Chapter 15
Innovative Techniques for Seismic Retrofitting of RC Joints

Idris Bedirhanoglu, Alper Ilki, and Nahit Kumbasar

Abstract The main target of this study is to develop simple, realistic and applicable retrofitting techniques by using innovative materials in existing deficient beam-column joints. Retrofitting is targeted at overcoming deficiencies such as usage of low-strength concrete, absence of stirrups in the joint and poor anchorage of beam longitudinal bars at the joint.

15.1 Introduction

The use of low-strength concrete, smooth (plain) reinforcing bars and insufficient transverse reinforcement in beam-column joints may cause severe damage to buildings during earthquakes. While structures with these deficiencies are common in developing countries, research on these types of joints is limited.

Early attempts to retrofit joints were made by using different types of steel and reinforced concrete jackets. More recently researches have concentrated on innovative techniques such as FRP (fibre reinforced polymers) retrofitting. On the other hand, the use of cementitious composites for joint retrofitting is very rare. The main purpose of this study is to investigate the behaviour of reference and retrofitted exterior beam-column joints constructed with low-strength concrete and smooth

I. Bedirhanoglu (✉)
Department of Civil Engineering, Structural Engineering Laboratory, Engineering Faculty, Dicle University, 21280 Diyarbakir, Turkey
e-mail: ibedirhanoglu@dicle.edu.tr

A. Ilki • N. Kumbasar
Faculty of Civil Engineering, Istanbul Technical University, Ayazaga Campus, 34469 Maslak, Istanbul, Turkey
e-mail: ailki@itu.edu.tr; kumbasarn@itu.edu.tr

M.N. Fardis (ed.), *Innovative Materials and Techniques in Concrete Construction: ACES Workshop*, DOI 10.1007/978-94-007-1997-2_15,

(plain) round bars. For the retrofitting of joints, innovative materials such as FRP sheets and precast HPFRCC (high performance fibre reinforced cementitious composite) panels are used. Beam-column joint specimens tested in most of the available studies are T-type joints without transverse beam and slab. In this study, joint specimens with transverse beam and slab are tested with the aim to represent the actual geometry more realistically.

15.2 Experimental Details

15.2.1 Specimens

Three large-scale specimens were tested to investigate the behaviour of reference and retrofitted beam-column joints against simulated earthquake excitations. Details of the reference and retrofitted specimens are given in Table 15.1. The specimens were designed to represent the exterior joint of a column and two beams at a corner of an intermediate floor in a reinforced concrete building. As can be seen in Fig. 15.1a, half of the column represented the lower half of the upper-story column and the other half of the column represented the upper half of the lower-story column. In laboratory conditions, the specimens were tested with the columns in horizontal position. Lateral displacement reversals were applied to the tip of the beam. The intersection between the column and the beam will be referred to as the beam-column joint. While there was no transverse reinforcement in the joint core, columns and beams were designed following recommendations given in the Turkish Seismic Design Code (DBYBHY-07 2007) and Reinforced Concrete Design Code TS 500 (TS 500 2000) to avoid their failure under shear forces, and enforce damage to occur in the joint core (Fig. 15.1b).

Specimens were constructed with low-strength concrete (the mean measured cylinder strength was $f_c = 8.3$ MPa for the testing days), and smooth (plain) round reinforcing bars. Sixteen mm and 8-mm smooth round bars were used as longitudinal and transverse reinforcement, respectively. The yield stresses of longitudinal

Table 15.1 Specimen details

			Retrofitting		
			Amount of FRP		Plies and designation
Specimen	Explanation	Welding of hooks of beam longitudinal bars	Total (m^2)	ρ_{FRP} [a]	
JO	Reference	No	–	–	–
JWC-D-5	Weld, repair mortar and FRP	Yes	5.4	0.0010	5 plies CFRP-200 mm diagonal strips
JWH	Weld, repair mortar and HPFRCC	Yes	40 mm thick precast HPFRCC panel		

[a]Ratio of cross-sectional area of FRP in the joint to the area of the joint in the diagonal direction

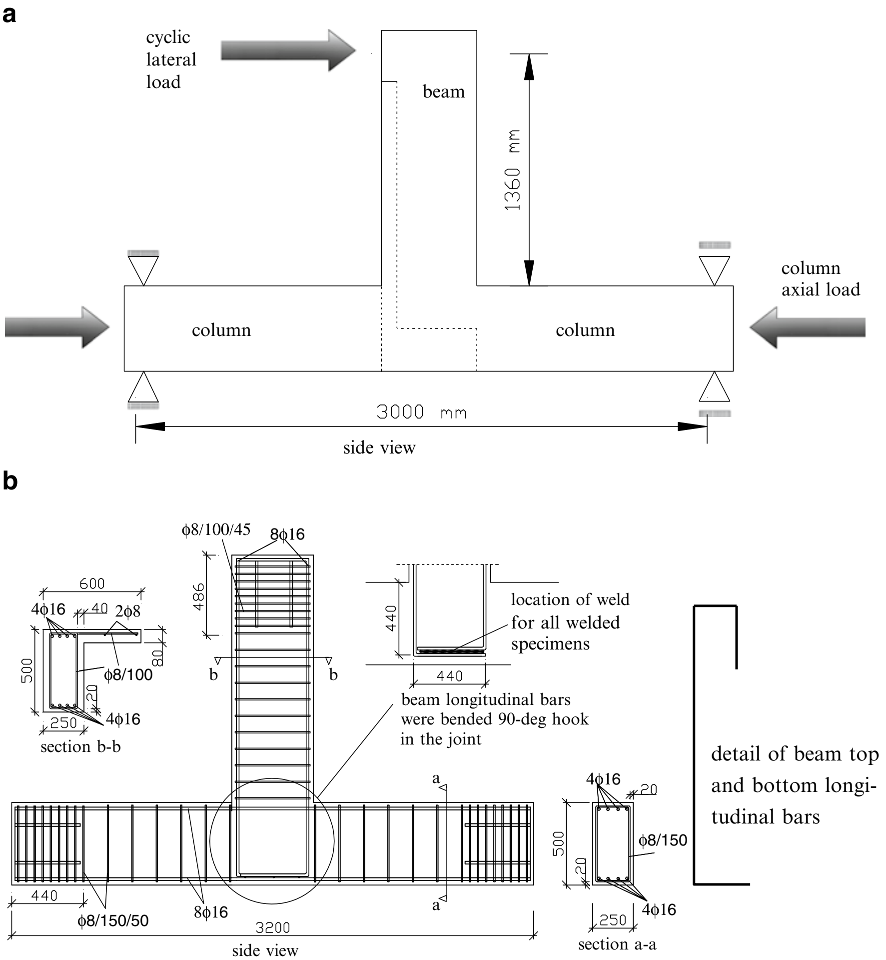

Fig. 15.1 Geometry, test setup and reinforcement details (**a**) Specimen at the test setup (**b**) Reinforcement details (dimensions are in mm and clear cover is 20 mm for all members)

and transverse bars were 333 and 315 MPa, respectively. Maximum stresses were measured to be 470 and 430 MPa and the rupture strains were 0.34 and 0.33 for longitudinal and transverse bars, respectively. These values are the average of five coupon tests for each series. It should be noted that the columns were stronger than the beam.

In all specimens, the longitudinal reinforcement of the column was continuous and the longitudinal reinforcement of the beam was anchored in the joint using 90° hooks (Fig. 15.1b). The anchorage length (including the length of the hook) was 880 mm, which corresponds to 55 bar diameters. According to TS 500 (TS 500 2000), the

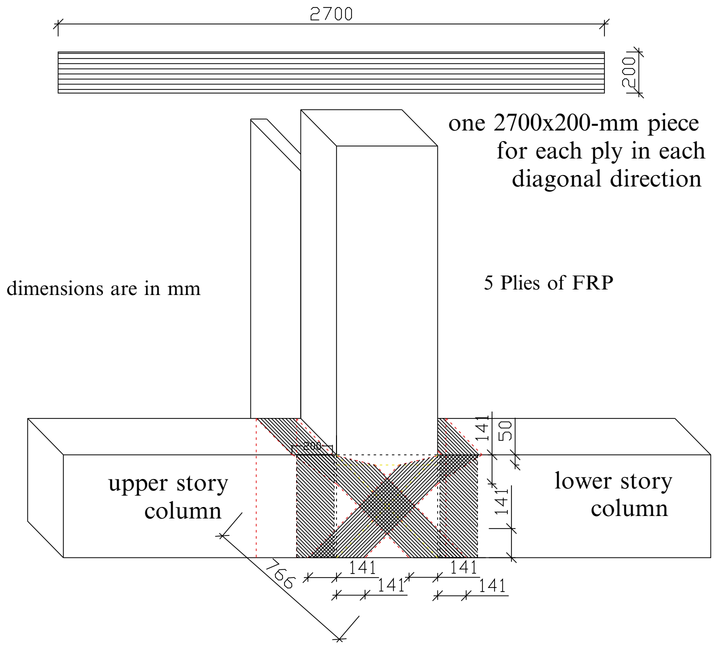

Fig. 15.2 FRP application details of specimen JWC-D-5

development length (l_{dh}) for smooth (plain) round bars with 90° hooks is 0.18 times the ratio of design yield strength of the bar to the design tensile strength of the concrete, where this length should not be smaller than $40d_b$ (d_b is the bar diameter). This definition yields a required anchorage length of 633 mm which corresponds to $40d_b$ for the tested specimens considering the design practice of the 1970s–1980s (i.e. considering a design yield strength of the bar of 220 MPa and a design compression strength of the concrete of 14 MPa).

15.2.2 Retrofitting

After testing the reference specimen, other undamaged joints were retrofitted by welding hooks of beam longitudinal bars and by shear strengthening the joint core, either with FRP sheets or with precast HPFRCC panels (Figs. 15.2 and 15.3). The design philosophy of retrofitting is to achieve a ductile failure through flexural failure of the beam. Slip of beam longitudinal bars and potential shear failure of the joint if the slip of beam longitudinal bars is to be avoided were observed to be the probable premature failure modes through the testing of a reference specimen and analytical predictions.

To prevent the slip of beam longitudinal bars in the joints, the hooks of top longitudinal bars were welded to the hooks of bottom bars in the joint. To place these

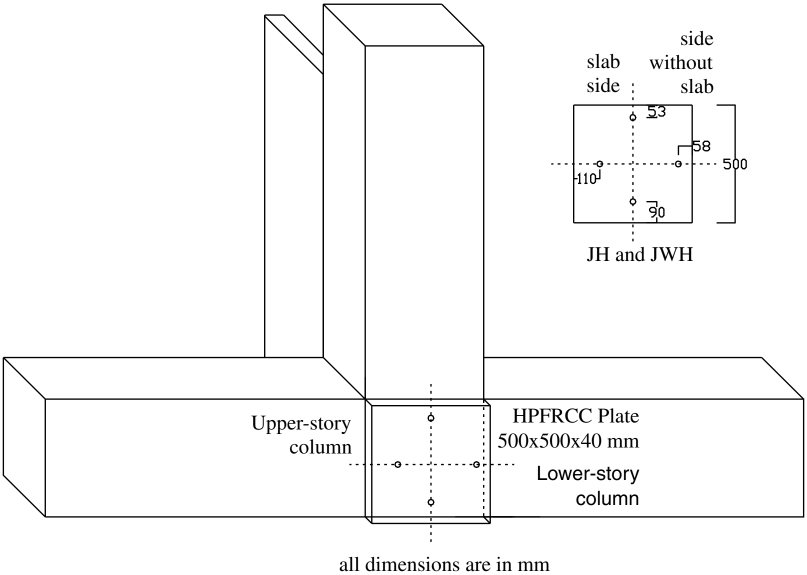

Fig. 15.3 Details of HPFRCC retrofitting (specimen JWH)

welds, a 130-mm thick layer of concrete was removed after constructing these specimens. After welding, the removed concrete was replaced with Emaco S88 high-strength repair mortar produced by BASF.

To prevent brittle shear failure of the joints, either FRP sheets or HPFRCC panels were bonded over the external surface of the joint. One of the specimens (JWC-D-5) was retrofitted with 200-mm wide FRP sheets in two diagonal directions of the joint core as can be seen in Fig. 15.2. FRP sheets were bonded only on the external side of the joints since on the other side the beam was framing into the joint (Fig. 15.2). In order to prevent stress concentrations, all corners were rounded before FRP application. Tensile strength, elasticity modulus, rupture strain, effective thickness and unit weight of carbon FRP sheets were 3,800 MPa, 240 GPa, 1.55%, 0.176 mm and 330 g/m^2, respectively. In HPFRCC retrofitting (specimen JWH), the prefabricated HPFRCC panel was bonded on the external side of the joint by an epoxy-based adhesive (Fig. 15.3). The tensile and compressive strength of the adhesive were 25 and 75 MPa, respectively, at an age of 7 days. As seen in Fig. 15.3, the dimensions of the HPFRCC panel (500 × 500 × 40 mm) were tuned to match the joint dimensions. The thickness of the bonding material between the HPFRCC panel and the joint surface was 3 mm. The contact surface was cleaned carefully before bonding and the epoxy adhesive was applied on the prepared surface with a trowel to ensure a uniform thickness of 3 mm of the epoxy adhesive layer. In addition, as a further precaution for appropriate connection of HPFRCC panel to the joint surface, four 16-mm diameter rods were used to anchor the HPFRCC panel to the joint. The embedment depths of these steel roods were 200 mm into the joint core. The bolts were fixed in the joint using an epoxy based anchorage mortar. For fixing the panels, 12 Nm torque was applied to the bolts.

The HPFRCC panel was cast in a wooden form and the form was placed on a vibration table to ensure a satisfactory compaction. It should be noted that relatively longer mixing time (≈30 min) with respect to normal concrete was necessary to obtain a workable HPFRCC. The panel was removed from the formwork after 1 day and was cured in 90°C water for 3 days and in 20°C water for 25 days. To obtain the optimum mix-proportion and high tensile strength, an extensive experimental study had been carried out beforehand. The HPFRCC mix-proportions are 925 kg cement, 204 kg water, 186 kg microsilica, 557 kg silica sand, 278 kg sand, 314 kg steel fibres and 33.6 kg admixture per cubic meter. The total volumetric ratio of steel fibres was 4%. The diameter, aspect ratio and tensile strength of the steel fibres were 0.55 mm, 55 and 1,100 MPa, respectively. The microsilica was produced by Elkem Materials with a mean particle size smaller than 500 μm and specific gravity of 2.3 kg/dm^3. The admixture was Glenium 51 hyperplasticizer produced by BASF.

In order to obtain the mechanical characteristics of HPFRCC mixture, standard cylinder compression and splitting tests were carried out at the ages of 28, 90, 180 and 360 days. The average compressive and splitting tensile strengths of the HPFRCC mixture around testing days were found to be approximately 129 and 17 MPa, respectively, and the modulus of elasticity was around 41,000 MPa (the age of the HPFRCC panels was around 60 days at the days of experiments). As shown in Figs. 15.2 and 15.3, special attention was paid for the retrofitting methods to be simple and practically applicable. Further details can be found elsewhere (Bedirhanoglu 2009; Bedirhanoglu and Ilki 2009).

15.2.3 Test Setup and Displacement History

The specimens were tested under the combined action of constant column axial load and static lateral displacement reversals were imposed on the tip of the beam. The test setup is shown in Fig. 15.1a. Nearly constant axial load of 130 kN was applied by a 600 kN-capacity hydraulic jack at one end of the column. Reversed cyclic lateral displacements were applied in the horizontal direction to the free end of the beam using a 250-kN servo-controlled hydraulic actuator. All tests were conducted under displacement control. The measuring system consisted of displacement transducers (LVDTs), electrical resistance strain gages bonded on steel bars, concrete surfaces and load cells.

Each test started with gradual application of the axial load. Subsequently, lateral displacements were imposed until the pre-defined drift ratios were reached. Drift ratios reported herein are the ratios of the displacements measured at the free end of the beam to the length of the beam. These ratios were then corrected by subtracting the rigid-body rotation associated with deformations of the supports. Specimens were subjected to 10 cycles at drift ratios increasing gradually from 1/4,000 to 1/25.

15.3 Test Results and Discussion

Test results are summarized in Table 15.2, and Figs. 15.4 and 15.5. The main deficiency observed during testing of the reference specimen was the slip of beam longitudinal bars together with shear damage at the joint after large drift ratios, such as 4%. Cracks showing slip of beam longitudinal bars at the intersection of the column and the joint and parallel to the beam longitudinal axis were observed during the test of reference specimen JO. The slip of beam longitudinal bars was also verified through a wide crack

Table 15.2 Test results

	Maximum load at tip of beam		Drift ratio at first		Diagonal deformation at 4% drift ratio (from LVDT over 480 mm gage length)				
Specimens	SWT (kN)	SWC (kN)	Flexural crack at beam	Inclined crack in the joint core		ε_{lmax}[a]	τ_v (MPa)[b]	Δ_L (%)[c]	V_{jh}/f_c[d]
JO	65.8	53.3	1/1,000	4/1,000	0.0064	0.0011	1.53	6.3	0.19
JWC-D-5	80.8	71.4	1/2,000	20/1,000	0.0013	0.0020	1.86	10.0	0.23
JWH	85.3	84.0	1/1,000	20/1,000	−0.0048	0.0017	1.97	8.5	0.24

SWT slab works in tension, *SWC* slab works in compression

[a]Maximum strain of beam longitudinal reinforcement at maximum lateral load

[b]Joint shear stress (slab works in tension)

[c]Δ_L: drift ratio corresponding to the 85% of the lateral load capacity on the descending branch

[d]Joint shear strength in horizontal direction, V_{jh} (slab works in tension). f_c is the mean measured cylinder strength

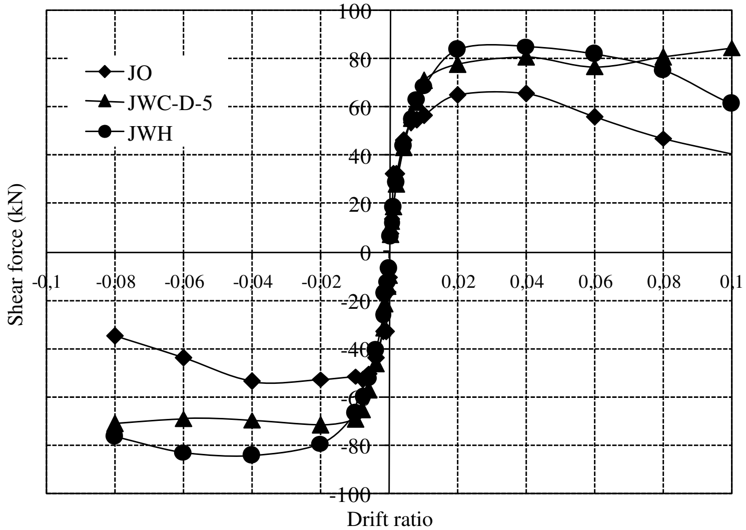

Fig. 15.4 Comparison of envelopes of shear force-drift ratio relationships of all specimens

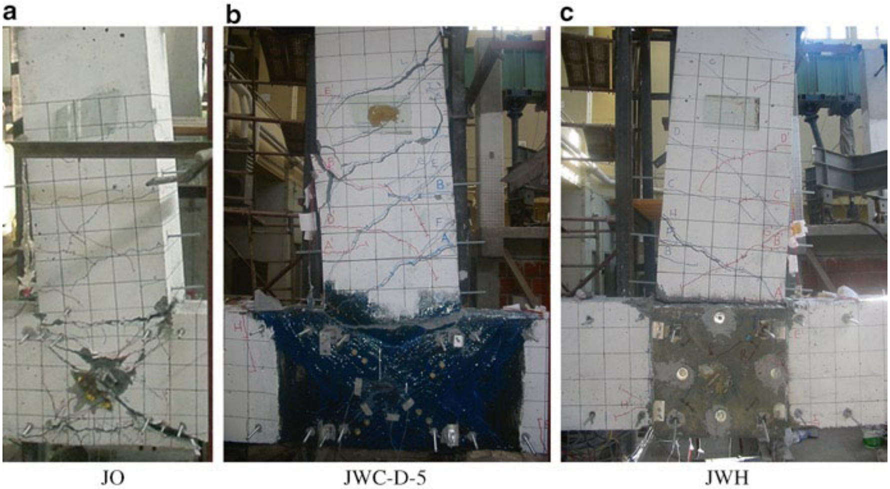

Fig. 15.5 Photos of damaged specimens (**a**) JO (**b**) JWC-D-5 (**c**) JWH

at the intersection of the beam and the joint, although the beam flexural capacity was not reached. However, the slip was not associated with the brittle failure of the specimen, and the specimen reached 4% drift ratios without any strength decay. The mechanism of slip can be explained briefly as follows. As lateral load is applied to the tip of the free end of the beam, beam longitudinal bars exert compression stress to the concrete around the corners of the 90° hooks of beam longitudinal bars. As this load increases and reaches a certain limit, gradual crushing of concrete around the corners of the 90° hooks begins and the crushing causes slip out of the beam longitudinal bars.

The slip of beam longitudinal bars, observed while testing reference specimen JO, was prevented by welding hooks of beam longitudinal bars at their hooks at the joint (Bedirhanoglu et al. 2010). As explained by Bedirhanoglu et al. (Bedirhanoglu et al. 2010), while the welding prevented slip of beam longitudinal bars, it did not improve the shear capacity of the joint core. Consequently, a ductile failure mechanism through beam hinging could not be obtained.

Therefore, in order to improve further the behaviour of the joint by preventing joint shear damage, the other specimens were retrofitted by bonding either FRP sheets or HPFRCC panel to the external face of the joint in addition to welding. Envelopes of shear force-drift ratio relationships both in positive and negative loading directions are given for all specimens in Fig. 15.4. As seen in this figure, bonding FRP sheets or HPFRCC panel to the external face of the exterior joint is a very effective way to prevent strength decay due to shear damage at the joint. The retrofitted specimens did not show any sign of strength degradation until the large drift ratios of 7–10%.

Since the amount of FRP used was significantly more than needed in specimen JWC-D-5, no damage was observed either in the middle of the joint or at the anchorage zones of carbon FRP sheets. As shown in Fig. 15.5, no important damage was observed on the HPFRCC panel, apart from a few very fine cracks.

It should be noted that the details of the behaviour of FRP retrofitted specimens can be found elsewhere (Ilki et al. 2011).

15.4 Concluding Remarks

The behaviour of existing typical deficient reinforced concrete joints before and after retrofitting was investigated. The main conclusions are summarized below:

All the specimens sustained their capacities to carry lateral loads during static displacement reversals with maximum drift ratios of up to 4% where the maximum strength degradation was less than 10%. The pseudo-ductile behaviour of the reference specimen is mainly due to local gradual crushing of low strength concrete around the 90° hooks of beam longitudinal bars.

It was clearly seen that through adequate design and detailing of FRP or HPFRCC retrofitting of joint cores together with rehabilitation of the anchorage of beam longitudinal bars through welding, the specimens could reach their flexural capacity and could keep their strengths until the drift ratios of 7–10%.

References

Bedirhanoglu I (2009) The behaviour of reinforced concrete members with low strength concrete under earthquake loads: an investigation and improvement. Dissertation, Istanbul Technical University, Istanbul

Bedirhanoglu I, Ilki A (2009) HPFRCC for rehabilitation of reinforced concrete members with low-strength concrete. ITU J/d Eng 8(6):146–156, in Turkish

Bedirhanoglu I, Ilki A, Pujol S, Kumbasar N (2010) Seismic behavior of joints built with plain bars and low-strength concrete. ACI Struct J 107(3):300–310

DBYBHY-07 (2007) Regulations for buildings to be constructed in earthquake prone areas, Turkish seismic design code, Ankara, Turkey

Ilki A, Bedirhanoglu I, Kumbasar N (2011) Behavior of FRP-retrofitted joints built with plain bars and low-strength concrete. ASCE J Compos Constr 15(3):312–327

TS 500 (2000) Requirements for design and construction of reinforced concrete structures. Turkish Standards Institute (TSE). Ankara, Turkey