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# Finite element development of a Beam-column connection with CFRP sheets subjected to monotonic and cyclic loading

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**Abstract.** Beam–column joints are recognized as the weak points of reinforcement concrete frames. The ductility of reinforced concrete (RC) frames during severe earthquakes can be measured through the dissipation of large energy in beam–column joint. Retrofitting and rehabilitating structures through proper methods, such as carbon fiber reinforced polymer (CFRP), are required to prevent casualties that result from the collapse of earthquake-damaged structures. The main challenge of this issue is identifying the effect of CFRP on the occurrence of failure in the joint of a cross section with normal ductility. The present study evaluates the retrofitting method for a normal ductile beam–column joint using CFRP under monotonic and cyclic loads. Thus, the finite element model of a cross section with normal ductility and made of RC is developed, and CFRP is used to retrofit the joints. This study considers three beam–column joints: one with partial CFRP wrapping, one with full CFRP wrapping, and one with normal ductility. The two cases with partial and full CFRP wrapping in the beam–column joints are used to determine the effect of retrofitting with CFRP wrapping sheets on the behavior of the beam–column joint confined by such sheets. All the models are subjected to monotonic and cyclic loading. The final capacity and hysteretic results of the dynamic analysis are investigated. A comparison of the dissipation energy graphs of the three connections shows significant enhancement in the models with partial and full CFRP wrapping. An analysis of the load-displacement curves indicates that the stiffness of the specimens is enhanced by CFRP sheets. However, the models with both partial and full CFRP wrapping exhibited no considerable improvement in terms of energy dissipation and stiffness.

**Keywords**: Beam-column connection; interior RC joint; earthquake; plastic hinge; CFRP sheet; retrofitting

## 1. Introduction

Beam-column joint is usually pondered critic zones for RC frames subjected to earthquake. The early failure of reinforced concrete (RC) beam–column joints can result huge deformations and subsequent collapse. In structural design, beam–column joints are considered as a critical element

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due tocrucial role in withstanding earthquake loading. Mostly, inadequate transverse reinforcement and intermitted beam-bottom reinforcement are leading to non-ductile joint (Li, Wu and Pan 2002). The structures in low to moderate seismic regions which rely on their inherent ductility are vulnerable during earthquakes. Common practice to address this issue could be the demolition of the structure and re-construct; however, such practice isn`t a feasible economic solution.

On the other hand, rehabilitation seems to be a more economical alternative, nevertheless demolition is generally preferred to structural rehabilitation due to the lack of discernment for performance of repaired building. As the matter of the fact, beam-column connection plays an important role in maintaining integrity of the whole structure against seismic loading, thus, the retrofitting of the joint considered as the main priorities. Fiber-reinforced polymer (FRP) composite materials have become increasingly popular in the past decade because of their high strength-to-weight ratios, corrosion resistance, ease of application, and constructability. Nowadays, implementing of carbon fibre reinforced polymer (CFRP) for strengthening of beam column joints appear as cost effective technique for retrofitting of RC structure although identifying the damage mechanism of joint after repairing is essential. Finite Element Method is recognized as a cost effective tool and technique to predict the behaviour of RC beam-column connections strengthened with CFRP.

Mahini and Ronagh (2011) performed nonlinear FE modeling and analysis of beam-column retrofitted by web-bonded CFRP in order to verify experimental tests. Omidi and Behnamfar (2015) proposed a numerical model consists of rigid offset element and beam and column elements with concentrated plasticity in order to simulate the elastic and inelastic behavior of RC beam-column connections. Therefore, through of this study, the seismic performance of beam–column joints with different rehabilitation methods is illustrated. The findings of this study can be useful in developing effective and economical techniques of rehabilitating such non-seismically designed beam-wide column joints. Engindeniz, Kahn, and Zureick (2005) studied about fiber orientation and proved the effect of fibre orientation on performance of retrofitted joint. Numerous studies from the previous researchers have been conducted on nonseismically design of beam-column joints strengthened by FRP composites (Ghobarah and Said, 2002; Tsonos and Stylianidis, 2002) and proved that externally bonded FRP composites mitigate the limitation in site (Gergely, Pantelides, and Reaveley, 2000) as well as maintain the member size (Balsamo *et al*., 2005) whereas joint shear capacity improved (Ghobarah and Said 2002). Also the similar investigations indicated the considerable increase of lateral load capacity (Karayannis, Chalioris, and Sideris, 1998), joint shear strength (El-Amoury and Ghobarah, 2002), and energy dissipation capacity, (Binici and Bayrak (2005).

Al-Salloum and Almusallam (2007) successfully developed a technique for analytical prediction of interior beam–column joints strengthened with externally bonded FRP sheets. On the other hand, Alsayed *et al*. (2010) proposed a rehabilitation method for the exterior beam–column joints. Their results indicated that the formation of localized plastic hinges are being shifted from the joint to the adjacent members like beams or columns in the strengthened joints. Pampanin, Bolognini, and Pavese (2007) proposed analytical model of predicting sequence of plastic hinge formation and they experimentally investigated the efficiency of retrofitted models for both beam–column joints and whole frame systems. Li and Chua (2009) conducted experimental test in full scales in order to develop an efficient and cost effective FRP-strengthening method for interior beam–column joints.

Also, Parvin *et al*. (2009) experimentally investigated the efficiency of joint rehabilitation by CFRP through conducting six full scale tests and evaluate the performance of existing beam–

column joints with inadequate shear and anchorage details. Bousselham (2009) developed a FEM model to predict the contribution of FRP sheet to the shear capacity through experimental testing for the seismic rehabilitation of RC frame with implementing FRP in the exterior joints. Alsayed *et al*. (2010) explored the efficiency of CFRP sheets in improving the shear strength and ductility of seismically deficient exterior beam–column joints. Le-Trung *et al*. (2011) proposed an analytical study on the modeling of exterior RC beam–column connections strengthened with CFRP composites under lateral loading. Sasmal *et al*. (2011) developed a hybrid retrofitting scheme that consists of CFRP wrapping and steel-plate jacketing to sustain the original strength of the damaged structure.

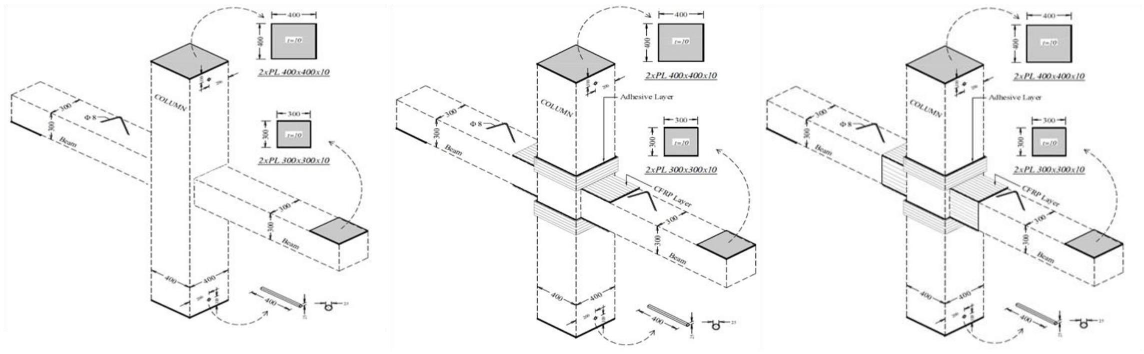
Alhaddad *et al*. (2011) evaluated the seismic behaviour of RC beam-column connections with FRP and textile reinforced material (TRM) through developing a nonlinear FE model. No transverse reinforcement at the joint core was implemented and the specimens were composed with the adjacent slab. Thus, column wrapping and web-bonding configuration were adopted to mitigate the shear deficiency. Halabi *et al*. (2012) realized that eccentric loading degraded the ultimate load capacity and ductility of the slab column connection.Nonlinear FE model developed by Eslami and Ronagh (2013) was used to to find out the minimum thickness of CFRP composites which needed to shift the beam plastic hinges away from the column face.

Dalalbashi, Eslami, and Ronagh (2013) numerically explored the seismic performance of reinforced concrete joints by strengthening CFRP sheets subjected to combined axial and cyclic loads. Ha *et al*. (2013) proposed a model that incorporated embedded CFRP bars with CFRP sheets in RC beam–column joints. This scheme mitigated damage and boost up the overall structural performance of beam–column joints subjected to cyclic load reversals. Vaghei *et al*., (2014, 2016) and Taheri *et al*. (2016) numerically developed a 3D finite element model of precast walls and connection in order to evaluate the performance of connection under imposed load. The most suitable FRP configurations have been investigated in RC beam–column joints strengthened with FRP systems by Realfonzo, Napoli, and Pinilla (2014). Agarwal, Gupta, and Angadi (2014) studied the axial behavior of concrete and external beam–column joints through effect of confinement provided by transverse reinforcement and FRP jacketing on the axial behavior of joints. As discussed in here, the previous studies proved that the exterior beam–column joints in RC buildings subjected to seismic loads are susceptible to experience local damages or diagonal cracks as a result of the yielding of bars and crushing of concrete in shear bending. Therefore, this type of joints should be investigated to determine precisely the effect beam–column connections strengthening with CFRP sheets subject to seismic excitation.

This study investigates the behavior of beam-column joint with partially and fully strengthened using CFRP during dynamic force excitation. For this purpose, three cross sections of RC beam–column joints are considered as frame with partial CFRP wrapping, frame with full CFRP wrapping and one frame with normal ductility and all three sections are subjected to monotonic and cyclic loading, and their capacity, energy absorption, stress, absolute plastic strain, and maximum displacement are studied and reported.

## 2. Interior beam–column joints

A detailed review of the literature shows that the beneficial effects of FRP composites on the seismic behavior of non-seismically detailed beam–column joints are relatively limited. The structural demand on joints is largely affected by the type of loading system and loading path in



(a) C1 (b) C2 (c) C3

Fig. 1 Three interior beam-column connection: (a) Normal Ductility-C1, (b) Partial Wrapping CFRP-C2 and (c) Full Wrapping CFRP-C3

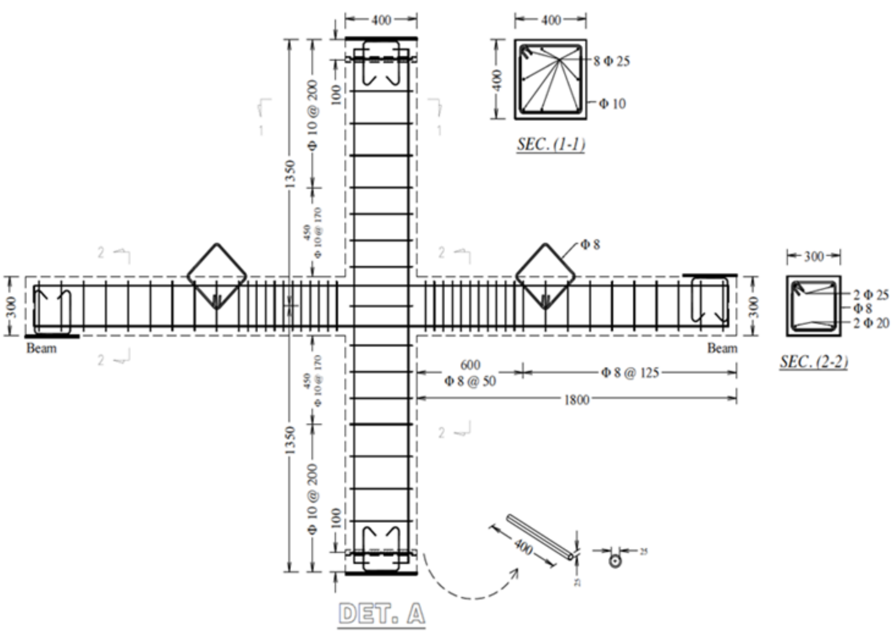



Fig. 2 Geometry and reinforcement details of specimens

any type of joint (i.e., interior, exterior, or corner). Therefore, using design procedures, in which the severity of each type of loading is properly recognized, is important. For example, the strength under monotonic loading without stress reversals is the design criterion for continuous RC structures subject to gravity loading only.

Using the finite element method, this study investigates three types of interior beam–column connections that are similar in terms of geometry (Fig. 1).These specimens are typical as-built joints extracted from existing buildings and are classified into three types depending on their respective CFRP sheet wrappings: one with partial CFRP wrapping (C1), one with full CFRP wrapping (C2), and one with normal ductility RC (C3). Fig.s 1(a), (b), and (c) illustrate the schematic dimensions of C1, C2, and C3, respectively.

The control specimen C1 is not wrapped with CFRP sheets. The top and bottom of the beam in C2 are partially wrapped with CFRP sheets. The front and back of the beam in C3 were partially wrapped with transverse CFRP sheets to observe the efficiency of the full CFRP wrapping method.

Fig. 2 shows the details of the interior beam–column connections. C1, C2, and C3 have a cross-sectional column of 400 mm × 400 mm and a cross-sectional beam of 300 mm × 300 mm.

## 3. Development of finite element modeling

The ABAQUS finite element software was used to evaluate the response of the interior beam–column connections subjected to monotonic loads. The finite element model of three aforementioned connections C1, C2, and C3 were developed according to detail which depicted in Fig. 1 and Fig. 2.

The all detail of finite element model is described in the next section:

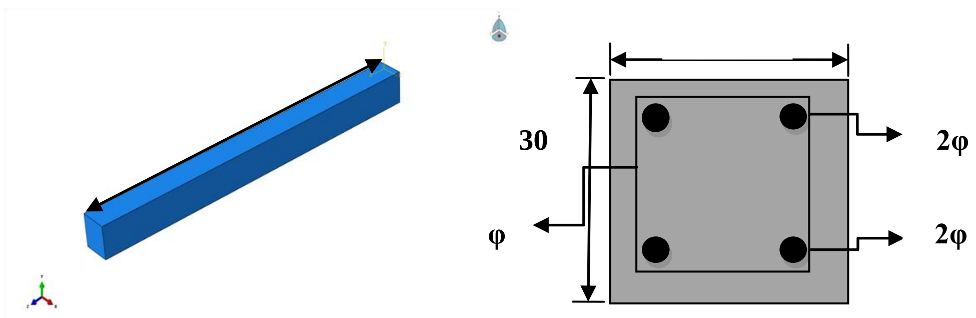
### *3.1 Components of the interior Beam-Column connection*

The finite element models of the concrete beam and column were developed with the actual geometry dimensions mentioned in the previous section.

#### *3.1.1 Concrete beam*

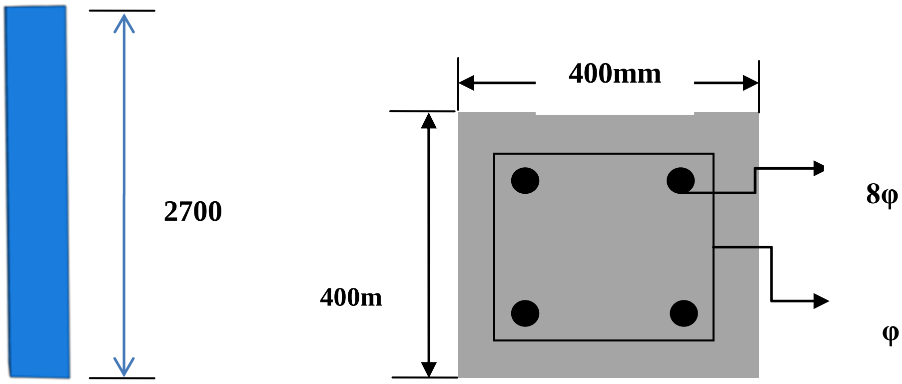
The reinforce concrete beam with 0.3m width and height and 4 m length is considered in this study. Four longitudinal reinforcement is allocated in beam with 25mm diameter in top and 20mm diameter in bottom of beam. The cover of concrete is considered as 20mm. Also the bar with 8mm diameter is used as stirrup in each 125mm of beam length unless in 0.6m beside beam-column joints which stirrups distance is reduces to 50mm. Two steel plates at top of right side beam and bottom of left side beam is located in order to apply incremental point load to the beam and avoid of having local damage because of imposing point load.

The concrete beam was created as a solid part as shown in Fig. 3.




(a) Isometric View (b) Cross section

Fig 1 Geometry details and dimensions of concrete Beam in Interior joint



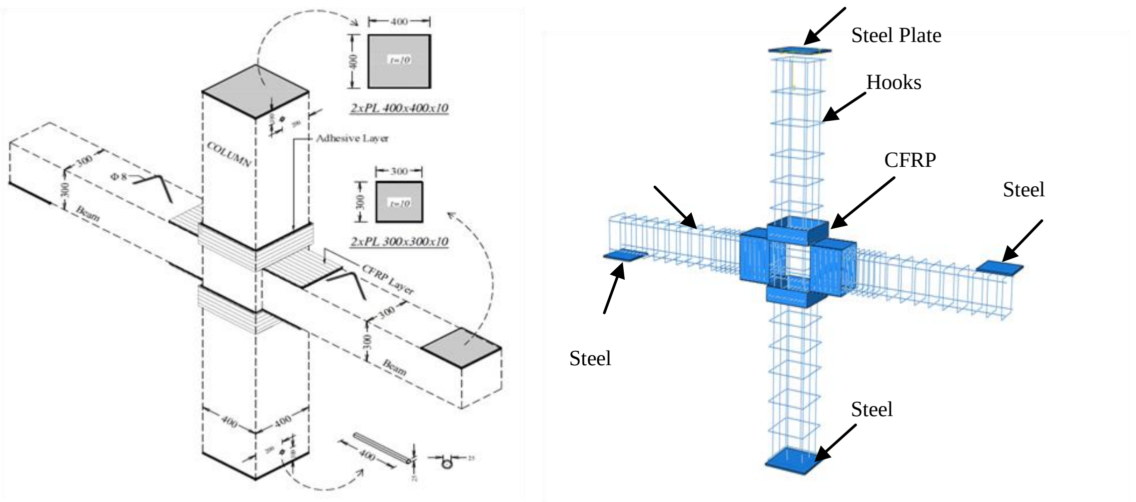

(a) Isometric View (b) Cross section

Fig. 2 Details and dimensions of concrete Column in Interior joint

### *3.1.2 Concrete column*

The column size is considered as 0.4m by 0.4m with 1.35m height in up and down of joint. Same as beam, 4 reinforcement with 25mm diameter are used as longitudinal bar and 10mm stirrups are implemented in each 200mm length of column. Although in 0.45m of column length in up and down of joint, stirrups are located in each 170mm. The steel plate is located in top of column for apply load from top of column. The solid part was used to create the concrete column and the details of the column are shown in Fig. 4.

The assemblage of the beam, column, reinforcement, and hook is shown in Fig. 5. The CFRP sheet was also set at specific positions to strengthen the joints with partial and full CFRP wrapping.

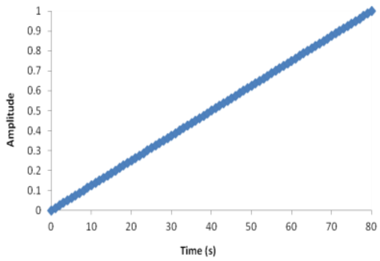



(a) Isometric view (b) Render view

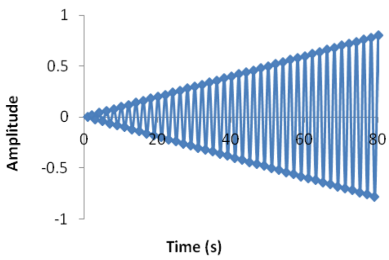
Fig. 3 Assemblage of beam column joint components

Table 1 Material properties

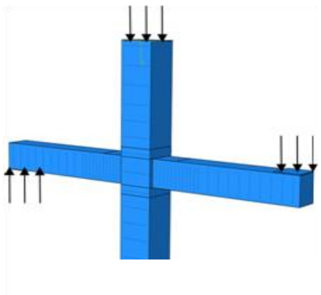
| Material | Mass Density(kg/mm$^3$) | Young's Modulus (MPa) | Poisson ratio | Yield stress (MPa) | Plastic strain |
|---|---|---|---|---|---|
| Concrete | 2.4E-006 | 41000 | 0.2 | 20 | 0 |
| | | | | 25 | 0.003 |
| Steel G10 | 7.85E-006 | 200000 | 0.3 | 559 | 0 |
| | | | | 619 | 0.25 |
| Steel G20 | 7.85E-006 | 200000 | 0.3 | 545 | 0 |
| | | | | 639 | 0.25 |
| Steel G25 | 7.85E-006 | 200000 | 0.3 | 603 | 0 |
| | | | | 701 | 0.25 |
| CFRP | 1.82E-006 | 198800 | 0.32 | - | - |




(a) monotonic loading




(b) cyclic loading



Roller Support

(c) boundary Condition

Fig. 6 Applied load on aforementioned specimens

### *3.2 Material properties*

The properties of concrete, steel, and CFRP for all three beam–column connections are listed in Table1.

### *3.3 Load and boundary conditions*

The load was applied as a uniformly distributed load from the top of the column through steel plate which assumed in top of column. Also two upward and downward loads imposed to end of left and right beam respectively. The monotonic and cyclic loading had a magnitude of 4 MPa as showed in Fig. 6 (a) and (b), respectively. The boundary condition was defined as the roller support at the bottom of the column, as shown Fig 6. (c).

### *3.4 Meshing*

The element types used for the concrete beam and column and for the reinforcement and hooks with the structured technique were the 8-node and 2-node linear beam elements, respectively as shown in Fig. 7.

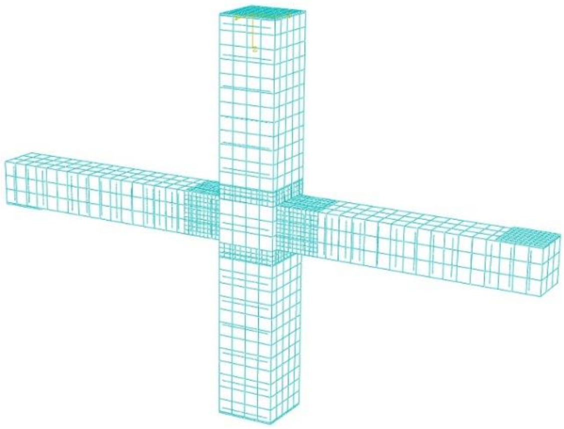


Fig. 7 Finite element meshing of components of beam column joint

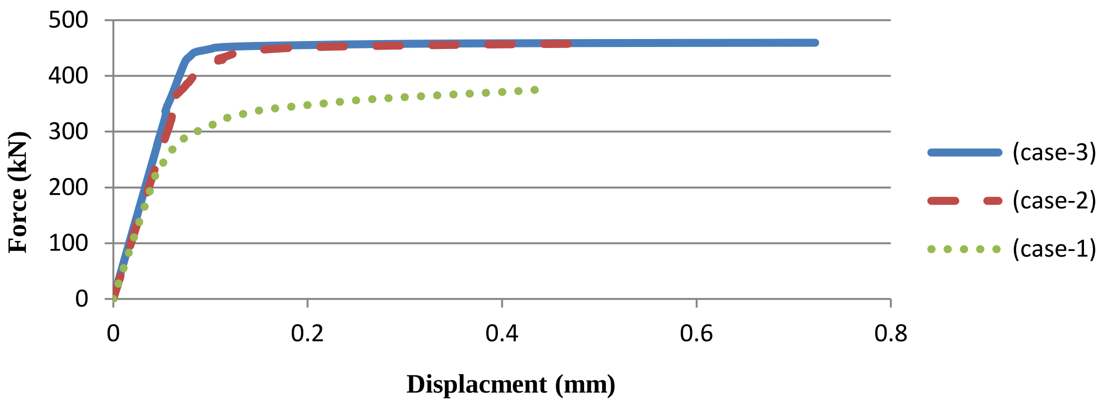



Fig. 8 Comparison of load–displacement curves for all considered models at ultimate load levels

## 4. Results and discussion

Inelastic pushover and cyclic analyses were conducted to investigate the effect of lateral displacements on the response of the interior beam–column connection. Three key features (i.e., maximum principal stresses, deformation, and absolute plastic strain) of the concrete panels and steel reinforcements were investigated in the three connection types (i.e., C1 = normal ductility, C2 = partial CFRP wrapping, and C3 = full CFRP wrapping) to determine the effect of incremental lateral movements. Fig. 8 compares the load-displacement curves of C1, C2, and C3 under extreme load levels. The graph clearly indicates the test frame behavior of the three connection types in terms of strength, deformation, and stiffness. The results of the ultimate loading test show an approximately 140% and 200% increase in the load carrying capacity of C2 and C3, respectively; by contrast, C1 exhibited no increase because of the presence of CFRP sheets. Fig. 9, depicts the energy dissipation of the models during monotonic loading.

The results show that both C2 and C3 achieved higher ultimate force than C1 under monotonic loading and that C2 exhibited better behavior than C3. A noticeable improvement in the displacement in C3 was observed, unlike in the other two cases.

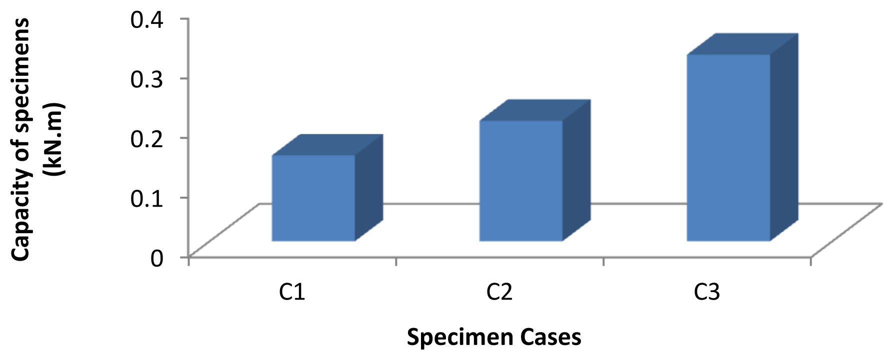
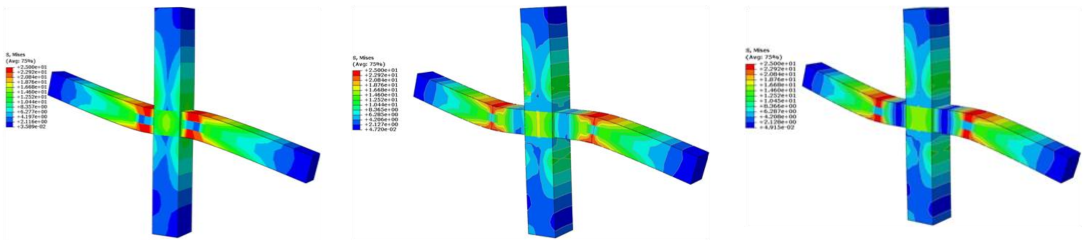
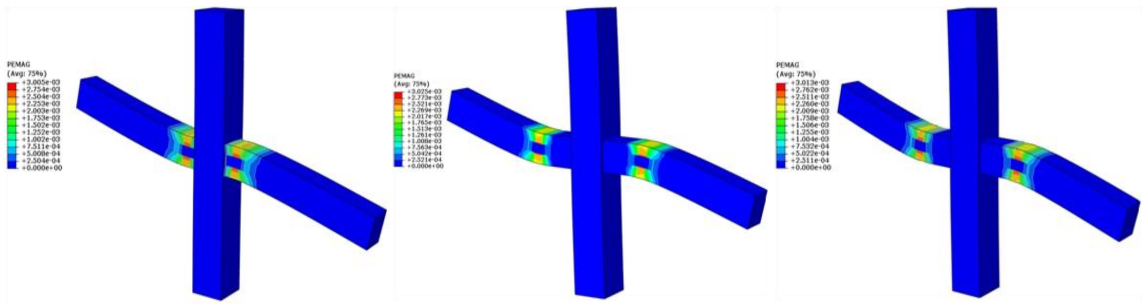



Fig. 9 Capacity of C1, C2 and C3 (kN.mm)



(a) C1 (b) C2 (c) C3

Fig. 10 Concrete stress distribution in normal ductility (C1), partial CFRP wrapping (C2) and full CFRP wrapping (C3)



(a) C1 (b) C2 (c) C3

Fig. 11 Absolute plastic strain distribution in normal ductility (C1), partial CFRP wrapping (C2) and full CFRP wrapping (C3)

The maximum principal stress of the concrete connection in all three interior beam–column connections reached 25 MPa, which indicates their status in the plastic range (Fig. 10). The reinforcement stress in C3 and C2 increased; by contrast, that in C1 exhibited no increase because of the presence of CFRP wrapping.

The plastic strain in all the three connection types was also compared; Fig. 11 shows the absolute plastic strain.

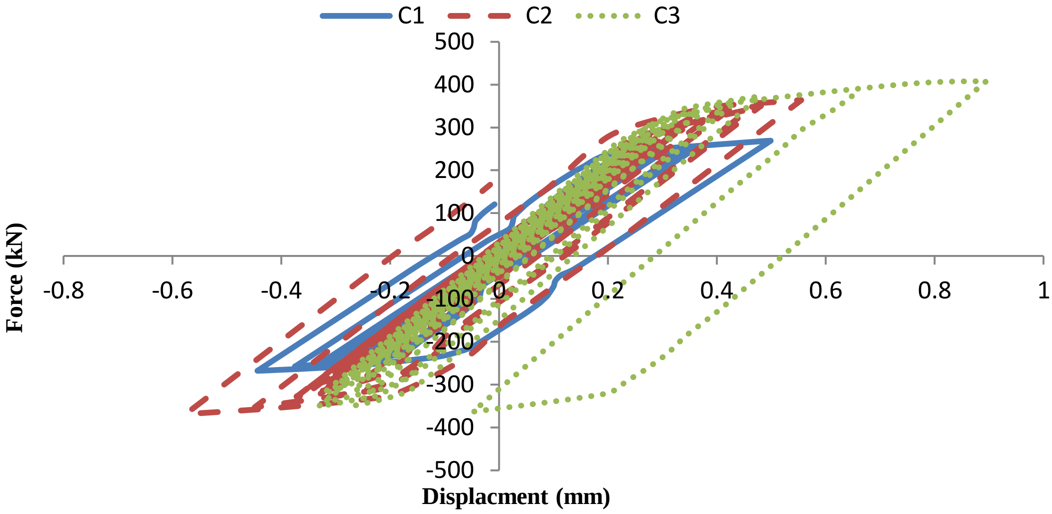



Fig. 12 Load-displacement hysteresis loops for control and retrofitted specimens

As expected, the concrete plastic strain representing the cracks was propagated across the beam and column of all the connection types. A significant enhancement of the plastic strain occurred beside the CFRP sheets in the retrofitted models (i.e., C2 and C3).

The hysteresis response of the considered models subjected to the cyclic load is shown in Fig. 12. Fig. 12 shows the total energy of each specimen. The beam–column joint specimens strengthened with CFRP had a higher total energy than C1. C2 and C3 were stronger and more ductile than C1 because they absorbed the highest total energy. The test results also indicated that the strengthening technique using CFRP sheets improved the strength and ductility of the beam–column joints relative to those of deficient beam–column joints and those built according to seismic codes, such as ACI 318-02.

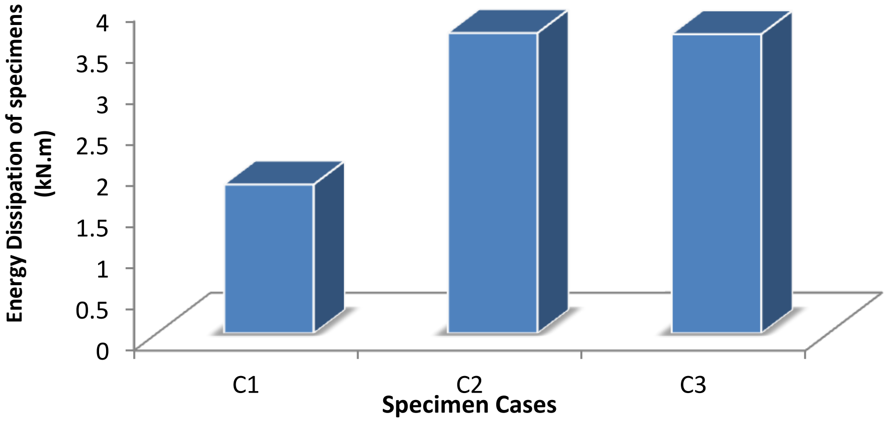
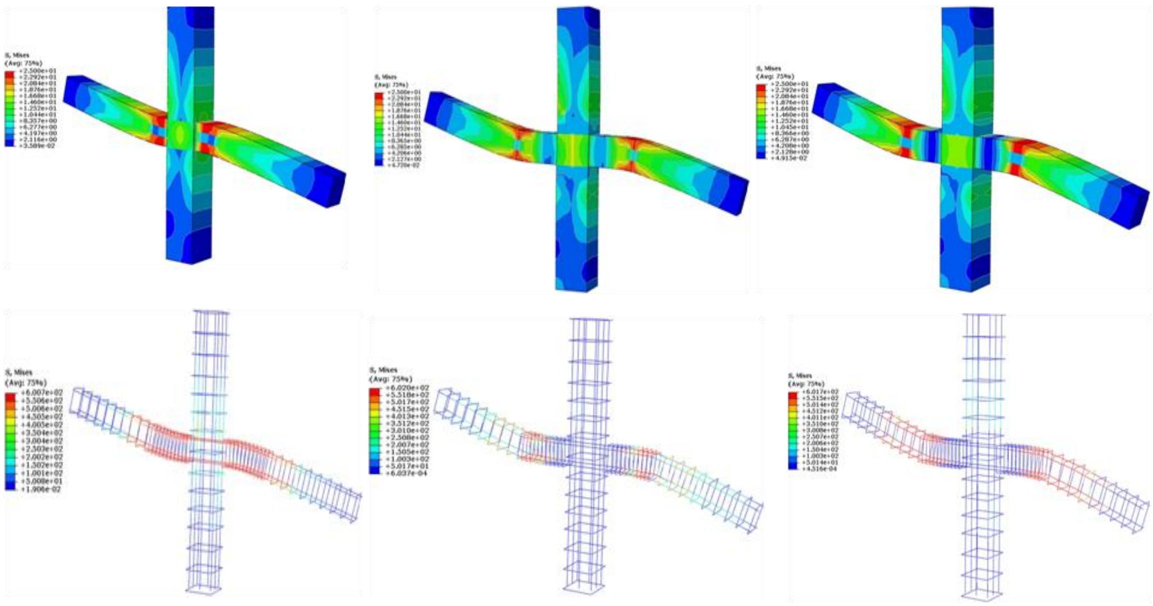
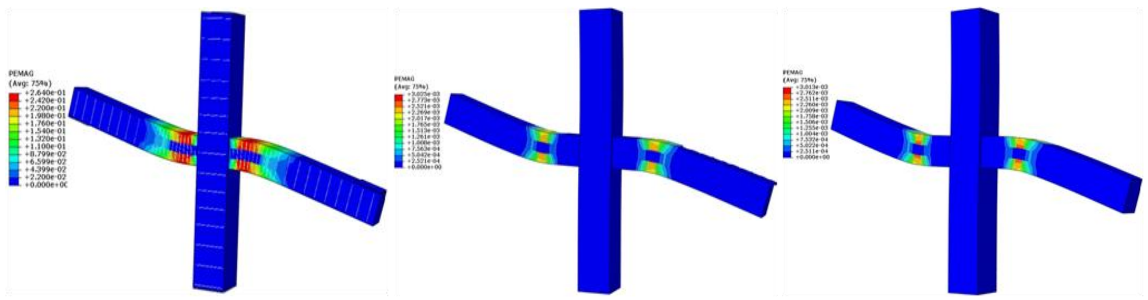



Fig. 13 Energy dissipation of C1, C2 and C3 (kN.mm)



(a) C1 (b) C2 (c) C3

Fig. 14 Concrete and reinforcement stress distribution in normal ductility (C1), partial CFRP wrapping (C2) and full CFRP wrapping (C3)



(a) C1 (b) C2 (c) C3

Fig. 15 Absolute plastic strain distribution in normal ductility (C1), partial CFRP wrapping (C2) and full CFRP wrapping (C3)

Fig. 13 shows the energy absorption of all the three models during cyclic loading under the curve area. The energy absorbed by the beam–column joints specifically relies on the strength and ductility of the joints under reversed cyclic loads.

The total energy of the joints consists of the accumulation of the areas under the curves of the force-displacement hysteretic loop up to the peak loads in both the pushing and pulling phases of the cyclic loads. Fig. 13 shows that the energy absorption of C2 and C3 was nearly twice more than that of C1 because of the CFRP sheets.

Fig. 114 explains the discrepancies in the maximum principal stress of the concrete and reinforcement of all the three aforementioned connection types. The maximum principal stress of all three types of connections was more than 600 MPa, which indicates that the reinforcements were in the plastic range. The stress in the concrete panels in C1, C2, and C3 reached 25 MPa. The reinforcement stress in C2 and C3 was more than that in C1 because of the function of the CFRP during vibration.

Fig. 15 illustrates the concrete plastic strain observed in all the interior connections. Unlike that of C1, the value of the maximum plastic strain of C2 and C3 was over 3e-3. Concrete plastic strain represents the crack propagation that occurred near the beam–column connection in C1 and around the CFRP sheets in C2 and C3.

## 5. Conclusions

This study investigated two retrofitting schemes for interior beam–column connections that were subjected to monotonic and cyclic loading. The performance of a connection with normal ductility (C1) was compared with that of connections with partial and full CFRP wrapping to verify the efficiency, high resistance function, and proper action against imposed force of the latter two connections. Finite element analysis was also conducted to investigate the capacity and high energy dissipation function of the aforementioned connections.

• The strength reduction and displacement improvement found in the cyclic loading analysis directly affect the amount of dissipated energy contained within the load–displacement curve. The retrofitted connections are highly flexible, and the CFRP sheets also effectively dissipate the energy.

• Pushover analyses reveal that the capacity of the connection with normal ductility (C1) to bear monotonic loading is considerably less than that of the retrofitted connections (C2 and C3). The capacity of the conventional beam–column connection improves by 140% compared with C2 and by 200% compared with C3.

• A significant enhancement is observed in the energy dissipation of C2 and C3, which represent a twofold improvement over that of C1.

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