life Safety, and Collapse Prevention performance levels, respectively. Therefore, the efficiency of the strengthening technique could be examined in terms of displacement response parameters of the bare and strengthened models.

The maximum inter-story drifts of the two stories of the original and FRP-strengthened models are shown in Figure 8 for the set of six seismic records. The results show that using FRP strengthening reduces significantly the maximum inter-story drift of the 2nd story, particularly for the higher energy input earthquake excitations (records 1, 3 and 5). However, the maximum inter-story drift of the 1st story is slightly higher in the strengthened structure. These results are congruent with those from the experimental tests (see Table 2), where the drifts for the 1st and 2nd stories decrease and increase respectively after the application of FRP.

Figure 8 indicates that FRP always results in a substantial reduction of inter-story drifts greater than 2% (i.e. Collapse Prevention performance level). This implies that the strengthening method is able to improve the seismic behavior of structures, especially when they are subjected to strong earthquake excitations. The results show that for the higher energy input records (records 1, 3 and 5), the seismic performance of the deficient frame was improved from Collapse Prevention to Life Safety performance level, thus resulting in a significant improvement in performance.

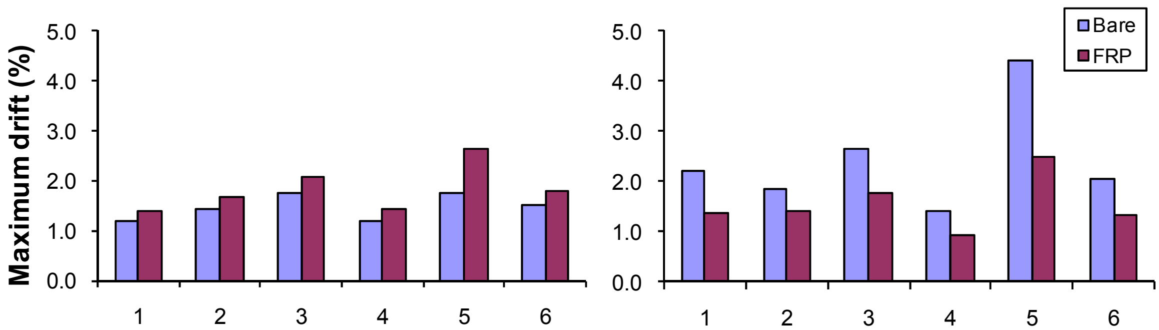

Figure 8. Maximum inter-story drift for 1st (left) and 2nd (right) stories for original and CFRP-strengthened models, six earthquakes.

Conclusions

This paper presented an experimental and analytical investigation on the efficiency of the CFRP strengthening method on deficient RC buildings. The results from the analyses show that the use of CFRP increases significantly the deformability capacity of the columns, hence reducing the expected structural damage of the strengthened structures. The use of CFRP also results in the reduction of maximum inter-story drifts, especially for the higher energy input records. Consequently, less structural and non-structural damage is expected to occur in the strengthened frame. The results show that CFRP strengthening can improve the seismic performance level of deficient RC frames subjected to strong seismic excitations.

Acknowledgments

The first author wishes to thank CONACyT and SEP/DGRI Mexico for providing the financial support for his research. The second author wishes to acknowledge the financial support provided by the EU through the Marie Curie International Incoming Fellowship.

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