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Strengthening of defected beam–column joints using CFRP



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ABSTRACT

This paper presents an experimental study for the structural performance of reinforced concrete (RC) exterior beam–column joints rehabilitated using carbon-fiber-reinforced polymer (CFRP). The present experimental program consists of testing 10 half-scale specimens divided into three groups covering three possible defects in addition to an adequately detailed control specimen. The considered defects include the absence of the transverse reinforcement within the joint core, insufficient bond length for the beam main reinforcement and inadequate spliced implanted column on the joint. Three different strengthening schemes were used to rehabilitate the defected beam–column joints including externally bonded CFRP strips and sheets in addition to near surface mounted (NSM) CFRP strips. The failure criteria including ultimate capacity, mode of failure, initial stiffness, ductility and the developed ultimate strain in the reinforcing steel and CFRP were considered and compared for each group for the control and the CFRP-strengthened specimens. The test results showed that the proposed CFRP strengthening configurations represented the best choice for strengthening the first two defects from the viewpoint of the studied failure criteria. On the other hand, the results of the third group showed that strengthening the joint using NSM strip technique enabled the specimen to outperform the structural performance of the control specimen while strengthening the joints using externally bonded CFRP strips and sheets failed to restore the strengthened joints capacity.

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Introduction

Occasionally, long after the structure has been completed, it is discovered that a contractor has left out some steel or some details are inadequately executed or the concrete is not what was specified. Fiber reinforced polymer, FRP, can be used in order to replace the missing steel or compensate the low concrete strength or structural faults in design. That is because FRP in the form of plates or fabric sheet has its strength in the direction of the fibers only and can be engineered to place the strengthening in the needed direction only. In addition, it can

provide an improved load carrying capacity and a higher rate of stiffness than that of un-strengthened specimens [1].

FRP composites have become more popular in the last two decades due to the reduction in their cost, combined with newer understanding of the versatility and benefits of the material properties. CFRP strips and fabric are generally constructed of high-performance carbon fibers which are placed in resin matrix. These composites can easily be externally bonded to RC elements. Strengthening with fiber-reinforced polymeric composite applications is one of the recent retrofitting and strengthening techniques [2].

The beam–column joint is considered as the most critical zone in a reinforced concrete moment resisting frame. It is subjected to large forces during earthquake excitation and its behavior has a significant influence on the response of the entire structure. As a result, a great attention has to be paid for good detailing of such joint. The absence of transverse reinforcement in the joint, insufficient development length for the beam reinforcement and the inadequately spliced reinforcement for the column just above the joint can be considered as the most important causes for the failure of the beam–column joint under any unexpected transverse loading on the building. Antonopoulos and Triantafyllou [3] demonstrated that externally bonded FRP reinforcement is a practical solution towards enhancing the strength, energy dissipation, and stiffness characteristics of poorly detailed, in shear, RC joints subjected to simulated seismic loads. Abdel-Wahed et al. [4] studied experimentally and analytically different CFRP strengthening configurations for beam–column joints having inadequate transverse reinforcement in the joint.

In the literature, many researches had been conducted experimentally in order to address the effectiveness of using FRP laminates for the strengthening of beam–column joints [5–9]. In addition, Ravi and Arulraj [10] studied numerically the behavior of beam–column joints retrofitted with carbon fiber reinforced polymer sheets. In the continuation, the effectiveness of composite fiber reinforced polymer layers for exterior beam–column connections was studied numerically considering strength and ductility enhancement of the RC joints [11].

Despite the fact that the defect of inadequate transverse reinforcement in the beam–column joint has been studied extensively in literature, other defects have to be studied in details. The current study conducts an experimental investigation on different strengthening configurations using CFRP for three defects encountered in the detailing of the exterior beam–column joints. In addition to the defect caused by the absence of transverse reinforcement in the beam–column joint, the insufficient bond length for the beam main reinforcement and inadequately spliced implanted column were also studied.

Strengthening material

One of the most important factors affecting the successful strengthening technique of structures is the selection of the strengthening material. The need to lower the cost of maintenance, repair and strengthening techniques, while extending the service life of the structures, has resulted in new systems, processes, or products to save money and time. The fiber-reinforced polymer (FRP) systems are recently used in the field of strengthening and restoration of the buildings. The most com-

monly utilized fiber-reinforced polymers (FRPs) are fibers made of carbon (C) or glass (G). These materials can be designed and used in the form of laminates, rods, dry fibers (sheets) adhesively bonded to the concrete, wet lay-up sheets mounted on the surface, or near surface mounted bars or laminate strips in the concrete cover [12]. The carbon fiber-reinforced polymer (CFRP) materials have a high potential for manufacturing effective strengthening systems to increase the flexural or shear strength of RC beams. The CFRP materials have a very low weight to volume ratio, are immune to corrosion, and possess high tensile strength. FRP systems may have thermal expansion properties that are different from those of concrete. In the fiber direction, CFRP systems have a coefficient of thermal expansion near zero, however previous research work [13] has indicated that the thermal expansion differences do not affect the bonding for small ranges of temperature change ($+/-50$ °C). Also, due to their electrical conductivity, Ghali et al. [14] concluded that carbon based FRP materials should not come in direct contact with steel to avoid potential galvanic corrosion of steel reinforcement and, a minimum concrete cover of about 10 mm was recommended.

The performance of the FRP system over time in an alkaline or acidic environment depends on the matrix material and the reinforcing fiber. Unprotected carbon fiber resists both alkali and acid environments while bare glass fiber can degrade over time in these environments [15]. However, a properly applied resin matrix may isolate and protect the fiber from the alkaline/acidic environment and retard deterioration.

Compared to the traditional strengthening techniques (externally bonded steel plates, near surface mounted steel bars and concrete jackets), the cost of the externally bonded CFRP system is relatively high but, in some special circumstances, and regarding the aforementioned advantages, the choice of the CFRP as a strengthening material may represent the best solution.

Experimental

Test specimens

A total of eleven half-scale beam–column *T*-joints were prepared and cast in the current study. The first specimen, J0, was considered as the base control specimen. It had an extruded beam of 900 mm length and cross-sectional dimensions of 200 × 300 mm. This beam was connected to a column at its mid-height point. The cross-section of the column was 200 × 300 mm. The total length of the columns was 2.3 m divided into two equal parts, lower part and upper part.

The upper and lower reinforcement of the beam in addition to the main longitudinal steel reinforcement of the column were made from high tensile steel. The main steel reinforcement of the beam was three bars of 16 mm diameter, while the secondary steel reinforcement was two bars of 12 mm diameter. On the other hand, the column was reinforced with four bars of 16 mm diameter at each corner of the column cross-section. The stirrups for both beam and column were mild steel bars of 8 mm diameter and spaced every 100 mm and 150 mm for the beam and the column, respectively. In addition, three stirrups were added at the beam–column joint. Fig. 1 shows the concrete dimensions and reinforcement detailing for the base control specimen provided that the loading

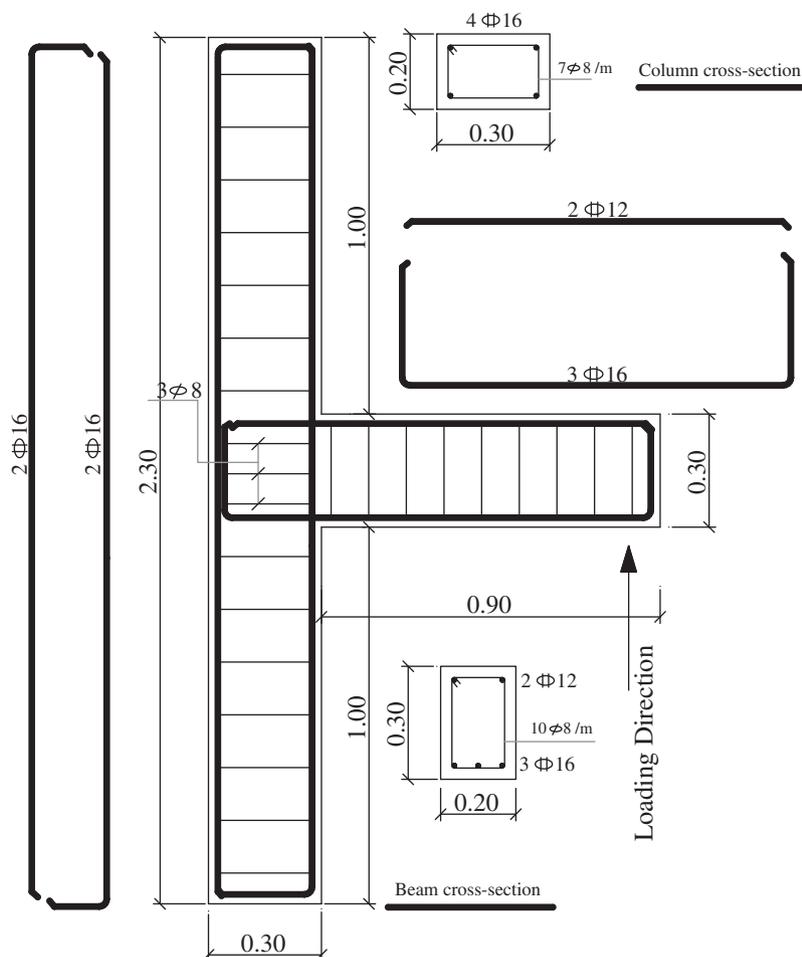


Fig. 1 Concrete dimensions and reinforcement details for the base control specimen, J0.

direction on the beam end was acted at the bottom side according to the adopted testing setup.

The remaining 10 specimens were divided into three groups representing the considered three defects as shown in Fig. 2. The first group, group #1, contained three specimens: JI0, JI1, and JI2, representing the reference specimen and two strengthening configurations, respectively. This group represented the first defect which was the absence of the stirrups at the beam-column joint. Group #2 represented the defect of insufficient bond length for the beam main steel reinforcement. This group contained three specimens: JII0, JII1, and JII2 representing the reference specimen for such group and two strengthening configurations, respectively. Group #3 represented the third defect that was deficiently executed implanted column on an old one. This group contained four specimens that were one reference specimen along with three different strengthening configurations. Fig. 3 represents the three defects of the beam-column joints. Group #3 was executed in three steps: the first step was the casting of the lower column along with the extruded beam monolithically. The second step was the drilling of four 50 mm depth holes to accommodate the longitudinal steel reinforcement for the upper column then the holes were filled with epoxy to hold the steel bars in their positions. The third step was the preparing of the surface of old concrete then casting the upper column as shown in Fig. 4.

All specimens were cast horizontally in wooden forms. Two days after casting, the standard cubes and the sides of the specimens were stripped from the molds and covered in wet Hessian until the seventh day, when the Hessian was removed and the specimens allowed air-drying until testing.

Strengthening scheme

In accordance with ACI 440 [15] recommendations, seven specimens were strengthened with either CFRP fabric or CFRP strips as shown in Fig. 2. Based on the failure patterns of the base control specimen and the reference specimens included into the three groups, the strengthening configurations were proposed. For both group #1 and group #2, two strengthening configurations were proposed, while three configurations were considered for group #3.

For group #1, the first strengthening configuration represented two perpendicular overlaying fabric sheets on the beam-column joint. One layer of 200 mm width and 1000 mm length parallel to the column axis was bonded to each side of the column. Then a horizontal U-shaped layer of 200 mm width and extended by 600 mm length parallel to the beam axis was bonded to each side. Finally, three 100 mm in width U-shaped sheets were used at the joint in order to prevent the premature peeling of the sheets at the

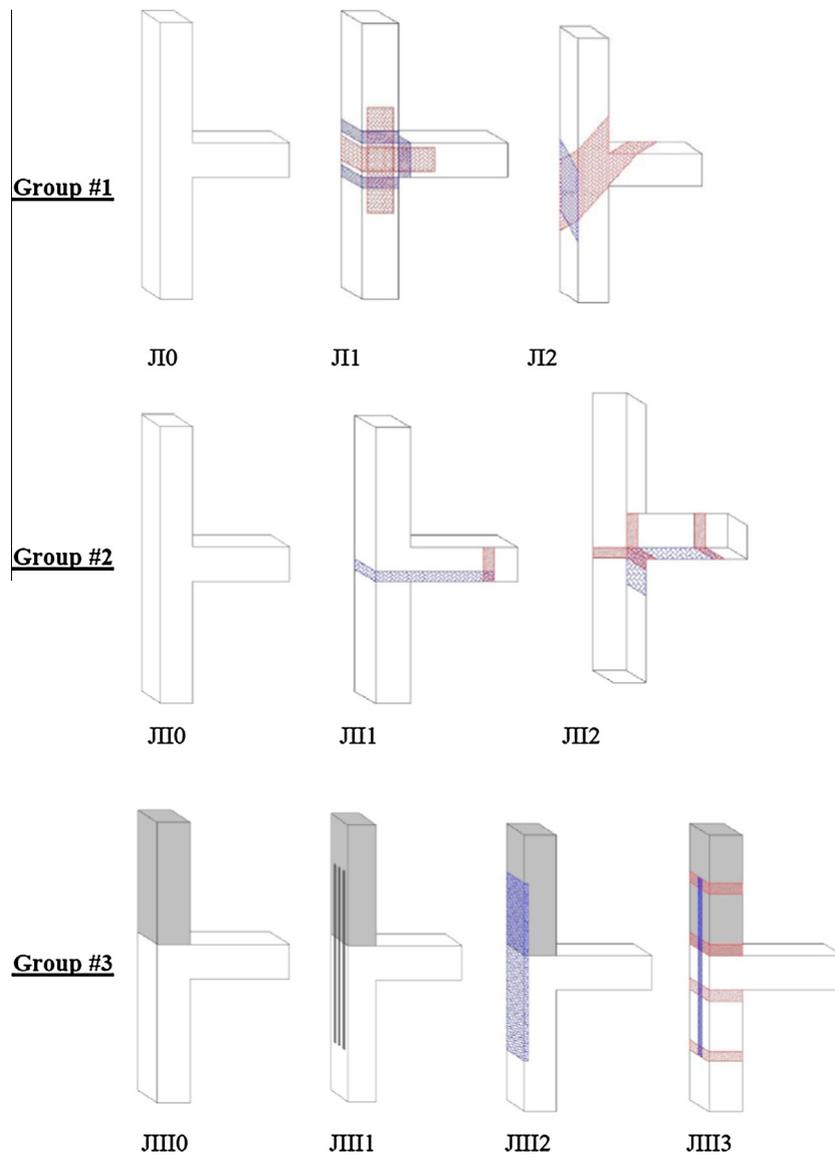


Fig. 2 Schematic representation for the considered three groups.

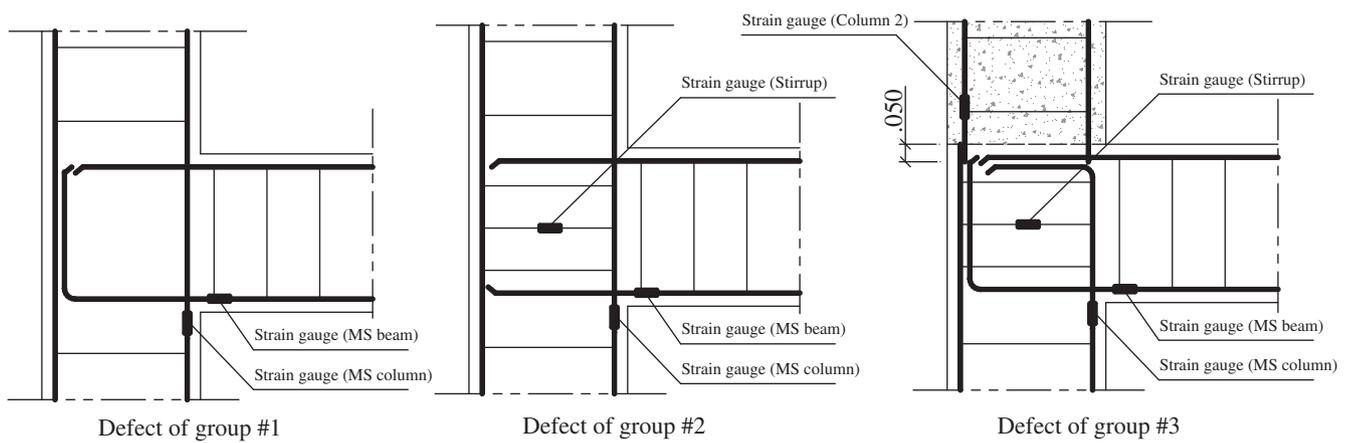


Fig. 3 Details of the three defects.

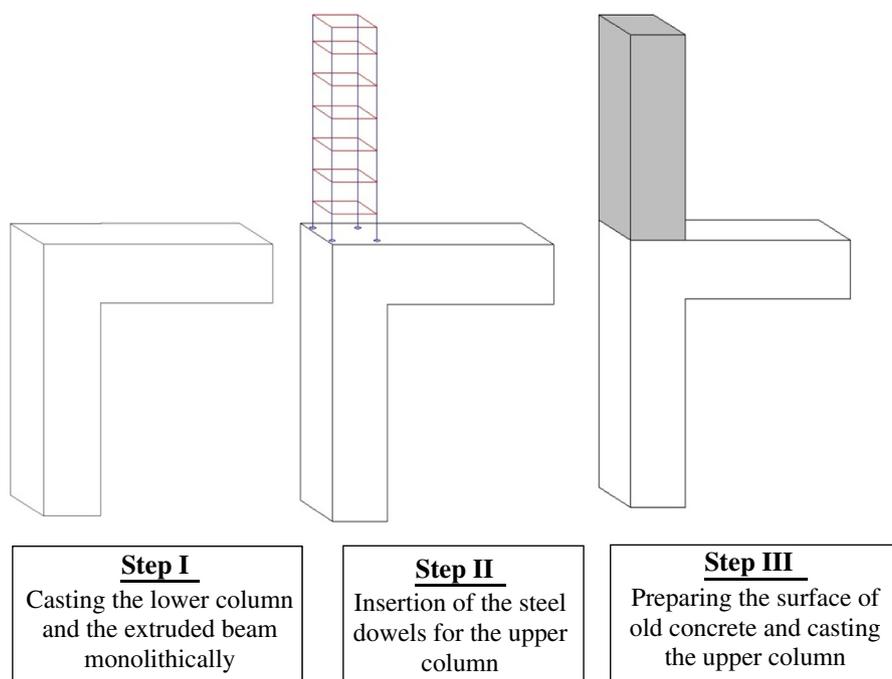


Fig. 4 Execution steps for the specimens of the group #3.

beam–column joint [16,17]. The three anchorages U-shaped sheets were bonded in the transverse direction of the beam at the column face and in the transverse direction of the column just below and up the beam faces. The second configuration represented a 45° inclined one layer of a 500 mm wide U-shaped bonded at each side covering the beam–column joint.

As for group#2, the first strengthening configuration represented four layers of 100 mm wide sheets in the form of U-shape that were bonded to the lower side of the beam parallel to its axis. The sheets were designed to compensate the main steel of the beam with an efficiency factor of 0.5. These sheets were placed so that they covered the column width and ran up both sides of the beam to a length of 1000 mm. 100 mm in width U-shaped sheet was used at the end of the horizontal layers in order to prevent the premature peeling of the sheets. The second configuration represented two 200 mm in width L-shaped sheets bonded to the lower face of the beam and extended by 300 mm in the direction of the lower column. Three 100 mm width U-shaped anchorage strips were used at the free end of the sheet, the beam at the column face and the lower column at the beam face, respectively.

Three strengthening configurations were used in group #3 making use of CFRP fabric sheets and CFRP plates. The most important criterion for the three configurations was that the area of either CFRP plates or the fabric sheet was the same. The first configuration represented near-surface mounted, NSM. NSM FRP technique does not require extensive surface preparation work and, after groove cutting, requires minimal installation time compared to externally bonded FRP laminates [18]. Three CFRP strips inserted into grooves cut at the outer surface of the column were used. The strips, as provided by the manufacturer have a nominal width of 50 mm and a total thickness of 1.2 mm. In order to insert the strips within a typical concrete cover used for concrete members, the strips

were cut into three equal parts each, 16.6 mm wide. Using a concrete saw, approximately 5 mm wide and 17 mm deep and 1500 mm long grooves were cut into the outer surface of the column [19]. The grooves were injected with epoxy adhesive to provide the necessary bond with the surrounding concrete. The strips were carefully placed into the grooves to ensure that they were completely covered with the epoxy. The second configuration represented two layers of CFRP fabric sheets having 1500 mm in length which were bonded to the outer column surface. In order to obtain the same area of the former configuration, the sheets were extended 15 mm in the column sides. The third configuration represented the externally bonded CFRP plate of cross-sectional dimensions of 12 × 50 mm and extended 1500 mm in the column direction. Four 100 mm U-shaped anchorage strips were used to hold in position the CFRP plate as shown in Fig. 2.

Material properties

The used concrete was normal strength concrete of 25 MPa target strength, which was the average of three standard cubes of 150 mm side. The concrete mix contained a blend of type I and type II crushed pink limestone as the coarse aggregate whose maximum aggregate size was 16 mm. The sand was supplied from a local plant around the site and its fineness modulus was 2.8%. The volumes of limestone and sand in one cubic meter were 0.82 m³ and 0.41 m³, respectively. The used cement is normal Portland cement (Type I) with 3 kN/m³ as cement content and the water–cement ratio was 0.42. The longitudinal reinforcement for both beam and columns was deformed bars of 400 MPa yield strength while the stirrups were ordinary mild steel of 240 MPa yield strength. The modulus of elasticity for both types of reinforcements was 200 kN/mm². As for the strengthening materials, Table 1 shows the mechanical proper-

Table 1 Mechanical properties of CFRP material.

Criteria	CFRP strips	CFRP fabric	Epoxy (for strips)	Epoxy (for fabric)
Tensile strength (MPa)	2800	3500	30	30
Modulus of elasticity (GPa)	165	230	12.80	21.40
Failure strain (%)	1.70	1.50	1.0	4.80
Shear strength (MPa)	–	–	30.0	15.0
Thickness (mm)	1.2	0.13	–	–

ties for both CFRP strips and fabric sheets along with the epoxy resins as provided by the manufacturer.

Test setup and test procedure

One bay of three-dimensional steel frame of Concrete and Heavy Structures Laboratory, Faculty of Engineering, Tanta University, was equipped and used to carry out testing as shown in Fig. 5. The specimens were considered hinged at both column ends. Steel caps were used at both ends of the column to distribute the column compression load at the upper end and to support the column lower end at the testing frame. In addition, a threaded rod was wrapped around the upper steel cap and fastened to the column of the testing frame to prevent any tilting of the specimen during testing.

100 mm LVDT was used in order to measure the vertical displacement at the tip of the beam. 10 mm strain gauges were used to measure the developed strains on reinforcement at the tension sides for beam, columns, stirrups and FRP. A compression load equals to 200 kN, simulating the load in a real structure, was first applied to the column before the beam was loaded. Such column load was kept constant during the loading phase. Therefore, in several steps the beam was loaded up to failure. The loads on both column and beam were mea-



Fig. 5 Test setup.

sured by a load cell of 600 kN capacity. Before fastening the specimen to the testing frame, a laser level was used to insure the verticality of the column and to ensure the coincidence of the axes of the specimen, the load cell on the upper column and the load cell on the beam.

After each loading step, the vertical beam tip deflection and the strains in the tension sides of the beam and the columns, the stirrup and the FRP were recorded. The loading rate for all specimens ranged from 5 to 10 kN/min. An automatic data logger unit (TDS-102) had been used in order to record and store data during the test for load cells, steel strain gauges, developed tensile strain on FRP, and LVDT.

Results and discussion

Table 2 summarizes the recorded failure characteristics after complete collapse of all specimens. In the following clauses, the criteria most related to the failure modes for the reference defected specimens and the CFRP-strengthened specimens are discussed in detail. The considered criteria include the mode of failure, load–deflection relationship, ultimate capacity, ductility and initial stiffness, along with the ultimate developed strains on the main reinforcing bars at the joint, ultimate strain on joint stirrup and ultimate strain on either CFRP sheets or plates.

Mode of failure

Cracks began to appear at the tension side of the beam at a vertical load of about 10 kN for the base control specimen in addition to all reference specimens for the three groups. For the base control specimen, J0, increasing the vertical load led to increasing the propagation of cracks on the beam tension side up to a vertical load of about 25 kN. Then, cracks began to appear at the tension side of the lower column at a vertical load of about 45 kN where diagonal shear cracks began to appear inside the beam–column joint. Increasing the load further increased the size of cracks further till the failure occurred due to the flexural mode at the interface of the beam to the column at about 80 kN.

As for the first group, group #1, that represented the defect of the absence of stirrups at the beam–column joint, the reference specimen, J10, showed approximately the same behavior of the base control specimen, J0, except that the diagonal shear cracks began to appear at a vertical load of about 40 kN which is lower than that of the base control specimen. This can be attributed to the absence of transverse reinforcement. The diagonal shear failure was controlled the failure of such specimen at about 67 kN vertical load which is lower than the failure load of the base control specimen. Strengthening the joint using CFRP sheets yielded the increased ultimate capacities of

Table 2 Experimental results and failure characteristics of all specimens after complete collapse.

Group#	Specimen	f_{cu} (MPa)	Cracking load (kN)		Pu (kN)	Deflection (mm)	Toughness (N mm)	Ultimate strain at failure, micro-strain				Dominant mode of failure			
			Beam	Column				Joint	MS Beam	MS Column	Stirrup		Column2	FRP 1	FRP 2
Control	J0	25.4	10	25	45	79.8	40.2	2655	2350	1479	1310	-	-	-	Flexural failure of the beam
	J10	24.8	10	25	40	67	29.6	1733	1600	1428	-	-	-	-	Shear failure at the joint
Group #1	J11	25.1	10	-	-	103.8	19.3	1276	2780	1462	-	-	-	-	Rupture of the CFRP sheet
	J12	25.8	58	-	-	107.8	26.9	2163	2900	1740	-	-	-	-	Rupture of the CFRP sheet
Group #2	J110	25.9	10	32	50	70	20.4	971	1720	1413	1000	-	-	-	De-bonding of main reinforcement
	J111	26.1	48	38	63	85	13.1	726	3301	1173	1110	-	-	-	Peeling off at the end of the CFRP sheets
	J112	25.4	35	-	65	89.7	14.8	953	3542	1633	1190	-	-	-	Rupture of the CFRP sheet
Group #3	J1110	25.3	10	40	50	65.1	15.2	629	1610	1220	850	1010	-	-	Splitting flexural failure at the upper implanted column
	J1111	26.1	10	45	65	85.1	26.8	1762	3102	1091	1075	1450	-	-	Peeling off at the end of CFRP plates
	J1112	26.5	10	25	40	73.7	13.3	621	1750	1273	1010	1300	-	-	Peeling off at the end of CFRP sheets
	J1113	25.8	10	-	42	66.2	20.4	989	1690	1260	890	1080	-	-	De-bonding of CFRP sheet followed by rupture of CFRP anchorage sheet

MS Beam: strain on the main steel of the beam, MS Column: strain on the tension main steel of the lower column, Column2: strain on tension main steel for the implanted upper column, FRP1: strain on the longitudinal direction of the strengthening CFRP, FRP2: strain on the horizontal CFRP sheet at the joint.

the joint by about 55% and 61% as for specimen J11 and J12, respectively, compared to that of the reference specimen, J10. Specimen J11 began to crack at the same load as that of the reference specimen but cracks began to appear at higher load for specimen J12 which was about 58 kN. The proposed configurations for strengthening this defect showed their efficiency in increasing the ultimate capacity of the joint. In addition, the failure of both configurations occurred on the CFRP sheets by rupture of the sheets at the joint.

Group #2 represented the second defect which is the insufficient bond length of the main tensile steel of the beam. The failure of the reference specimen J110 was flexural failure accompanied with debonding of the beam main steel. The failure of this specimen occurred at lower vertical load compared to that of the base control specimen, J0. The ultimate capacities of the two proposed configurations were higher than that of the reference specimen J110 by about 21% and 28%, respectively for specimen J111 and specimen J112. On the other hand, these increases were 6.4% and 12.5%, respectively, compared to that of the base control specimen J0. This means the proposed configurations were not strong enough for significant gain in strength and additional layers of CFRP are recommended. The failure of the first strengthening configuration was characterized by the peeling off the CFRP layers while the rupture of the CFRP sheets characterized the failure of the second configuration.

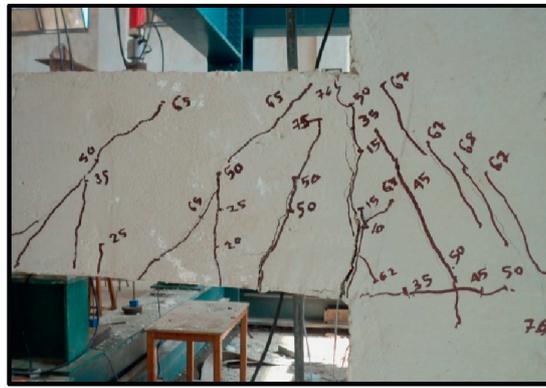
Group #3 represented the defect of poorly spliced implanted column. The failure of the reference specimen, J1110, was characterized by splitting of the upper implanted column at about a vertical load of about 65 kN which is lower than that of the base control specimen by about 19%. The most appearing phenomenon for all specimens of this group was that all of them began to crack at the same vertical load, and then different behavior was noticed till the complete failure had occurred. The failure of both specimens J111 and J112 was due to the peeling off of either the CFRP NSM or CFRP sheet. On the other hand, rupture of the anchorage U-shaped characterized the failure of specimen J1113.

For all specimens, the cracking load for the beam, column and the joint in addition to the mode of failure are included in Table 2. In addition, failure shapes of all specimens are presented in Fig. 6.

Load–deflection relationship

Fig. 7 shows the load–deflection relationship for all specimens of group #1 in addition to the response of the base control specimen. At the beginning of loading, all specimens approximately showed the same deflection till a value of about 10 kN then, both specimens J0 and J10 showed identical response up to about 45 kN. On the other hand, the strengthened specimens, J11 and J12, showed different response up to failure where specimen J12 (strengthened using diagonal CFRP sheets) showed lower deflection compared to that of specimen J11 at the same loading level. In the continuation of the loading phase, the reference specimen J10 showed the highest corresponding deflection at the same loading level compared to that of the base reference specimen in addition to the strengthened specimens.

Fig. 8 shows the same trend for all specimens of group #2. The reference specimen J110 showed the highest corresponding



(a) Base control specimen, J0



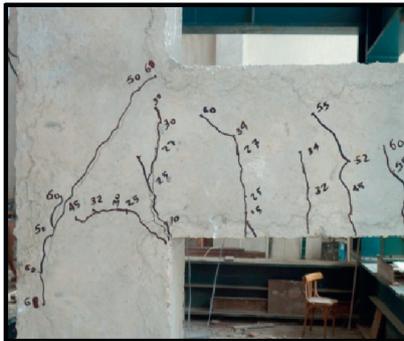
(b) Specimen JI0



(c) Specimen JI1



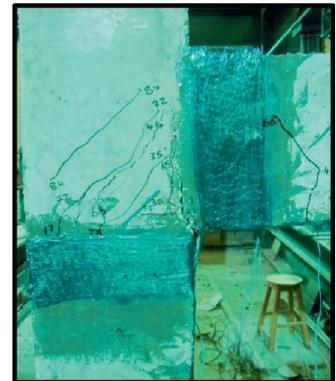
(d) Specimen JI2



(e) Specimen JII0



(f) Specimen JII1



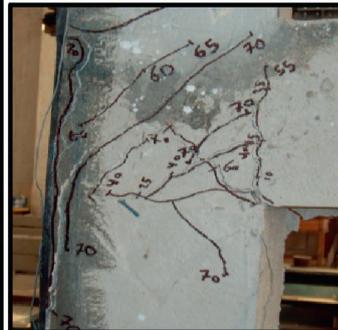
(g) Specimen JII2



(h) Specimen JIII0



(i) Specimen JIII1



(j) Specimen JIII2



(k) Specimen JIII3

Fig. 6 Failure shapes of all specimens.

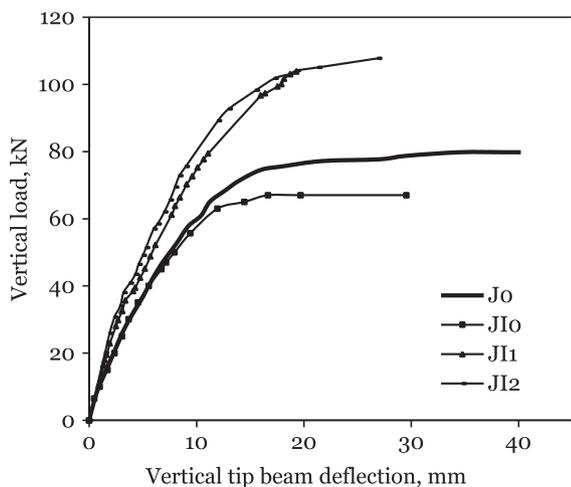


Fig. 7 Vertical load versus vertical deflection at the beam free end for all specimen of group #1.

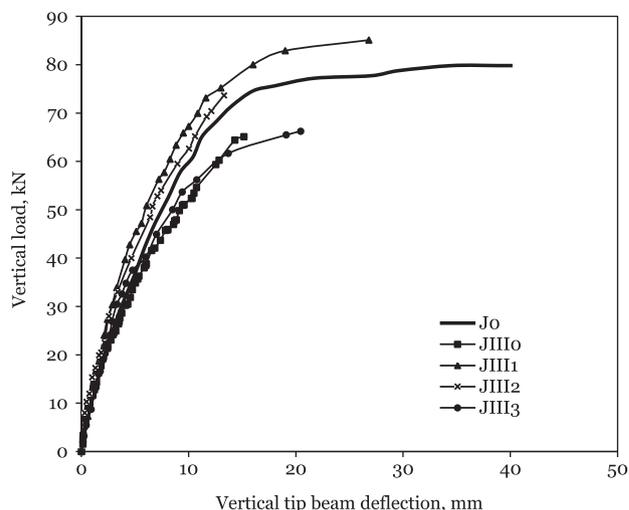


Fig. 9 Vertical load versus vertical deflection at the beam free end for all specimen of group #3.

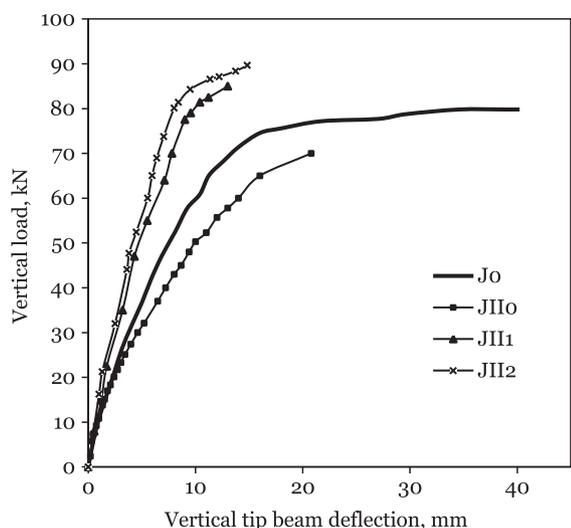


Fig. 8 Vertical load versus vertical deflection at the beam free end for all specimen of group #2.

vertical deflection at the same loading level. In the same way, the strengthened specimens showed lower deflections compared to that of both the reference specimen, JII0, and the base control specimen, J0. However, the responses for group #2 showed noticeable variations in contrast to that of group #1.

Fig. 9 shows the load–deflection response for group #3. The same trend as that of the former groups regarding the reference specimens and the strengthened specimens was noticed. However, the variations among the responses were smaller than that exhibited by the former groups. This is the only group that used both CFRP sheets and plates. NSM strips, showed the lowest vertical deflection at the same loading level among the group.

Ultimate capacity, initial stiffness and ductility

The most important factors in the design process are the ultimate capacity and ductility. It is experienced that using CFRP

as strengthening technique increases ultimate capacity and reduces ductility in the same time. This means that it is a matter of compromise to choose the most mandatory criteria for the structure under consideration.

The most evident phenomenon is that all defected reference specimens exhibited lower ultimate capacity than that of the properly detailed base reference specimen, J0. The comparisons here will be considered in two levels; with respect to the reference specimen inside the group, and with respect to the base control specimen, J0.

As for group #1, the strengthened specimens, JI1 and JI2, showed an increase in the ultimate capacity by 55% and 61% compared to the reference group JI0, respectively, while these increases were 30% and 35%, respectively, compared to that of base control specimen, J0. This indicates that the strengthening configuration for that defect is properly chosen based on the ultimate capacity viewpoint. In addition, the diagonal overlaying sheets showed higher performance than that of the orthogonal overlaying. The strengthened specimens of group #2 showed lower percentages of capacity gain compared to that of group #1. This is reasonably good because the strengthening configurations were chosen in order to compensate only the beam main steel not to increase the ultimate capacity. The percentages of increases in the ultimate capacities for specimens JII1 and JII2 were 21% and 28%, respectively, compared to that of reference specimen JII0 and were 6.5% and 12.4%, respectively, compared to that of the base control specimen, J0.

As for group #3, one configuration (NSM) only gave higher capacity compared to that of the base reference specimen, J0. That increase was about 6.6% compared to that of the specimen J0. While, the specimens having overlaying sheet or plate exhibited higher capacity compared to the reference specimen JIII0, they did not convey the capacity to that of the base reference specimen, J0. Although, the specimen JIII1 and specimen JIII3 had the same area of the CFRP plate, NSM showed higher capacity due to the good orientation of the plates in the direction of higher stiffness of these plates. Fig. 10 shows a comparison among all the specimens considering the ultimate capacity. It can be noted that specimen JIII1

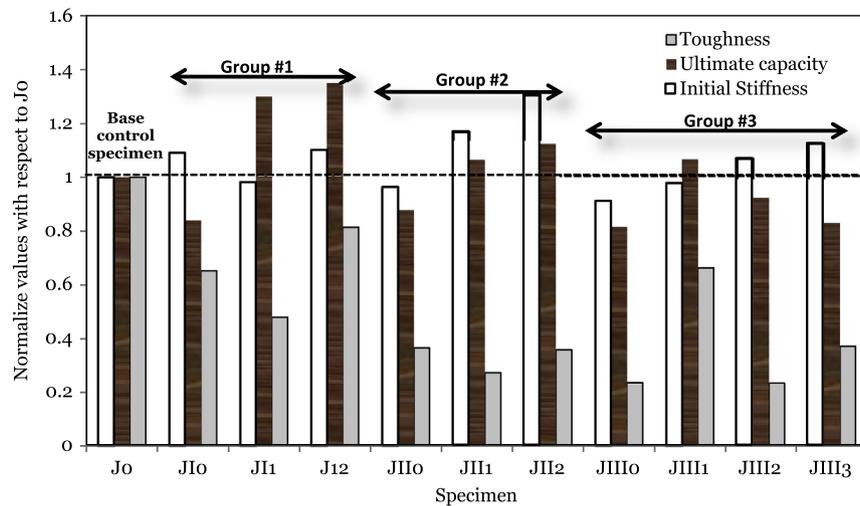


Fig. 10 Comparison among all specimens concerning ultimate capacity, initial stiffness and ductility.

showed a reasonable increase in the ultimate capacity compared to that of the properly detailed specimen, J0. However, specimens JIII2 and JIII3 showed lower ultimate capacities compared to that of specimen J0. This means that those former two strengthening configurations do not represent the best solution.

As for the initial stiffness, Fig. 10 shows that all reference specimens (JI0, JII0 and JIII0) exhibited lower initial stiffness compared to that of the base control specimen, J0. In addition, approximately all strengthened specimens showed higher initial specimen compared to that of specimen J0. In addition, Figs. 7–9 show that during the loading course all strengthened specimens yielded higher instantaneous rate of stiffness compared to that of the base control specimen, J0.

Ductility may be broadly defined as the ability of a structure to undergo inelastic deformations beyond the initial yield deformation with no decrease in the load resistance. Toughness of the system can be defined as the maximum energy that can be sustained by the system up to failure. It can be used as an indicator for the ductility where higher toughness means higher dissipation of energy, until the failure occurred leading to higher ductility. The toughness can be defined as the area under the load–deflection curve. Fig. 10 shows comparison among all specimens from the ductility viewpoint. It can be concluded that both reference un-strengthened specimens and strengthened specimens for the three defects exhibited lower ductility compared to that of the base control specimen J0. The most important phenomenon is that it is not necessarily that the specimen having higher strength yielded lower ductility. That happened for specimens JIII2 and JIII3 where both of them showed lower strength and lower ductility compared to specimen J0. In addition, the strengthened specimen JI2 which had the maximum capacity showed also the highest ductility among its group.

Ultimate strains

Fig. 3 shows the position of the measured strain on the reinforcing steel as included in Table 2. In this part, only the developed strains at the failure state will be compared. As shown in Table 2, the strain on the middle stirrup of the joint has only

yielded for the base control specimen, J0 (yield strain = 1200 micro-strain). However, the strengthened specimens of the second defect showed high values approaching the yield strain. The strain on stirrup for all strengthened specimens of the three defects showed higher values compared to those of the un-strengthened relevant reference specimen. In contrast, neither the lower column strain nor the strain on the spliced upper column reached the yield strain for both reference specimens and the CFRP-strengthened specimens.

The strain on the main tensile steel of the beam had shown different behavior where the strain for all reference specimens did not reach the yielding point except the base control specimen. In addition, the beam tensile strain for the first and second defects exceeded the yield strain. That reflects the adequacy of the chosen strengthening configuration which assisted the concrete section to reach up to its limit. However, the last two strengthening configuration of the third defect showed that they do not represent the best configuration for that defect, where the strain on the beam main steel did not yield. This observation was confirmed through the recorded tensile strain on the CFRP sheets and plates that were below the failure strain of CFRP as given in Table 1. On the other hand, the recorded strains on the CFRP sheets for the group #1 affirm that it was the best choice of the strengthening configuration.

Conclusions

Based on the studied dimensions of the beam–column joint and the considered defects along with the proposed CFRP strengthening configuration subjected to incrementally monotonic static loading, the following conclusions can be drawn:

- (1) Using either CFRP fabric sheets or plates as strengthening material showed its efficiency in enhancing the failure characteristics of the defected beam–column joints if only the proper configuration was chosen.
- (2) The diagonal overlaying sheets was observed to be the better configuration to strengthen the defect of the absence of joint stirrups. While, the L-shaped fabric sheet showed its adequacy to strengthen the defect of insufficient bond length for the beam main steel

provided that anchorage U-shaped layers were used in the joint. On the other hand, NSM-CFRP plates showed the highest performance in case of inadequate spliced column.

- (3) The orientation of the CFRP plates has a great effect on the performance of the strengthened joint. Comparing the responses of both specimens JIII1 and JIII3 which had the same volume of the CFRP plate assure that evidence. Specimen JIII1 has NSM plates while JIII3 has an overlaying plate.
- (4) Generally, using CFRP as a strengthening material led to increased ultimate capacity and decreased ductility compared to those of un-strengthened joints.
- (5) End anchorage sheets manifested its advantage especially in case of member under flexure. The visual observation of the failure of specimen JIII1 showed that the joint can sustain additional loading if the anchorage U-shaped remained unpeeled off the beam soffit.

Conflict of interest

The authors have declared no conflict of interest.

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