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Rehabilitation of Reinforced Concrete Corner Beam-Column Joints Subjected to Damage

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Abstract

Important structural properties related to strength and deformations of corner beam-column joints damaged and strengthened were studied in this investigation. For this purpose a total of 18 corner beam-column joints divided into five groups were cast and tested. The cross sections were 150mm X 200mm and 150mm X 250mm for the beam and column, respectively. The legs of each beam and column were 900mm. Group1 specimens were consisted of two control specimens tested to failure. Other groups specimens each consisted of four specimens, two of which were tested up to first crack loading and the other two were tested up to 80% of the expected ultimate load. After preloading, the damaged specimens were strengthened in such a way that Group2 were strengthened with strip of CFRP, Group3 with steel plate strip, Group4 with ferrocement, and Group5 with full CFRP sheets. After strengthening process, the strengthened specimens were retested to failure. The test results indicated that the process of the rehabilitations is successful. After strengthening, the ultimate load was increased by 38.3, 4.25, 6.3, and 42.5 % for Group2, 3, 4, and 5, respectively. The stiffness of the strengthened specimens increased compared to the control specimens. The better strengthening technique was those specimens strengthened with full CFRP sheets. The effect of cracking due to preloading on the flexural behavior of strengthened beam-column joint was found to be insignificant.

Introduction

The idea of strengthening structural concrete members and rehabilitation of damaged structures is not new and goes back to the early 1960's [1]. However, the problem is the invention of economical materials with high performance. Since the total cost of rehabilitating existing structures is usually lower than that provided to rebuild them and some structures are historical in nature, there is a need for strengthening damaged locations for such structures. The state of the art report of the ACI 440 Committee is a useful matter for the practical applications of strengthening concrete structures, and can be followed for this purpose. The noticeable property for strengthening layers is their high tensile strength and low self-weight, while a successful binder epoxy is that which provides excellent bond between the sheet and concrete surface with good durability. Carbon fiber sheet is mostly used for strengthening purpose. Carbon fiber sheet is formed by laying out fibers in single or multiple directions and embedding in a protective epoxy resin. A carbon fiber sheet receives particular attention due to its higher strength, stiffness, and corrosion resistance with reasonable cost. One advantage of using a fiber composite material is the negligible increase in dead load due to its light weight, in addition, it can be easily carried to construction site in rolls. Carbon fiber sheet has a very high unidirectional tensile strength but has stiffness close to that of steel. Typical values are between 2500-4600 MPa for tensile strength and 235-269 GPa for Young's modulus [2]. Carbon fiber sheet exhibits no effective compression or bending stiffness. Plate bonding has a considerable success in repair, strengthening and upgrading of concrete structures in spite of variety of their loadings and functions. The fame of this technique came from his capability of increasing the structural member capacity and restraining its deformability[3]. After 1948, ferrocement was utilized in a number of practical applications such as repair of shear damaged reinforced concrete beams, beams and slab with excessive deflection, joints, repair/strengthening of brick masonry columns as well as plain concrete column, it has been found that use of ferrocement is advantageous in terms of enhancement of load carrying capacity, better cracking behavior, ductility, energy absorption properties, stiffness and flexural capacity [4]. The objective of this research is to develop an economic, effective and easily applicable rehabilitation method and to reduce intensive labors, for the rehabilitation of damaged R.C beam-column joints. For this purpose the efficiency of Carbon Fiber Reinforced Polymer (CFRP), steel plate strips, and ferrocement was studied. The effects of the proposed rehabilitation techniques on the load carrying capacity and the behavior of RC beam-column joints were investigated by making a comparison among the results of specimens with regard to the ductility.

Strengthening of Beam-Column Joints: There are many techniques for strengthening and rehabilitation of beam-column joints such as epoxy repair, removal and replacement, reinforced or pre-stressed concrete jacketing, concrete masonry unit jacketing, steel jacketing and/or addition of external steel elements, and fiber reinforced polymer (FRP) composite. Each method needs some different level of artful detailing, consideration of labor, cost, disruption of building occupancy, and range of applicability. It is important to use materials which are effective for rehabilitation of beam-column joints, adaptation of these materials used in strengthening the existing materials, and the cost and availability of materials. Beam-column joint retrofitting is an important aspect of improving the seismic performance of a structure. Almusallam et al [5] (2009) tested a rehabilitated full scale corner beam-column joint with carbon fiber reinforced polymer (CFRP) sheets after damaging it. They concluded that the proposed method of repair can improve the load carrying capacity and ductility of joint by 88% and 97%, respectively. Ramakrishna and Ravindra [6] (2012), tested eight strengthened interior beam column joint. Four of the specimens were strengthened with glass fiber reinforced polymer sheets, and the other specimens were strengthened with carbon fiber reinforced polymer sheets. Experimental results showed that the rehabilitated specimens of glass fiber reinforced polymer and carbon fiber reinforced polymer are successful. Load carrying capacity increased by 37%, and 55%, for GFRP and CFRP strengthening, respectively. After review of previous research, several conclusions were developed. It has been noted that the majority of past studies on rehabilitation of beam-column joint were concentrated on increasing load carrying capacity and changing the failure mode after strengthening.

Materials and experimental set up and program

A concrete mix of 32MPa cylinder compressive strength has been used in constructing the specimens. Steels of 538MPa yield strength (8mm diameter bars) for stirrups, and 690MPa (10mm dia. bars) for longitudinal bars were used, as shown in the Fig.(1). The materials used for rehabilitation were carbon fiber reinforced polymer (CFRP) (uni-direction), steel plate strips, and ferrocement. Reinforced concrete beam-column joint (corner joint or L-shape joint) was chosen as a specimen model, the length of each leg was (900mm) with cross-sections of (150mm X 200mm) and (150mm X 250mm) for both the beam and the column, respectively. The experimental work consisted of casting and testing 18 reinforced concrete corner beam-column joints which were divided into four main groups according to the strengthening materials, as shown in Table (1). The specimens were designed to insure that it would fail in bending. In all specimens the beam's depth was smaller than column's depth to insure that failure occurs at the joint zone on the beam region same as a frame with strong column-weak beam to insure the formation of plastic hinge at the beam end. The specimens were tested under a load for opening the joint. The test set-up is shown in Fig.(2) Cracks were monitored and the displacements were measured. The cracking and ultimate loads were recorded. The rehabilitation process consisted of external application of CFRP sheets, steel plate strips, and ferrocement, individually.

Table (1) Testing program

Group no.		***Symbol of specimens***	***Preloading***	***Material of strengthening***
1	G1	C1*	failure	--------
		C2*	failure	
2	G2	BC1	80% Ultimate load	Strip of CFRP
		BC2	80% Ultimate load	
3		BC3	first crack	
		BC4	first crack	
4	G3	BC5	80% Ultimate load	Steel plate strip
		BC6	80% Ultimate load	
5		BC7	first crack	
		BC8	first crack	
6	G4	BC9	80% Ultimate load	Ferrocement
		BC10	80% Ultimate load	
7		BC11	first crack	
		BC12	first crack	
8	G5	BC13	80% Ultimate load	Full CFRP
		BC14	80% Ultimate load	
9		BC15	first crack	
		BC16	first crack	

* Control specimens.

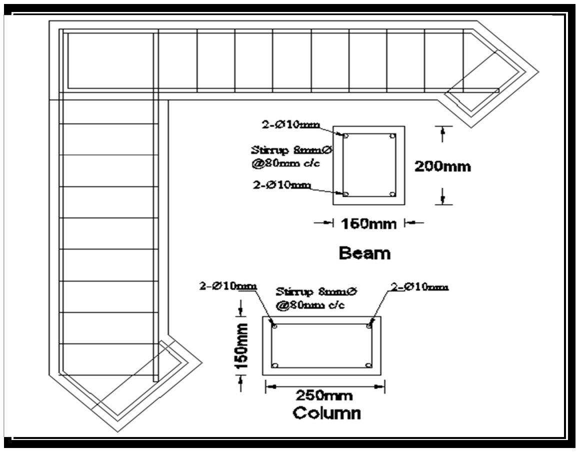

Fig.(1) Dimensions and reinforcement detail of the corner Beam-Column joint specimen.

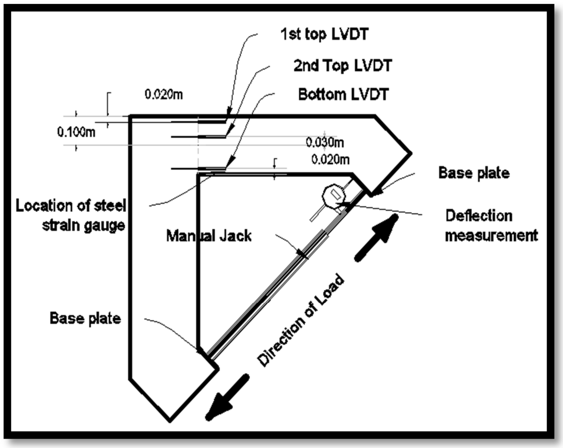

Fig.(2) Test set-up and loading arrangement of the specimens

Table (2) Results of corner beam column joint specimens

Specimens		***Initial loading***	***First cracking load (kN)***	***Ultimate load capacity (kN)***	***Average ultimate load capacity (kN)***	***% Load capacity increase***	***Mode of failure***	***Group average %load capacity increase***
G1 (Control specimens)	C1	failure	6	24.5	24.0		Splitting of concrete (diagonal tension crack)	——
	C2	failure	6	23.5			Splitting of concrete (diagonal tension crack)	
G2 (Strengthened with Strip of CFRP)	BC3	first crack	18	32.6	33.7	40.4%	Bond failure between concrete & CFRP	38%
	BC4	first crack	18	34.7			Bond failure between concrete & CFRP	
	BC1	80% loaded	18	32.6	32.6	36%	Bond failure between concrete & CFRP	
	BC2	80% loaded	18	32.6			Bond failure between concrete & CFRP	
G3 (Strengthened with steel strip plate)	BC7	first crack	10	23.5	24.5	2.1%	Bond failure between concrete & steel plate	4.2%
	BC8	first crack	10	25.5			Bond failure between concrete & steel plate	
	BC5	80% loaded	10	26.5	25.5	6.3%	Bond failure between concrete & steel plate	
	BC6	80% loaded	10	24.5			Bond failure between concrete & steel plate	
G4 (Strengthened with ferrocement)	BC11	first crack	8	26.5	25.5	6.3%	Mesh delamination	6.3%
	BC12	first crack	8	24.5			Mesh delamination	
	BC9	80% loaded	8	24.5	25.5	6.3%	Mesh delamination	
	BC10	80% loaded	8	26.5			Mesh delamination	
G5 (Strengthened with Full-CFRP)	BC15	first crack	14	34.7	34.7	44.5%	Beam concrete cover delamination	42.5%
	BC16	first crack	14	34.7			Beam concrete cover delamination	
	BC13	80% loaded	14	34.7	33.7	40.4%	Beam concrete cover delamination	
	BC14	80% loaded	14	32.6			Beam concrete cover delamination	

Test results

In the control specimens (C1 and C2) the first crack was formed at the inside face of the joint (beam-column intersection line) as expected at a load of 6kN opening the joint, inclined diagonally at 45 degrees to both the column and the beam due to the maximum bending tensile stress produced by the load opening the joint, as shown in Fig. (3). As the load increased to about 10kN, the crack split into two cracks, one paralleled to the beam and the other paralleled to the column, followed at load of 14-20kN by other cracks away from the joint mostly tensile flexure type of cracks due to maximum moments at faces of the column and the beam. At late stages (23-24kN), and close to the ultimate load, a diagonal crack (splitting) appeared approximately perpendicular to the line connecting the two inside and outside points of intersection of the joint and close to the outside joint point resulting from the high compressive stress induced at this region splitting that part of the joint away from the confinement reinforcement. This could be a tensile split inside the joint or a rapture of the portion of the concrete above and at the reinforcement bars under compression, in this case which is an opening column-beam connection. In the first case confinement steel bars (stirrups) would be most effective to control or minimize this type of failure. In the latter case also confinement bars in diagonal direction and good detailing would be effective and prevents cover rupture. Of course in other types of connections such as (T-joint) or (interior-joint) this type of failure would be most unlikely.

Fig.(3) Crack pattern of the control specimen (C2)

Fig.(4) Crack pattern of the specimen preloaded until 80% ultimate load and strengthened with strip of CFRP (BC1)

In the specimens strengthened with strips of CFRP (BC1 to BC4) after reloading, the first new crack formed at the tension face of the beam at a load of 18kN due to mostly tensile flexure stress, as shown in Fig.(4), outside the strengthened region. With increasing the load from 20 to 30kN other cracks occurred at tension face of the beam and the column away from the joint and went toward the top of the beam and column. At ultimate load 32-34kN the bond between CFRP strips and the concrete face failed and a diagonal crack (splitting) or cover rupture occurred similar to the control specimens. Compared to control specimens, for strengthened specimens, the first crack appeared outside the strengthening area (away from the joint) at a load of 18kN, whereas for control specimens the first crack occurred inside the joint (beam-column intersection line) at a load of 6kN, due to CFRP strip resistance against opening of the joint and increasing the tensile strength of the joint and causes stress increase at other locations and consequently cracks at these new locations. The ultimate load for control specimens were approximately 24kN while for strengthened specimens that preloaded up to first crack load and 80% of failure load were 33.7kN and 32.6kN, respectively, it means an increase of about 40.4% and 36% in load carrying capacity, respectively. The control specimens failed by diagonal cracks, G2 specimens failed in the same pattern after the bond failure between CFRP strips and concrete surface. The load carrying capacity after strengthening for specimens that damaged up to 80% ultimate load have smaller load carrying capacity than the specimens that damaged up to the first crack load. This indicates that the preloading process has a slight effect on ultimate load carrying capacity.

For G3, the specimens strengthened with steel plate strips (BC5 to BC8), and after reloading, the first new crack formed at the tension face of the beam at a load of 10kN due to tensile flexural stress, as shown in Fig. (5), with increasing the load from 12 to 18kN other cracks formed at tension face of both beam and column. At the later stage of 20kN cracks occurred at the joint zone and between the steel plate strips. At ultimate load of 23-26kN, bond failure occurred between steel plate and the concrete surface and a diagonal crack (splitting) occurred. In comparison with control specimens, the first crack in G3 appeared outside the strengthening area (away from the joint) at a load 10kN similar to G2, whereas for control specimens the first crack occurred at inside face of the joint (beam-column intersection line) at a load of 6kN. The ultimate load for control specimens were approximately 24kN while for specimens preloaded to first crack load and for specimens loaded to 80% of ultimate load were 24.5kN and 25.5kN, showing an increase of 2.1% and 6.3% in load carrying capacity, respectively. The final failure pattern was similar to G2 specimens which were a bond failure between the steel strip and concrete surface followed by the concrete split. The failure load was more than that of the control specimens, although the increase in load carrying capacity is less than G2 specimens strengthened with CFRP strips.

Fig.(5)Crack pattern of the specimen preloaded until first crack load and strengthened with strip steel plate(BC8)

Fig.(6) Crack pattern of the specimen preloaded until 80% ultimate load and strengthened with ferrocement (BC9)

For G4 the specimens strengthened with ferrocement (BC9 to BC12), the first crack formed at the inside faces of the joint (beam-column intersection line) at a load of 8 kN opening the joint inclined diagonally or extended parallel to the beam or the column as the load increased, as shown in Fig. (6), followed at the load of 14-20kN by other cracks away from the joint mostly tensile flexure type of cracks. At a load of 24-26kN and close to ultimate load, the cracks at the joint extended toward outside. Compared with control specimens, in G4 the first cracks appeared inside the joint (beam-column intersection line) same as the control specimens but at a load of 8kN instead of 6kN for control specimens, due to small resistance of ferrocement against the opening of the joint. The ultimate load for G4 was 25.5kN, with an increase of about 6.3% in load carrying capacity for both specimens preloaded to first cracking load and to 80% ultimate load compared to the control specimens. In specimens of G4, the diagonal cracks or the final cracks causing failure is not visible because the ferrocement layer may have covered it. In this group, the ultimate load was reached and enhanced by 6.3%.

For G5 the specimens were strengthened with full CFRP coverage of the joint (BC13 to BC16). After reloading, the first new crack formed at the edge of the CFRP sheet or outside the strengthened region at the tension face of the beam at a load of 14kN, as shown in Fig.(7). With increasing load from 18kN to 30kN other cracks occurred at tension face of both of the beam and the column. At ultimate load of 32kN-34kN failure occurred at the beam tension face at the edge of the CFRP sheet, and this caused delamination of concrete cover in some cases. Compared to the control specimens, the full coverage of the joint with CFRP resulted in shifting the cracks away from the joint and the strengthened region with an increase in load carrying capacity by 44.5% and 40.4% for strengthened specimens preloaded up to first crack load and 80% ultimate load, respectively, which is the largest ratio among the groups. It can be observed that load carrying capacity for the specimens subjected to preloading up to first crack is greater than those preloaded up to 80% ultimate loading before strengthening.

Fig.(7) Crack pattern of the specimen preloaded until first crack load and strengthened with full-CFRP (BC15)

Fig.(8) Delamination of concrete cover for specimen preloaded until first crack load and strengthened with full-CFRP (BC16)

Fig.(9) and Fig.(10) show the load-displacement curve for the control specimens and strengthened specimens before and after strengthening. All the load-displacement figures indicate a small increase in stiffness of strengthened specimens during the initial stages of loading. After cracking, and at any loading value, the displacements in the strengthened specimens are much less than the displacement in the non-strengthened specimens. However for the non-strengthened specimens (control tests and others preloaded) the displacements are almost similar or have little differences. This indicates that the strengthening process has made the joint more stiff and more ductile. The reduction in displacement is maximum for the specimens strengthened with full CFRP strips, and the least in the case of ferrocement strengthening.

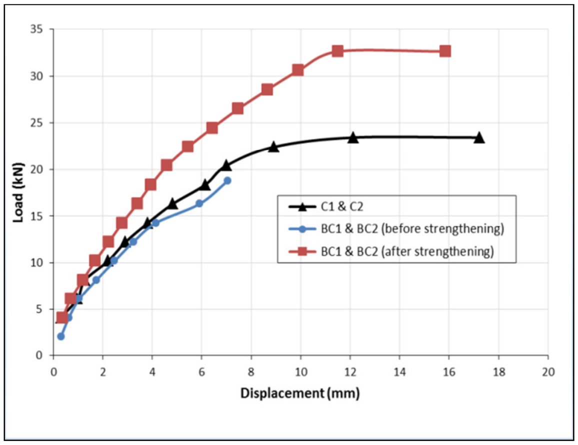

Fig.(9) Load-Displacement relationship for control specimens (C1 and C2) and strengthened specimens with strip of CFRP (preloaded up to 80% ultimate load)

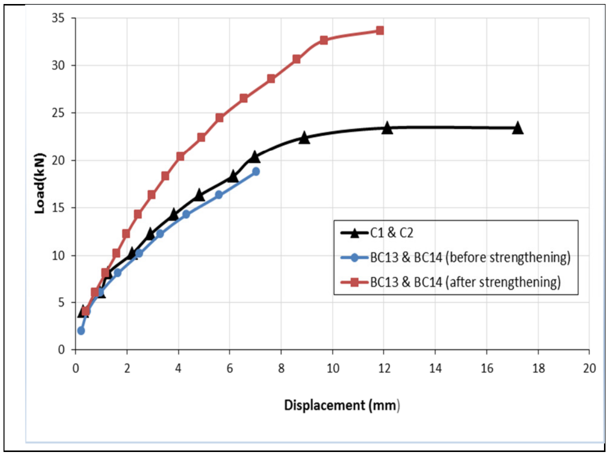

Fig.(10) Load-Displacement relationship for control specimens (C1 and C2) and strengthened specimens full-CFRP(preloaded up to 80% ultimate load)

Figs.(11) to (14) show the strain distribution across the cross section of the beam at a load level of 0.8 of the ultimate load for some of the specimens before and after strengthening process. The figures show clearly the compressive and tensile strain at top and bottom fibers respectively and the shift of the neutral axis after the strengthening process.

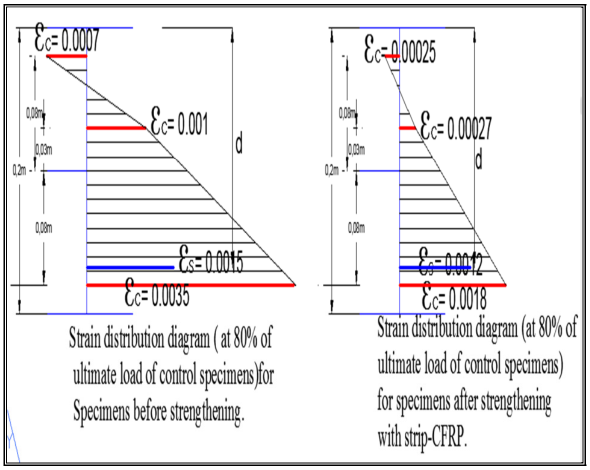

Fig.(11) Strain distribution diagram for specimens strengthened with strip of CFRP

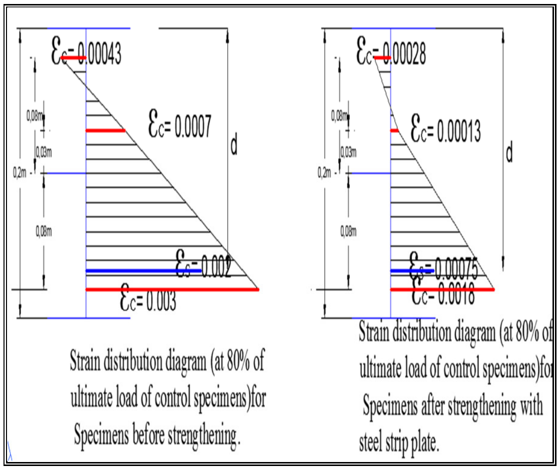

Fig. (12) Strain distribution diagram for specimens strengthened with steel strip plate

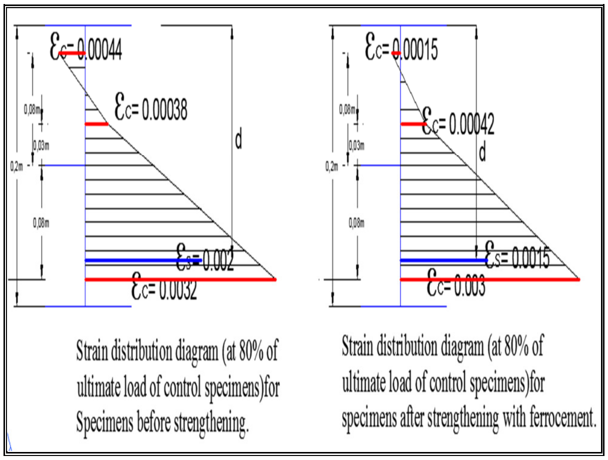

Fig.(13) Strain distribution diagram for specimens strengthened with ferrocement

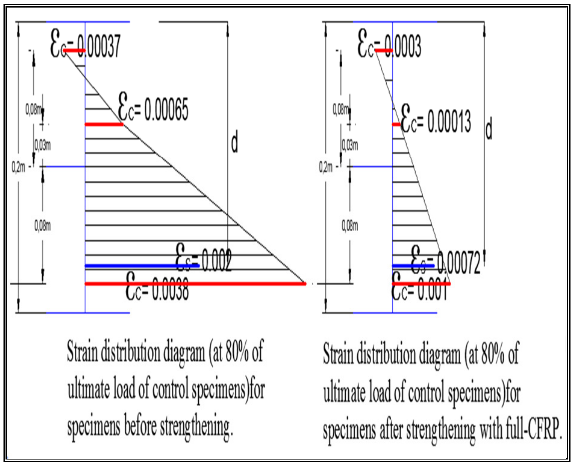

Fig.(14) Strain distribution diagram for specimens strengthened with full-CFRP

Conclusions

From the experimental tests carried out in this investigation, the following conclusions can be drawn.

1- The process of fixing CFRP sheets and steel strips to the concrete surface using bonding epoxies is successful and can furnish -- good composite actions.

2-A comparison between the control specimens and rehabilitated specimens showed that for all the methods of strengthening used, the ultimate joint strength is achieved.

3- Strengthening reinforced concrete beams-column joint with CFRP strips after damage can be done successfully for obtaining the ultimate load fairly higher than that of the control specimens. After strengthening, an increase in the ultimate load of 38.31 to 42.57 % was obtained for strengthening with strip of CFRP and full CFRP wrap, respectively.

4- Strengthening reinforced concrete beam-column joint with steel strips bonded using bonding epoxies with concrete surface after damage can be done successfully to recover the ultimate load of the joint.

5- Rehabilitating damaged beam-column joint with ferrocement through covering the whole damaged area is also a successful method for strengthening in order to recover the ultimate strength of the joint.

6-The stiffness of the strengthened specimens was increased compared to the control specimens. The specimen strengthened with full CFRP wrap exhibited the largest displacement reduction.

7- Preloading effects on load carrying capacity of beam-column joint can be considered as insignificant based on the obtained results. In some tested specimens loaded to 80% a slight reduction in load capacity was observed, compared to those preloaded to first crack loading.

8- Among the strengthening methods used in this investigation, the full coverage of the joint with CFRP strips is the best solution. Next is the CFRP strips provided very good care is taken in bonding the strips to the concrete surface.

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