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Finite Element Analysis of Retrofitting Techniques for Non-Ductile Reinforced Concrete Beam-to-Column Joints Subjected to Cyclic Loading: A Parametric Study

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Abstract

The experience of past earthquakes has shown that the beam-column joints in moment frames are one of the key members in the path of lateral load transfer and their poor performance results in the total collapse of the building. In this study, four reinforced concrete beam-column joints were simulated using the nonlinear finite element analysis ABAQUS software. Different parameters were taken into consideration including Carbon fiber reinforced polymer (CFRP) grid and engineered cementitious composites (ECC), externally bonded reinforcement (EBR) of CFRP sheets, EBR of CFRP sheets combined with steel anchor bolts, externally bonded reinforcement on grooves (EBROG) of CFRP sheets, Hybrid FRP grid (GFRP+CFRP) and ECC, and steel jacketing. The nonlinear finite element analysis results showed that the steel jacketing method can significantly enhance the beam-column joint performance leading to an 87% higher lateral load capacity.

Keywords: Engineered cementitious composites, Carbon fiber reinforced polymer, Steel jacketing, Finite element analysis, ABAQUS

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1. Introduction

Existing reinforced concrete (RC) structures may need to be repaired or strengthened, as a result of material deterioration, usage changes, new building codes, or new design specifications [1]. The performance of beam-column joints is a key factor in the structural behaviour and integrity of RC buildings exposed to lateral cyclic loads. The lateral load capacity of the beam-column joint depends on the strength of the connection's three main components (beam, column, and joint). All design codes must include special provisions for the beam-column joints of moment resistant frames in order to guarantee a certain ductility level with controlled damage when resisting seismic forces. Internal moments with the same direction develop as a result of this loading condition in the beams on both sides of the joints [2]. As a result, the moments pull the bottom in the opposite direction from the top bars, which are continuous within the joint [3]. The top and bottom bars produced tensile forces cause bond stresses to form between the steel and concrete bars. Weak column-strong beam condition will result in bond failure. The steel bars will slide into the joint region as a result, decreasing the beam's carrying capacity. Additionally, the joint deforms, with one dimension growing while the other shortens, as a result of the pull and push forces that originate near the top and bottom ends of the joint. If the column's size cannot accommodate this deformation, diagonal cracks start to appear in the concrete near the connection [4]. In order for the B-C connection to function as intended for RC moment resistant frames subjected to earthquake loading, it must have additional reinforcing to control the diagonal fractures and prevent crushing of the concrete in the joint area [5], [6], [7]. The primary reasons of the B-C connections collapsing are a lack of transverse reinforcement in the joint area and inadequate splicing. To improve the lateral stiffness and strength of old structures, the traditional retrofitting procedure involves constructing or strengthening shear walls, braces, or frames [8]. These methods are commonly used as a part of a worldwide strategy to increase the seismic resilience of structures constructed in the 1970s [9]. Column jacketing is another well-known RC strengthening technique. Composite columns are created by laminating pre-existing columns with a layer of RC, offering a lateral load-carrying structure with a higher load capacity [10]. This was the preferred option for retrofitting multiple medium-rise structures affected by earthquakes [11]. This method, like every other method, has some advantages and disadvantages. For instance, while it is beneficial in providing a uniform increase in strength and stiffness [12], it causes significant disruption to the building's occupants during construction [10]. New methods for seismic retrofitting of structures have been proposed throughout the past few decades. One of these techniques relies on the FRP composites. FRP have an advantage over other materials, due to their better strength-to-weight ratio, ease of installation, minimal increase in the size of the strengthened elements, and non-corrosive nature [13], [14].

The most effective FRP material for strengthening structural elements is the carbon fiber reinforced polymer (CFRP), which has unique qualities such as high strength to weight ratio, high stiffness, excellent corrosion and alkali resistance. It was proven that FEA was a suitable method for structural simulations. FEA programs can solve linear and nonlinear, static and dynamic, implicit and explicit structural problems and provide a comprehensive set of solutions [15], [16], [17].

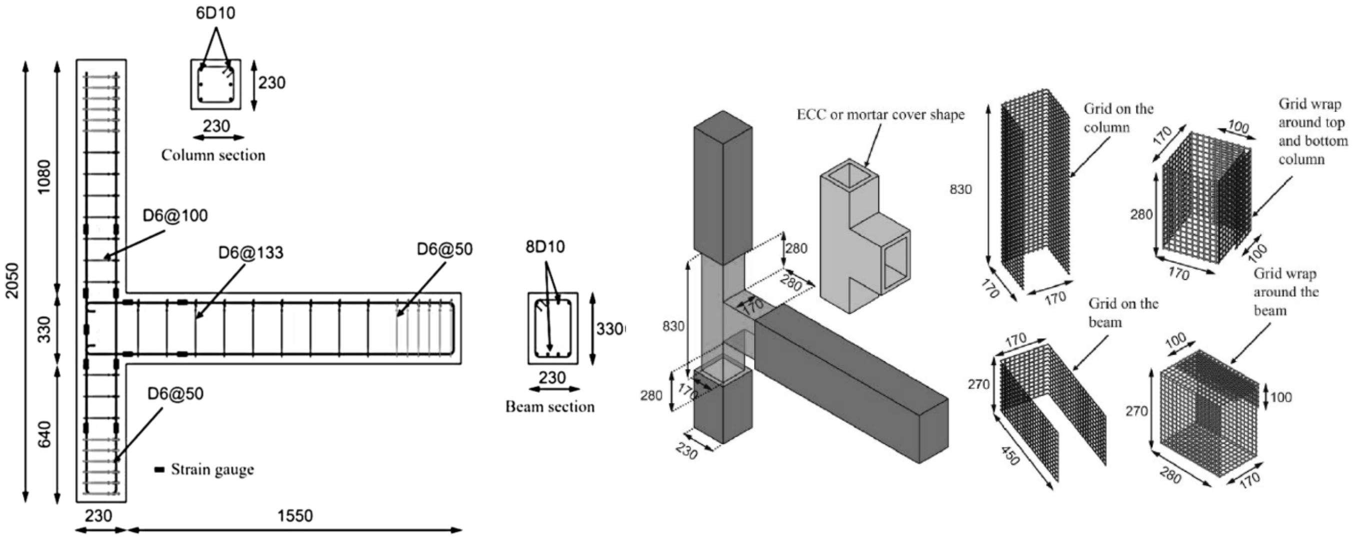
In this work, a variety of parameters that are related to the response of the beam-column joint when exposed to cyclic loading are examined by systematic nonlinear FEA using ABAQUS software [18]. The study considers the impacts of the configuration of the strengthening technique. The typical design philosophy is to produce a strong column-weak beam action in beam-column joints that are a part of moment resisting frames, which provides the level of ductility needed by the joint and ensures plastic hinge creation in the beam away from the joint core.

1. Experimental work verification

Figure. 1 shows the dimensions and reinforcement details of the previously experimented beam-column joints [19]. The column had a length of 2050mm and cross-sectional dimensions of 230×230mm, while the beam was 1550mm long and had cross-sectional dimensions of 230×330mm. Simple rollers were used to support the beams ends, and the hinged support was installed on the bottom end of the column while the other end was left free to allow the relative drift. Deformed bars with yield strengths of 400 and 300 MPa were used as main reinforcements and stirrups, respectively. The concrete strength of each specimen is 26.13 MPa.

1.1. Finite Element models

Ten specimens have been conducted using Abaqus software [19]. The first specimen (T-0-0) is the verified control specimen without horizontal stirrups in the joint, which was similar to the parametric study non-seismic specimen (NS). Various strengthening techniques have been applied as listed in Table 1; EBR of CFRP grid and engineered cementitious composites (ECC), externally bonded reinforcement (EBR) of Carbon fiber reinforced polymer (CFRP) sheets, externally bonded reinforcement on grooves (EBROG) of CFRP sheets, EBR of CFRP sheets combined with steel anchor bolts, and Hybrid FRP grid (GFRP+CFRP) and ECC. The last specimens were retrofitted using steel jacketing either by steel plates or plates and angles. The materials properties of the CFRP grid, CFRP sheets and the characteristics of the ECC used in these techniques are listed in Table 2. Figure. 2 illustrates the schematic of the non-seismic and retrofitted specimens.



(a) **(b)**

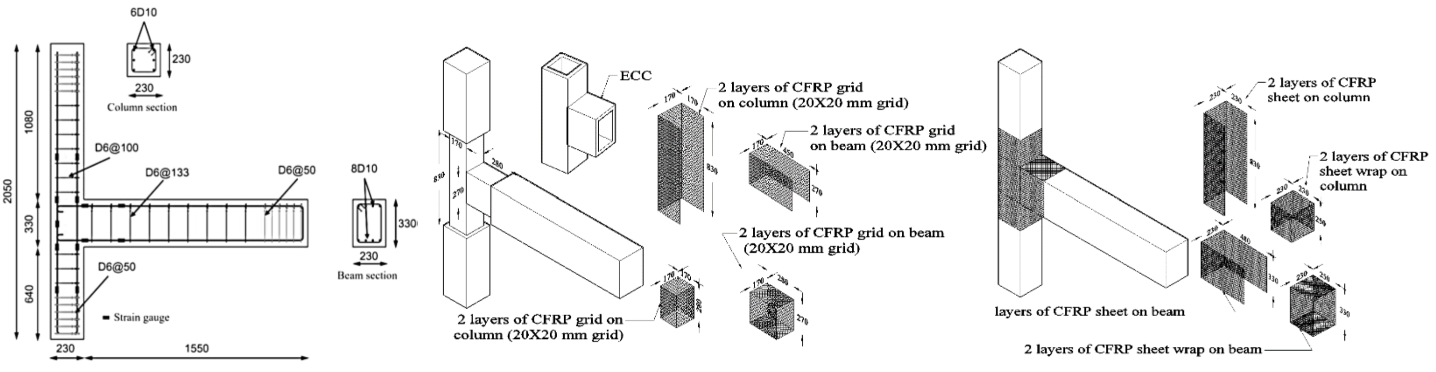
Figure. 1. Details of previously experimented specimens [19], **a.** non-seismic specimen (T-0-0), **b.** The retrofitted specimen (T-E-W).

Table 1. Summary of the specimens.

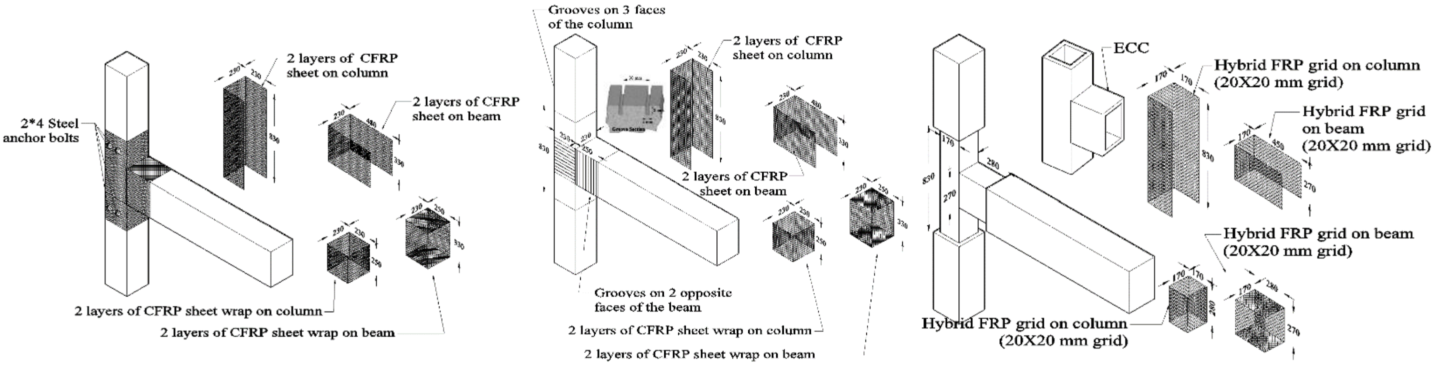
No.		Specimen	Retrofitted	CFRP	Hybrid FRP	ECC	Steel anchor bolts	Steel plates	Steel angles
1	Verification	T-0-0	No						
2	Verification	T-E-W	Yes	√		√			
3	Parametric study	NS	No						
4	Parametric study	T-EBR-2W	Yes	√					
5	Parametric study	T-EBROG-2W	Yes	√					
6	Parametric study	T-EBR-2WB	Yes	√			√		
7	Parametric study	T-E-2W	Yes	√		√			
8	Parametric study	T-E-HW	Yes		√	√			
9	Parametric study	T-STC	Yes					√	
10	Parametric study	T-STP	Yes					√	√

Table 2. Mechanical Properties of Materials

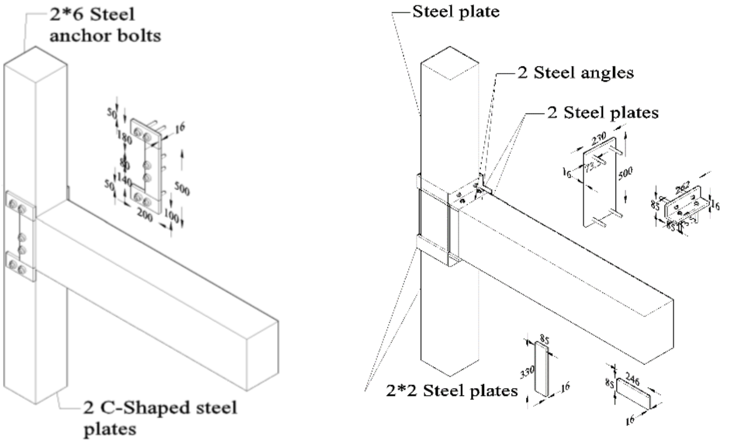
Material	Compressive strength (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Fiber orientation	Thickness (mm)	Area density (g/m2)	Elongation at failure
CFRP/fiber SikaWrap®230C	–	3650	238	Unidirectional	0.13	225	1.5%
GFRP/fiber SikaWrap®430G	–	2200	76	Unidirectional	0.17	430	2.8%
Hybrid fabric HFRP(Hexcel)	–	–	–	Bidirectional	–	274	–
Sika Carbodur	–	2800	165	Unidirectional	1.2	–	1.7%
Epoxy Sika330	–	30	3.8	–	1	500 g	0.9%
Epoxy Sika30	–	30	12.8	–	1	–	–
ECC	47.5	2.43	15.454	–	–	–	–



a. NS **b.** T-E-2W **c.** T-EBR-2W



d. T-EBR-2WB **e.** T-EBROG-2W **f.** T-E-HW



g. T-STC **h.** T-STP

Figure. 2. Details of the tested specimens.

1.2. Loading and boundary conditions

Boundary conditions were assigned to the finite element models in a certain way to match the experimental specimens; Pinned joint was considered for the bottom end of the column, and a displacement controlled loaded upper end. A roller joint was considered for the beam end. A constant axial load of 112-KN was applied on the top of the column, and the horizontal displacement was applied in X-direction on the top end of the column. Details of boundary conditions and cyclic loading scheme of the beam-column joint which are applied to FE models are illustrated in Figure. 4, and Figure. 5, respectively [19].

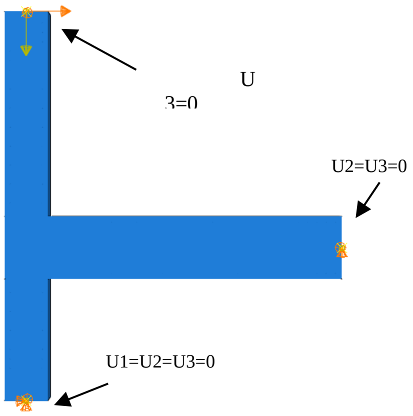
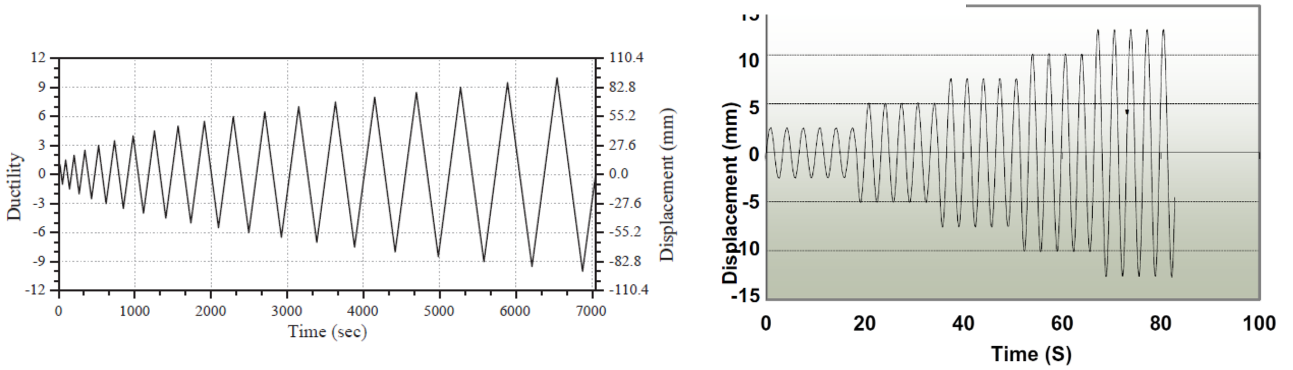


Figure. 3. Boundary conditions of the FE models [19].



a. Cyclic-loading of the previously experimented specimens [19].

b. Cyclic loading scheme of the FE specimens [50].

Figure. 4. Cyclic-loading of the studied specimens.

1.3. **Mesh independent method**

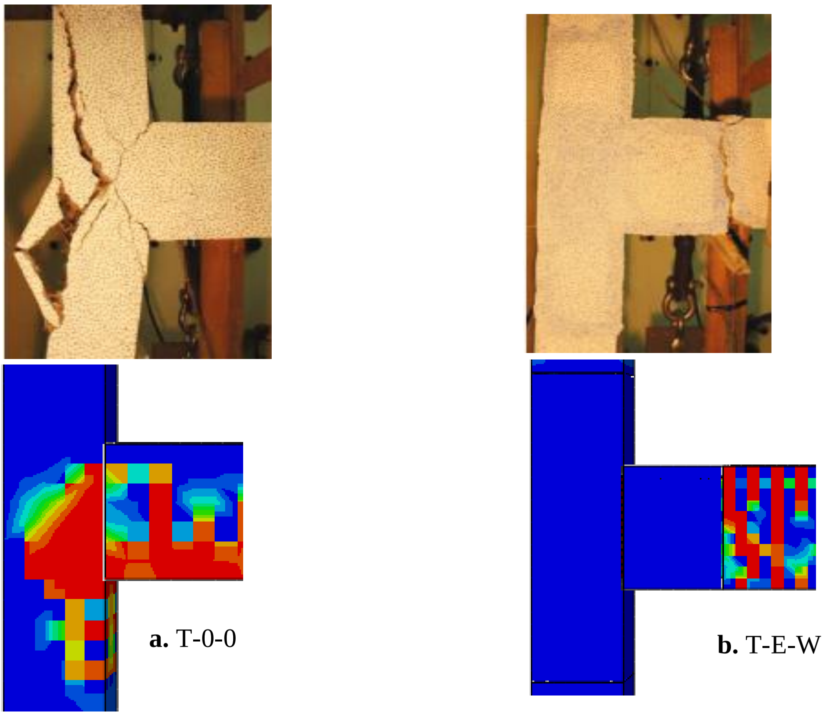
The mesh sensitivity analysis has been performed for the beam-column joints to choose a proper and stable mesh size for each specimen. The mesh convergence started from a coarse mesh to fine meshes. The mesh element sizes were 50, 40, 30, and 20 mm. The results indicate that the elements with mesh size 25 mm offer acceptable convergent results.

2.**Results**

Table 3 shows the Comparison between experimental and FEM results of the tested specimens. Figure. 5 illustrates the failure mode of the specimens. Figure. 6 depicts load versus displacement curves of the validated specimen.

Table 3. Comparison between experimental and FEM results of the tested specimens.

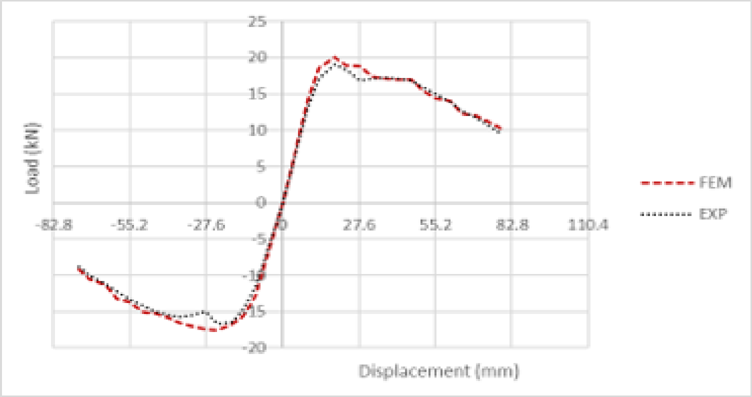
No.	Specimen	Pu (kN)	Pu (kN)	Pexp / PFEM	Δu (mm)	Δu (mm)	Δuexp / ΔuFEM	Ductility μ = Δu/Δy	Cumulative dissipated energy (kN.mm)	Failure mode
		Experiment	FEM		Experiment	FEM				
1	T-0-0	19.13	20.04	0.95	18.05	18.56	0.97	5.79	7703.28	Joint shear failure
2	T-E-W	22.07	22.93	0.96	50.1	50.98	0.98	8.92	21291.18	Beam flexural failure



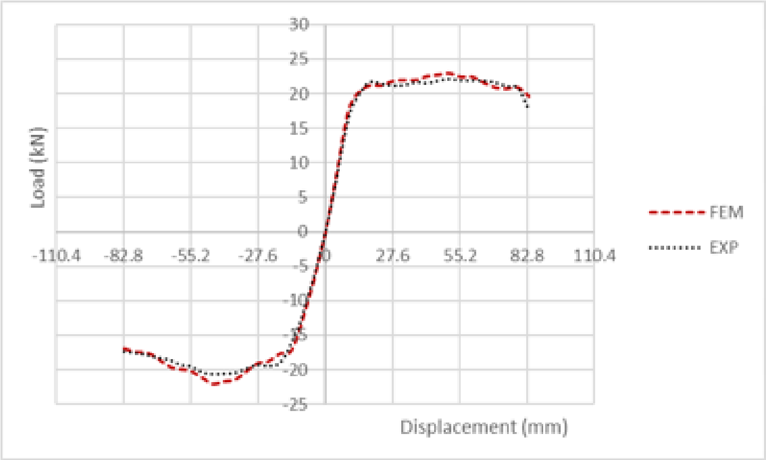
a. T-0-0

b. T-E-W

Figure 5. Failure mode of the tested and simulated beam-column joints of the first experimental study.



a. Envelop of the hysteresis loops of the experimental and FE T-0-0 specimen.



b. Envelop of the hysteresis loops of the experimental and FE T-E-W specimen.

Figure 6. Load-displacement curves of the validated specimens.

Table 4 shows the FEM results of the studied specimens. Figure. 7 illustrates the failure mode of the specimens. Figure. 8 depicts load versus displacement curves of the retrofitted specimen. The parametric study results showed that the considered factors vary in their effect on the behavior of the non-ductile RC BCJs as follows:

2.1. Specimen retrofitted with CFRP

The influence of using EBR of CFRP sheets is studied. The lateral load capacity of the retrofitted NS specimen increases, whereas the ultimate displacement decreases. Compared to NS, the lateral load capacity increased by 15%, while the displacement decreased by 27%.

2.2. Specimen retrofitted with (EBR) of CFRP sheets

EBROG method helped increasing the lateral load capacity of the specimen. The lateral load capacity is higher than that of specimens NS and T-EBR-2W by 18% and 2%, respectively. Which proves that using the EBROG method to delay the FRP sheets debonding has a positive impact in increasing the load bearing capacity of a deficient beam-column joint.

2.3. Specimen retrofitted with CFRP sheets combined with steel anchor bolts

Typically, using steel can be a great retrofitting technique to increase load bearing capacity and prevent failure of structures. In this case we studied adding steel anchor bolts to the CFRP sheets to increase load bearing capacity and improve the bond between the CFRP sheets and concrete to prevent the detachment of the sheets. It is observed that lateral load capacity increases with the use of steel anchor bolts compared to regular EBR CFRP sheets retrofitting technique. The lateral load capacity of specimen T-EBR-2WB is higher than the lateral load capacity of specimens NS, T-EBROG-2W, and T-EBR-2W by 35%, 14%, and 17%, respectively.

2.4. Specimen retrofitted with ECC and CFRP grid

The specimen retrofitted with ECC and two layers of CFRP grid, namely, T-E-2W is considered in this parametric study. Using ECC and CFRP grid was able to increase the lateral load capacity. The ultimate lateral load capacity of specimen T-E-2W is higher than that of specimen NS by 42%. In addition, the ability to modify the failure mode from shear failure in the joint to bending failure in the beam.

2.5. Specimens retrofitted with hybrid FRP (GFRP+CFRP) and ECC

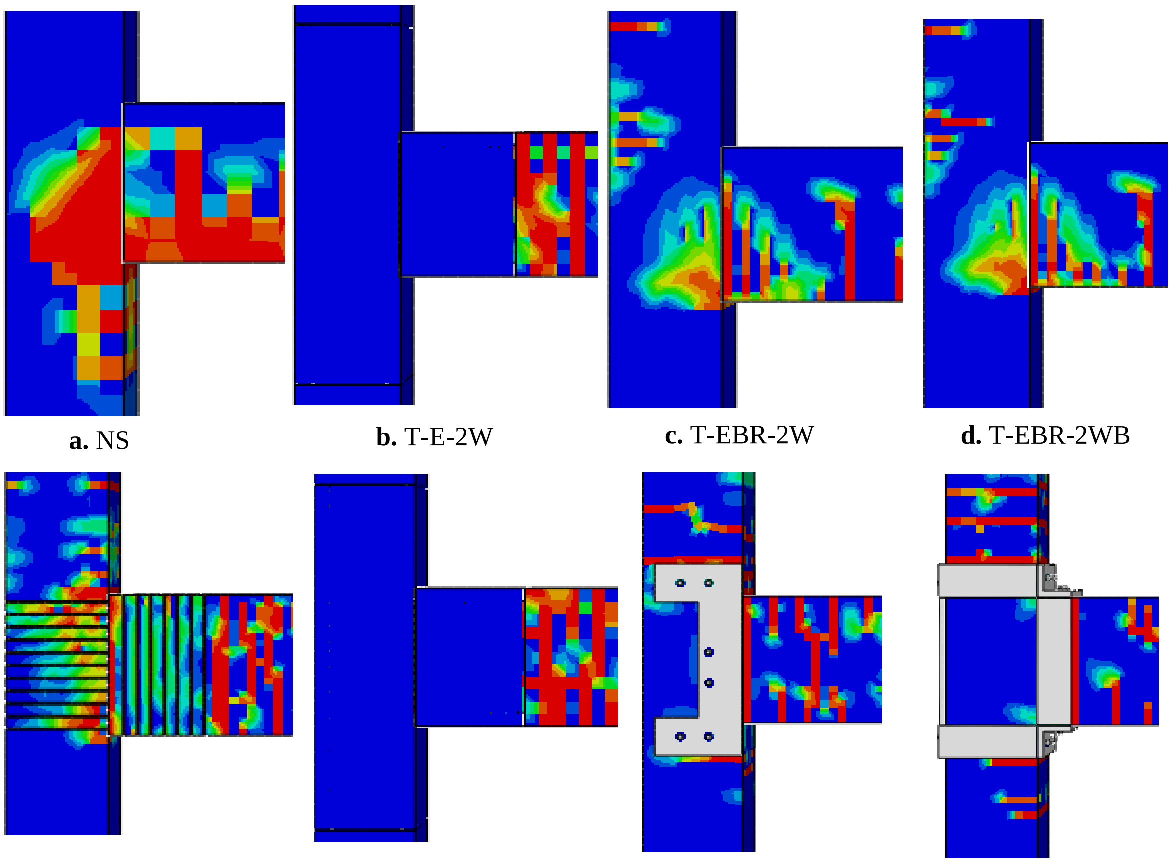
Hybrid fibers could increase the lateral load capacity of the non-ductile reinforced concrete beam-column joints specimens. With ECC and hybrid FRP grid the lateral load capacity is higher than that of specimen NS by 36 %, and 5% less capacity than of specimen T-E-2W. Using hybrid FRP was the best choice to achieve the maximum strain along with a higher load bearing capacity compared to the non-seismic specimen.

2.6. Specimens retrofitted with steel jacketing

Steel jacketing is typically used for strengthening reinforced concrete structures due to its high capability of increasing the load bearing capacity of structure elements. As shown in Figure.8, it can be found from the curves that the use of steel elements leads to improving the lateral load capacity of the non-ductile beam-column joints. The failure load increased from 20.04 kN in the case of the non-seismic specimen to 27.88 kN and 37.49 kN (increased by 39% and 87%, respectively) when the steel jacketing elements increased from C-shaped steel plates to steel plates and steel angles.

Table 4. FEM results of the studied tests.

No.	Specimen	Pu (kN)	Strength gain	Δu (mm)	Ductility μ = Δu/Δy	Cumulative dissipated energy (kN.mm)
1	NS	20.04	-	18.05	1.8	8309.74
2	T-E-2W	28.55	42%	9.29	2.65	12779.01
3	T-EBR-2W	23.14	15%	13.03	2.61	8490.12
4	T-EBROG-2W	23.59	18%	13.47	2.28	8707.6
5	T-EBR-2WB	26.99	35%	12.83	2.14	10194.22
6	T-E-HW	27.2	36%	17.34	2.62	14023.28
7	T-STC	27.88	39%	13.44	3.19	17158.78
8	T-STP	37.49	87%	13.05	3.46	18627.899



a. NS **b.** T-E-2W **c.** T-EBR-2W **d.** T-EBR-2WB

e. T-EBROG-2W **f.** T-E-HW **g.** T-STC **h.** T-STP

Figure. 7. Failure mode of the simulated beam-column joints of the parametric

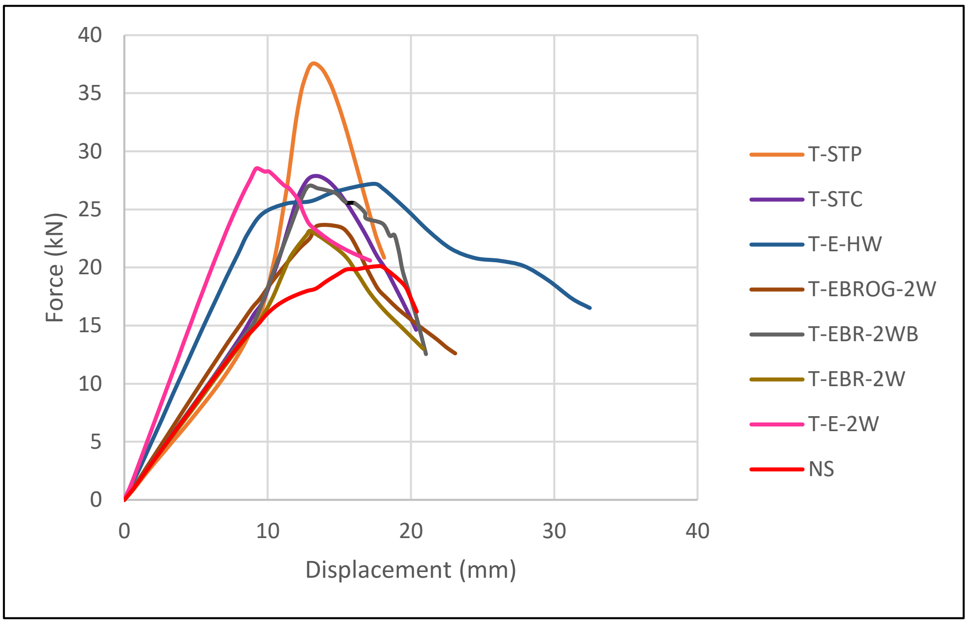


Figure 8. Envelop of the hysteresis loops of the parametric study specimens.

3. Discussion

The findings of the current research emphasized that insufficient reinforcement can significantly affect the RC BCJs' behavior. Non-ductility due to deficient reinforcement reduces the lateral load capacity of BCJs, the cracks due to reinforcement deficiency lessen the shear capacity of reinforced concrete columns. Besides, the shear resistance of rebars also weakened due to the reduced development lengths of the beams' reinforcement. Moreover, this caused internal moments on the BCJ which led to the formation of cracks. The cracks widened with the increase in lateral loading causing a negative effect on the capacity. Furthermore, the bond loss between concrete and reinforcement has a negative impact on the shear capacity of the BCJs.

3.1. Effect of EBR

The research revealed that the shear capacity increases as the layout dimensions and number of layers on the beam-column joint increase. This is caused by stress distribution to the CFRP sheets along with the high tensile strength of carbon fibers which make up for the deficiency or absence of tensile reinforcement in BCJs. As a result, shear stresses are distributed uniformly throughout the loaded area. These results are consistent with another research [2], [20], [21], [22], [23], [24].

3.2. Effect of EBROG

The resin-filled grooves can transfer the bond shear stress to the deep concrete layers, leading to postponing FRP debonding and an increase in the EBROG bond capacity compared to the EBR bond capacity. The EBROG method can postpone or eliminate the debonding failure mode and exploit the tensile capacity of the FRP material effectively. In other words, the EBROG method can resolve the primary EBR method drawback [25], [26], [27], [28], [29], [30].

3.3. Effect of EBR combined with steel anchor bolts

Anchor bolts increased the load at failure by 35% as compared to the control specimen, and it increased the load at failure by 17%, as compared to the EBR beam-column joint without anchorage. Moreover, it is apparent that steel anchorage was effective in confining concrete to the CFRP sheets and helped in delaying concrete-fiber bond failure and thereby increasing load bearing capacity. This finding agrees with previously conducted research where anchorage systems were used for the flexural CFRP layer [31], [32], [33], [34].

3.4. Effect of ECC

The results indicate a positive effect of using ECC and CFRP grid as a retrofitting method on the shear capacity. Past studies concluded that the use of ECC to replace conventional jacket leads to a more uniform strain distribution in the joint. This is attributed to the dense hairline crack pattern usually observed in the ECC jacket that reduces the strain concentration in the joint core [19], [35], [36], [37], [38], [39], [40], [41], [42], [43], [44].

3.5. Effect of hybrid FRP

The research revealed that the shear capacity increases using hybrid FRP sheets by 36% for specimen T-E-HW. This result is consistent with another research [45], [46]. On the other hand, when compared to the specimens retrofitted with two layers of CFRP sheets it was observed that hybrid fiber had a better influence on the beam-column joint strain. Furthermore, with the more amount of carbon fiber, the stress increases. The increase in capacity is due to the mechanical nature of carbon fiber which has tensile stress more excellent than that of glass fiber.

3.6. Steel jacketing effect

The research revealed that as the surface area of concrete covered by steel jacket increases the effect of confinement also increases. Using steel jacketing techniques for strengthening RC beam-column joints has been proven to be effective since it increases the BCJ capacity to a minimum of 39%. This finding agrees with previously conducted research [47], [48], [49]. The rapid loss of joint capacity and the brittle joint shear failure of the deficient beam-column joints were prevented by the suggested retrofitting methods. transforming its failure mode to beam flexural failure and improving the capacity of the beam-column joint; thus, we suggest applying these schemes to retrofit deficient interior beam-column joints as a suggestion for future research.

4.Conclusions

According to the outcomes of this research, the following conclusions can be drawn:

1. The non-seismic specimen without seismic details in the joint core and without transverse reinforcement and with insufficient shear strength in the joint core, suffers the formation of the shear hinge in the joint core.
2. In case of the non-seismic specimens retrofitted with EBR of FRP sheets the lateral load capacity was 15% higher than NS specimen until cracks started to occur on the interface of the beam-column joint propagating to the joint core causing shear failure at the joint due to failure of FRP bond to the concrete of the beam-column joint.
3. In case of the non-seismic specimens retrofitted with EBROG technique the lateral load capacity was 18% higher than NS specimens, the cracks also occurred on the interface of the beam-column joint propagating to the joint core causing shear failure at the joint due to failure of FRP bond to the concrete face
4. The non-seismic specimens retrofitted with EBR of FRP sheets combined with steel anchor bolts had a 35% higher lateral capacity compared to NS specimen, the cracks also occurred on the interface of the beam-column joint propagating to the joint core causing shear failure at the joint due to failure of FRP bond to the concrete but with approximately 17% higher capacity than using only EBR of FRP sheets.
5. In case of the non-seismic specimens retrofitted with ECC and FRP grid the cracks occurred on the beam at the end of the ECC retrofitted area due to bending causing the development of a beam plastic hinge leading to a higher lateral load capacity with 42% more capacity compared to NS specimen. Also, using two layers of CFRP grid improved the joint shear strength capacity by 5% compared to the case of using hybrid FRP grid (GFRP+CFRP).
6. The non-seismic specimens retrofitted with steel jacketing method resulted in the highest lateral load capacity leading to a 39-87% more capacity compared to NS specimen. Confinement conditions in the joint core area prevented shear failure and plastic hinge formation in the column with the formation of a flexural plastic hinge in the beam and the absence of damage to the joint core area.

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