Recent Developments in Seismic Strengthening of RC Beam-Column Joints with FRP Materials

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SUMMARY
In this contribution, the latest research findings into the effects of multiaxial loading on the seismic performance of as-built and FRP retrofitted three-dimensional (3D) beam-column joints with and without floor slabs are presented. For this purpose, the experimental results of four 2/3 scale, deficient RC beam-column joints are presented and conclusions are drawn on the basis of observed global and local performance. Special emphasis is given to the feasibility and efficiency of a retrofitting intervention using glass fibre reinforced polymer (GFRP) composites. A performance-based retrofit approach is adopted with attention given to the targeted specific limit states or design objectives. In addition, a numerical study is presented to calibrate and develop versatile finite element (FE) model, based on microplane concrete, to simulate the response of the 3D corner as-built joint under bidirectional loading with concurrent varying axial load and compared with the experimental results.

Keywords: Reinforced concrete, beam-column joint, seismic retrofit, biaxial tests, finite element

1. INTRODUCTION

Many examples of damage are found in recent earthquakes (Japan, 1978; Algeria, 1980; Italy, 1980; Greece, 1981; Mexico, 1985; Taiwan, 1999; Turkey, 1999 and 2002; Italy, 2009; New Zealand, 2011) which can only be interpreted by the complex three-dimensional (3D) behaviour of frames subjected to multi-dimensional earthquake excitations. The partial or complete collapse of reinforced concrete (RC) buildings have demonstrated the vulnerability and need for retrofitting of RC corner columns and beam-column joints built without seismic considerations, in order to withstand the multidirectional nature of seismic excitations and response. As a consequence, either for well-designed and older designed structures, the problem of designing or assessing/retrofitting these structural components which are subjected to bidirectional loading, imposes a great challenge to structural engineers. If these effects are not taken into account properly, particularly in the assessment and retrofitting of older and poorly detailed reinforced concrete structures, disastrous results for the overall stability of the system may occur. (Pampanin et al., 2007; Akguzel, 2011a)

The behaviour of beam-column joints in plane frames under seismic loading has been extensively investigated by experimental testing since the 1960`s. Most of these studies were undertaken with the aim of verifying the design of new space frame joints, whilst there has been far less experimental investigation into the behaviour of under-designed (e.g. following older code of practice when compared to current one and prior to capacity design principles were introduced) beam column joints in space frames either in as-built or retrofitted configurations (Hertanto, 2006; Chen, 2006; Akguzel et al., 2010b; Engindeniz, 2008).

Several experimental tests have been conducted to investigate the behaviour of deficient full-scale RC buildings strengthened with FRPs using uni-directional (Balsamo et al., 2005) and bi-directional pseudo-dynamic (Ludovico et al., 2008) or quasi-static lateral load tests (Della et al., 2006). Garcia et al. (2010) also reported the experimental results of uni-directional shake table tests performed on a full-scale RC frame with poor detailing in the beam-column joints in as-built and CFRP retrofitted configuration.

Recently, as part of a more extensive research program on the Seismic Retrofit Solutions for RC buildings in New Zealand (Pampanin, 2009; FRST website), a non-ductile 3-storey 2/5 scale RC frame model structure was tested under unidirectional horizontal input shaking on the shake table of the University of Canterbury to assess the effectiveness of the proposed FRP retrofit technique and validate the adopted design procedures (Akguzel et al., 2011a; Quintana-Gallo et al. 2011, 2012).

It is important to note that, almost all the works cited previously were basically investigating the behaviour of retrofitted beam-column joint with floor slabs under uni-directional loading input and validating the proposed design methodology based on the obtained test results. For a clear understanding of adverse multiaxial effects and for the establishment of rational retrofit design procedures, it is crucial to add test data on the behaviour of these structural elements subjected to bidirectional loading reversals with concurrent variation of axial load. Furthermore, it is noteworthy that, to the best knowledge of the authors, there have been no numerical studies conducted on the effects of biaxial loading and varying axial loading, neither on as-built nor on retrofitted corner beam-column joints. This study mainly aims to fill these gaps in literature and contribute to knowledge development in the earthquake engineering community.

2. EXPERIMENTAL STUDY

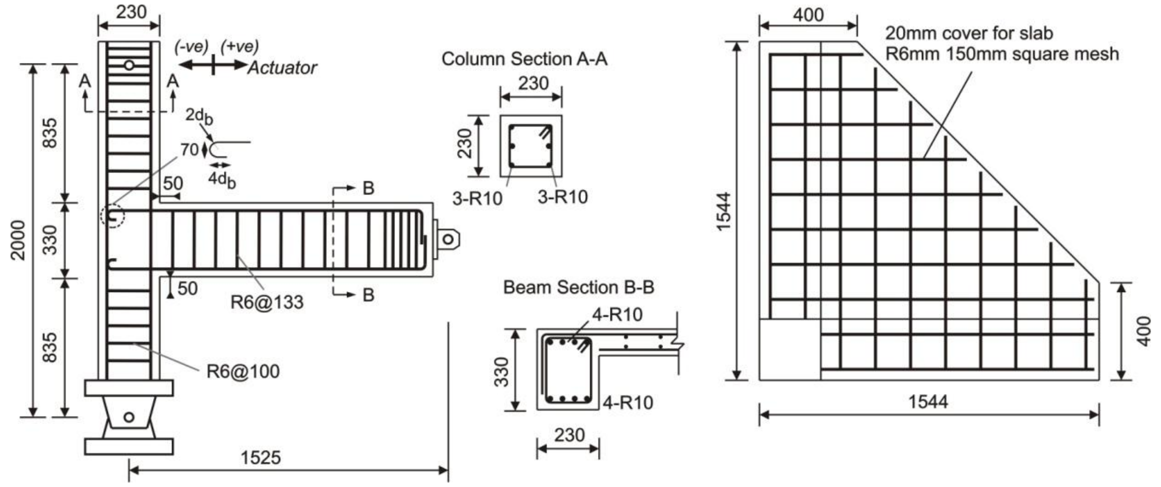
2.1. Specimen Description, Test Setup and Loading Protocol

A series of experimental quasi-static cyclic tests were performed on a total of four RC 2/3 scale specimens comprising an as-built plane frame joint (2DB), an as-built space frame joint (3DB), a retrofitted space frame (3DF) and a retrofitted space frame with floor slab (3DFS). All specimens were produced to pre-1970s construction practice, i.e., plain round bar with end-hook, and no joint stirrups (Figure 2.1a). The detailing of 3D specimens is equivalent to those of the x-direction face of the 2D specimen. The cast-in-situ slab had thickness of 100mm with R6 mesh on 150mm square. The reinforcement detailing of the slab onto the b-c joint was consistent with typical gravity-designed one-way slab, with continuous or tension anchorage for top mesh bars, and discontinued or hooked bottom mesh bars. The average values of the yield stress, f_y, and ultimate stress, f_{su}, of the longitudinal bars were 430 and 530 MPa for the as-built specimen 2D1 and 340 and 440 MPa for the remaining specimens, respectively. The concrete compressive strengths, f_c`, are given in Table 2.1.

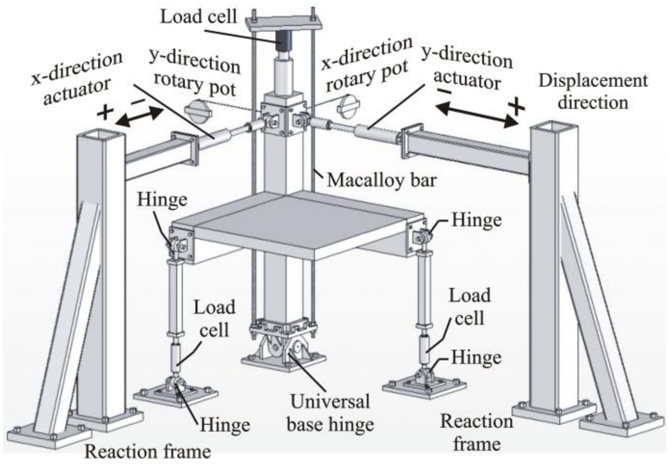
Figure 2.1b illustrates the test set-up and loading protocol adopted. Beam and column elements were extended between points of contra-flexure where pin connections were introduced. The as-built specimen 2DB tested under in-plane displacement-control cyclic loading consisted of two cycles at increasing drift levels (0.1, 0.2, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 and 4.0%) imposed to the top of the column. In the 3D tests, the 2D loading protocol was extended to 3D dimensions by adopting a cloverleaf loading path (Figure 2.1c). The axial load was varied around the gravity load value based on tributary area in proportion to the lateral force as it would occur during the frame lateral sway: $N=N_{gravity}(=110\ kN)\pm\alpha V_c$ where α, is a function of the geometry of the building (Pampanin et al., 2002). The as-built specimen, 2DB was tested under varying axial load with α coefficient of 4.63 whereas to prevent any axial tension effect in the columns in the 3D tests an upper value of α=2.35 was used. According to the adopted sign convention, positive drift and positive lateral force correspond to decrease in the axial load.

2.2. Performance Based Seismic Retrofit

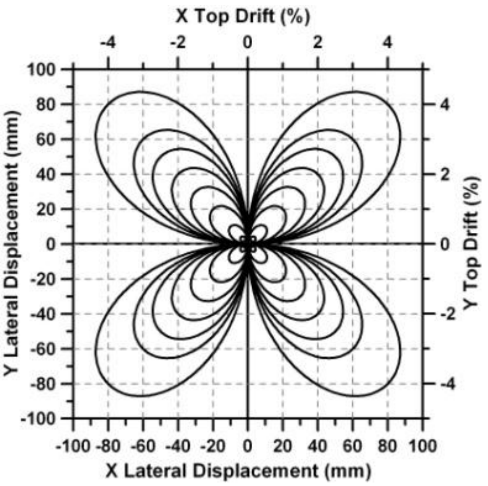
The main aim of the retrofit strategy adopted in this study was to provide adequate protection to the joint region in such a way that the ‘hierarchy of strength’ in the as-built subassembly could be converted into a more favorable ductile failure mechanism. In order to achieve this, the retrofit strategy is performed under the umbrella of Performance-Based Seismic Retrofit criteria (Pampanin, 2009; Pampanin et al., 2011). More specifically, the retrofit design methodology for exterior beam-column joints proposed by Akguzel et al. (2012) has been adopted. Hence, different levels of performance (i.e., Collapse Prevention, Life Safety, etc.) are taken into account along with the other parameters (i.e., damage, cost of repairing and invasiveness). Furthermore, by adopting this methodology, the performance of different retrofit techniques can be easily identified by the designer by rearranging the sequence of events to achieve capacity design provisions. A summary of the assessment and retrofit design procedure is given in Figure 2.2 as a flowchart in a step-by-step fashion. More information can be found in Akguzel et al. (2011a).



a) Specimen details



b) Test setup for bidirectional loading



c) Bi-directional loading pattern

Figure 2.1. Specimen details, test setup and loading pattern (all dimensions in mm)

Step 1: Capacity assessment of RC elements (ie, shear strength of exterior corner beam-column

↓

Step 2: Determination of demand curves (ie, effects of varying axial load due to the lateral

↓

Step 3: Generation of M-N performance domain and the evaluation of the sequence

↓

Step 4: Shear strength check of critical sections

↓

Step 5: FRP dowel anchorage design

↓

Step 6: Determination of the amount of FRP material and application

Figure 2.2. Flowchart for assessment and retrofit methodology

2.3. FRP Application Scheme

The retrofit strategy followed was aimed to protect the beam-column joint regions in a RC frame, which are regarded as one of the most vulnerable and critical structural elements in pre-1970s as observed in the recent earthquakes. Within the same frame of approach, for the existing deficient RC frames, a partial retrofit strategy can be implemented which consists of protecting only the exterior joints, forming plastic hinges in beams framing into exterior columns (Pampanin et al., 2011). In recent years, the feasibility and efficiency of the proposed FRP retrofit strategy for the exterior beam-column joints have been studied extensively by the authors (Akguzel et al., 2010a and 2012). Furthermore, similar approach was validated successfully in the uni-directional shake table testing of a FRP retrofitted RC frame performed at the University of Canterbury (Quintana-Gallo et al., 2011a and 2012; Akguzel et al., 2011a).

In Figure 2.3, the application sequence, dimensions of uni-directional glass fiber sheets (GFRP SikaWrap-100G) are presented in a step-by-step fashion for corner 3D beam-column joints with slab (3DSF). For the specimens 3DF (corner joints without floor slab) and 3DSF the first two steps consisted of the installation of vertical laminates on the exterior sides of the column faces (Step 1) and horizontal laminate on the joint region (Step 2). Primary objectives were to increase the flexural capacity of these elements, in order to increase the joint shear strength and to prevent the expulsion of a concrete wedge which was observed in the tests of the as-built specimens. In specimen 3DF, FRP strip anchorages were wrapped around the column similar to Step 6 for Specimen 3DFS and beams to increase the efficiency of the horizontal (main) strips. In Specimen 3DFS the vertical and horizontal FRP laminates were installed inside the column and beam faces (Step 3 and Step 4). The next step of application involves the enhancement of flexural as well as the shear strength of the beam and slab elements. In order to achieve this, in the corner joint the beams were wrapped with FRP sheets which were starting from the bottom face of the slab and extended to the top of the slab (Step 5). In the following step, FRP confinement was provided by wrapping the column element.

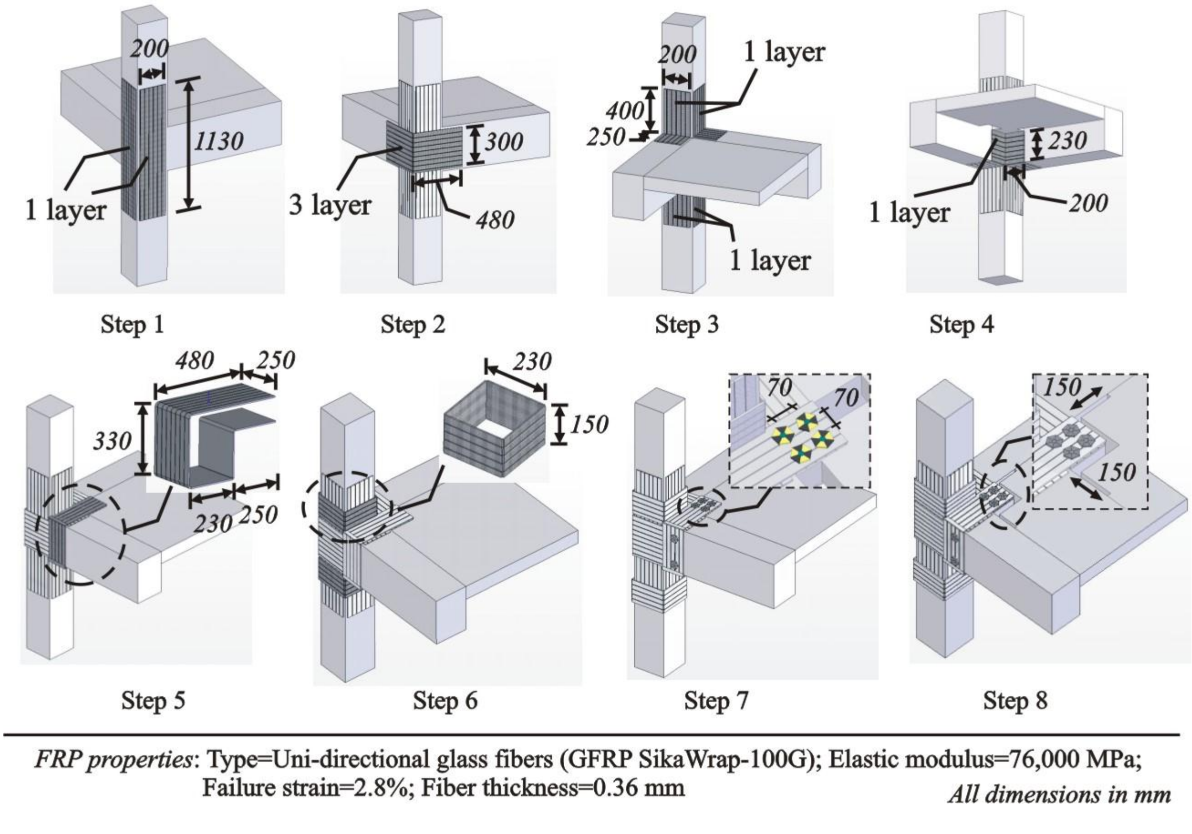


Figure 2.3. Dimensions and application sequence of FRP sheets for Specimen 3DFS

Due to the premature debonding of the FRP system from the concrete surface of the strengthened member, the FRP material might not be fully exploited. Previous experimental studies conducted by the authors (Akguzel et al., 2010a and 2010b) have shown that the proposed retrofit scheme for exterior 2D RC joints yielded successful results. However, the same retrofit solution applied to a 3D corner joints without floor slabs Specimen 3D2, proved to be insufficient mainly due to debonding of FRP layers in the beam and joint region as a result of severe multiaxial loading demands. Detailed information can be found in Akguzel (2011b). In this study one of the main objectives was to investigate the efficiency of the revised FRP retrofit proposal for the 3D corner joints without slabs. In order to mitigate the early FRP debonding problem as well as to strengthen the laminates against buckling under compression loads, in Specimen 3DF (corner joint with no floor slab), FRP anchor dowels were adopted as part of an improved FRP retrofit solution for corner beam-column joints. In the next stage the same approach was also adopted in the 3D corner joint with slab, Specimen 3DFS, (see Figure 2.3 Step 7) to investigate the possible negative effects of bidirectional loading on the performance of the retrofitted corner joints with floor slabs.

The FRP anchor dowels were prepared by twisting the strips of GFRP sheets, folding into two and epoxing into pre-drilled holes in the beams and slab. Afterwards, epoxy saturated GFRP anchor dowels are plugged into the holes by means of tie-wires and are initially injected into the holes during the placement of the epoxy resin. The ends of the anchor dowels remaining outside the holes, either in the beam or joint faces, are glued to the horizontal GFRP sheets already applied on the beam and joint faces for proper anchorage. The dimensions and detailing of the sections with anchor dowels are given in Figure 5.

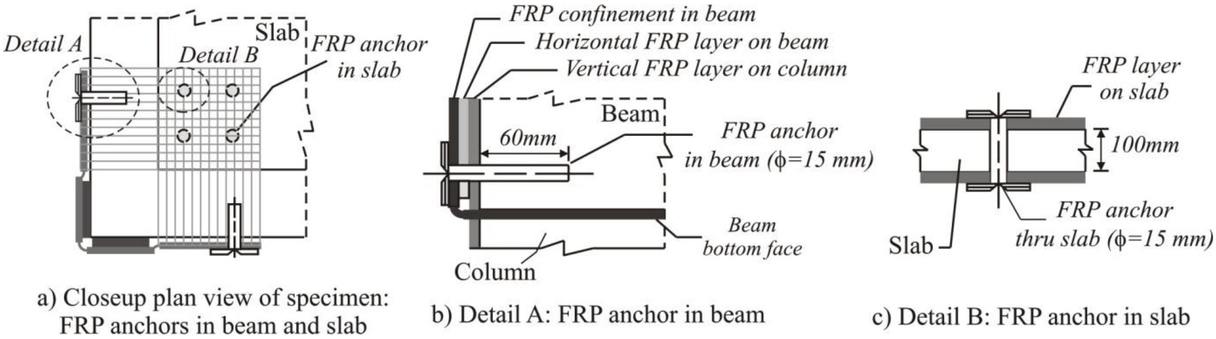


Figure 2.4. Details of FRP anchor dowels installed in slab and beam elements

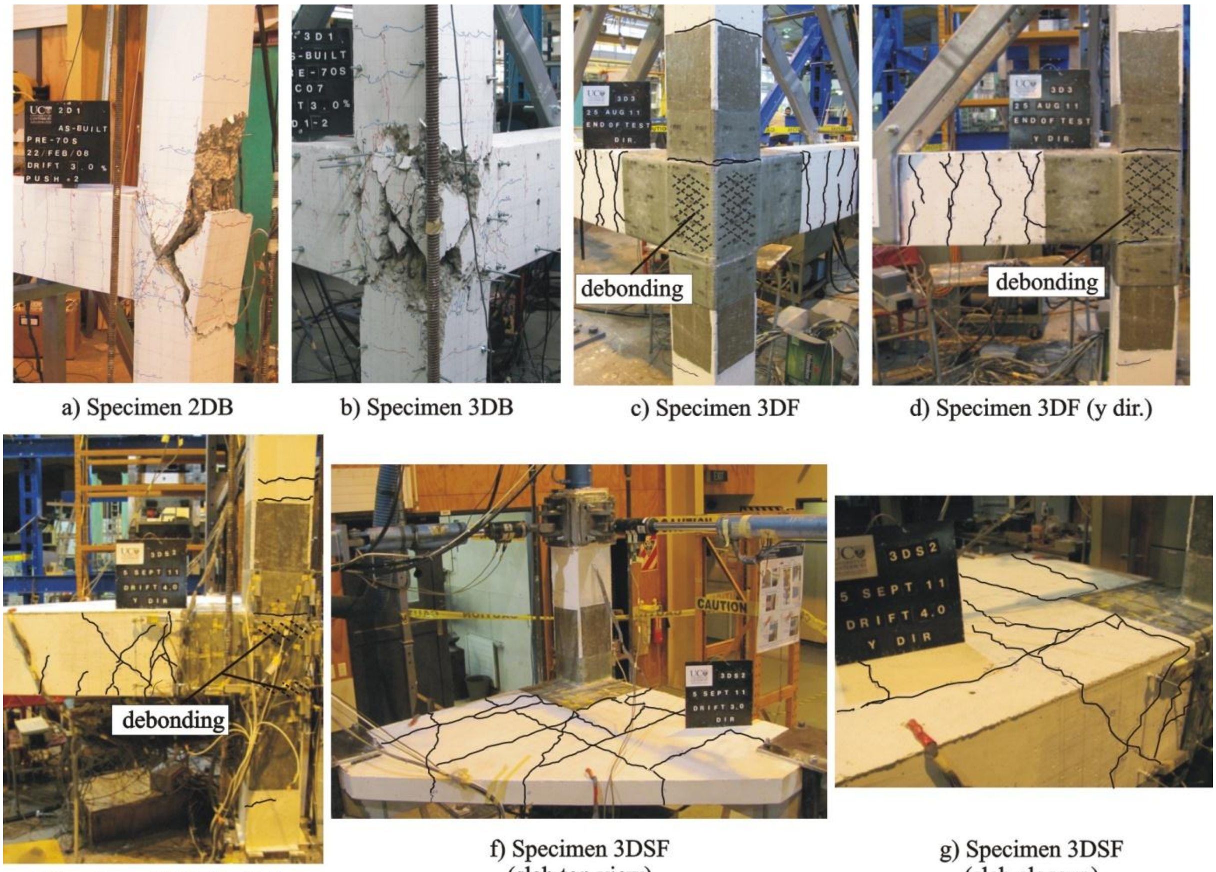
As last step of the retrofit scheme, in order to reduce the over-strength effects and deformation demand from the adjacent slab element in Specimen 3DSF, the slab flexural reinforcement was cut (selectively weakened) in the perimeter of GFRP layers anchored into the top of the slab (Step 8).

2.4. Experimental Results and Discussion

In this section the test results are analysed and discussed based on the observed damage and recorded instrumentation readings. In order to provide useful information on the effect of bidirectional loading on the corner joints, the performance of the 3D benchmark specimen, 3DB, is compared with its 2D counterpart, 2DB. Next, experimental findings of the retrofitted 3D corner beam-column joint specimens with and without floor slabs, 3DF and 3DSF, are presented along with the comparison of benchmark unretrofitted specimen corner joint 3DB. To facilitate the discussion of the test results of Specimen 3DF, the experimental observations of an additional corner beam-column joint unit without slab, referred to as Specimen 3D2, selected from the previous work of the authors (Akguzel et al., 2010a), is also added.

The test results of Specimen 2DB confirmed the inherent structural inadequacies of poorly detailed beam-column joints designed according to pre-1970s codes provisions. The first cracking was observed in the joint panel at 1% drift. A particularly brittle mixed failure mechanism, typically referred to in the literature as "concrete wedge mechanism" (Pampanin et al., 2002) was observed to initiate after 1.5% drift level. This mechanism was characterized by the combination of two separate mechanisms in the joint region: 1) shear damage in the form of cross diagonal cracking due to the inadequate confinement, 2) slippage of the beam longitudinal plain round bars leading to a concentrated compressive force at the end-hook anchorages. As a result, a wide concrete wedge was developed resulting in the expulsion of the outer face of the column after 3% drift level (Figure 2.5a). Summary of test results are given in Table 2.1.

Specimen 3DB exhibited the worst seismic performance amongst the tested 2D and 3D joint units. Biaxial loading combined with significant variations in axial load imposed more deformation and strength demand on the specimen. As a result, a drastic drop in the strength capacity with lesser energy dissipation was noticed after the first cracking in the joint panel (at around 0.5% drift) in spite of the partial confinement effect provided by the orthogonal beam. The specimen 3DB failed after heavy flexural and inclined cracking, spalling of concrete and disintegration of the core concrete (mainly due to propagation of inclined cracks and grinding along the concrete) followed by buckling of the column bars. A complex three-dimensional concrete wedge mechanism was observed to initiate during the early stages of the test (after 1% drift), following the shear cracking of joint panel zone at 0.5% drift.



a) Specimen 2DB b) Specimen 3DB c) Specimen 3DF d) Specimen 3DF (y dir.)

e) Specimen 3DSF (y dir.) f) Specimen 3DSF (slab top view) g) Specimen 3DSF (slab closeup)

Figure 2.5. Final damage states of the specimens

As anticipated, in the previous studies performed by the authors (Akguzel et al., 2010a) a 3D corner beam-column joint with no slab, Specimen 3D2, was retrofitted with a similar FRP scheme but without the no installation of FRP anchor dowels at the beam faces. During the test of Specimen 3D2, it was observed that detachment of the GFRP started in the beams at 1.5% drift in both directions of loading. At 2.5% drift, an abrupt complete detachment in the beam GFRP sheets occurred. In the following cycles, due to the development of a diagonal compression strut in the joint, bulging of the column sheet started to become evident. As a result of the extension of the GFRP debonding from the vicinity of the joint to the upper and lower column regions, buckling and partial rupture of the column sheets occurred. Post-test observation revealed a severe damage in the joint region.

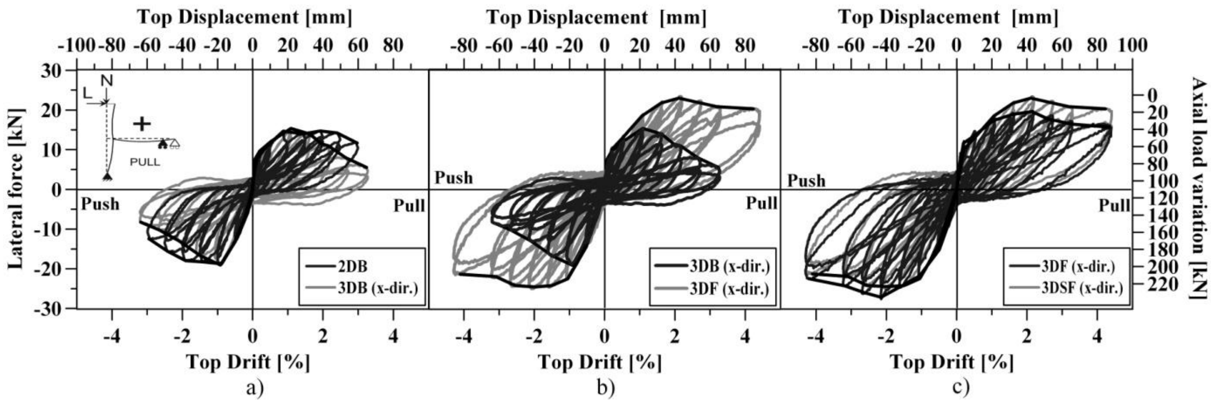


Figure 2.6. Hysteresis response comparison: a) 2DB vs. 3DB; b) 3DB vs. 3DF; c) 3DF vs. 3DSF

Table 2.1. Summary of Test Results

Spc.	f_c` (MPa)	Column shear force, V_c		Nominal axial load ratio, $N/(A_g f_c`)$	
		Pull dir. (+ve)	Push dir. (-ve)	Pull dir. (+ve)	Push dir. (-ve)
2DB	17.9	14.9	-18.9	0.05	0.21
3DB	17.4	15.3 (x-dir.) 13.6 (y-dir.)	-18.8 (x-dir.) -18.3 (y-dir.)	0.07 (x-dir.) 0.08 (y-dir.)	0.21 (x-dir.) 0.19 (y-dir.)
3DF	21.3	23.4 (x-dir.) 23.2 (y-dir.)	-24.8 (x-dir.) -24.8 (y-dir.)	0.08 (x-dir.) 0.08 (y-dir.)	0.18 (x-dir.) 0.18 (y-dir.)
3DSF	19.4	19.7 (x-dir.) 21.4 (y-dir.)	-27.6 (x-dir.) -27.5 (y-dir.)	0.10 (x-dir.) 0.09 (y-dir.)	0.20 (x-dir.) 0.20 (y-dir.)

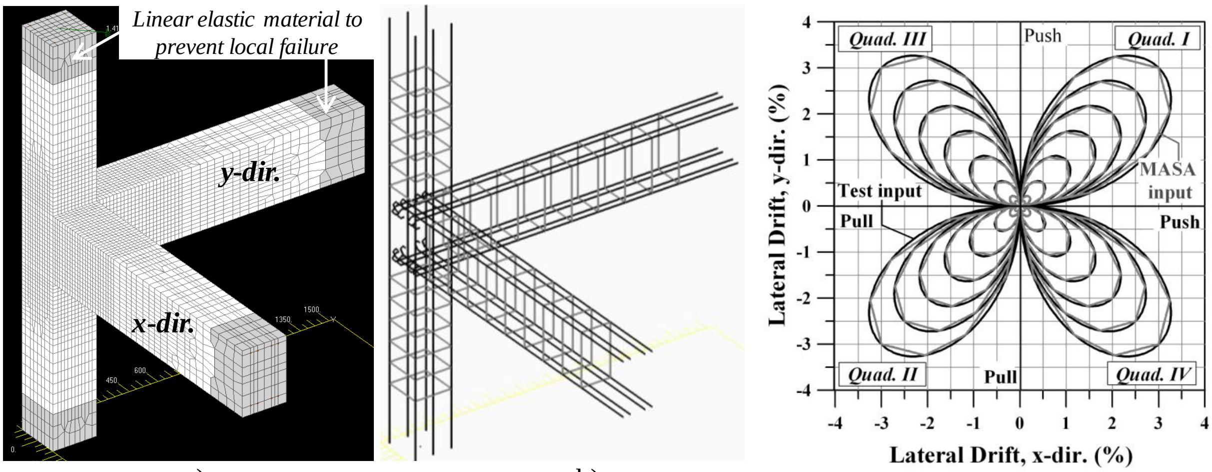
On the other hand, test results indicated that the behaviour of the specimen 3DB was very satisfactory in terms of seismic performance compared to its counterpart specimen 3D2 and benchmark specimen 3DB. Due to the use of special FRP anchors with wider FRP anchorage application in the beams and column, detachment of the FRP from the beam surfaces as well as buckling of the FRP column sheets as observed in the previous tests were totally eliminated. Following an elastic drift of 0.1%, hairline flexural cracks formed under the beam face at 0.2% drift. During the cycles performed between 0.5% to 2.5% drift levels no critical event occurred in the specimen. The specimen reached its maximum lateral strength of around 24 kN in each directions at around 2.2% drift. After 3% drift cracks barely developed in the beams and columns and FRP sheet in the joint region started to debond lead to 13% reduction in the load carrying capacity (Figure 2.5c). Nevertheless, stable hysteresis loops were observed till the end of the test. In overall, Specimen 3DF exhibited approximately 40% increase in strength and enhanced displacement ductility compared to Specimen 3D2. The post-examinations of the joint panel also showed minor damage of concrete within the joint region.

Experimental findings of an as-built counterpart of the retrofitted corner beam-column joint with floor slab, specimen 3D-B, have been previously reported by the authors (Kam et al., 2010). Test results showed very poor seismic performance characterized by severe degradation in stiffness, pinching and loss of strength starting at the very early stages of loading. After 1.5% drift level, the damage was accumulated in the joint region. The retrofitted corner joint specimen 3DSF showed significant improvements resulting in preventing brittle joint shear damage mechanism. At 2-2.5% drift levels, flexural tension cracks formed evenly with a crack width ranging from hairline to 2mm in the beams, around the columns and in the floor slab. In the following cycles, the FRP started to detach and rupture in the vicinity of upper and lower joint region close to the column ends (Figure 2.5e). After 3% drift level, the damage mainly concentrated in the interface of upper column and joint region leading to a 20% capacity reduction at the end of test. Apart from the fracture of the horizontal sheet at the column-joint intersection region, which led to the formation of pivot point at the column end, no debonding of FRP was observed in the beam and column sheets. To prevent this type of unfavourable behaviour, the FRP retrofit scheme can be refined such as adding extra sheets in the critical regions and/or installing anchor dowels. This study shows that employing independent planar frame analyses along a building`s principal axes is deemed to be a potentially inadequate and unconservative method because it ignores the significant increase in deformation and stress demand associated to the three dimensional response.

3. FINITE ELEMENT MODELLING OF SPECIMEN 3DB

3.1. FE Model and Discretization

The FE code, MASA, developed at the Institute of Construction Materials (IWB) of the University of Stuttgart (Ožbolt, 2001), was used in the study. In the code, the concrete is modeled according to a microplane model and the smeared-crack concept was used for the modeling of the cracking of the concrete. A typical finite element mesh, boundary conditions and loading pattern are shown in Figure 3.1. The same geometry and dimensions are employed in the FE model as the test specimen. The spatial discretization of concrete is performed by 4-node solid elements whereas the reinforcement is modeled by 2-node truss elements with a trilinear stress-strain constitutive law. The bond between the longitudinal reinforcement and the concrete is modeled using discrete bond elements consisting of 1D nonlinear springs. In order to simulate the bond characteristics of plain round bars, the existing bond-slip model, originally developed for deformed bars, was calibrated on the monotonic experimental behavior of plain round bars, assuming similar cyclic behavior (Genesio, 2011).



a) b) c)

Figure 3.1. FE model of joint: a, b) discretization of concrete and reinforcement; c) load input

A complex and realistic bi-directional lateral loading history, with interaction between the two orthogonal directions was generated. Each loading step was coupled with a varying axial load to simulate the triaxial loading as performed in the testing.

3.2. Results of Analysis and Comparison with Experimental Data

Due to stability problems, mainly because of the excessive damage in the joint region occurred, the results of FE analyses up to 2% drift level are presented in the following paragraphs. The hysteresis loops in x-direction show a similar trend of strength degradation for both directions of loading (Figure 3.2). The numerical initial stiffness values for x-direction loading were 3360 kN/m and 3118 kN/m for the pull and push loadings, respectively. These values were 5% lower and 48% higher than the experimental values. As seen in Figure 3.3, FE model matches quite well in the push direction of loading up to 2% drift and in pull direction up to 0.5% drift. However, in the following drift levels of the pull direction (decreasing axial load) the response of the specimen is underestimated.

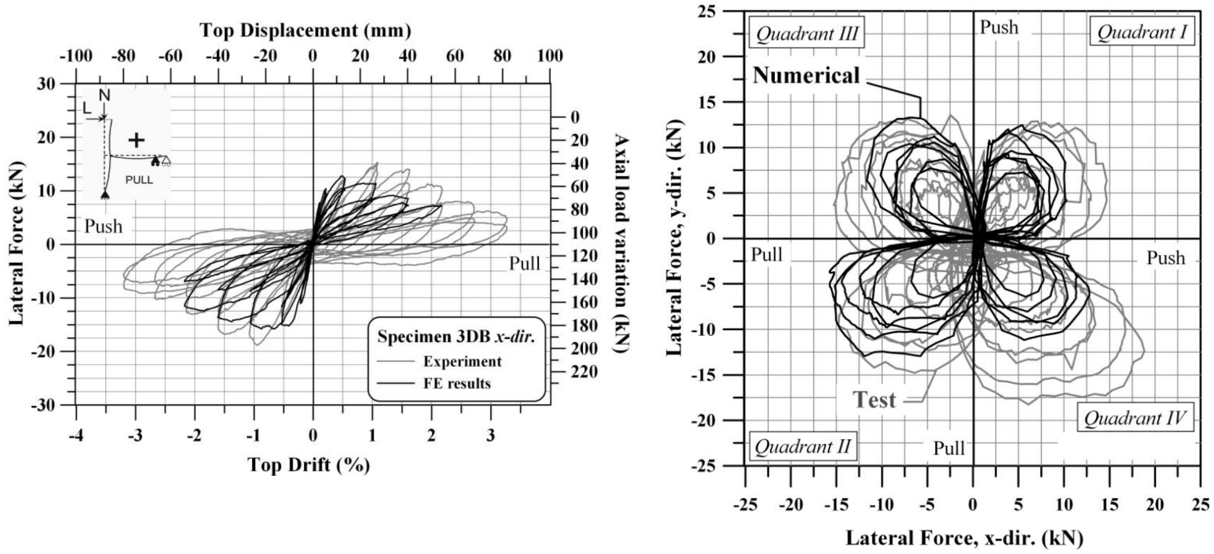


Figure 3.2. Comparison of load-displacement response: FE versus test results

In spite of the good simulation of the dominating damage pattern by the FE model, the flexural cracking in the beams and columns observed in the test up to a 1% drift level could not be reproduced by the FE analysis. Nevertheless, in Figure 3.3, some FE snapshots are presented as an example of the good prediction of cracking patterns.

a) Joint panel - trimetric view

b) Beam bottom, x-dir.: test versus FE results

Figure 3.3. Comparison of various cracking patterns after 2% drift - Specimen 3DB

4. CONCLUSIONS

In this contribution the question of efficiency of FRP retrofitting technique used to strengthen under-design corner beam-column joints with and without floor slabs subjected to multiaxial loading conditions is addressed. As demonstrated by experimental and Finite Element analysis studies, simultaneous multiaxial (i.e. multi-directional) loading can significantly change the nature of the seismic response of structures from what would be predicted using a uniaxial view. If bidirectional and varying axial load effects are taken into account in the design, the retrofit solutions proposed in this study have been proven to provide satisfactory improvements in the seismic performance of the specimens. The analysis of the 3D corner beam-column joint showed that the FE model reproduced the overall hysteretic behaviour and cracking pattern of the test unit with sufficient accuracy.

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