Shear strengthening of 3D corner beam-column connection using bidirectional GFRP layers

E. Esmaeeli & F. Danesh
K.N. Toosi university, Tehran, Iran

ABSTRACT: Almost all the previously-done studies, on shear strengthening of RC beam-column connections, using FRP materials, have been conducted on the simplified specimens like the one-way joints. These proposed methods cannot be used practically in the strengthening of existing buildings. In this study, the enhancement of the joint shear strength of 3D RC corner beam-column connections has been attempted by using FRP layers through an applicable pattern. To this purpose, two identical 3D full-scale corner beam-column connections were made, with shear deficiency in the joint region. In both specimens, the column was subjected to a constant axial load and two cyclic loads were applied at the beam tips. One of these two specimens was tested as a reference specimen. The second one was strengthened, using bidirectional GFRP layers at the joint panel. The edges of the applied composites at the beam-column interface were anchored to the series of steel angles, without making perforations of any kind in the concrete of the joint panel. The strengthened specimen was tested and the results were compared with the results of the control one. The results indicated the good performance of the proposed strengthening method. In addition, the proposed method could be easily used for rehabilitation of the existing buildings.

1 INTRODUCTION

Insufficient or no transverse shear reinforcement in the joint region of most RC structures designed based on only gravity loads and also those with practical problems in spite of good seismic design can lead to brittle shear failures in the joint region under the severe earthquake loads. This kind of premature failure will prevent developing full plastic capacity of RC frames subjected to transverse loads, even though the connection had been designed based on weak beam-strong column theory.

The use of FRP materials for strengthening RC structures is the most desirable method studied recently. The literature on FRP-strengthened joints mainly consists of simplified two-dimensional tests (Engindeniz et al. 2004). These proposed methods cannot be used practically for strengthening of existing buildings due to the existence of spandrel beams. In this study, the enhancement of the shear strength in 3D RC corner beam-column connections has been attempted, using bidirectional GFRP layers and a steel cage through an applicable pattern.

2 EXPERIMENTAL PROGRAM

2.1 Test specimens

Two identical 3D reinforced concrete corner beam-column connections with no transverse shear reinforcement in the joint region were made and tested. The specimens, designated TS and
TSR, were full scale RC corner beam-column joint subassemblages. The models had beams in orthogonal directions, and were subjected to both unidirectional and bidirectional loadings.

The geometry and reinforcement details of both beams were identical. Columns were reinforced to have greater flexural strength than beams to satisfy the ACI-318 (2002) weak beam-strong column requirements. In order to produce shear stresses large enough in the joint region, beam depth-to-bar diameter ratio recommended by ACI-ASCE Committee 352 recommendations (2002) was ignored. Similar to the most gravity loads designed RC structures, no transverse reinforcement was placed within the joint region of the column. To prevent shear failure outside the joint, sufficient shear reinforcement with close spaces was provided in the beams and columns. Bidirectional GFRP layers anchored to a steel cage were used for joint shear rehabilitation of specimen TSR. The rehabilitated specimen test results were compared with the control one. More information about test specimens can be found elsewhere. (Esmaeeli et al. 2007)

2.2 Test setup and instrumentation

The beam-column joint was tested in a column horizontal position, as shown in Fig. 2. Partial-spherical contact bearing was used at each end of the column. This condition allows the free rotation at the end of columns in both orthogonal directions. A constant axial load of 370 kN was applied through a static jack, installed at the end of the column. A load cell was used at the other end of the column to measure column axial load and to transfer it to the support. Two cyclic loading actuators were installed at the beam-tips in order to apply reversal quasi-static loads. Similar loading pattern was applied in the USA-New Zealand-Japan-China Cooperative Program on Design of RC Beam-Column Joints (Jirs1 1991). Load history is controlled by drift-angle. Drift-angle was defined as the ratio of beam-tip displacement to the beam length. The plane of the vertically-placed beam was selected as the main loading direction and the effects of bidirectional loading applied on the horizontal beam were studied in the main direction.

Twenty four strain gauges were installed to measure the strains in the steel reinforcement bars of the beam-column connection. The locations of the strain gauges were determined based on the place where yielding and hinging were expected to occur in the specimen during the test. Also, seven strain gauges were installed on different points of the GFRP surface to measure the strains in the composite layers of rehabilitated specimen. Two diagonal linear-variable differential transducers (LVDT’s) were mounted on the joint region in plane of the vertical beam. Using these LVDT’s, shear deformations of the joint region were measured during the test. Beams-tip load were measured by load cells of cyclic load actuators. Column axial load variations due to the beam shears were recorded by a load cell installed at the end of the column. More information about test setup and instrumentation can be found elsewhere (Esmaeeli et al. 2007).

2.3 Rehabilitation scheme

The proposed rehabilitation method for shear deficient joint consists of wrapping 4 layers of FRP composite around the reinforced concrete joint and anchoring the free edges of the FRP layers to a steel-angles cage. The composite was made of bidirectional glass fibers which were placed in the $90^\circ$ and $0^\circ$ directions in a matrix of epoxy resin. The main difficulty of applying FRP composite layers in 3D RC connections is at the interface of the beam and column. Because of the existence of orthogonal beams, the wrapping of the composite layers in this zone is not possible. Previous experimental results (Gergely et al. 2000; Ghobarah & Said 2002) on rehabilitation of simplified two-dimensional joints using FRP materials emphasized on the use of mechanical anchors to avoid premature de-bonding of layers. This difficulty was overcome in this study through using a configuration of L80 steel angles. The steel angles were placed around the column at the joint region on the composite surface, after applying the GFRP layers. One leg of both steel angles was cut along the interface of each beam by column. Three 12 mm diameter threaded holes were made on the other leg of these angles. Two steel angles were mounted on the top and bottom edges of the intersection of three members where the beams and the column reached together. Also another steel angle was used on the last edge of the column. A series of L40 steel angles with a length of 50 mm had been welded to the L80 steel angles. A 12 mm diameter hole was drilled at the center of the free leg of each L40 steel angles. L80 steel angles
were connected to each other by threaded rods crossed through the holes of L40 steel angles. Then, the free surface of the composite layers at each side of the joint was turned over the welded bar surface and lied down on the leg of the steel angles. This 14 mm diameter steel bar was used to prevent stress concentration and avoid the rupturing of the GFRP layers due to the sharp edge of the steel angle. A steel plate with three drilled holes was put over this region and the short bolts were fastened while the free surface of the composite was placed between the L80 steel angle and the steel plate surfaces. The strengthening scheme using bidirectional GFRP layers inside and outside of the joint region and applied steel cage are shown in Figure 2.

Steel Cage Efficiency: In order to examine the composite effect independently in confinement of the joint, the bolts of steel cage were fastened just by hand; therefore, it has the less effect on shear strengthening of the joint. However, the cage will be remained effective in reducing the column’s ends rotation close to the joint region.

3 EXPERIMENTAL RESULTS

3.1 Specimen TS

Cracks formed in the plane of the vertical beam at the end of the test of control specimen are shown in Figure 3. As illustrated in this figure, except for few flexural cracks that have been formed on the beam surface close to the beam-column interface, almost all the cracks concentrated on the joint region due to the zone’s shear deficiency.

The maximum beams tip loads for the control specimen occurred at the 2 percent drift angle for both push and pull load cycles. The values of these loads are listed in table 1. After the peak
load was reached, beam bars' slip occurred and only the width of the previously-formed cracks increased, without forming any new crack. The maximum load was followed by concrete crushing and spalling in the joint region of the column. Beam-tip load, for both vertical and horizontal beams, was dropped significantly (more than 30 percent) at a drift angle equal to 3 percent and the test terminated at this point. None of the beam or column reinforcement yielded during the test. The specimen failed in the joint shear failure mode.

Figure 3. Cracks formed at the end of the test for specimen TS.

<table>
<thead>
<tr>
<th>Table 1. Maximum Beams-tip load for specimen TS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical beam</td>
</tr>
<tr>
<td>Beam-tip peak load (kN)</td>
</tr>
<tr>
<td>Drift angle (percent)</td>
</tr>
</tbody>
</table>

3.2 Specimen TSR

Cracks formed at the end of the test for rehabilitated specimen are shown in Figure 4. Flexural cracks are developed in the overall length of both beams during the test. Before the peak load of the specimen in each direction was reached, the top and bottom longitudinal bars for both beams had been yielded. Values of corresponding loads and drift angles are listed in Table 2. The yielding of the beam longitudinal reinforcement led to the formation of plastic hinges at the interface of each beam by column. The specimen reached its peak loads at a drift angle equal to 2.5 percent for vertical beam in both push and pull cycles. Corresponding values for the horizontal beam were 3.0 and 2.5 percent for push and pull loads, respectively. Values of the peak loads for the specimen TSR are listed in Table 3. Beam longitudinal bars' slip was the main factor of the specimen's shear strength degradation. Finally, while the maximum drift angle reached up to 7.5 percent, the peak load was dropped down about 38 percent and the test terminated. The GFRP layers remained undamaged till the end of the test.

As shown in figure 4, a wide crack at the interface of the vertical beam-column was formed, the place where the beam hinge occurred. This crack did not penetrate in the joint region and was formed along the edge of the composite at the interface of the beams and column as a flexural crack.

Figure 4. Cracks formed at the end of test (specimen TSR).
Table 2. Beams-tip load at the first yield of longitudinal beams bar

<table>
<thead>
<tr>
<th></th>
<th>Vertical beam</th>
<th></th>
<th>Horizontal beam</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Push cycle</td>
<td>Pull cycle</td>
<td>Push cycle</td>
<td>Pull cycle</td>
</tr>
<tr>
<td>Beam-tip load (kN)</td>
<td>95.2</td>
<td>97.7</td>
<td>91.4</td>
<td>101.0</td>
</tr>
<tr>
<td>Drift angle (percent)</td>
<td>2.0</td>
<td>2.5</td>
<td>3</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 3. Maximum Beams-tip load for specimen TSR

<table>
<thead>
<tr>
<th></th>
<th>Vertical beam</th>
<th></th>
<th>Horizontal beam</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Push cycle</td>
<td>Pull cycle</td>
<td>Push cycle</td>
<td>Pull cycle</td>
</tr>
<tr>
<td>Beam-tip peak load (kN)</td>
<td>112.6</td>
<td>97.7</td>
<td>99.5</td>
<td>108.3</td>
</tr>
<tr>
<td>Drift angle (percent)</td>
<td>2.5</td>
<td>2.5</td>
<td>3.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

4 DISCUSSION

In this section, the response envelopes, the contribution of the shear deformation of the joint in the vertical-beam tip displacement and the ductility factor of both specimens are discussed.

Envelops of the hysteretic loops for both control and rehabilitated specimens are depicted in Figure 5. Specimen TSR showed an average increase, about 50%, in the load-carrying capacity compared with control specimen. Specimen TSR reached a higher load level and maintained the load carrying capacity at a drift angle higher than that of the control specimen.

![Figure 5. Vertical-Beam tip load - tip displacement.](image)

The contribution of the joint shear deformation in the vertical-beam tip displacement for both TS and TSR specimens are shown in Figure 6. According to this figure, while this contribution for control specimen at 3 percent drift angle is 54 %, the corresponding value for rehabilitated specimen is only about 20 percent. The flat region for specimen TSR between 2 and 5 percent drift angle is corresponding to the beam plastic hinging deformation. The sudden increase in the contribution of the joint shear deformation in vertical-beam tip displacement of specimen TSR after 5 percent drift angle is due to beam bars’ slip and, therefore, significantly degrades the joint shear strength.

Displacement ductility factor for both specimens were calculated based on Mirmiran et al.’s (1999) definition. Based on this definition, the yield-based ductility index follows the conventional definition of ductility, as the ratio of ultimate-displacement to the yield-displacement of the specimen. The ultimate deflection is generally considered to be the deflection at the time of collapse as long as the load drop is not more than 15% of the capacity of the member (Park & Paulay 1975). The yield deflection is defined as that of an equivalent elasto-plastic system with the same elastic stiffness and ultimate load as those of the real system. Based on the above definition, average displacement-based ductility factor for specimens TS and TSR were equal to 2
and 3.2, respectively. These values indicated an increase about 60 percent in ductility of the rehabilitated specimen compared with the control one.

![Figure 6. Contribution of joint shear deformation in vertical-beam tip displacement.](image)

5 CONCLUSIONS

The proposed rehabilitation method using bi-directional GFRP layers and a steel cage as a mechanical anchor can be used in shear strengthening of RC beam-column joints in existing buildings. This technique could be used practically without any disruption in building occupancy. Lack of need to perforation of any kind in concrete members made this method simple and easy to install. Anchoring the GFRP layers to the steel cage provided an effective confinement to the concrete joint until the full shear strength of the confined concrete was achieved. The rehabilitated specimen performed a much higher ductility than the control one. Joint shear distortion was decreased significantly in this specimen due to delay in cracking of the joint concrete. The strengthened specimen carried 50 percent more loads at both beam-tips than the original specimen. Also, the enhancement in the joint shear strength of specimen TSR led to the formation of the partial-plastic flexural hinges at the end of both beams by yielding the longitudinal steel bars.

REFERENCES


Building code Requirements for Structural Concrete (ACI 318-02).


