RETROFIT OF RC JOINTS WITH FRP COMPOSITES

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ABSTRACT

The exterior and corner beam-column joints are among the weakest members of RC (reinforced concrete) frames in terms of seismic resistance. Poor seismic performance of inadequately detailed exterior/corner joints can lead to total or partial collapse of reinforced concrete frame structures. Since most of the existing beam-column joints in relatively old structures have not been constructed properly in terms of reinforcement detailing, they are in urgent need of retrofitting, particularly in terms of shear strength. To address a solution for this problem, from beginning of 1990s up to now, many researchers spent efforts on retrofitting of beam column joints by making use of conventional and FRP (fibre reinforced polymer) materials. In this study, after the introduction of typical failure modes of RC beam-column joints, available results on the behaviour and retrofitting of exterior and corner beam-column joints are reviewed and discussed. Furthermore, the most common FRP retrofitting schemes and contribution of the FRP retrofitting to behaviour of exterior beam columns are examined. Additionally, the large-scale structural tests conducted at Istanbul Technical University (ITU) on retrofitting of beam column joints built with low strength concrete and plain bars by FRP sheets are summarised. Based on the available research, it seems possible to include the issue of seismic retrofit of RC joint using FRP materials in the future versions of seismic design/retrofit guidelines/codes.

KEYWORDS

FRP, RC beam-column joints, seismic retrofit, strengthening.

INTRODUCTION

Although exterior beam-column joints are one of the most critical regions of the buildings during earthquakes, insufficient transverse reinforcement details, low quality of materials and problematic anchorage details in beam-column joints are quite common in relatively old existing buildings (particularly in developing countries). For many times, these deficiencies have caused severe damages or partial/total collapse of structures during earthquakes (Fig. 1). In this study, typical failure modes of unretrofitted and FRP retrofitted beam-column joints are briefly introduced and the FRP retrofitting schemes applied to RC beam-column joints are reviewed. Additionally, two recent studies carried out at ITU on RC beam-column joints constructed with low strength concrete and plain reinforcing bars are summarised. The first study consists of 16 full-scale RC beam-column subassemblage tests. This study focused on the effects of the column axial loads and existence of the transverse beam and slab. In this study, specimens and tests have been designed for reaching a design methodology for retrofit of joints. In the second study, five full-scale three-dimensional reinforced concrete frames, constructed with the same details as the first group of specimens, were tested under constant axial loads and reversed cyclic lateral displacements. This study was mainly carried out to confirm the results obtained from the first study on the full-scale three-dimensional (3D) tests.

Figure 1. Damages observed during the 2011 Van Tabanli earthquake (Figure 1(a) from Onen et al. 2011).
RC BEAM-COLUMN JOINT FAILURE MODES

Basically, six different failure modes can be observed on beam-column joints. These failure modes are described below;

J type failure: Joint panel reaches to shear strength, Fig. 2 (a).
AJ type failure: Longitudinal bars of the column buckle at joint panel region, Fig. 2 (b).
BJ type failure: Soon after beam longitudinal bars yield, beam-column joint reaches shear capacity, Fig. 2 (c).
CJ type failure: Soon after column longitudinal bars yield, beam-column joint reaches shear capacity.
BCJ type failure: It is combination of the BJ and CJ type failures. In this failure mode, soon after beam and column longitudinal bars yield, beam-column joint reaches shear capacity.
SJ type failure: Firstly beam longitudinal bars slip due to anchorage conditions. After large deformations, shear capacity of the beam-column joint region reduces and dominates behaviour, Fig. 2 (d).

On the other hand, the following failure modes can also be encountered if the beam column joint capacity is higher than the capacities of connected beams and columns:

B type failure: No failure is observed at beam-column joint region, beam reaches its capacity.
C type failure: No failure is observed at beam-column joint region, column reaches its capacity.

In case FRP retrofitted deficient RC joints, the target is to achieve B or C type failures (preferably B type failure) rather than a brittle joint failure. However, two failure modes related to FRP can also accompany these failure modes. These FRP related failure modes are described below;

FD type failure: Debonding of fiber reinforced polymer layers, Fig. 3 (a).
FR type failure: Fracture of fiber reinforced polymer layers, Fig. 3 (b).

OVERVIEW OF RC BEAM-COLUMN JOINT LITERATURE

A timeline of the exterior RC beam-column joint studies in literature and their research parameters such as concrete quality, stirrup ratio in joint, column axial load ratio, hook details of beam, existence of transverse beams and slabs, etc. are presented in Figure 4. In this figure, the blue circles define joint shear behaviour oriented studies, the green
ones define FRP retrofitting studies, the yellow circles define pure analytical studies and the red circles define the other types of retrofitting studies performed without FRPs. As seen in this figure, researches on RC beam-column joints have been conducted since 1967 until today (e.g. Hanson and Connor 1967; Marques and Jirsa 1975; Meinheit and Jirsa 1977; Uzumeri 1977; Paulay et al. 1978; Meinheit and Jirsa 1981; Zhang and Jirsa 1982; Songchau and Jirsa 1983; Ehsani and Wight 1985; Ammerman and French 1988; Soroushian et al. 1988; Pessiki et al. 1990; Zerbe and Durani 1990; Alcocer and Jirsa 1991; Tsonos et al. 1992; Beres et al. 1992; Priestley and Hart 1994; Hwang and Lee 1999; Tsonos and Stylianidis 1999; Vollum and Newman 1999; Hakuto et al. 2000; Clyde et al. 2000; Gergely et al. 2000; Liu and Park 2001; Ghobarah and Said 2002; Pantelides et al. 2002; Pampanin et al. 2002; Lowes and Altoonash 2003; Murty et al. 2003; Antonopoulos and Triantafillou 2003; Ghobarah and El-Amoury 2005; Wong 2005; Kuang and Wong 2006; Pampanin et al. 2007; Engindeniz 2008; Gokgoz 2008; Barnes and Jigoral 2008; Favatta et al. 2008; Karayannis et al. 2008; Tsonos 2008; Karayannis and Sirkelis 2008; Park and Mosalam 2009; Shrestha et al. 2009; Bedirhanoglu et al. 2010; Parvin et al. 2010; Alsayed et al. 2010; Akguzel and Pampanin 2010; Bouselham 2010; Al-Salloum et al. 2011; Ilki et al. 2011; Hassan 2011; Helal 2012; Park and Mosalam 2012; Sezen 2012; Prota et al. 2014).

**Retrofitting of Exterior RC Beam-Column Joints by FRPs**

Although the number of experimental studies for seismic retrofitting of deficient RC beam-column joints with FRPs tends to increase in recent years, the complexity of the problem and the variety of the parameters still make the subject challenging for the engineering community (Bouselham 2010). The complexity of research on FRP retrofit of beam-column joints mainly stems from the following issues:

- The location and configuration of the beam-column joint: Observations after destructive earthquakes show that exterior beam-column joints (particularly corner joints) experience more damage than the interior ones, which are generally confined by the transverse beams and slabs.
- Presence of transverse beams and slabs: Presence of transverse beams and slabs plays a crucial role in the retrofitted joint behaviour, not only due to the mechanical complexity introduced to the joint behaviour (in means of the force components), but also due to the difficulties in application of the FRP retrofitting.
- Variety of deficiencies: Although the absence of transverse reinforcement in the joint region is the main deficiency observed in the joints of sub-standard structures, other deficiencies related to the column bearing capacity, concrete quality, anchorage of longitudinal bars and reinforcement detailing are among other problems, some of which sometimes may exist simultaneously. Apparently, presence of multiple deficiencies would also affect the design for FRP retrofit. Moreover, other failure modes, such as debonding or rupture of FRP materials are also introduced to the joint after retrofitting.
- FRP retrofit schemes: FRP is a highly tailor able material with several products (such as sheets, laminates, bars, profiles produced from unidirectional or multi directional fibres made of materials such as carbon, glass, aramid, basalt, etc.) available in the market. The variety of potential retrofit schemes that can be developed with these materials requires design approaches that considers the mechanics and failure characteristics of each respective retrofit scheme.

In this study, an overview of the available experimental studies for seismic retrofitting of deficient exterior RC beam-column joints with FRPs is presented as well as a brief summary analytical modelling of retrofit of joints. For this purpose, some main aspects of the available experimental studies, that include the RC joint type, test parameters, concrete quality, reinforcement type, presence of transverse beams and slabs, FRP type, short description of the tested retrofit scheme and failure modes of the reference and retrofitted specimens are summarized in Table 1. In order to visualize different retrofit schemes outlined, illustrations of joint retrofit schemes from these studies are also compiled in Fig. 5. Accordingly:

- Majority of the studies, except Antonopoulos and Triantafillou (2003); Engindeniz et al. (2008); Tsonos. (2008); Alsayed et al. (2010); Akguzel and Pampanin (2010); Ilki et al. (2011); Al-Salloum et al. (2011) and Prota et al. (2014), include 2D T-type beam-column assemblages that do not have any transverse beams or slabs. Although, neglecting the transverse beam and slab may lead to a worse performance than for actual unretrofitted joints with transverse beam and slab and does not reflect the actual geometric conditions for the application of FRP retrofitting (Ilki et al. 2011), studies with T-type specimens may help in the direction of an in-depth understanding of the effectiveness of the FRP retrofitting solutions (Karayannis and Sirkelis 2008). Moreover, studies on 3D beam-column assemblages tend to increase in recent years.
Figure 4. Timeline of research on exterior and corner RC beam-column joints
Almost all studies include specimens constructed with medium strength concrete that vary between 17 and 37 MPa compressive strength. Only exception to this observation is the study reported by Ilki et al. (2011) in which the specimens have been constructed with a concrete with compressive strength as low as 8 MPa (which is not an exceptionally low strength concrete for existing buildings in developing countries). It should be noted that, the concrete quality becomes a major concern particularly for bond critical type FRP retrofit applications, where debonding of the composite material may highly influence the efficiency of the retrofit. In addition, the bond and anchorage of the internal steel bars within concrete is also poor in case of low strength concrete that has a great potential to influence the behaviour of both unretrofitted and retrofitted joints.

Most of the researchers preferred to utilize deformed bars for construction of the specimens. This sounds appropriate for developed countries where plain bars are not in use for several decades. However, for many developing countries, round plain bars are still being used or have been widely used until recent years. Studies such as those carried out by Liu and Park (2001); Pampanin et al. (2002); Pampanin et al. (2007); Akguzel and Pampanin (2010) and Ilki et al. (2011) include plain round bars which caused significant bond-slip effects during testing. These researches proposed other measures such as confinement, addition of steel ties or FRP strips, welding and replacement of weak concrete in the regions of deficient anchorage zones for further improving the contribution of FRP retrofit in case of plain round reinforcing bars.

Carbon FRPs seem to be the most common type composites investigated in the presented studies, probably due to their superior strength and stiffness characteristics. However, glass fibres have also been widely used for retrofitting of test specimens. Although weaving technology for fibres is already advanced notably and multi-axial sheets are available in the market, majority of the researchers have preferred to use unidirectional FRPs.

Retrofit schemes can mainly be classified as,

- schemes that wrap the joint area with sheets or strips along the beam axis (Fig. 5). These wraps are generally U-shaped for T-type specimens, and L-shaped for 3D specimens with transverse beams or slabs. Since these applications are bond critical, in many cases, researchers have proposed measures to delay or avoid the debonding of composite layers, either by applying confinement at the beam end or by using mechanical anchors (generally done with steel plates and bolts). In addition to the wrapping of the critical regions of the joint and beam, axis of columns are also generally confined, and in some cases, additional longitudinal FRPs are bonded along the columns to ensure a strong-column weak-beam behaviour.
- schemes that wrap the joint area with diagonal sheets (Ghobarah and Said 2002; Ilki et al. 2011 and Sezen 2012). This approach aims to confine the joint area with fibres that are aligned along the joint with angles closer to the principal tensile stresses that occur in the joint panel.

A wide range of test parameters such as; surface preparation, heat curing of FRP composites, beam bar anchorage, column axial load level, presence of joint reinforcement, pre-damage and repair, presence of transverse beams and slabs and loading type (cyclic, monotonic, bidirectional) have been investigated and coupled with different retrofit schemes (Table 1).

Reference specimens tested by various researchers generally failed in joint shear (J) as they were intentionally designed for this type of failure. However, depending on the anchorage status of the longitudinal bars and capacities of beams and columns, joint shear failure modes accompanied with slip effects (SJ) and column or beam hinging (BJ and CJ).

As seen in Table 1, tested retrofit designs were generally successful in enhancing the shear behaviour of deficient beam-column joints and in transferring the damage to the beams (B type failure mode). In cases where FRP debonding was a concern (bond critical applications without additional anchors) joint performance was relatively less improved. In other words, the joint shear failure was only delayed. However, mechanical anchorages proved to be successful in preventing or delaying the debonding (although local debonding of FRP layers was still generally observed) and leading to hinging of the beams (B type failure mode). In some cases, rupture of FRP sheets or strips was also observed at relatively higher drifts. Finally, column hinging (C type mode) was also observed after retrofitting, depending on the relative bending capacities of beams and columns.

A number of shake table tests were conducted to understand behaviour of exterior RC beam-column joints after retrofit FRP materials under dynamic loads (Garcia et al. 2010 and Quintana Gallo et al. 2012). Both studies noted to the efficiency of FRPs for enhancement of the behaviour deficient joints as also observed during the quasi-static member tests.
Code Recommendations

Many codes and guidelines are currently available for seismic safety assessment of existing structures (e.g., ASCE 41 2006; ACI 352 2011; Eurocode 8 2004; Turkish Seismic Design Code 2007; Architectural Institute of Japan 1999; ACI 352 2002). All of these documents include or refer to equations for calculating the shear strength of RC beam-column joints (which is required for checking whether the joint capacities are adequate or not). Moreover, some of these documents identify the backbone rotation curves to simulate actual behaviour of the structures more realistically (ASCE 41 2006; ACI 369 2011). In the last decade, a number of design guidelines and regulations for strengthening of RC structures by FRP materials have been published and became available (e.g. CSA S806-02 2002; Eurocode 8 2004; CNR-DT200 2004; ACI 440.2R 2008; Turkish Seismic Design Code 2007). However, except the Italian CNR-DT200 (2004) document, none of these documents includes design principles for strengthening of RC beam-column joints by FRP materials. The CNR-DT200 (2004 and 2012) guideline indicates that the beam-column joints of RC members can be effectively strengthened with FRP only when FRP reinforcement is applied with the fibres running in the direction of principal tensile stresses and provided that FRP reinforcement is properly anchored. This guideline limits the maximum tensile strain for FRP reinforcement to 0.4% for which a similar value was also observed during the tests of Ilki et al. (2011). For the next generation seismic design/retrofit guidelines/codes, there seems to be a need to include the issue of seismic retrofit of RC joint using FRP materials.

Modelling Approaches

Generally, the design procedures for FRP retrofit of exterior beam-column joints without transverse reinforcement aim to resist excessive tensile stresses that the concrete cannot bear and transfer the damage from the joint core to more ductile members (particularly to beams). For this purpose, the contribution of FRP sheets or strips \( T_{frp} \) are considered as a tensile force in the horizontal force equilibrium (Eq. 1) which is shown in Figure 6. In this figure, \( V_c \) is the joint shear force, \( V_{jn} \) is the joint shear force, \( T \) is the force in the tension bars of the beam, \( A_t \) is the FRP cross-section area in the fibre direction, \( E_t \) is the modulus of elasticity of FRP and \( e_{frp} \) is the effective strain of the FRP. The effective FRP strain value \( (e_{frp}) \) may vary depending on the selected FRP retrofit scheme and expected failure mode. The retrofit schemes can affect the efficiency of FRP retrofit by anchorage details and fibre directions. The possible failure modes which can influence the effective strain of FRP;

- the FRP would fail by the rupture of fibres when the tensile strength of FRP is reached,
- the FRP would debond from the concrete surface without reaching its tensile capacity.

Among the available design procedures, the following effective FRP strains are proposed as constant values: Tsonos and Stylianidis (1999) 0.0035; Gergely et al. (2000) 0.0033 for surfaces prepared with brushing and 0.0021 for surfaces prepared with pressured water; Ilki et al. (2001) 0.004; Ghabarah and Said (2001) and Sezen (2012) uses the ultimate elongation of FRP since they use mechanical anchorages to prevent debonding. Alternatively, a number of proposed design procedures such as Antonopoulos and Triantafillou (2002); Pampanin et al. (2007); Tsonos (2008); Shrestha et al. (2009); Bousselham (2010); Akguzel and Pampanin (2012) require the calculation of the effective FRP strain. It should be noted that diagonal compression strut mechanism is another concern for design of FRP retrofit of RC joints.

\[
V_{jh} = T + T_{frp} - V_c
\]  

ITU TESTS ON FRP RETROFIT OF JOINTS

Between the years 2002 and 2013, two major studies on behaviour and retrofit of reinforced concrete beam-column joints have been carried out at the Istanbul Technical University, Structural and Earthquake Engineering Laboratory. All specimens were constructed with low strength concrete and plain reinforcing bars to represent existing building stock of relatively old buildings in Turkey. These studies were funded by TUBITAK (The Scientific and Technological Research Council of Turkey). The first study consisted of 16 RC T-shaped beam-column joint tests. This study has focused on the effect of different details of FRP retrofit and different amounts of FRP used for retrofit, as well as the effects of the axial load level column and the effect of the transverse beam and slab on the behaviour of deficient RC beam-column joints. In the second study, five full-scale three-dimensional reinforced concrete frames were constructed with similar details with the first group of specimens which were tested under constant axial load and reversed cyclic lateral displacements. This study mainly targeted to confirm the results obtained from the first study by utilizing full-scale three dimensional tests. Further details
for the first and second studies can be found elsewhere (Bedirhanoglu et al. 2010; Ilki et al. 2011, Cosgun et al. 2012).

**Specimen Properties**

The specimens of the first test group were designed to represent the beam-column joints at a corner of an intermediate floor in a reinforced-concrete building. The specimens consisted of two columns and a beam perpendicular to them. In addition to these members, a transverse beam and slab was also connected to the joint (Fig. 7(a)). One of the columns represented the lower half of a hypothetical upper-story column. The other column represented the upper half of a hypothetical lower-story column (Fig. 7(a)). The specimens were supported at the ends of the columns and lateral displacement reversals were applied to the beam (Fig. 7(b)). The properties of materials and their details were chosen to represent those of the buildings constructed in Turkey before 1990s. Consequently, the specimens were constructed with low-strength concrete (the mean measured cylinder strength was f_c=8.3 MPa at around the testing days), and plain round reinforcing bars. The characteristic yield strength of the longitudinal and transverse bars were 333 MPa and 315 MPa, respectively.

The specimens of the second test group were designed to represent corner parts of actual frames in an intermediate floor. The specimens consisted of eight half-height columns (from mid-height to mid-height at two sequential stories), beams in two orthogonal directions and slab (Fig. 8(a)). The plan dimensions of specimens were 2 m and 3 m in two principle axes. The total height of the specimens was 3 m. The columns and beams were dimensioned as 250 x 500 mm x mm and the slab thickness was 80 mm. All specimens were constructed with plain surface reinforcing bars. The compressive strength of the concrete was 11.5 MPa and the yield strength of the reinforcing longitudinal and transverse bars were 347 MPa and 357 MPa, respectively. The lower story columns were supported at the ends with pin connections (point of contra-flexure was assumed to be at mid-height of actual columns). Cyclic displacement reversals were applied to the ends of upper story columns with moment releases (Fig. 8(b)).

In design of all specimens (for the first and second groups), a certain pre-determined member strength hierarchy was considered. According to this hierarchy, the columns are the strongest members and beam-column joint regions are the weakest members of the specimens. In the applied displacement scheme, it is expected that the columns would not exceed the elastic deformation range and the beam-column joints are critical in terms of diagonal shear stresses and the slip of the longitudinal bars of the beams. If these deficiencies could be prevented, the structural behaviour would be dominated by flexural capacity of the beams. In both test group, one specimen was tested as a reference specimen. Other specimens were either tested after FRP retrofit or were tested for investigation of other parameters.

**Retrofit Procedure**

**Welding of Beam Longitudinal Bars and Replacement of Poor Concrete**

The reference specimens have failed due to slip of the beam longitudinal bars as a result of crushing of low strength concrete in front of the 90-degree hooks of the beam main bars (both the first and second groups). In order to overcome the bond-slip effects observed during the initial tests, the hooks of the top and bottom beam longitudinal bars of the specimens were welded (WELD) (Fig. 9). Welding procedure was applied after a 130 mm thick layer of concrete was removed from the external backside of beam in the joint core. After welding, the removed concrete was replaced with a high strength repair mortar (REP) (Fig. 9).

**Bonding of External Surface and Diagonal CFRP Sheets**

Some of the specimens in both test groups were retrofitted with carbon FRP sheets to enhance the shear capacity of beam-column joints and/or to try to reduce/avoid slip of the longitudinal bars of beams. The basic design philosophy of retrofitting targeted to achieve ductile failure of the specimens through flexural failure of the beams. For this purpose, beam-column joints were retrofitted with different plies of CFRP sheets that runs diagonally (200 mm width) over the external faces of the joints (fibre orientation was 45 degree) (DIAG) (Fig. 10). The retrofit interventions of the first and second groups of specimens are presented in Table 2 and Table 3, respectively. The modulus of elasticity, the failure strain and the thickness of carbon FRP sheets used in retrofit is 240000 MPa, %1.55 and 0.176 mm, respectively. To prevent stress concentrations, all corners were rounded to a radius of 25 mm before the FRP application. It should be noted that the FRP sheets were bonded on the internal and side faces of the columns to ensure proper anchorage.
<table>
<thead>
<tr>
<th>Study</th>
<th>Type of joint</th>
<th>Test parameters</th>
<th>Concrete comp. str. (MPa)</th>
<th>Reinf. type</th>
<th>Transverse members</th>
<th>FRP type</th>
<th>Retrofit scheme</th>
<th>Reference failure mode</th>
<th>Failure mode after retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gergely et al. (2000)</td>
<td>2D Exterior</td>
<td>Retrofit scheme, fiber orientation, surface preparation, curing</td>
<td>20, 34</td>
<td>Deformed</td>
<td>No</td>
<td>Carbon</td>
<td>Inclined sheets on joint and beam + with and without confinement sheets on beams and column</td>
<td>J</td>
<td>FD, C</td>
</tr>
<tr>
<td>Ghobarah and Said (2002)</td>
<td>2D Exterior</td>
<td>Retrofit scheme</td>
<td>25</td>
<td>N/A</td>
<td>No</td>
<td>Glass</td>
<td>U-shaped wrap parallel to beam with and without mechanical anchors + column confinement, diagonal sheets</td>
<td>J</td>
<td>FD &amp; J for unanchored, B for anchored</td>
</tr>
<tr>
<td>Antonopoulos and Triantafillou (2003)</td>
<td>2D Exterior and 3D Corner</td>
<td>Joint reinforcement, retrofit scheme, axial load level, pre-damage, transverse beam</td>
<td>19-29</td>
<td>Deformed</td>
<td>Beam and slab</td>
<td>Glass and Carbon</td>
<td>Strips along beams and columns, strips along beams and columns + mechanical anchors at the beam ends, sheets on columns + U-shaped sheet on beam, sheets along columns + U-shaped sheet on beam + column and beam confinement for debonding, L-shaped sheets on beam</td>
<td>J</td>
<td>FD without measures against debonding, FR with anchors</td>
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<td>Pampanin et al. (2005)</td>
<td>Bottom beam bar anchorage, retrofit scheme</td>
<td>30</td>
<td>N/A</td>
<td>No</td>
<td>Glass</td>
<td>U-shaped wrap parallel to beam with mechanical anchors + column confinement + steel ties to support beam bottom bars</td>
<td>SJ</td>
<td>B</td>
<td></td>
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<tr>
<td>Karayannis and Sikelis (2008)</td>
<td>2D Exterior</td>
<td>Retrofit scheme, joint reinforcement, pre-damage</td>
<td>36</td>
<td>Deformed</td>
<td>No</td>
<td>Carbon</td>
<td>U-wrap around joint parallel to beam + confinement on beam + with and without confinement on columns</td>
<td>J</td>
<td>B</td>
</tr>
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<td>Engindeniz et al. (2008)</td>
<td>3D Corner</td>
<td>Bidirectional loading, retrofit scheme</td>
<td>26</td>
<td>Deformed</td>
<td>Beam and slab</td>
<td>Carbon</td>
<td>Sheets along columns and beams</td>
<td>CJ + SJ</td>
<td>FD, B</td>
</tr>
<tr>
<td>Tsinos (2008)</td>
<td>3D Corner</td>
<td>Pre-damage, retrofit scheme</td>
<td>16-22</td>
<td>Deformed</td>
<td>Beam and slab</td>
<td>Carbon</td>
<td>Sheets along columns and beams + column confinement + strips at beam end to prevent debonding</td>
<td>J</td>
<td>B</td>
</tr>
<tr>
<td>Shrestha et al. (2009)</td>
<td>2D Exterior</td>
<td>Loading type, retrofit scheme</td>
<td>26</td>
<td>N/A</td>
<td>No</td>
<td>Carbon</td>
<td>Strips along the columns + confinement for debonding, U-shaped strips along the beam + confinement for debonding</td>
<td>J</td>
<td>FD, J</td>
</tr>
<tr>
<td>Alsayed et al. (2010)</td>
<td>3D Exterior</td>
<td>Retrofit scheme, pre-damage</td>
<td>30</td>
<td>Deformed</td>
<td>Slab</td>
<td>Carbon</td>
<td>U-wrap around the joint + U-wrap on beam + columns confined, U-wrap around the joint + mechanical anchors</td>
<td>J</td>
<td>B, FD</td>
</tr>
<tr>
<td>Akguzel and Pampanin (2010)</td>
<td>2D and 3D Corner</td>
<td>Bidirectional loading, axial load level, joint reinforcement, retrofit scheme</td>
<td>17-31</td>
<td>Plain</td>
<td>Beam</td>
<td>Glass</td>
<td>Sheets along the columns + U- or L-shaped sheets parallel to beams + strips to confine columns and beams and to prevent debonding</td>
<td>J</td>
<td>B, FD</td>
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<td>Parvin et al. (2010)</td>
<td>2D Exterior</td>
<td>Retrofit scheme, axial load level, column bar lap splice</td>
<td>25</td>
<td>Deformed</td>
<td>No</td>
<td>Carbon</td>
<td>Sheets along columns and beam + column confinement + strips around the beam and joint</td>
<td>SJ</td>
<td>FD, FR</td>
</tr>
<tr>
<td>Ilki et al. (2011)</td>
<td>3D Corner</td>
<td>Retrofit scheme, beam bar anchorage, transverse beam</td>
<td>8</td>
<td>Plain</td>
<td>Beam and slab</td>
<td>Carbon</td>
<td>Diagonal sheets over the joint wrapped to columns + sheets on joint</td>
<td>SJ, BJ (for welded)</td>
<td>B</td>
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<tr>
<td>Al-Salloum et al. (2011)</td>
<td>3D Exterior</td>
<td>Retrofit scheme</td>
<td>30</td>
<td>Deformed</td>
<td>Slab</td>
<td>Carbon</td>
<td>U-wrap around the joint + U-wrap on beam + columns confined</td>
<td>J</td>
<td>B, FD</td>
</tr>
<tr>
<td>Sezen (2012)</td>
<td>2D Exterior</td>
<td>Reinforcement ratio, retrofit scheme</td>
<td>28</td>
<td>Deformed</td>
<td>No</td>
<td>Carbon</td>
<td>Diagonal strips over the joint region + longitudinal strips anchored on beam</td>
<td>BJ</td>
<td>FR, B</td>
</tr>
<tr>
<td>Prota et al. (2014)</td>
<td>3D Corner</td>
<td>Retrofit scheme</td>
<td>17</td>
<td>Deformed</td>
<td>Beam</td>
<td>Carbon</td>
<td>L-shaped quadaxial sheet over joint + column confinement + U-wrap on beam</td>
<td>J</td>
<td>FD, C</td>
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</tbody>
</table>
Figure 5. Retrofit schemes tested for deficient exterior and corner RC beam-column joints
Figure 6. Free body diagram of FRP retrofitted beam-column joints

Figure 7. Specimen and test setup details of the first group of tests (Ilki et al. 2011)

Figure 8. Specimen and test setup details of the second group of tests (Cosgun et al. 2012)

Figure 9. WELD and REP interventions (Ilki et al. 2011)
Test Results for the First Group of Specimens

The results of the first test group of specimens are given in Table 2. The lateral load – drift (deflection) responses of three specimens of the first test group are presented in Figures 11 and 12. In these figures, marks indicate significant stages in each test. While one of these specimens is tested as a reference specimen (JO), the others are tested after the retrofit interventions (JW - Welding of the beam longitudinal bars and replacement of poor concrete around the hooks of beam longitudinal bars and JWC-D-5 – Intervention made for JW and bonding of 5 plies of the diagonal CFRP strips). As can be seen in Figures 11 and 12, the lateral strength of the reference specimen was significantly less than the retrofitted specimens. This difference (worse performance of specimen JO) can be attributed to the slip of the beam longitudinal bars due to local crushing of concrete inside the hook regions of these bars. The concrete crushing leads to slip of beam longitudinal bars in parallel with the beam axis. Thus, it decreases the stress carried by the beam longitudinal bars, as a result, the shear stress carried by beam column joint region reduces. Welding of the top and bottom beam longitudinal bars at the 90 degree hook region and replacement of poor concrete around the hooks of beam main bars reduces the local crushing of the concrete and prevents the slip of the beam longitudinal bars significantly. Additionally, while no distributed damage was observed on the beams, large shear cracks were seen in the beam column joint region. As seen in Fig. 12(b), this problem was solved by bonding of 5 plies of diagonal CFRP strips to the beam-column joint surface. The FRP-retrofitted specimen preserved 100% of the lateral load capacity up to 8% drift while the reference (JO) and welded specimens preserved the lateral force capacity approximately 55% and 60% at 8% drift, respectively. As seen in Table 2, the maximum measured joint shear stress of the welded specimen JW is higher than approximately 30% of the reference specimen JO. In case of the FRP retrofitted specimen, the measured maximum joint shear stress was slightly less than the welded specimen, since the damage was dominated by the flexural strength of the beams and a more ductile response was observed.

Table 2. Definition of retrofit intervention and test results for the first group of tests (Ilki et al. 2011)

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Retrofit Interventions</th>
<th>$\tau_{jh}$*</th>
</tr>
</thead>
<tbody>
<tr>
<td>JO</td>
<td>-</td>
<td>1.53</td>
</tr>
<tr>
<td>JW</td>
<td>WELD + REP</td>
<td>1.97</td>
</tr>
<tr>
<td>JWC-D-5</td>
<td>WELD + REP + 5 plies DIAG</td>
<td>1.86</td>
</tr>
</tbody>
</table>

* Joint shear stress (slab work in tension) (MPa).
Test Results for the Second Group of Specimens

The lateral load-drift (deflection) responses of the three specimens of the second test group are given in Figures 13-15. In addition, the test results are summarised in Table 3. While one of these specimens was tested as a reference specimen (B-REF), the others were tested after the retrofit interventions (B-WELD - Welding of the beam longitudinal bars and replacement of weak concrete around hooks of beam longitudinal bars with high strength repair mortar and B-WELD-FRP-H – The intervention made for specimen B-WELD and bonding of 6 plies of the diagonal CFRP strips). As seen in these figures, all specimens exhibited some pinching and the lateral force capacity was increased significantly (approximately 40%) when the top and bottom beam longitudinal bars were welded and poor concrete layer was replaced. In parallel with the observations done for the first test group, the slip of beam longitudinal bars limited the lateral force capacity of the reference specimen B-REF (followed by shear failure at around 4% drift). In case of specimen with welded longitudinal bars (B-WELD), the lateral force capacity was limited by the shear strength of beam-column joints. In FRP-retrofitted specimen, damage was transferred from the beam-column joints to beams as targeted. While the reference frame preserved approximately 75% of the lateral force capacity at 8% drift, the FRP-retrofitted specimen preserved approximately 95% of the lateral force capacity at 8% drift. As seen in Table 3, the maximum measured joint stress of the welded specimen B-WELD was approximately 20% higher than of the reference specimen B-REF. For the FRP retrofitted specimen, the maximum joint shear stress was approximately 16% higher than the welded specimen, since the damage was dominated by the flexural strength of the beams and a more ductile response was observed.

Table 3. Definition of retrofit intervention and test results for the second group of tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Retrofit Intervention</th>
<th>$\tau_{jh}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-REF</td>
<td>-</td>
<td>1.93</td>
</tr>
<tr>
<td>B-WELD</td>
<td>WELD + REP</td>
<td>2.30</td>
</tr>
<tr>
<td>B-WELD-FRP-H</td>
<td>WELD + REP + 6 plies of DIAG</td>
<td>2.68</td>
</tr>
</tbody>
</table>

*$\tau_{jh}$ Joint shear stress (MPa)
CONCLUSIONS

There are many studies on FRP retrofit of sub-standard RC joints. These studies have shown that FRPs can contribute significantly to the performance of RC joints against various deficiencies through various mechanisms. It is expected that the available studies and future work will enable different approaches of FRP retrofit of joints to be included in retrofit design and documents and guidelines.

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