Transaction A: Civil Engineering
Vol. 16, No. 2, pp. 138–146

Bending Response of HSRC Beams Strengthened with FRP Sheets

S.H. Hashemi[1,*], A.A. Maghsoudi[2] and R. Rahgozar[2]

Abstract. *The repair and strengthening of RC structures has become a major problem for civil engineers in the past few decades. To satisfy this problem, a previous method for the repair and strengthening of RC beams included bonding steel plates to the inferior structure. However, bonding steel plates to concrete presents disadvantages, including corrosion of the steel/adhesive joints and the heavy weight of the material. These problems increase installation and maintenance costs. The bonding of Fiber Reinforced Plastics (FRP) to structures provides an attractive alternative to steel plates. This material is corrosion resistant and lightweight, has a high strength-to-weight ratio and possesses nonconductive properties. The use of Fiber Reinforced Plastics (FRP) in repairing and strengthening RC beams has been researched in recent years. In particular, attaching unidirectional FRP to the tension face of RC beams has provided an increase in the stiffness and load capacity of the structure. However, due to the brittle nature of unidirectional FRP, the ductility of the beam decreases. Consequently, the safety of the structure is compromised, due to the reduction in ductility. The purpose of this research is to investigate the behavior of high strength reinforced concrete beams strengthened with FRP sheets. The major test variables included the different layouts of CFRP sheets and the tensile reinforcement ratio. More particularly, change in the strength and ductility of the beams, as the number of FRP layers and tensile reinforcement bar ratios are altered, is investigated. Eight under-reinforced concrete beams were fabricated and tested to failure. With the exception of the control beam, one or four layers of CFRP were applied to the specimens.*

Keywords: *Beams; Ductility; FRP; High strength concrete; Tensile bars.*

INTRODUCTION

High strength-to-weight ratio, resistance to electrochemical corrosion, larger creep strain, good fatigue strength, potential for decreased installation costs and repairs, due to lower weight, in comparison with steel, and the nonmagnetic and non-metallic properties of Fiber Reinforced Polymer (FRP) composites, offer a viable alternative to the bonding of steel plates. The emergence of high strength epoxies has also enhanced the feasibility of using CFRP sheets and a carbon fiber fabric for repair and rehabilitation.

The failure modes of concrete beams retrofitted with FRP materials and the techniques used in analyzing the failure modes were reviewed by Toutanji et al. [1]. The behavior of concrete beams strengthened with externally bonded steel plates [2], FRP plates [3,4], carbon fiber fabric [5,6] and GFRP sheets [7] was studied both experimentally and analytically. Malek et al. [8] presented analytical procedures to calculate the flexural strength of RC beams bonded with FRP plates. To date, extensive research work has been conducted on the flexural strength of concrete beams bonded with various types of FRP composites.

Advances in concrete technology in many countries have now made practical use of concrete with strengths up to 90 MPa. These concretes, with very high compressive strength, can result in less ductile responses of structural members. It has been found that flexural ductility, in terms of maximum curvatures attainable, may be smaller in HSC beams [9]. In seismic areas, ductility is an important factor in the design of HSC members under flexure. The use of HSC beams strengthened with CFRP and ductility has not been the focus of much previous experimental research work and, consequently, will be focused on

1. *Faculty of Engineering, Arak University, Arak, P.O. Box 38139, Iran.*
2. *Department of Civil Engineering, University of Kerman, Kerman, P.O. Box 76169-14111, Iran.*
*. *Corresponding author. E-mail: h-hashemi@araku.ac.ir*

Received 5 December 2006; accepted 13 May 2008

in this study. Although external strengthening of RC beams using epoxy-bonded FRP has been established as an effective tool for increasing their flexural and/or shear strength, the method still suffers from some drawbacks. Many of these drawbacks are attributed to the characteristics of currently available commercial FRP strengthening systems. Although FRPs have high strengths, they are very brittle. When loaded in tension, FRPs exhibit a linear stress-strain behavior up to failure without exhibiting a yield plateau or any indication of an impending failure.

The objective of this investigation is to study the effectiveness of FRP sheets on the ductility and flexural strength of High Strength Reinforced Concrete (HSRC) beams. This objective is achieved by conducting the following tasks:

1. Flexural testing of HSRC beams strengthened with different amounts of cross-ply FRP sheets with different amounts of tensile reinforcement;
2. Calculating the effect of different layouts of FRP sheets on flexural strength;
3. Evaluating the failure modes.

HSRC LABORATORY BEAM SPECIMENS

Beam Detail, Instrumentation and Test Procedure

Four-point bending flexural tests were conducted up to failure on two HSRC control beams and six HSRC beams strengthened with externally bonded FRP sheets on the tension face. The length, width and depth ($L \times b \times h$) of all beams were kept as $3000 \times 150 \times 250$ mm. Each concrete beam was reinforced with two 16-mm diameters for A series and two 22-mm diameters for B series steel bars, for tension, and two 10-mm-diameter steel bars for compression, along with 10-mm-diameter bars at a spacing of 90 mm center-to-center for shear reinforcement. The spacing of stirrups and maximum and minimum reinforcement ratios are in accordance with the provisions of the American Concrete Institute (ACI).

Electrical resistance disposable strain gauges, manufactured by the TML Measurements Group (Japan), were pasted on the CFRP sheets and on internal reinforcing bars at different locations. The demec and electrical gauges were also attached along the height of the beams to measure the concrete strains; these values can be used to find the strain distribution and the moving neutral axis depth of the beams tested. All beams were loaded in four-point bending to failure with a clear span of 2.7 m, and loading points were located at 450 mm on either side of the mid-span location. The load was applied step-by-step up to failure in the load control manner of test beams. During the test, the strains on steel and concrete, and vertical deflections were measured using LVDTs. The strain gauges, LVDTs and the load cell were connected through a data acquisition system to a computer and the data was recorded and stored in the computer (Figure 1).

For all beams, the shear-span-to-depth ratios are 4.18 and the length of the bonded plate is 2600 mm, which covers almost the full-span length between the supports of the beams. The reason for the full-span-length strengthening with FRP plates is to maximize the strengthening effects by delaying FRP separation.

Material Properties

The concrete in the beams was designed for a mean 28-day cube strength of about 100 MPa. For each beam, three 100 mm$\times$100 mm$\times$100 mm concrete cube specimens were made at the time of casting and they were kept with the beams during curing. The average 28-day concrete cube strength (f_{cu}) was 96.2 MPa. The

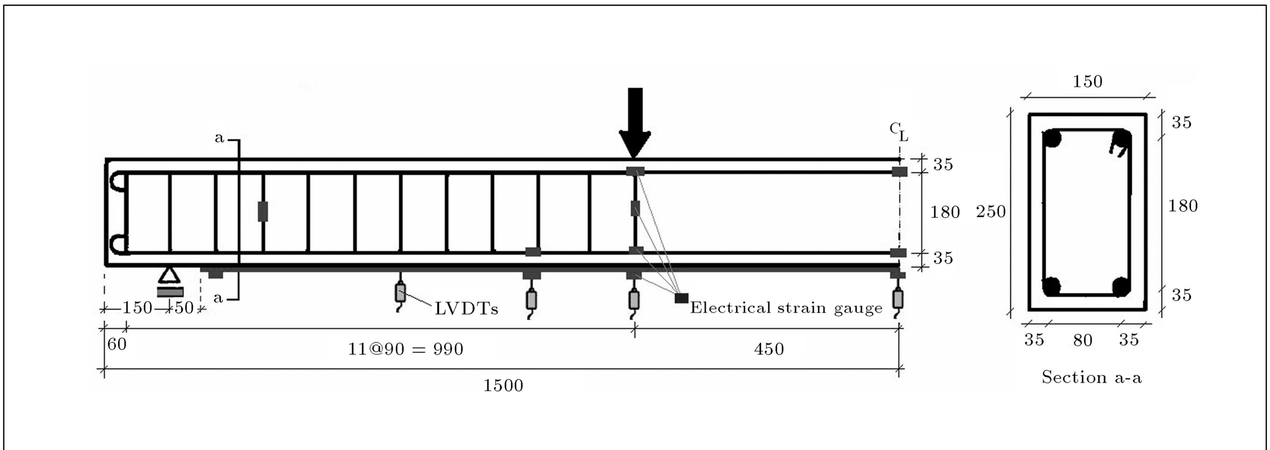


Figure 1. Beam details and measurement schemes for half of the test specimen (unit: millimeter).

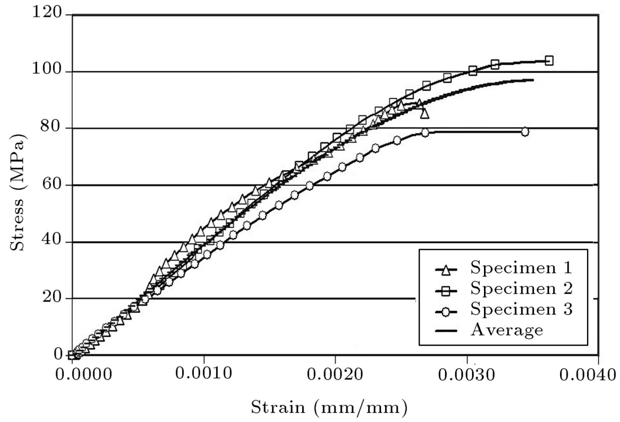


Figure 2. Stress-strain diagram for high strength concrete.

relationship of cylinder strength (f'_c) and cube strength was assumed as ($f'_c = 0.8 f_{cu}$) and the stress-strain curves for the cube specimens are shown in Figure 2. Thus, the average compressive strength (f'_c) was 77 MPa.

The measured yield and maximum tensile strength of the 10 and 16 mm rebars were 420.6, 634.1 and 412.5, 626.4 MPa, respectively. The density and thickness of the CFRP and GFRP material was 1.78 ± 0.1 gr/cm^3, 0.045 mm and 2.6 ± 0.1 gr/cm^3, 0.114 mm, respectively, and 2600 mm long for both of them. The Young's modulus (E_{fu}), ultimate tensile stress (f_{fu}) and elongation (ε_{fu}) of the FRP sheets were 230 GPa, 3850 MPa and $1.7 \pm 0.1\%$ for CFRP and 71 GPa, 2900 MPa and $4.5 \pm 0.5\%$ for GFRP, respectively. FRP sheets were externally bonded to the tension face of the concrete beams using a two-component structural epoxy named EP-TX at a 1:1 ratio for the first layer and a two-part epoxy named EP-IN at a 1:1 ratio for the next layer(s) of FRP. Strengthened concrete beams were cured for at least seven days at room temperature before testing.

Major Test Variables

The main test variables considered in the present study include the FRP sheet layers and tensile bars. The FRP sheet layers vary from 0 to 6 and the bar reinforcement ratio varies from 1.2% to 2.4%. The test program is summarized in Table 1. Of the eight beams tested, two were set aside as control beams and were not strengthened (AH0, BH0), two beam were strengthened with one layer of CFRP (AH1, BH1) and two beams were strengthened with four layers of CFRP (AH4, BH4), where the width of CFRP was 150 mm. The remaining two beams were strengthened with three layers of CFRP first and then with three layers of GFRP (ACG3, BCG3); the width of CFRP and GFRP being 100 and 150 mm, respectively.

TEST RESULTS AND DISCUSSIONS

Failure Pattern

The cracking patterns and failure for various test beams are shown in Figure 3. The control beams without strengthening plates was designed to fail in flexure. For the control beams (AH0, BH0), failure was by crushing the concrete in the compression zone after tension steel yield. For the strengthened beams, as the amount of FRP reinforcement increased, the failure mode of the strengthened beams transferred from an FRP rupture in the constant moment region to the delamination of FRP from the concrete substrate (ACG3, BCG3).

Most of the test beams exhibited the rupture of FRP sheets (AH1, AH4, BH1 and BH4) and failed in the same manner. We attended to the failure of a concrete cover along the tensile reinforcement. The concrete was not initially precracked and the development of cracks during the reinforcement test was highly influenced by the number of CFRP layers. The occurrence of the first crack was delayed and

Table 1. Test parameters and specimen identifications.

Series	Test Beam	A_S	A'_S	A_{SV}	A_{FRP} (mm^2)	FRP Detail (Layers×Thickness×Width)
A	AH0	2Φ16	2Φ10	Φ10@9 cm	0	0
	AH1	2Φ16	2Φ10	Φ10@9 cm	$(6.75)_{\mathrm{CFRP}}$	$(1 \times 0.045 \times 150)_{\mathrm{CFRP}}$
	AH4	2Φ16	2Φ10	Φ10@9 cm	$(27)_{\mathrm{CFRP}}$	$(4 \times 0.045 \times 150)_{\mathrm{CFRP}}$
	ACG3	2Φ16	2Φ10	Φ10@9 cm	$(13.5)_{\mathrm{CFRP}}+$ $(51.3)_{\mathrm{GFEP}}$	$(3 \times 0.045 \times 100)_{\mathrm{CFRP}}+$ $(3 \times 0.114 \times 150)_{\mathrm{GFRP}}$
B	BH0	2Φ22	2Φ10	Φ10@9 cm	0	0
	BH1	2Φ22	2Φ10	Φ10@9 cm	$(6.75)_{\mathrm{CFRP}}$	$(1 \times 0.045 \times 150)_{\mathrm{CFRP}}$
	BH4	2Φ22	2Φ10	Φ10@9 cm	$(27)_{\mathrm{CFRP}}$	$(4 \times 0.045 \times 150)_{\mathrm{CFRP}}$
	BCG3	2Φ22	2Φ10	Φ10@9 cm	$(13.5)_{\mathrm{CFRP}}+$ $(51.3)_{\mathrm{GFEP}}$	$(3 \times 0.045 \times 100)_{\mathrm{CFRP}}+$ $(3 \times 0.114 \times 150)_{\mathrm{GFRP}}$

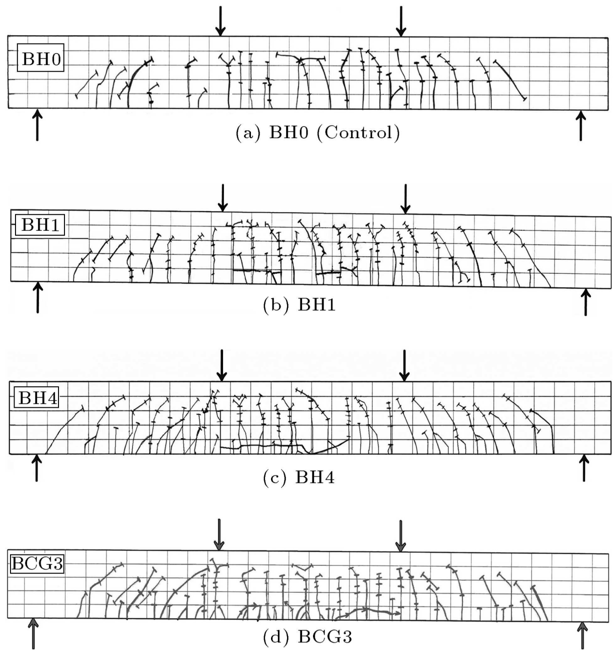


Figure 3. Failure configuration of control and FRP beams at ultimate state.

more diffuse. Shear cracks occurred in the shear span length of the beams for an applied load, which was between 70% and 80% of the ultimate load. Finally, the sudden propagation of horizontal cracks in the concrete-steel bond region occurs. This type of cracks runs along the weakest surface, which is the concrete-steel interface. It leads to the failure of the beam as soon as the cracks open and separates the concrete cover from the rest of the beam. It is interesting to note that the weakest point of the assembled concrete-bond-composite material is not the concrete-composite interface but the concrete-internal steel interface. Figure 3 also indicates that the strengthened beams show many diagonal cracks, which were caused by the increase of flexural capacities, due to CFRP sheets.

From experimental observation, the debonding failure can be explained as follows: Due to the flexural cracks formed in the constant moment region as the load increased, the bond between FRP and concrete started to fracture at a certain load level and the failure propagated towards the shear span until most parts of FRP composites detached from the concrete beams. It can be seen that the bond between FRP and concrete is not strong enough to ensure the rupture of the composites with more than four layers of carbon fiber sheet; thus, the FRP-concrete bond strength controls the failure mode when five or six layers of fiber sheet are bonded. When four, or less than four, layers of carbon fiber are applied, the bond problem is not the controlling factor for failure, thus, the force in FRP will reach its ultimate tensile capacity when the beam fails.

Discussions on Flexural Behavior

Table 2 summarizes the test results for the peak loads, displacements and strains at peak loads for the tested beams. Table 2 also shows the increase of peak load, according to the various strengthening layers of FRP. The rates of increase of peak loads varied from 1% to 44%, depending upon the strengthening method.

For the strengthened beams in this study, as seen in Table 2 the highest measured strain in the concrete at the beam top surface was 2700 $\mu\varepsilon$, which was reported in beam BH4. This constituted about 77% of the ultimate concrete strain at 3500 $\mu\varepsilon$, which clearly demonstrated that failure due to concrete crushing was not possible.

The load deflection response for each of the test beams is plotted in Figure 4. In general, the

Table 2. Test results of the control and CFRP strengthened beams.

Series	Test Beam	Failure Modes[a]	Peak Load P_{ult} (kN)	Ratio to Unstrengthened Beam	P_{ult}				
					Displacement (mm)	Strain (micron)			
						CFRP	Tensile Rebar	Stirrup	Concrete
A	AH0	C.C	81.25	1	102	-	2316	48	3600
	AH1	C.R	89.96	1.11	50.42	844	3341	441	2500
	AH4	C.R	117.33	1.44	32.85	2581	9557	954	2100
	ACG3	D.L	104.7	1.29	26.2	8663	15413	638	1738
B	BH0	C.C	149.52	1	95.7	-	17843	644	4200
	BH1	C.R	150	1.01	63.24	1066	17330	790	2600
	BH4	C.R	167	1.12	30.92	3367	4512	-	2700
	BCG3	D.L	162.23	1.09	26	4327	10375	887	2240

[a] C.C: Concrete crushing; C.R: CFRP rupture; D.L: FRP delamination.

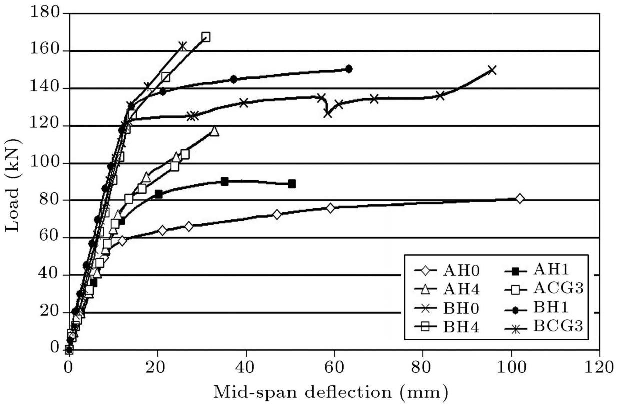


Figure 4. Load deflection responses of test beams.

strengthened beams were stiffer and less ductile than the control specimens with a higher ultimate load. As a result, compared to a beam reinforced heavily with steel only, beams reinforced with both steel and CFRP have an adequate deformation capacity, in spite of their brittle mode of failure.

The tension steel in control beams AH0 and BH0 reached its yield strength before the compressive strain in concrete reached 0.003 and the beams failed by the crushing of the concrete. Even though the control beams failed by the crushing of concrete, since the failure was initiated by the yielding of tension steel, the mode of failure was mentioned to be under reinforced tension failure, thus, the behavior of the two control beams was a ductile flexural response. For control beams, after the first visible cracks were observed, the cracking became extensive and crack widths increased steadily. The shape of the load deflection curves indicates a loss of stiffness at a load of approximately 64 kN for AH0 and 122 KN for BH0. This was due to the yielding of the tensile reinforcement and occurred at a midspan deflection of 21 mm for AH0 and 13.3 mm for BH0. After this point, large flexural cracks opened during the test and eventual ultimate collapse occurred by concrete crushing within the compression zone; a photograph of which is presented in Figure 5. The ultimate loads recorded were 81.25 and 149.5 kN for AH0 and BH0, respectively.

In this study, for AH1, AH4, BH1 and BH4, the bond problem is not the controlling factor for failure, thus, the force in CFRP will reach its ultimate tensile capacity when the beam fails and the failure mode of the strengthened beams is a CFRP rupture in the constant moment region. Figure 6 shows such a typical failure mode.

As the amount of FRP reinforcement increased, the failure mode of the strengthened beams transferred from a FRP rupture in the constant moment region to the delamination of FRP from the concrete substrate. In this study, beams ACG3 and BCG3 failed by the debonding of sheets from the concrete surface. Figure 7 shows photographs of this failure mode.

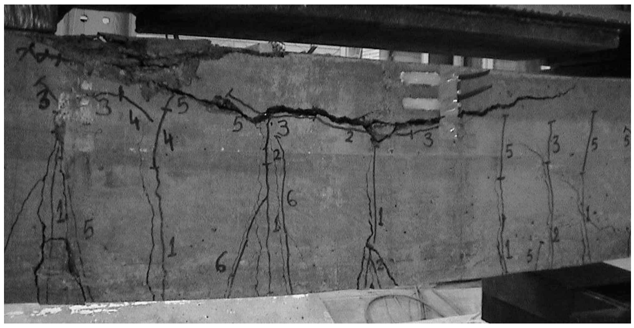
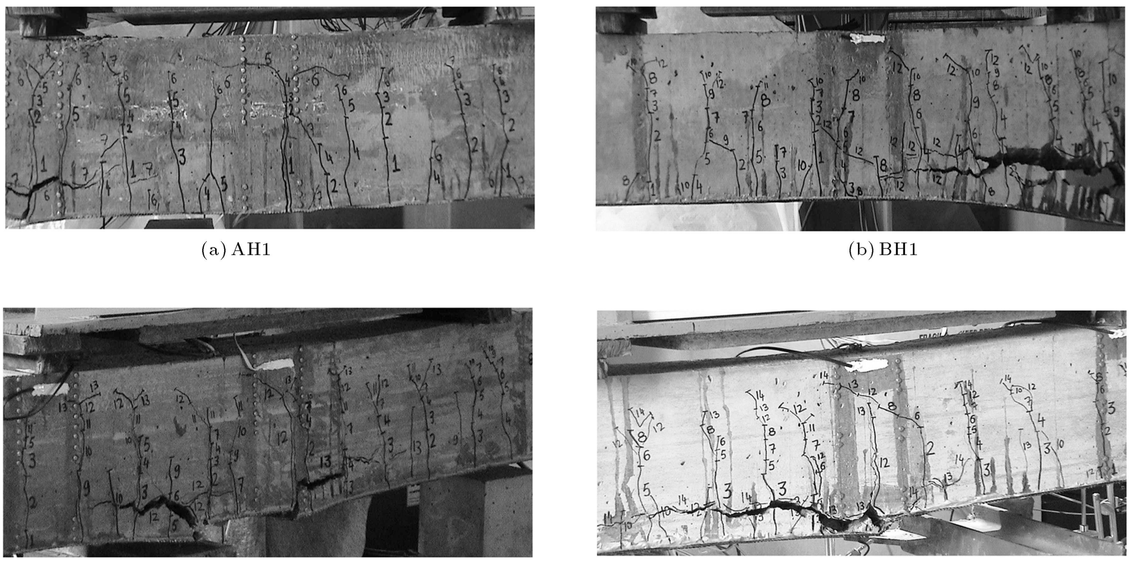


Figure 5. Flexural failure of control beam AH0.



(a) AH1

(b) BH1

(c) AH4

(d) BH4

Figure 6. Rupture of FRP in beams AH1, AH4, BH1 and BH4.

Table 3. Mid-span deflection and load in yield and ultimate stage of R/C beams strengthened with FRP sheets.

Series	Test Beam	Yield Stage				Ultimate Stage			
		Load P_y (kN)	Increase over Control (%)	δ_y (mm)	Decrease over Control (%)	Load P_u (kN)	Increase over Control (%)	δ_u (mm)	Decrease over Control (%)
A	AH0	63.93	–	21	–	81.25	–	102	–
	AH1	69.5	8.7	13	38	89.9	11	50.42	51
	AH4	64.7	1.2	9.83	53.2	117.3	44.4	32.85	67.8
	ACG3	67.33	5.3	10.37	51	104.66	28.8	26.2	74.4
B	BH0	122.2	–	13.325	–	149.52	–	95.7	–
	BH1	130	6.4	14.11	-5.9	150	0.5	63.24	33.9
	BH4	118	-3.4	12.86	3.6	167	11.7	30.92	67.7
	BCG3	130.66	6.9	13.8	-3.6	162.33	8.5	26	72.9

Table 3 shows a summary of the flexural behavior of all test beams, in terms of flexural loading capacity and deflection. The results clearly demonstrated the accepted beneficial effects of CFRP layers, with regard to the stiffening and strengthening of the beams.

The strain response of FRPs is different from that of conventional steel, which yields after elastically deforming to relatively small values of strain (0.2% for Grade 60 [410 MPa] and 0.14% for Grade 40 [280 MPa]). FRP materials exhibit elastic deformation to relatively large strain values before rupture. As a result, when FRPs are used for the flexural strengthening of concrete beams reinforced with conventional steel, the steel reinforcement may yield before the FRP contributes any additional capacity to the beam. Therefore, it can be difficult to obtain a significant increase in yield load or stiffness for a beam. When an increase in beam yield load or stiffness is required, larger cross sections of FRPs must be used (before the steel yields), which generally increases the cost of strengthening. Although using some special low-strain fibers, such as ultra-high-modulus carbon fibers, may appear to be a solution, they can result in brittle failures, due to fiber failure. Taking advantage of the high strength of FRPs during the flexural strengthening of RC beams is limited by the bond capacity between them and the concrete surface. In many cases, debonding occurs [8,10] at stress levels that are a small fraction of the FRPs' strength.

As the amount of steel reinforcement increases, the additional strength provided by the carbon FRP external reinforcement decreases. The same amount of CFRP reinforced the flexural strength of a lightly reinforced beam (ρ = 1.2%) by more than 44%, but only increased the strength of a moderately reinforced beam ($\rho = 2.4\%$) by 11.7%.

Ductility

Ductility is an important factor for any structural element or structure, especially in seismic regions. A ductile material is one that can undergo large strains while resisting loads. When applied to RC members, the term ductility implies the ability to sustain significant inelastic deformation prior to collapse [11]. In the case of beams strengthened with FRP laminates, there is usually no clear yield point. However, it was shown that deflection and energy, based on tension steel yielding, can be used as a criterion of ductility to evaluate the comparative structural performance of FRP bonded RC beams [12].

The ductility index in this study is obtained, based on deflection (μ_d) and curvature (μ_ϕ) computation, and is defined as the mid-span deflection or curvature at peak load, divided by the mid-span deflection or curvature at the point where the steel starts yielding. Table 3 shows the test results of the beams for yield and ultimate stage, and Table 4 shows the experimental deflection and curvature ductility ratio and percent decrease of ductility, with respect to the control beam, for each of the specimens.

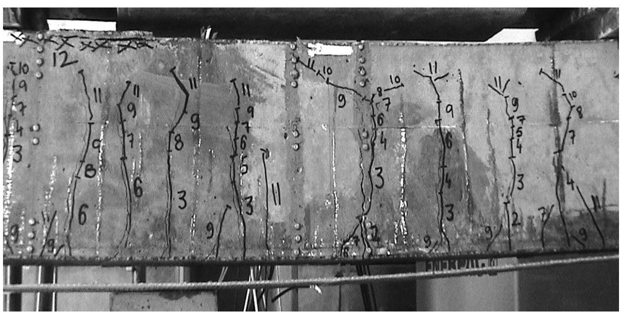


Figure 7. Debonding failure of FRP in beams ACG3 and BCG3.

Table 4. Experimental ductility ratio of the test beams.

Series	Test Beam	Deflection Ductility Ratio $\left(\mu_\delta = \frac{\delta_u}{\delta_y}\right)$	Decrease over Control Beam (%)	Curvature Ductility Ratio $\left(\mu_\phi = \frac{\phi_u}{\phi_y}\right)$
A	AH0	4.86	–	6.37
	AH1	3.87	20.4	–
	AH4	3.34	31.3	3.91
	ACG3	2.53	47.9	2.56
B	BH0	7.19	–	6.2
	BH1	4.48	37.7	–
	BH4	2.4	66.6	2.37
	BCG3	1.9	73.6	3.67

Considering HSC members, displacement ductility, μ_d, in the range of 3 to 5, is considered imperative for adequate ductility, especially in the areas of seismic design and the redistribution of moments [13]. Therefore, assuming that a μ_d value of 3 represents an acceptable lower bound for ensuring the ductile behavior of HSC flexural members, it appears that, for ACG3, BH4 and BCG3 beams would not meet that requirement [14].

Concrete and Tensile Bar Moment-Strain Response

The relationship between concrete strains (measured on the compression face at mid-span) and applied moments for both A and B series are plotted in Figure 8. There is a similar increase in strain for all the beams at low moments. However, cracking of the concrete in the tension zone results in larger increments of strain in the control specimens (i.e., for control beam AH0, the extreme layer of concrete compressive strain at failure is $\varepsilon_{cu} = 0.0036$). For these beams, concrete strain varies almost linearly with moment, after initial cracking, until the yielding of the tension steel. Following yield, steel strain increases rapidly, with each increment of moment, and, finally, the concrete crushes as the beam collapses (see Table 2). On the other hand, the extreme compressive strain of concrete fiber in the strengthened beams, with the increased number of layers of the CFRP sheet, remains more or less linear up to failure and is not significantly affected by concrete cracking or a yielding of the tension steel. These results demonstrate that the effect of the strengthening plate is to reduce strain in the compression fibers of the concrete. The presence of the plate draws the neutral axis lower in the section and, hence, places a greater volume of concrete in compression, resulting in lower strain (see Table 2) and enabling a more efficient use of the existing material. Thus, externally bonded CFRP

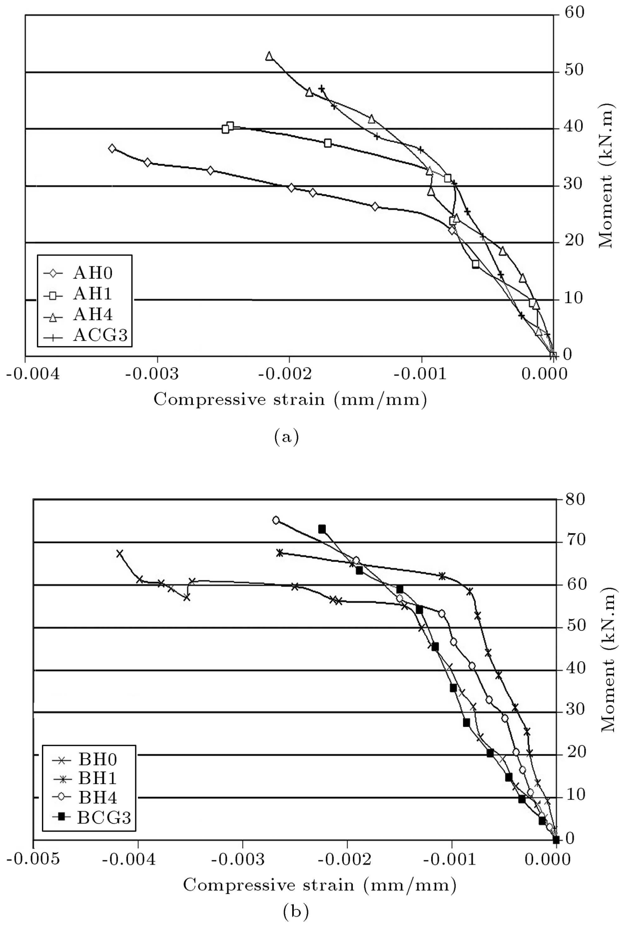


Figure 8. Moment vs concrete strain at mid-span.

plates may also be beneficially used to reduce concrete compressive stresses, in addition to acting as additional tensile reinforcement.

Figure 9 indicates that each curve consists of almost three straight lines with different slopes. The first turning point, A, indicates the cracking of concrete in the tension zone. The second turning point, B, refers to the yielding tension steel. The yielding and

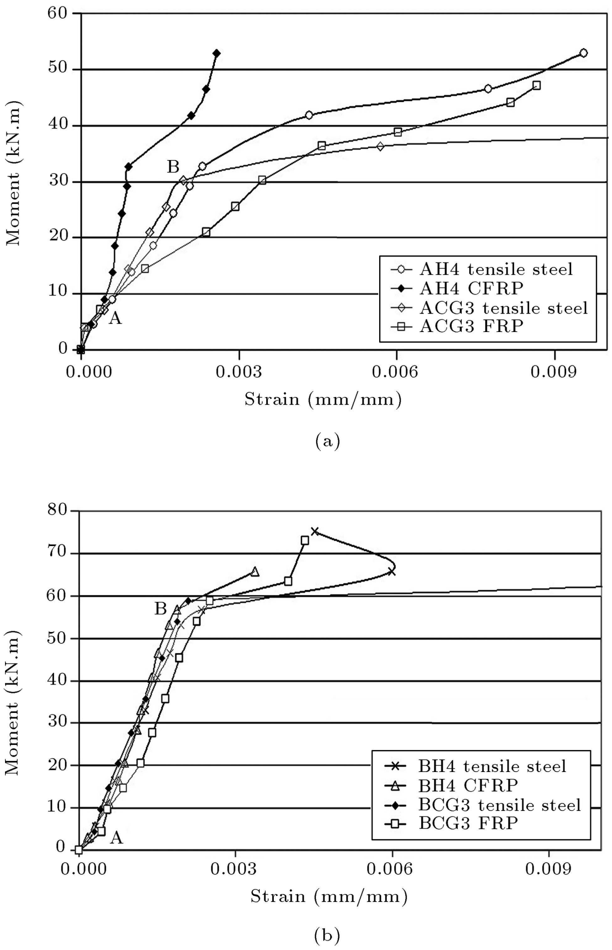


Figure 9. Moment-strain curves of CFRP, tensile steel and extreme top concrete fiber for beams AH4, BH4, ACG3 and BCG3 at mid-span.

maximum load (ultimate load) can be found for each beam from its load - strain curve.

For beams AH4 and BH4, the tensile steel and CFRP strains are essentially the same at loads below cracking of the concrete. After cracking, the strains in steel exceeded those of the CFRP laminate. As the load approached the yielding load for the strengthened beam, the strains in steel increased more rapidly than those in the CFRP. This is because the CFRP had begun to debond from the nearby cracks of the concrete surface. It was noted that tensile steels strains were always higher than CFRP strains.

CONCLUSIONS

The major conclusions derived from this experimental study are given as follows:

- The results of tests performed in this study indicate that significant increase in flexural strength can be achieved by bonding CFRP sheets to the tension face of high strength reinforced concrete beams. The gain in ultimate flexural strength was more significant in beams with lower steel reinforcement ratios. In addition, strengthening reduced the crack width in beams at all load levels.
- The extreme compressive strain of concrete fiber in the strengthened beams, with the increased number of CFRP layers, remains more or less linear up to the failure of the beam and is not significantly affected by concrete cracking or a yielding of the tension steel. These results demonstrate that the effect of the strengthening plate is to reduce strain in the compression fibers of the concrete.
- Compared to a beam reinforced heavily with steel only, beams reinforced with both steel and CFRP have an adequate deformation capacity, in spite of their brittle mode of failure.
- As the amount of tensile steel reinforcement increases, the additional strength provided by the carbon FRP external reinforcement decreases. The same amount of CFRP reinforced the flexural strength of a lightly reinforced beam by more than 44.4% (20% of balanced ratio), but only increased the strength of a moderately reinforced beam by 11.7% (40% of balanced ratio).

REFERENCES

1. Toutanji, H., Zhao, L. and Anselm, E. "Verifications of design equations of beams externally strengthened with FRP composites", *Journal of Composites for Construction*, **10**(3), pp. 254-264 (2006).
2. Oh, B.H., Cho, J.Y. and Park, D.G. "Static and fatigue behavior of reinforced concrete beams strengthened with steel plates for flexure", *Journal of Structural Engineering*, **129**(4), pp. 527-535 (2003).
3. Toutanji, H., Zhao, L. and Zhang, Y. "Flexural behavior of reinforced concrete beams externally strengthened with CFRP sheets bonded with an inorganic matrix", *Journal of Engineering Structures*, **28**, pp. 557-566 (2006).
4. Chahrour, A. and Soudki, K. "Flexural response of reinforced concrete beams strengthened with end-anchored partially bonded carbon fiber-reinforced polymer strips", *Journal of Composites for Construction*, **9**(2), pp. 170-177 (2005).
5. Alagusundaramoorthy, P., Harik, I.E. and Choo, C.C. "Flexural behavior of R/C beams strengthened with carbon fiber reinforced polymer sheets or fabric", *Journal of Composites for Construction*, **7**(4), pp. 292-301 (2003).
6. Brena, S.F. and Marci, B.M. "Effect of carbon-fiber reinforced polymer laminate configuration on the behavior of strengthened reinforced concrete beams", *Journal of Composites for Construction*, **8**(3), pp. 229-240 (2004).
7. Saadatmanesh, H. and Ehsani, M.R. "RC beams strengthened with GFRP plates, I: experimental

study", *Journal of Structural Engineering*, **117**(11), pp. 3417-3433 (1991).

8. Malek, A.M., Saadatmanesh, H. and Ehsani, M.R. "Prediction of failure load of R/C beams strengthened with FRP plate due to stress concentration at the plate end", *ACI Structural Journal*, **95**(2), pp. 142-152 (1998).
9. American Concrete Institue (ACI) "State-of-the-art report on high-strength concrete", ACI Committee 363 Report, Detroit (1992).
10. Fanning, P.J. and Kelly, O. "Ultimate response of RC beams strengthened with CFRP plates", *Journal of Composites for Construction*, **5**(2), pp. 122-127 (2001).
11. Maghsoudi, A.A., *Design for Ductility of Structures*, Shaheed Bahonar University Publications, Kerman, Iran (1996).
12. Mukhopadhyaya, P., Swamy, N. and Lynsdale, C. "Optimizing structural response of beams strengthened with GFRP plates", *Journal of Composites for Construction*, **2**(2), pp. 87-95 (1998).
13. Hashemi, S.H. "Analytical and experimental study of HSC members strengthened with CFRP", PhD Thesis, Civil Eng. Dept. of Kerman University, Iran (2007).
14. Maghsoudi, A.A. and Akbarzadeh, H. "Flexural ductility of HSC members", *Structural Engineering and Mechanics, an International Journal*, **24**(2), pp. 195-213 (2006).