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Shear Strengthening of 3D RC Beam-Column Connection Using GFRP: FEM Study

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Abstract: In this research, effectiveness of GFRP layers for joint shear strengthening of two-way corner beam-column connection is studied through a finite element model. To this purpose, a model based on previous experimental test on one-way strengthened connection is made using general purpose finite element code ABAQUS. The FEM results are validated by comparison with the test results. This model was developed to analyze rehabilitation of two-way corner RC beam-column connection. Two models including the original and strengthened specimen (with L shape GFRP layers) were analyzed. Comparing the results of the models indicated the effectiveness of the proposed strengthening scheme in reducing story drift, increasing ultimate load carrying capacity and changing the shear failure mode to a relatively ductile mode.

Key words: Shear deficient joint, FEM, two-way, reinforced concrete, beam-column, rehabilitation

INTRODUCTION

Beam-column connections of the most gravity load designed reinforced concrete structures and some of the new structures designed based on the seismic provision but with no transverse reinforcement in the joint (because of poorly field construction) are vulnerable against lateral loading. Failure of such connections occurs under sever earthquake loading in beam-column joint by forming shear diagonal cracks instead of forming the plastic hinge at the end of the beam; as a desirable ductile failure mode proposed by weak beam-strong column theory (Fig. 1). This vulnerability leads to a reduction in shear stiffness of the joint; therefore, it increases the story drift.

In the recent years, applying Fiber Reinforced Polymers (FRP) is the most interested method for shear strengthening and repairing of RC beam-column connections that has been studied (Karayannis and Sirkelis, 2008; Antonopoulos and Triantafillou, 2003; Ghobarah and Said, 2002; El-Amoury and Ghobarah, 2002; Tsonos and Stylianidis, 2002). Also there is only an analytical model available among the papers (Antonopoulos and Triantafillou, 2002). Most of these proposed methods can not be used directly for practical purposes because of neglecting spandrel beams, slabs or bidirectional loading effects.

In this study, 3D corner beam-column connection, the most vulnerable connection under seismic loads, has been selected and a practical strengthening method using GFRP layers was studied and developed.

For this purpose, the ability of finite element method in presenting the behavior of this kind of structures is verified through modeling a previously tested one-way exterior beam-column connection by Ghobarah and Said (2001). The FEM model was made using general purpose finite element program

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Fig. 1: Beam-column connection failure, Izmit, Turkey, 1999 earthquake

ABAQUS. After obtaining good accuracy, the applied model was developed to study joint shear strengthening of the two-way specimen.

Research Significant

While no practical method for shear strengthening of 3D RC beam-column joint using FRP layers is available and also some of the literatures (Engindeniz *et al*., 2005; ACI-ACSE Committee 352, 2002) emphasize that more study is required on the behavior of such connections and rehabilitation techniques, An experimental test program was designed to evaluate a proposed strengthening scheme for two-way RC beam-column connection as a cooperative program between the International Institute of Earthquake Engineering and Seismology (IIEES) and department of civil engineering of Khajeh Nasir Toosi University (KNTU). This program is now under conducting in structural engineering laboratory of IIEES. The present FEM study is a preliminary study for evaluating the efficiency of the proposed scheme and calculating the number of required GFRP layers for shear strengthening of the joint.

FINITE ELEMENT MODELING

Ghobarah and Said (2001) has tested a full-scale exterior beam-column connection without transverse reinforcement in the connection joint (specimen T1); a 600 kN constant column axial load and a semi-static cyclic load at the free end of the beam were applied. During this test, the structure failed by forming diagonal shear cracks at the joint without yielding beam longitudinal reinforcement and occurring plastic hinge at the end of the beam. This specimen was repaired and strengthened by applying U shape mechanical anchored GFRP layers in the joint connection (specimen TR1), then it was tested again.

The comparison of the hysteretic beam load-displacement curves of the two specimens indicated efficiency of the applied strengthening scheme in enhancing the shear stiffness and strength of the joint; therefore, beam reinforcement close to the column face was yielded and the plastic hinges with increasing ultimate load and ductility of the structure were formed. But it should be noted that proposed strengthening scheme can not be used directly in practical cases because of the existence of transverse beam and the neglect of the simultaneous bidirectional loading effects. Specimen T1 and TR1 were simulated using a finite element model in ABAQUS program. In these models, a monotonic downward load, instead of cyclic loading, was applied at the tip of the beam and the results were compared with the downward loading response envelope curves of the experimental tests. In the following section FE modeling is defined briefly.

Elements and Materials Properties

Concrete

Concrete damaged plasticity model (CDP) was used for defining concrete behavior in plastic range. This model is based on the Lubliner *et al*. (1989) studies and modifications made by Lee and Fenves (1998). Tensile cracking and compressive crushing are two main mechanisms of the concrete failure in CDP model. Concrete stress-strain behavior under uniaxial compression after elastic range (0.7f'c) should be defined in terms of stress versus inelastic strain (crushing strain). Concrete behavior under uniaxial tension is assumed to be linear until forming the initial macroscopic cracks at the peak stress (failure stress).

Post failure behavior should be defined in terms of stress versus cracking strain. This behavior allows defining the effects of the reinforcement interaction with concrete by introducing some tension stiffening to the softening branch. As shown in Fig. 2, Saenz (1964) and Shirai and Sato (1984) relations were used to introduce compression and tension-stiffening behavior, respectively. A solid element with eight nodes and three translational degrees of freedom at each ones, called C3D8R, was used to model concrete elements.

Steel Reinforcement

Longitudinal and transverse steel reinforcement behavior was defined as an elastic-plastic material using a bilinear curve. Slope of the plastic range was assumed to be about one percent of steel modulus of elasticity. To introduce the plasticity, kinematics hardening option was used. A truss element called T3D2 was used to model reinforcement elements.

This element has two nodes with three translational degrees of freedom at each ones. The Embedded Element option was used for connecting reinforcement elements to the surrounding concrete. This option could constrain translational degrees of freedom of the embedded element nodes (steel reinforcement) to the degrees of freedom of set of surrounding element nodes called the host elements (concrete).

Glass Fiber Reinforced Polymers

While the properties of GFRP materials along the fibers are different with transverse directions, these materials are assumed to be orthotropic materials. Therefore Elastic lamina option

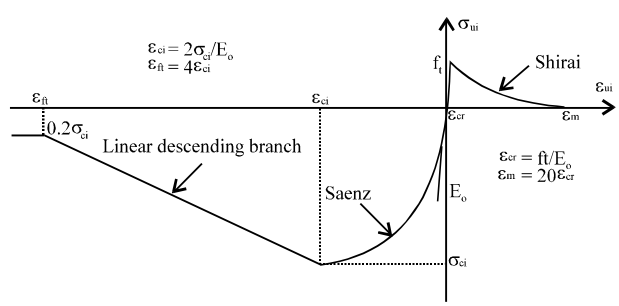


Fig. 2: Concrete stress-strain behavior

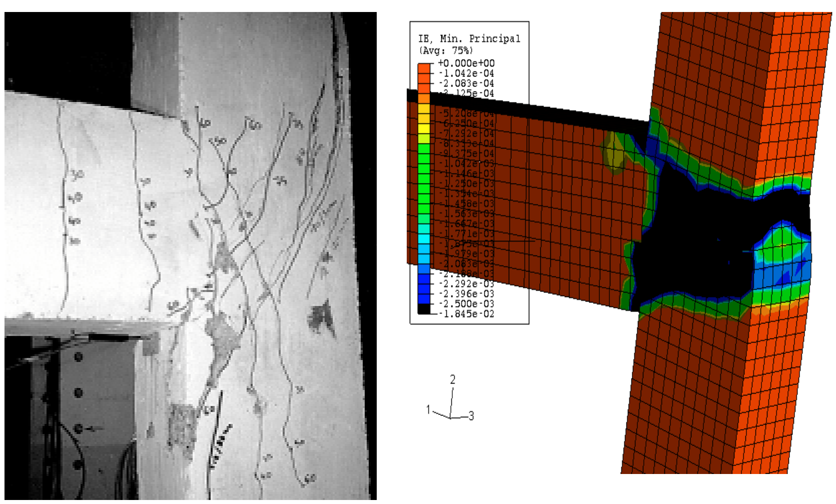
was used and the following parameters were defined as the mechanical properties: E_1, E_2, v_{12}, G_{12}, G_{13}, v_{21}, v_{12}. Failure criterion for orthotropic materials (with plane stresses) is generally defined in the stresses space. The required data for defining such a failure criterion are based on the ultimate compressive and tensile strength in two principal orthogonal directions and shear strength. For the FRP materials, Tsai and Wu (1971) criterion is a well-known failure index which was used in this study. Four node shell elements (S4R) with composite section were used for modeling the GFRP elements. By using composite section, various materials could be defined in optional directions with various thicknesses. Each layer has three sectional integration points. The composite layers of the experimental test specimen were anchored to the steel plates, therefore; Full bond assumption was made for the interaction between GFRP and concrete surfaces for simplicity of the FE model.

FEM VALIDATION

Specimen T1

As expected according to the experimental test results the original specimen failed in shear failure mode at the joint before the yielding of the beam longitudinal reinforcements. Excessive transverse tensile strains in the joint region reduced the compressive strength of the diagonal compression strut formed in this zone (the only resistance mechanism against the shear stresses) and finally a shear failure occurred.

In order for recognize the crushed elements in the FEM, assuming the ultimate compressive strength to be equal to 0.0035; therefore, a value equal to 0.0025 will be the minimum crush strain of the concrete. The comparison of the damage pattern demonstrated in Fig. 3 shows good accuracy between the damaged elements of the FEM and the experimental specimen. As shown in Fig. 3a and b ures for both specimens, damaging initiated from the top of the beam close to the column face, progressed towards the joint by forming diagonal cracks and finally transferred to the back of the column face at the failure.



(a) (b)

Fig. 3: Damaged concrete zones: (a) Experimental; (b) FEM

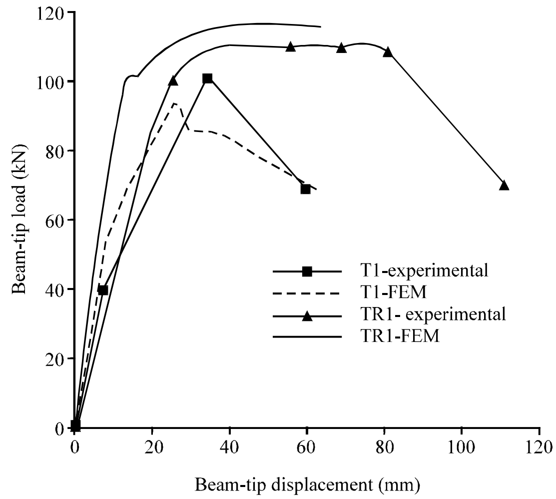


Fig. 4: Beam-tip load versus displacement curves

Beam load-displacement curve of the finite element model and the envelope of the hysteretic downward load-displacement response of the experimental test are compared in Fig. 4. This indicates accepted accuracy for the FEM results. Lower stiffness of the test specimen in the initial loading steps toward the FEM could be explained as follow:

- Assumptions made for the material properties at the FE model due to lack of the sufficient data
- Existence of the microscopic cracks in the test specimen
- Differences between the actual support system and the modeled one
- Comparison of the envelope curve for the experimental cyclic loading results with the monotonic loading of the FEM.

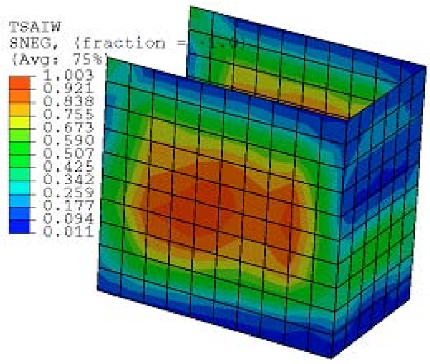
Specimen TR1

According to the test results, GFRP layers used in the beam-column joint region led to a relatively ductile failure mode of the specimen. The yielding of the beam longitudinal reinforcement was occurred and the plastic hinge formed in the beam close to the column face.

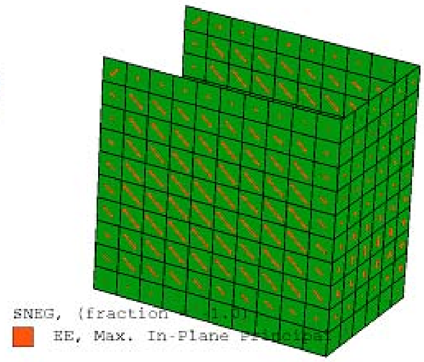
Although the GFRP layers rupture was the main factor in reducing the joint shear stiffness and preventing the full plastic hinge from development, it was just due to insufficient layers in design step. Since there was no possibility of stiffness degradation or element removal for the GFRP elements used in the FEM, the Tsai and Wu (1971) failure criterion was checked and when this criterion reached a value equal to 1, the analysis terminated, considering this as the failure point of the structure (Fig. 5a). Figure 5b shows the GFRP layers principal strains directions close to 45 degrees. The rupture direction could be found perpendicular to these directions.

Comparison with the experimental results, shown in Fig. 5c, confirms the accuracy of the model and GFRP behavior.

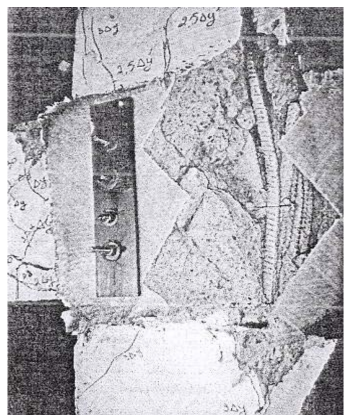
The beam tip load-displacement response of the FEM and the same envelope curve of the experimental test for downward loading are shown in Fig. 4. The FEM response curve showed a greater ultimate beam tip load than the experimental one. It could be explained in a reasonable way as a full bond assumption for the GFRP and concrete surfaces made in the model, which could transfer more shear stresses to composite and cause higher shear strength of the joint in spite of the experimental test.



(a)



(b)



(c)

Fig. 5: GFRP rupture (a) FEM: Tsai and Wu (1971) criterion; (b) FEM: maximum principal strain direction and (c) Experimental test

3D BEAM-COLUMN CONNECTION

Using the validated finite element model that presented good accuracy for modeling RC beam-column connections and the strengthened specimen by GFRP layers, the RC beam-column connection with two orthogonal beams and no hoops in the joint (reference specimen: A1) was modeled and analysis performed under bidirectional loading.

In order for joint shear failure mode to definitely occur, as the study purpose is, this model was deigned based on ACI Committee 318 (2002), except for joint hoops and the bar-diameter to beam depth ratio. Furthermore, to prevent any kind of shear failure in the beams or column, transverse reinforcements spacing was designed conservatively close to each other.

Concrete cylindrical compressive strength was 30 MPa. All the reinforcement has the yield value equal to 498 MPa. The geometrical model is shown in Fig. 6a. In the first step of analysis, an axial load about 0.1 f'_cA_g equal to 370 kN, was applied on the top of the column as the gravity load. In the next step, two displacement control loads were applied simultaneously to the free end of both beams.

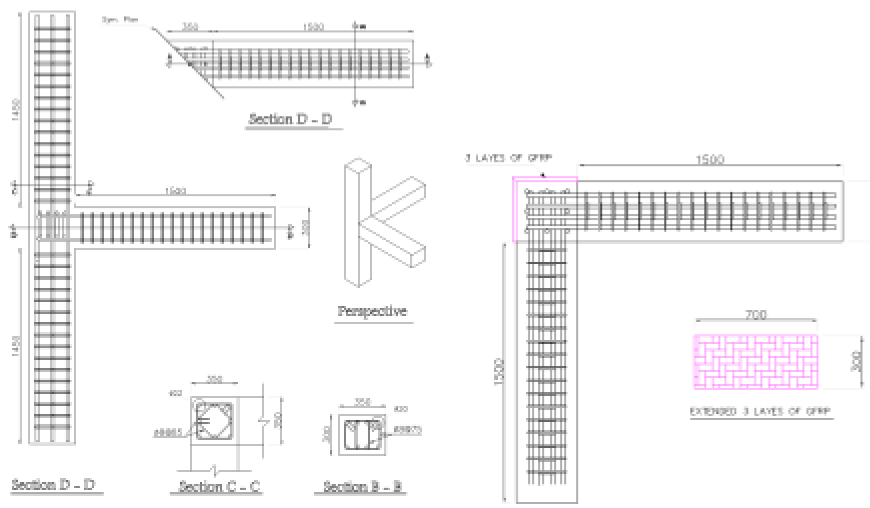
The results of the analysis showed excessive damage in the joint due to crushing the concrete without yielding any one of the beam reinforcement, which indicated shear failure mode at the joint.

The ultimate load was 85 kN at the end of each beam while the beam tip displacement reached a value equal to 31 mm.

Another model was built just the same as the reference specimen but it was strengthened with three layers L shape bidirectional GFRP(0°, 90°) and applied to the joint (Fig. 6b). This specimen designated AR1.

Each ply had a thickness equal to 0.15 mm, an ultimate tensile strength about 1073 MPa and a rupture strain equal to 1.5%. Again, a full bond assumption was made between FRP and the concrete which indicated the requirement of the mechanical anchors for the practical purposes.

As shown in Fig. 7 and 8, the strengthened specimen failed with excessive damages in the joint panel while partial plastic hinges were formed at the end of both beams close to the column face.



(a) (b)

Fig. 6: (a) Specimen geometry and details; (b) Rehabilitation scheme

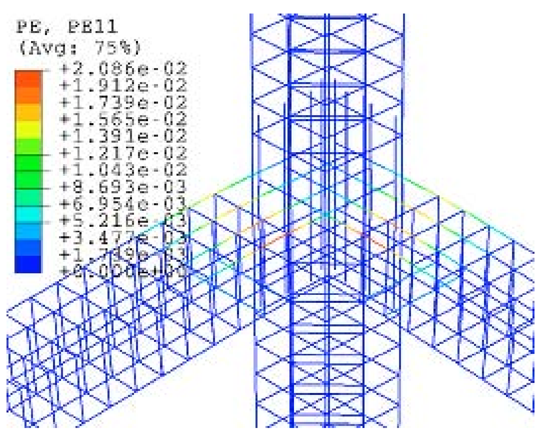


Fig. 7: Reinforcement plastic strain

Figure 9a shows Tsai and Wu (1971) criterion for the FRP layers at the failure. According to this criterion, no FRP rupture occurred during the analysis. As expected from previous results, the maximum principal strains, showed in Fig. 9b, were in the diagonal direction.

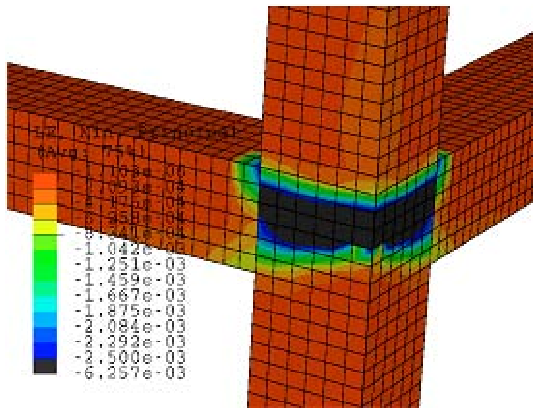


Fig. 8: Damaged concrete elements

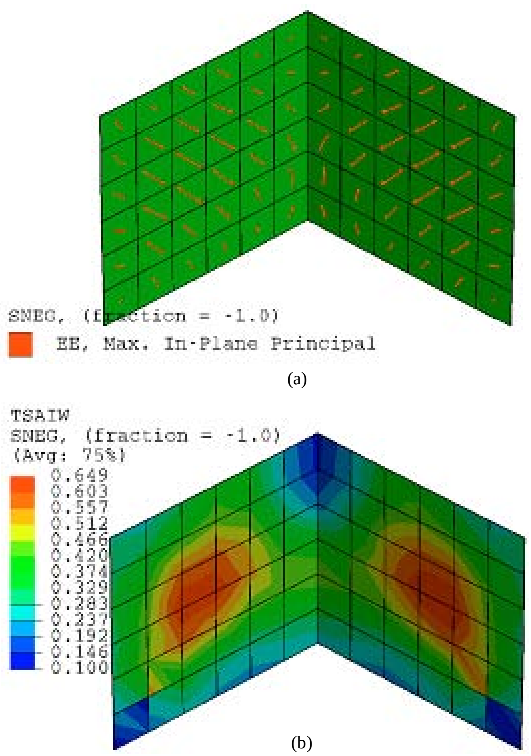


Fig. 9: GFRP rupture at the end of the analysis (a) Tsai and Wu (1971) criterion (b) Maximum principal strains directions

DISCUSSION

As shown in Fig. 10, specimen AR1 exhibited greater ultimate load, about 54%, than specimen A1. The contribution of the joint shear deformation in the total beam-tip displacement is calculated based on the distortion of the joint in plane of a beam and plotted in Fig. 11.

The comparison of these results demonstrates a reduction in contribution of the joint shear deformation in the beam-tip displacement for specimen AR1. This shows the efficiency of the applied GFRP layers in increasing stiffness and shear strength of the joint and a reduction in story drift as a desirable global strengthening purpose.

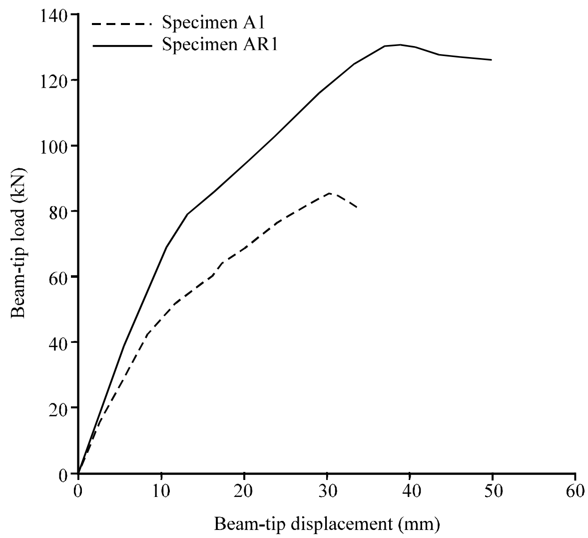


Fig. 10: Beam-tip load-displacement curves

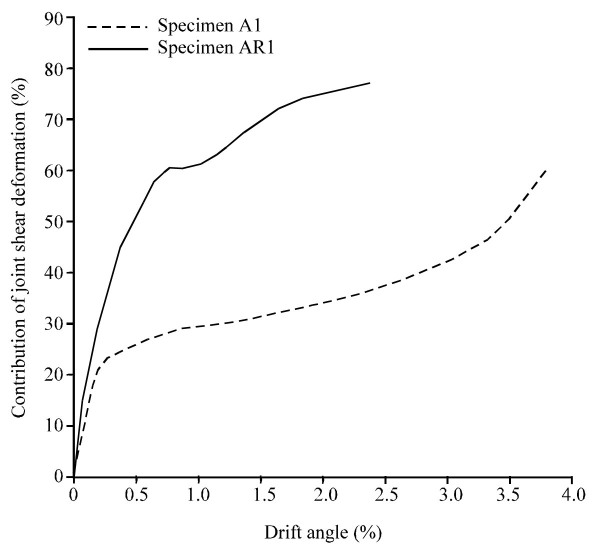


Fig. 11: Contribution of joint shear deformation in the beam-tip displacement

This effect could be explained as:

- Contribution of the GFRP layers' in shear resistance mechanism of the joint
- Confining the concrete of the joint; therefore, enhancing the compression strength of the diagonal concrete compression strut and finally increasing the shear strength of the joint
- Reducing in softening effects of compression zone by delaying in cracking of the joint

CONCLUSIONS

The nonlinear finite element model was used successfully to model one-way exterior beam-column connection, with and without FRP layers. The FEM results compared with the experimental one that indicates a good agreement. The FE model of the one-way exterior beam-column connection was developed to study the behavior of 3D corner beam-column connection under bidirectional loading without transverse reinforcement in the joint (shear deficient connection). The results were compared with the shear strengthened ones in which the L shaped GFRP layers were applied to the joint. The strengthened specimen exhibited greater ultimate beam-tip load and the partial plastic hinge was formed at the end of both beams close to the column face. There was a relative failure transfer from the joint in the original specimen to the beam by yielding the reinforcement of the beams in rehabilitated specimen. GFRP layers were effective in reducing the shear deformation of the joint which directly contributed to the beam-tip displacement and led to a reduction of story drift at the failure, as is the global purpose in strengthening of the structures. Followings are the most important conclusions:

- Finite element model used in this study has good agreement with the experimental results; therefore, it could be developed and used for further studies in the future
- The GFRP layers can effectively interact with the concrete and make more shear resistance due to confining the joint concrete and contributing to shear resistance mechanism of the beam-column connection
- Also, the applied strengthening method could not completely change the shear failure mode of the joint to a desirable ductile failure mode by forming full plastic hinges at the end of the beams close to the column face, but it was effective to demonstrate a better behavior than that of the original specimen. In addition, these layers could increase shear stiffness of the joint by delaying the concrete damage and controlling the cracks' width; therefore, reducing story drift
- As presented in this study, L shape GFRP scheme proposed for strengthening the shear deficient corner RC beam-column connection could apply effectively for practical purposes if the sufficient bond between GFRP and concrete is provided; for example, mechanical anchors can be used

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