SEISMIC UPGRADING OF 3-D EXTERIOR R.C. BEAM COLUMN JOINTS SUBJECTED TO BI-DIRECTIONAL CYCLIC LOADING USING GFRP COMPOSITES

Stefano PAMPANIN [1] Umut AKGUZEL[1] Gabriele ATTANASI [2]

[1] Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand
[2] ROSE School, Pavia, Italy

Keywords: seismic retrofit, GFRP, exterior beam-column joints, bi-directional cyclic loading

1 INTRODUCTION

The implementation of Fiber Reinforced Polymers composite materials for the retrofit or rehabilitation of existing reinforced concrete buildings subjected to seismic loading has received, in the recent past, a substantially increased level of attention within the earthquake engineering community, after gaining a well-recognized confidence on the behaviour and practical application for non-seismic civil engineering applications. The development and continuum updating of code guidelines or provisions in major seismic prone countries related to the use of FRP composites for the seismic rehabilitation of existing buildings [1-4] have provided valuable documentations for the design and practical implementation of these solutions. Valuable experience has been gained in the laboratory, by experimental testing and validation of theoretical models, as well as on site, by facing and addressing practical constructability and invasiveness issues.

While a quite considerable understanding have been achieved and reported on the flexural and shear strengthening as well as on the confinement of single structural elements or plane configuration of beam-column connections, relatively less attention has been given to the development and validation of retrofit strategy able to control, within a performance based approach, the highly complex 3-Dimensional structural behaviour.

Several studies on the seismic response of existing reinforced concrete frame buildings (typically with masonry infills) designed for gravity loads only, as typically found in most seismic-prone countries before the introduction of adequate seismic design code provisions in the 1970s, have confirmed the inherent weaknesses of these systems as observed in past earthquake events [5-9]. Controversial issues have been raised when evaluating the effects of joint damage mechanism and the presence of infill frames to the overall system [10-13].

Most of the studies available in literature on the seismic assessment and retrofit of existing poorly detailed frame building have, in fact, concentrated on the 2-dimensional response, thus subjecting the specimen or subassemblies to uni-directional cyclic loading testing protocol (e.g. [14]). Similar considerations can be derived when referring to the extensive literature specifically related to the use of FRP as a retrofit technique for frame systems and subassemblies [15-22]. Even when the 3D response under combined bi-directional loading has been taken into account in experimental testing, the attention has been typically given to interior (fully or partially confined) joints. A particularly limited information on the response of exterior 3D corner existing beam column joints is available, in spite of their intrinsically higher vulnerability under lateral cyclic loading) due to the lack of reliable joint shear transfer mechanisms, as confirmed in past earthquake event (i.e. Turkey, 1999, Taiwan, 2001).

In this contribution, as part of a more extensive research program on seismic retrofit solutions for reinforced concrete buildings, the effects of bi-directional loading on the assessment and design of the retrofit intervention using FRP composite materials will be discussed. Based on experimental evidences on the performance of exterior 3D (corner) under-designed beam-column joints, the limits and drawbacks of standard assessment methodologies when evaluate the hierarchy of strength and expected sequence of events prior to define the retrofit intervention, will be discussed. In addition, the preliminary results of an on-going experimental campaign to further investigate and address this issue will be presented, based on the quasi-static tests under bi-directional loading on two 3D exterior (corner) joint specimens, comprising of a pre-1970s as-built specimen (benchmark) and a “minimum” retrofitted configuration using GFRP sheets. The feasibility and efficiency of the adopted retrofit strategy and solution, aiming at controlling the hierarchy of strength within the beam-column joint system by protecting the panel zone and relocating the plastic hinge in the beam, will be discussed. Considerations and suggestions on the additional design criteria to account for the actual 3-D response under bi-directional loading will be also given.

2 BEHAVIOUR OF PRE-1970 EXTERIOR BEAM COLUMN JOINTS UNDER BI-DIRECTIONAL CYCLIC LOADING

2.1 Performance of inadequately detailed beam-column joints

As anticipated, previous studies on the seismic response of pre-1970s designed reinforced concrete buildings, have underlined the critical vulnerability of the joint panel zone region, observed to be, in most of the cases, the critical "weak link" of the structural system "chain".
Different damage and failure mechanisms can be expected depending on the typology of joints (interior vs. exterior) and on the structural detailing adopted (i.e. plain round or deformed bars, anchorage solutions, total lack or presence of a minimum amount of transverse reinforcement) (Fig. 1, after [23] In particular, brittle local failure mechanisms, likely to impair the overall system loading bearing capacity leading to global collapse modes, have been observed in exterior beam-column joint with lack of adequate transverse reinforcement in the joint, due to the intrinsic lack of alternative and reliable sources of shear transfer mechanism within the panel zone region after first diagonal cracking.

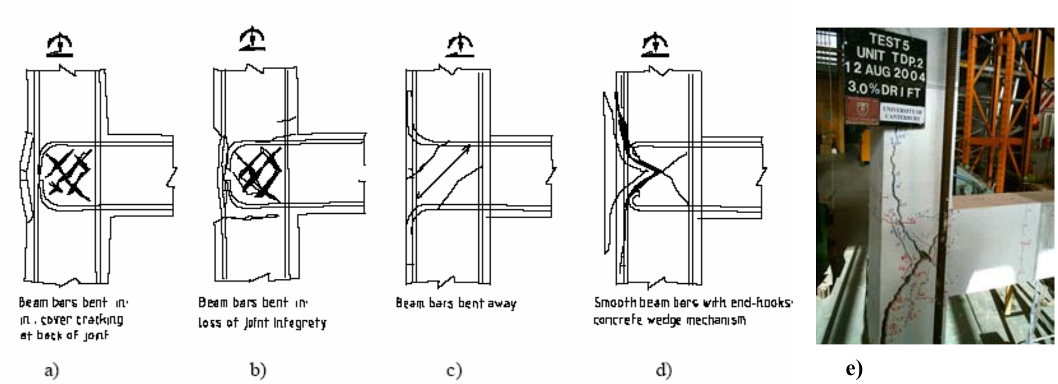


Fig. 1 Alternative damage mechanisms for exterior tee-joints [21]:
a,b) beam bars bent inside the joint region; c) beam bars bent outside the joint region; d,e) plain round beam bars with end-hooks: "concrete wedge" mechanism and damage in 2DB specimen [22]

Based on experimental evidences and numerical investigations, the concept of a shear hinge mechanism has been proposed as an alternative to the well-known flexural plastic hinge for beam and column elements [9,23]. A critical discussion on the effects of damage and failure of beam-column joints in the seismic assessment of frame systems has been given in [10]. When considering the global response of a frame system, even when joint shear damage or failure is not expected, the absence of any capacity design principles might lead to the formation of a weak-column strong-beam mechanism (possible soft storey). *Ad-hoc* retrofit strategies are therefore required, being capable of providing adequate protection to the joint region, while modifying the strength hierarchy between the different components of the beam-column connections according to a capacity design philosophy.

2.2 Effects of bi-directional loading

As part of a more extensive research project undergoing at the University of Canterbury on seismic retrofit solutions for existing reinforced concrete buildings, based on either numerical and experimental investigations, a series of quasi-static tests under uni- ad bi-directional testing protocol have been carried out on exterior 2D and 3D (corner) beam-column joint subassemblies, 2/3 scaled and representative of pre-1970s construction practice with different structural detailing (i.e. plain round with end hook anchorage or deformed bars, deep beams or shallow/wide beams, minimum or total lack of transverse reinforcement in the joint) [24].
In addition to providing valuable information on the effects of alternative structural detailing on the seismic response of poorly detailed beam-column joints, to be used for calibration/refinement of existing analytical and numerical models, the tests highlighted an expected significantly lower performance of exterior joints subjected to a bi-directional loading protocol (more representative of the expected seismic demand) when compared with their 2D counterparts subjected to uni-directional,

more typically adopted, testing loading protocol. Refinement of current assessment procedures and ad-hoc retrofit strategies are thus crucially needed when dealing with the actual 3D response of reinforced concrete buildings. Figure 2 and Table 1 show the geometry and reinforcement details of the 3D specimens adopted for either the as-built configuration (benchmark, herein referred to as 3DB) and the retrofit configurations using GFRP or other techniques. The testing results related to the first retrofitted configuration (specimen GFRF1) are herein reported. The properties of the corresponding 2D specimen in the as-built, benchmark, configuration (herein referred to as 2DB) are equivalent to those of the X-direction face in Figure 2. Plain round bars with end hook were adopted. In the joint region one single stirrup was used. Material properties are shown in Table 2.

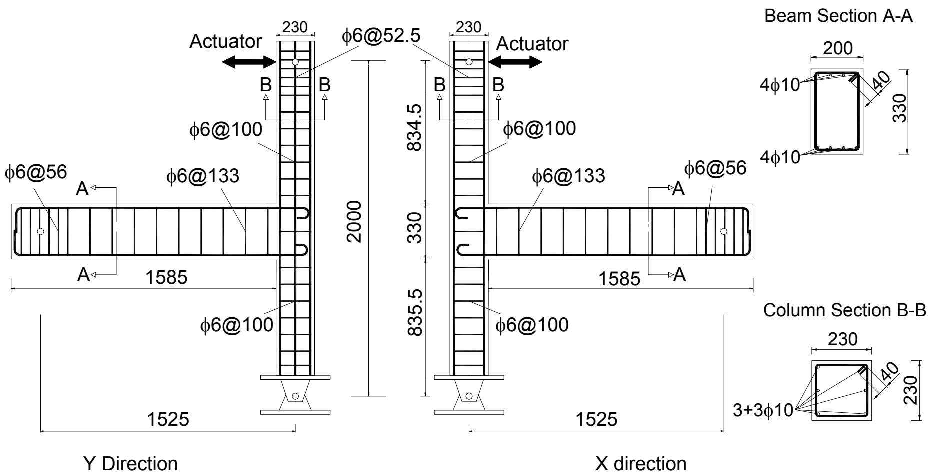


Fig. 2. Geometry and reinforcement details of the 3D exterior (corner) joint specimens 3D-B (pre-1970 as built, benchmark) and 3D-GF1 (GFRP retrofitted)

Table 1 Section and reinforcement details for the as-built and retrofitted specimens

Specimens		Section Details	Longitudinal Reinforcement	Transverse Reinforcement
3D-B (As-built)	Beam	330 x 200 mm	Top 4φ10 + 4φ10; Bottom	φ6@133
3D-GF1 (retrofitted)	Column	230 x 230 mm	3φ10 + 3φ10	φ6@100

Table 2 Material properties

Specimen	Concrete strength (f'c) (MPa)		Reinforcing bars strength (fy) (MPa)	
	28 days	Test day	Plain Φ10	Plain Φ6
3DB	24.2	24.8	388	344
3DGF1	24.5	30.1	385	386

2.3 Test set-up and loading protocol

Fig. 3 and 4 illustrate the test set-up and loading protocol adopted. Beam and column elements were extended between points of contra-flexure (assumed to be at mid-span in the beams and at mid-height in the columns) where pin connections were introduced. Simple supports at the beam ends were obtained connecting pin-end steel members to the strong floor. In general, the testing loading protocol consisted of increased level of lateral top displacements in each direction (series of two main cycles plus a smaller elastic one). More specifically, for the 3D configuration, in order to better simulate the actual displacement-imposed response under a real ground motion, a four clove loading protocol (Fig. 4) was adopted. It is worth noting that the two-cycles protocol used for the Y-direction component correspond to the protocol adopted for the 2D configurations. In order to provide a more realistic representation of what would occur on a beam-column subassembly during the sway of the prototype frame building under uni- or bi-directional excitations, the axial load was in general varied as a function of the lateral force. The relationship between the lateral force F and the variation of axial load N ($N = N_{gravity} \pm \alpha F$) is function of the geometry of the building (i.e. number of bays and storeys) and can be derived by simple hand calculations or pushover analyses on the prototype frame. The importance of a proper estimation of the variation of the axial load, particularly when dealing with the assessment and retrofit strategies of poorly detailed R.C. frames, has been highlighted in [21-22]. Further considerations related to the 3D response effects will be given in the following paragraphs, when discussing the evaluation of hierarchy of strength and sequence of events.

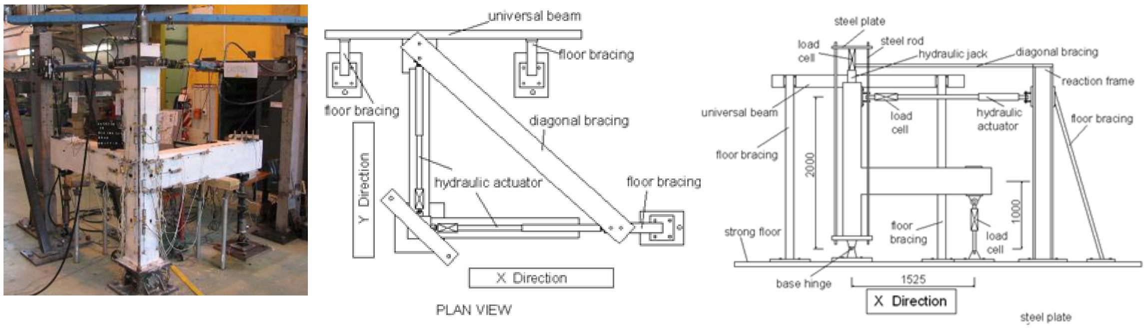


Fig. 3 Test set-up for quasi-static cyclic testing under uni- or bi-directional loading regime

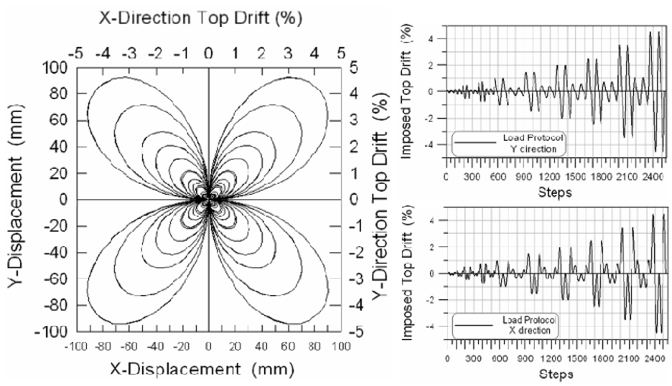


Fig. 4 Imposed four-cloves top-displacement loading protocol for bi-directional testing

2.4 Experimental results on as-built configurations

The experimental results on the as-built specimens confirmed the higher vulnerability of exterior 3D corner joints, subjected to a more realistic bi-directional loading protocol, when compared to their 2D counterpart subjected to a traditional uni-directional loading protocol.
Figures 1e, 5 and 6 show the observed damage and a comparison of the subassemblies hysteresis loops. In both 2D and 3D specimens a shear hinge mechanism developed in the joint region, providing the main source of the observed inelastic deformation and behaviour.

The 2D specimen exhibited the formation of a concrete wedge mechanism (Fig. 1e), due to the combined effects of shear cracking and extensive damage in the joint (lack of sufficient transverse reinforcement acting as ties to counteract the diagonal compression strut) and the stress concentration at the edge of the beam end-hook. On the exterior face of the column, the concrete wedge mechanism combined with the cover concrete spalling due to the compression action on the column outer face reinforcement. It is worth noting that the presence of one single stirrup in the joint allowed for a less dramatic strength degradation and loss of load-bearing capacity of the overall system, when compared to what expected in an equivalent configuration with no stirrup in the joint.

The 3D specimen, subjected to the aforementioned four clove loading regime, exhibited a more complex three-dimensional concrete wedge mechanism (Fig. 5), well in line with the damage observed in recent earthquake events (e.g. Izmit-Kocaeli, Turkey, 1999). A critical level of joint damage and a more rapid strength degradation were observed when compared to the 2D equivalent counterpart, in spite of the partial confinement effect provided by the orthogonal beam. Again, the presence of one single stirrup in the joint allowed to limit the extent of expulsion of the concrete wedge at earlier stage when compared to the equivalent solution with no transverse reinforcement in the joint core.

It is worth noting that, in addition to the damage in the joint panel zone, extensive flexural cracking and concrete crushing was observed in the inner side of the joint at the beam-column vertical interface (Fig. 5a), due to the joint asymmetrical behaviour under opening or closing actions as well as to the different shear strength of the panel zone (partially confined on one face only). As shown in Figure 6, the 3D configuration and bi-directional loading resulted into a reduction of the overall lateral load capacity (when compared to the 2-D specimen) of approximately 33% and 15%, in the positive and negative direction, respectively (corresponding to decreasing and increasing of the axial load).

Fig. 5 Damage observation in the as-built specimen 3DB
a) inner corner at 2.5% drift; b) end of testing after 4.5% drift

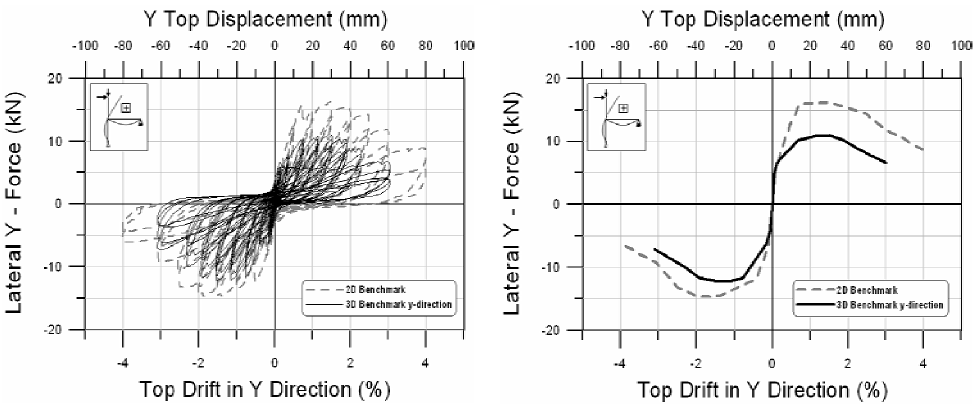
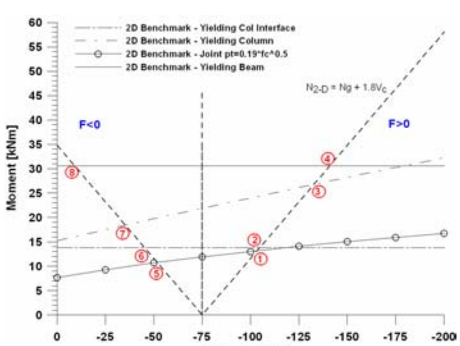


Fig. 6 Experimental hysteresis rules and envelope curves for the 2D and 3D exterior joint subassemblies in their as-built configurations (benchmark specimens 2DB and 3DB)

3 RETROFIT STRATEGY AND SOLUTION

3.1 Evaluation of hierarchy of strength and sequence of events

A simple procedure to compare the internal hierarchy of strengths within a beam–column-joint system has been proposed in [21-22]. The evaluation of the expected sequence of events can be carried out through comparison of capacity and demand curves within a M-N (moment-axial load) performance-domain. Figure 7 shows, as an example, the M-N performance domain adopted to predict the sequence of events and the level of damage in the joint panel zone expected for the 2D exterior specimen. The capacities of the beam, column and joint are referred to given limit states (e.g. for the joints: cracking, equivalent “yielding” or extensive damage and collapse are associated to increased levels of principle tensile or compression stresses) and evaluated in terms of equivalent moment in the column at that stage, based on equilibrium considerations within the b-c joint specimen.



Specimen 2DB – As-built 2D exterior joint - Benchmark				
	No	Event	Lateral Force [kN]	
			Analytical	Exp.
Positive loading	1	Joint First Diagonal Crack	11.08	11.37
	2	Column Interface Yielding	11.42	
	3	Column Yielding	22.36	
	4	Beam Yielding	25.50	
Negative loading	1	Joint First Diagonal Crack	9.06	8.75
	2	Column Interface Yielding	11.42	
	3	Column Yielding	15.40	
	4	Beam Yielding	25.50	

Fig. 7 Evaluation of hierarchy of strengths and sequence of events for the 2D benchmark specimen: M-N Performance Domain

3.2 Hierarchy of strength and sequence of event: variation of axial load effects

Given the aforementioned considerations, it is evident that major consequences can derive by confusing the "hierarchy of strength", better represented by capacity curves and independent of the demand [22], with the actual "sequence of events" (evaluated only when and if the correct demand is considered). As a general example, a set of beam-column joint subassemblies with same geometric and mechanical properties but located at different floor levels within different frame geometry configurations, would have same capacity curves (inherent hierarchy of strength) while resulting into completely different sequence of events, due to the different internal forces (axial load-lateral force) demand. More specifically, demand curves should account for the actual variation of axial load due to the effects of lateral loads on a frame system, which can be not-negligible when referring to exterior joints, as shown in Figure 7. In particular, as clearly shown by the figure, the reduction of axial load from the constant value due to gravity load (i.e. 75 kN in this case) due to the frame effects under lateral loading could lead to a premature (either in terms of lateral force and of interstorey drift) damage in the joint or column when not to a more severe modification of the internal hierarchy of strength. Incorrect and non-conservative assessment of the sequence of events can otherwise result, leading to inadequate design of the retrofit intervention. As a controversial result, it would be possible to activate, after a retrofit intervention, a global failure mechanism (i.e. due to the formation of a soft storey) which would have not occurred in the as-built configuration.

The lack of consideration of such a variation of axial load due to the seismic effects has to be considered a major limit and drawback of current assessment procedures for beam-column joint. Moreover, most of the experimental quasi-static tests on subassemblies (including column-to-foundation connections) available in literature have been carried out, and still are for simplicity, under a constant axial load in the column. Whilst this simplified testing procedure based on constant axial load is not expected to substantially affect the behaviour of well designed specimens, in the case of poorly detailed subassemblies, the effects on damage level and mechanisms could be significant.

3.3 Additional effects due to the 3D response under bi-directional loading

The aforementioned effects on the hierarchy of strength and sequence of event in a beam-column joint subassemblies are further critically emphasized when dealing with the 3D response of bi-directionally loaded subassemblies. When underestimating or overlooking such effects, significant modification to the response could derive, with the likely undesirable effects of impairing the efficiency of an expensive retrofit intervention. As a controversial result, it would be possible to activate, via the retrofit intervention, a global failure mechanism (i.e. due to the formation of a soft storey) which would have not occurred in the as-built configuration.

As demonstrated by the experimental evidences, three additional aspects should be considered:

1) the variation of axial load due to the 3D response of the building: in the case of a corner joints, the effects on the variation on the axial load in the two orthogonal direction (say x- and y-) will in fact have to be combined and algebraically summed. Thus, a more significant reduction of the axial load (typically leading to premature failure of the joint and column when compared to the beam, not affected to axial load) can be expected in corner joints;

2) the reduction of flexural strength in the column due to bi-directional loading. This effect, qualitatively recognized also for newly designed structural column and typically represented by means of M_y-M_z-P interaction surfaces (Fig. 8), is however typically neglected in the analyses. Higher strength reduction effects can be expected when dealing with poorly detailed columns, representative of older construction practice. Futhermore, the higher potential of buckling of the vertical longitudinal bars due to the higher spacing and limited amount of the transverse reinforcement should be considered;

3) the reduction of joint shear strength due to bi-directional loading: very few information are available in literature in particular when referring to under-designed beam column joints. Based on the aforementioned concept of shear hinge mechanism preliminary investigations for the evaluation of an equivalent M_{jy}-M_{jz}-P interaction diagram (again based on principle tensile stresses considerations) for under-designed joints have been carried out in [25].

$$\left(\frac{M_y}{MY_y}\right)^{\alpha} + \left(\frac{M_z}{MY_z}\right)^{\alpha} = 1.0$$

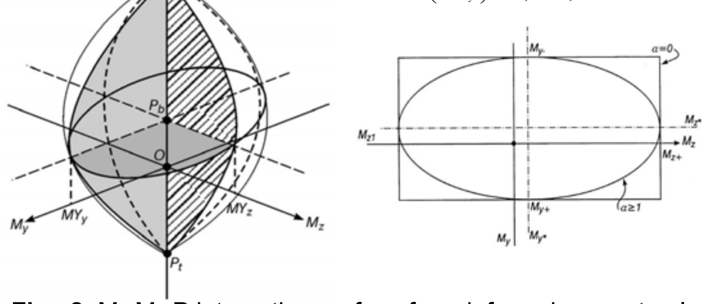


Fig. 8 M_y-M_z-P interaction surface for reinforced concrete elements subjected to bi-axially loading

According to the proposed assessment procedure based on M-N performance domain, it is possible to properly visualize such 3D effects on the expected sequence of events, prior and after the retrofit intervention. Figure 9 shows the M-N performance domain referred to the 3D specimens in the as-built configuration (left side) or considering alternative options for the retrofit intervention (right side). The enhancement of the flexural strength of the beam and column critical sections and of the joint core shear strength (in terms of principle tensile strength) has been evaluated according to the aforementioned procedure presented in [21-22]. Figure 10 shows the strength enhancement on beam and joint panel zone for the retrofit solution 3DGF1 described in the following paragraph. The effects of neglecting some of the aforementioned effects on the sequence of events can be clearly derived. As part of the on-going experimental investigation, alternative retrofit solutions are tested either considering or neglecting some of the 3D effects in the design phase in order to demonstrate the possible effects on the overall behaviour of the retrofitted specimens as well as to validate the suggested solution able to guarantee a full inversion of the hierarchy of strength.

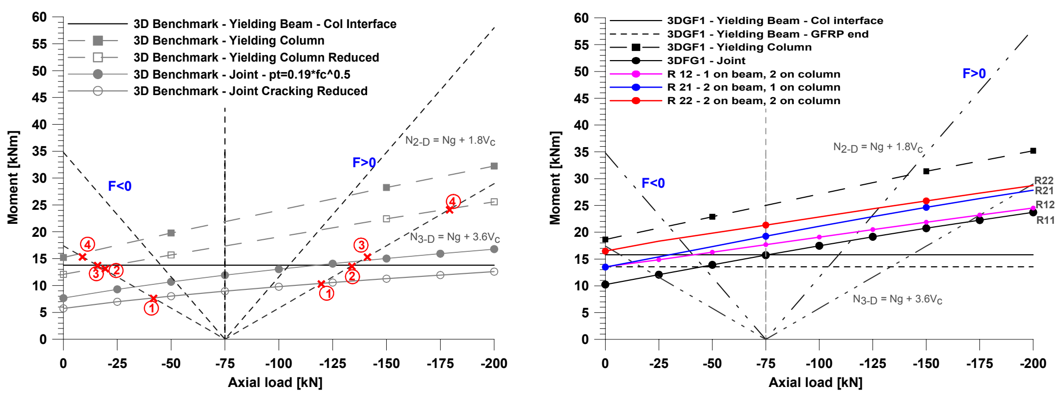


Fig. 9 Evaluation of hierarchy of strength and sequence of events within M-N performance domain for the as-built specimen 3DB and for alternative retrofit configurations

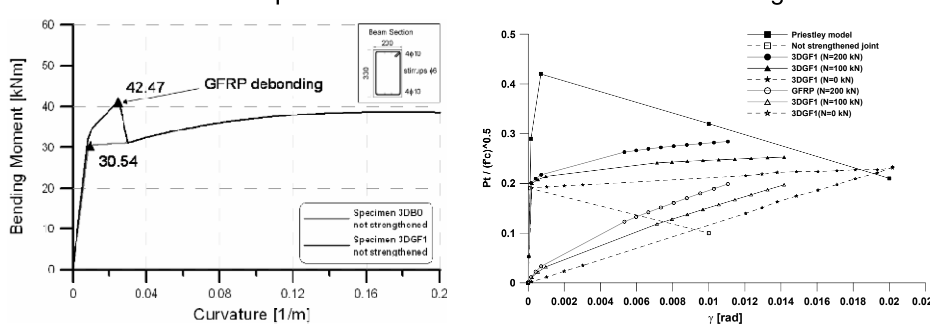


Fig. 10 Evaluation of the effects o the GFRP retrofit on the specimen 3DGF1:
beam moment-curvature at the column interface (left); joint strength degradation curve (right)

3.4 Design of the retrofit solution

The first retrofit solution, referred to as 3DGF1, has been designed based on the M-N performance domain of Figure 9 and using what would be considered a "minimum" retrofit solution (no variation of axial load was accounted for) to avoid extensive damage in the joint and protect from the collapse.

Uni-directional glass fiber laminates (Table 3) were adopted as shown in Figure 11. One vertical FRP layer was used on each of the two external faces of the column (supposed to be accessible in a real building without disruption of the internal activities) in order to increase the column flexural capacity as well as the joint shear strength. In addition, one U-shape horizontal laminate, wrapped around the exterior face of the specimen at the joint level, was used to increase the joint shear strength as well as to prevent the expulsion of the concrete wedge observed in both the 2D and 3D as-built configuration. Although the evaluation of the beam and column strengthening effects (Fig. 11) was carried out including debonding effects (when non-conservative), additional smaller strips were used to provide better anchorage to the main FRP laminates in the beam and column.

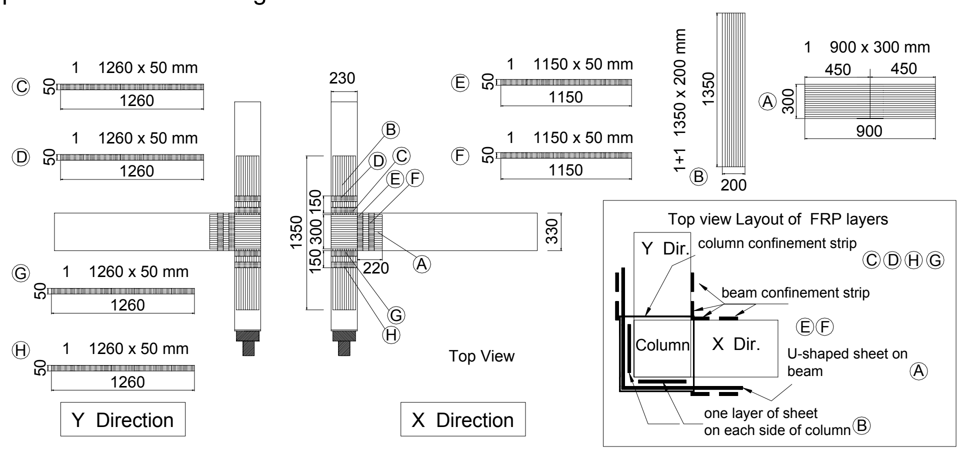


Fig. 11 FRP retrofit configuration 3DGf1 for the 3D corner joint specimen

Table 3. Properties of the un-idirectional glass fibres adopted (GFRP SikaWrap -100G)

Fibre Type	Areal Weight (g/m^2)	Fiber Density (g/cm^2)	Fabric Design Thickness (mm)	Characteristic Tensile Strength (N/mm^2)	Characteristic Modulus of Elasticity (N/mm^2)	Ultimate Strain (%)
High strength E-glass fibres	935 ± 47	2.56	0.36	2,300	76,000	2.8

3.5 Experimental behaviour of the retrofitted specimen 3DGF1

As part of the investigation on the effects of the variation of axial load on the efficiency of the retrofitted solution, this first test was carried out following the design assumptions (i.e. no variation of axial load), thus maintain a constant level of axial load to simulate the gravity component only. In the following undergoing phases of the project, the same retrofitted solution will be tested under the expected upper bound case scenario corresponding to a variation of axial load in each orthogonal direction. The performance of the retrofitted specimen 3DGF1 was in general satisfactory and in line with the expected targeted "minimum" performance level. The joint panel zone was properly protected by excessive damage (only hairline diagonal shear cracking observed after peeling the FRP from the joint core at the end of the test) with most of the inelastic behaviour concentrating in the beam element partially at the end of the GFRP laminate and partly, as observed in the as-built solution, at the inner face of the joint at the column interface.

As a result of the improved internal hierarchy of strength, a significant increase in the overall lateral force capacity (more than double) of the retrofitted specimen when compared to the as-built one was achieved (Fig. 13). A stable hysteresis behaviour was observed until 2% of drift. At the subsequent following cycling to 2.5% strength degradation occurred due to the increased separation crack at the inner side of the joint as well as to the debonding of the FRP laminate at the outer face of the beam. The test was interrupted after cycling at more than 4.5% of drift.

Fig. 12. Damage observation in the 3DGF1 retrofitted specimen at 2.5% of drift (left and centre) and at the end of the test (right)

Overall, the targeted performance, consisting of preventing the joint from excessive damage while relying on the flexural inelastic deformation capability of the beam, was achieved. Had the variation of the axial load been applied during the tests, lower performance would have been expected particularly in the direction corresponding to the reduction of axial load. Further tests will be carried out to validate this theory. Furthermore, it is expected that a marked improvement in the overall behaviour could be achieved by better controlling the debonding of the GFRP in the beam as well as further protecting the beam-to-column inner section from the peculiar damage observed in both the as-built and the retrofitted configuration. The next retrofit configurations under preparation, will target a more clear relocation of the plastic hinge region in the beam section at the end of the GFRP which should also allow for a longer development length of the beam plain round bars and thus increase the overall hysteretic energy dissipation capacity.

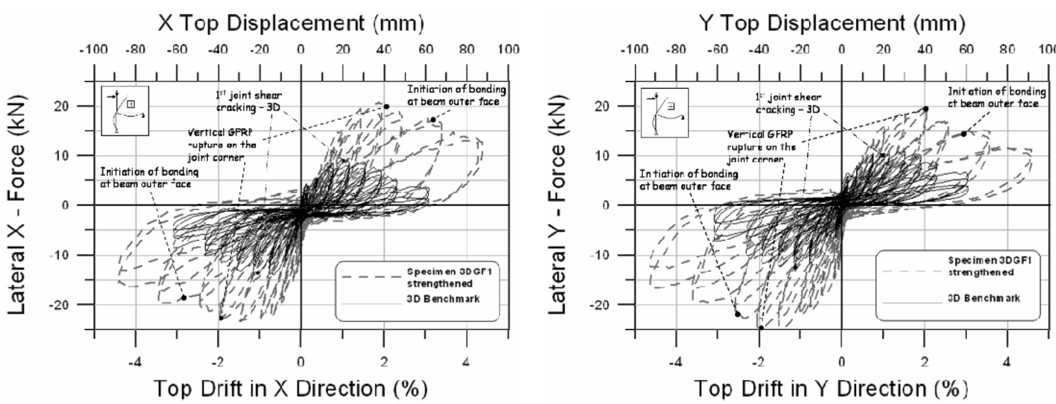


Fig. 13 As-built vs. GFRP1 retrofit solution : comparison of global hysteresis response

4 CONCLUSIONS

The effects of bi-directional loading on the implementation of retrofit intervention on pre-1970s designed exterior (corner) beam-column joints using FRP composite materials has been discussed.
The limits and drawbacks of standard plane frame behaviour assumptions when evaluating the hierarchy of strength and expected sequence of events prior to define the retrofit intervention have been discussed. A simplified procedure based on M-N (moment –axial load) performance domain, previously proposed by the authors, has been herein extended to better account for the 3D response of beam-column joint subassemblies. The results of quasi-static tests on exterior 3D (corner) poorly detailed beam-column joints subjected to a four clove displacement loading protocol before and after a retrofit intervention with GFRP, have been presented. Very promising confirmations on the efficiency of the adopted retrofit strategy and solution were provided.

ACKNOWLEDGEMENTS

The financial support provided by the NZ Foundation for Research, Science and Technology (FRST) through the Research Program "Retrofit Solutions for NZ" is gratefully acknowledged.
The material support and technical assistance provided by Sika-NZ and BBR-Contech NZ (Construction Technique Ltd) is also kindly appreciated.

REFERENCES

[1] *fib* International federation of concrete, "Externally Bonded FRP Reinforcement for RC structures.. *fib Bulletin n.14*, Lausanne, 2001

[2] *fib* International federation of concrete, "Retrofitting of concrete structures by externally bonded FRPs with emphasis on seismic applications" *fib Bulletin n.35*, Lausanne, 2006

[3] AIJ, Architectural Institute of Japan, "Design Guidelines for Earthquake-Resistant R.C. Buildings Based on Inelastic Displacement Concept (draft, in Japanese)" 1997

[4] ACI Committee 440, "Guide for the Design and Construction of Externally bonded FRP systems for strengthening concrete Structures", 2001

[5] Aycardi, L., E., Mander, J., B. & Reinhorn, A.M., "Seismic Resistance of R.C. Frame Structures Designed Only for Gravity Loads: Experimental Performance of Subassemblages", *ACI Structural Journal*, 91, 5, 1994, pp. 552-563

[6] Beres, A., Pessiki, S., White, R., Gergely, P., "Implications of Experimental on the Seismic Behaviour of GLD RC Beam-Column Connections", *Earthq. Spectra*, 12, 2, 1996, pp. 185-198.

[7] Hakuto, S., Park, R. and Tanaka, H., "Seismic Load Tests on Interior and Exterior Beam-Column Joints with Substandard Reinforcing Details", *ACI Struct. J.,* 97, 1, 2000, pp.11-25.

[8] Park, R. 2002. "A Summary of Results of Simulated Seismic Load Tests on R.C. B-C Joints, Beams and Columns with Substandard Reinforcing Details", *JEE*, 6, 2, 2002, pp. 1-27.

[9] Pampanin, S., Calvi, G.M. & Moratti, M., "Seismic Behaviour of R.C. Beam-Column Joints Designed for Gravity Loads" *12th ECEE*, London, paper n. 726, 2002

[10] Calvi G.M., Magenes G. & Pampanin S., "Relevance of Beam-Column Damage and Collapse in RC Frame Assessment", *JEE*, Special Issue 1, 2002, pp.75-100

[11] Crisafulli, F.-, Carr, A. and Park, R. "Analytical Modelling of Infilled Frame Structures – a General Review", *Bulletin of the NZSEE*, 33,1, 1997, pp. 30-47

[12] Magenes,.G. Pampanin, S., "Seismic Response of Gravity load Designed Frame Systems with Masonry Infills", Proceedings *12th WCEE*, Vancouver, August, 2004, Paper n.4004

[13] Dolsek, M and Fajfar, P. "Soft Storey Effects in Uniformly Infilled Reinforced Concrete Frames", *Journal of Earthquake Engineering*, 5, 1, 2001, pp.1-12.

[14] *fib* International federation of concrete "Seismic assessment and retrofit of reinforced concrete buildings" *Bulletin n. 24*, Lausanne, Switzerland, 2003

[15] Antonopoulos, C.P.& Triantafillou, T.C., "Analysis of FRP-strengthened RC beam-column joints", *Journal of Composites for Construction*, 6, 1, 2002, pp. 41-51.

[16] Antonopoulos, C.P.& Triantafillou, T.C. "Experimental Investigation on FRP-Strengthened RC Beam-Column Joints", *J. of Composites for Construction*, 7, 1, 2003, pp. 39-43.

[17] Gergely, J., Pantelides, C. P. & Reaveley, L.D., "Shear Strengthening of RC T-Joints Using CFRP composites", *Journal of Composite Constructions*, ASCE, 4(2), 56-64., 2000

[18] El-Amoury, T., Ghobarah, A., "Seismic Rehabilitation of beam-column joitns using GFRP sheets", *Engineering Structures*, 24, , 2002, pp.1397-1407

[19] Prota, A., Nanni, A., Manfredi, G., and Cosenza, E. "Selective upgrade of under-designed r.c. concrete beam-column joints using CFRP" *ACI Struct. J.*, 101, 5, 2004, pp. 699-707.

[20] Clyde, C. & Pantelides, C. P. "Seismic evaluation and rehabilitation of R/C exterior building joints" *7NCEE*, EERI, Berkeley, Calif, 2002

[21] Pampanin, S., Bolognini, D., Pavese, A., Magenes,. G., Calvi, "Multi-level seismic rehabilitation of existing frame systems and subassemblies using FRP composites", *CICE2004*, Adelaide.

[22] Pampanin, S., Bolognini, D., Pavese, "Performance-based Seismic Retrofit Strategy for Existing R/C Frame Systems using FRP composites", *ASCE*, Special Issue, in press, 2006

[23] Pampanin, S., Magenes, G. and Carr, A., "Modelling of Shear Hinge Mechanism in poorly Detailed RC Beam-Column Joints", *fib Symposium*, Athens, 2003, paper n. 171

[24] Hertanto, "Seismic Assessment of pre-1970s Reinforced Concrete Beam-Column Joint Subassemblies", *M.E. Dissertation*, Dept. of Civil Eng. Univ. of Canterbury, Chch, NZ, 2006

[25] Trowland, M., 2004, "Modeling the Shear Hinge in Beam-Column Joints", *3rd Professional year Report*, Univ. of Canterbury Department of Civil Engineering, Christchurch New Zealand