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Seismic strengthening of a non-ductile RC frame structure using GFRP sheets

U. Akgüzel, P. Quintana Gallo & S. Pampanin

University of Canterbury, Christchurch, New Zealand.

ABSTRACT: An as-built reinforced concrete (RC) frame building designed and constructed according to pre-1970s code design construction practice has been recently tested on the shake table at the University of Canterbury. The specimen, 1/2.5 scaled version of the original prototype, consists of two 3-storey 2-bay asymmetric frames in parallel, one interior and one exterior, jointed together by transverse beams and floor slabs. Following the benchmark test, a retrofit intervention has been proposed to rehabilitate the tested specimen. In this paper, detailed information on the assessment and design of the seismic retrofit procedure using GFRP (glass fibre reinforced polymer) materials is given for the whole frame. Hierarchy of strength and sequence of events (damage mechanisms) in the panel zone region are evaluated using a moment-axial load (M-N) interaction performance domain, according to a performance-based retrofit philosophy. Specific limit states or design objectives are targeted with attention given to both strength and deformation limits. In addition, an innovative retrofit solution using FRP anchor dowels for the corner beam-column joints with slabs is proposed. Finally, in order to provide a practical tool for engineering practice, the retrofit procedure is provided in a step-by step flowchart fashion.

1 INTRODUCTION

As part of the FRST (Foundation of Research Science and Technology)-funded Project "Retrofit Solutions for New Zealand Multi-storey Buildings", extensive research has been done on beam-column joint subassemblies in order to develop and improve feasible seismic retrofit solutions for RC non-ductile frame structures. The several tests carried out on b-c joint subassemblies under uni- and bi-directional quasi-static loading, using constant and varying axial load in the column, and with and with out floor slabs, have confirmed the likelihood of developing brittle failure modes in the panel zone region, as observed in the past (Aycardi et al., 1994; Beres, 1996; Hakuto et al., 2000; Park, 2002; Pampanin et al., 2002). Further tests have also allowed to develop and validate recently proposed and improved retrofit techniques for frame systems (Pampanin et al., 2006; Marriott et al., 2007; Kam and Pampanin, 2008; Akgüzel and Pampanin, 2008). In particular, GFRP sheets have been adopted for the seismic retrofit of 2D and 3D, plane and corner beam column joints, respectively. validating on a subassembly level the proposed performance-based design procedures (Akgüzel and Pampanin, 2010). On going work is focusing on the extension of the GFRP retrofit solution to beam column joints with slabs.

In order to validate the results on the seismic vulnerability of such structures and to confirm the feasibility of non-invasive retrofit techniques, a non-ductile RC frame building has been designed and tested on the shake table of the Structures Laboratory of the University of Canterbury (Quintana Gallo et al., 2011a, b). Following the experimental tests in its as-built configuration (presented in companion paper, Quintana Gallo et al. (2011a), the test building, which comprise two parallel frames and include floor slabs and transverse beams, will be rehabilitated using GFRP laminates with the design presented in this contribution. For that, hierarchy of strengths and sequence of events for the more vulnerable exterior beam column joints are evaluated in an M-N performance domain (Pampanin et al., 2007). Using recently developed seismic strengthening procedures for the seismic upgrading of corner beam column joints using GFRP materials (Akgüzel and Pampanin, 2010), a retrofit scheme is presented.

The GFRP design scheme is expected to protect the panel zone region from extensive damage, as expected and observed in the as-built test specimen, and relocate the inelastic behaviour into the beams, by means of reversing the hierarchy of strengths. A performance-based assessment of exterior beam column joints considering the effect of the floor slab and transverse beam in an asymmetric M-N performance diagram is presented as well as the expected performance (predicted prior to the test) of the retrofitted specimen (Quintana Gallo et al., 2011b). Anchorage, as well as bond of laminates with the concrete surface is proposed to be improved by means of GFRP dowels. Finally, a design flowchart for the implementation of the retrofit scheme is proposed.

1.1 Specimen description

The specimen consists in a 3D model composed by two 2-bay 3-storey RC frames in parallel, one exterior and one interior, jointed together by transverse beams and floor slabs (Quintana Gallo et al., 2011a). It was conceived as a 1/2.5 scale extended version of a plane frame designed by Marriott et al., (2007) which was used as the conceptual basis for the whole research campaign. Corner and exterior-interior beam column joints are basically a 3/5 version of the 2/3 scale beam column joints with floor slabs tested in 2009 at the University of Canterbury (Kam et al., 2010). Therefore, the mechanical behaviour of the beam-column joint in the model is analogous to that simulated in beam column joint subassemblies in terms of failure mode of the as-built specimen and implementation issues and features of the retrofit technique, with the obvious limitations of reduction in size. In Figure 1 the geometry of the experimental model, together with details of the two types of exterior beam column joints represented in the test-model are presented. These types of joints are, respectively, corner 3D joints, belonging to the exterior frame, and exterior – interior joint, belonging to the interior frame. Since longitudinal spans are different, it will be shown later that a different layout of the GFRP sheets must be used in the beam in order to relocate the plastic hinge into the inelastic behaviour to the beams. Material properties are shown in Table 1. For details refer to Quintana Gallo et al., 2010 and 2011b.

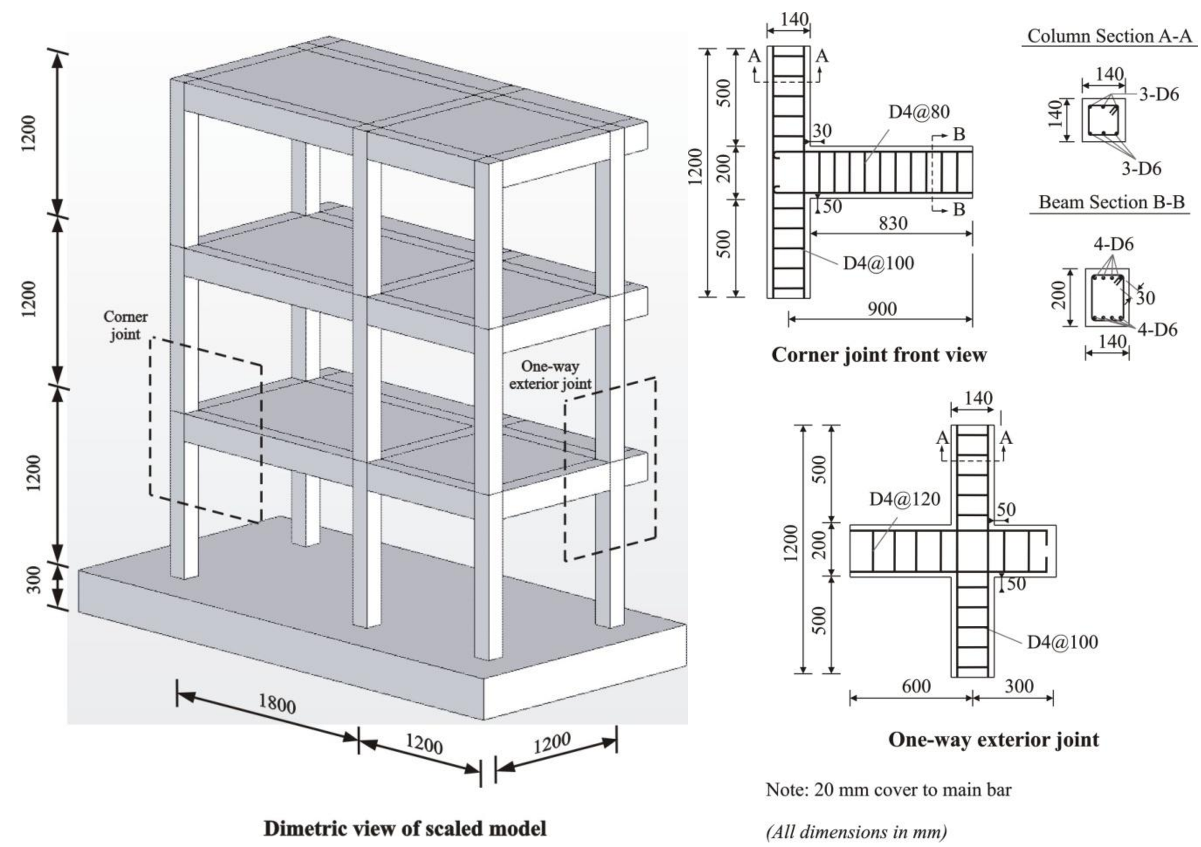

Figure 1. Model dimensions and reinforcement details

Table 1 Material Properties

Location	Concrete: At 28 days f_c (MPa)	Mortar: At 28 days f_c (MPa)
1st floor	27	30
2nd floor	23	30
3th floor	8	30

Reinforcement	Longitudinal Bars (including mesh steel)	Stirrups
Bar Type	10 mm plain round	6 mm plain round
Yield Strength, f_y (MPa)	385	585
Ultimate Stress, f_u (MPa)	500	600

Fibre Type	Density [g/cm^3]	Effective thickness of one layer, t_f [mm]	Ultimate Tensile Strength f_{fu}* [MPa]	Modulus of Elasticity E_f [MPa]	Design Rupture Strain (%) $\varepsilon_{fu}=C_E{}^{1}\varepsilon_{fu}^{*}$
GFRP SikaWrap - 100G High Strength E-Glass	2.56	0.36	2300	76,000	(0.65)(2.8%)=1.8%

[1] The joint specimens in this study are located in an exterior space. Therefore, environmental reduction factor of 0.65 is used for GFRP according to FRP design code of ACI-440.2R-08.

2 PERFORMANCE-BASED RETROFIT STRATEGY

The retrofit strategy follows in general terms capacity design principles (Paulay and Priestley, 1992). Selected components of the beam-column subassembly are thus upgraded to achieve ductile behaviour through the development of plastic hinge mechanisms in the beam (weak-beam strong-column mechanism), while other regions are protected from inelastic brittle mechanisms. In order to achieve this, the retrofit strategy implemented in this study is performed under the umbrella of Performance-Based Seismic Retrofit criteria (Pampanin, 2009; Pampanin and Akguzel, 2011). 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). A summary of the assessment and retrofit design procedure is given in Figure 2 as a flowchart in a step-by-step fashion. In the following sections, these steps are explained briefly.

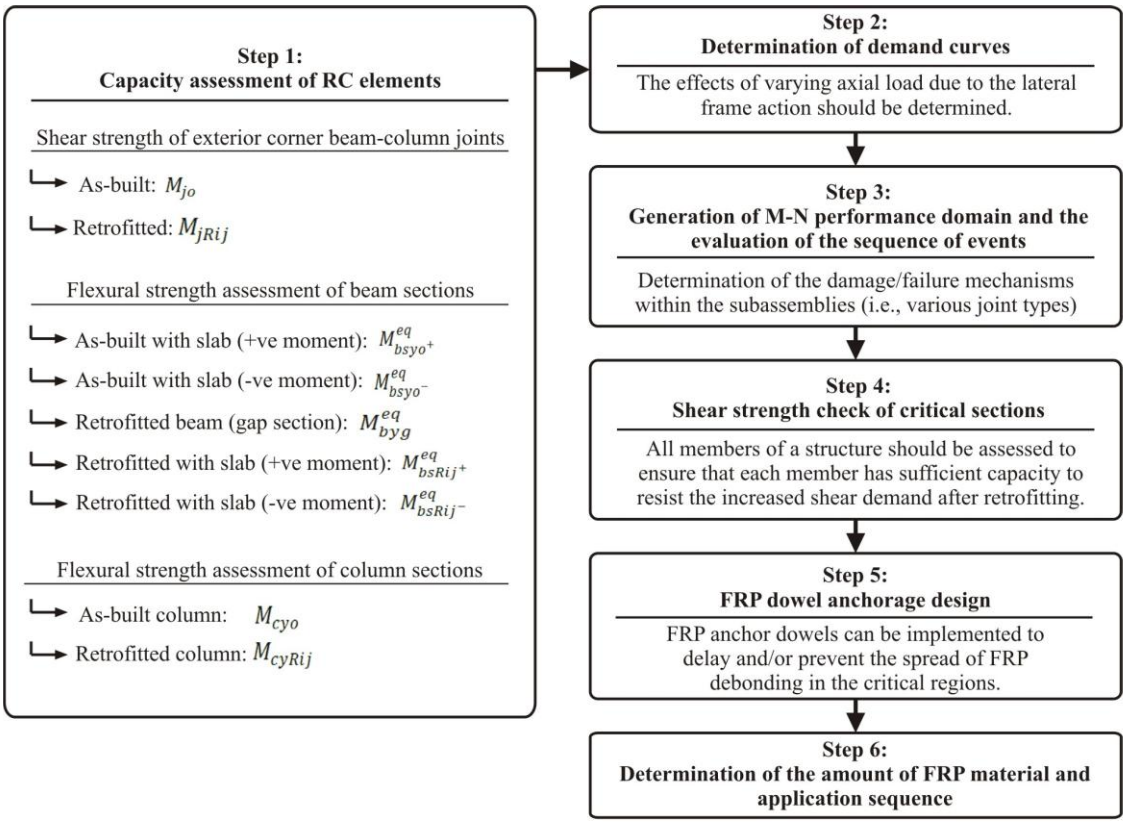

Figure 2. Flowchart for assessment and FRP retrofit of RC model structure

2.1 *Capacity Assessment of RC Elements: M-N Performance Domains and Sequence of Events*

As a critical step before any retrofit solution is designed, the assessment of the expected behaviour and performance (damage under a given intensity of loading) of an existing structural elements in RC frame in its as-built configuration needs to be properly carried out. The internal hierarchy of strength of the structural elements, combined with the likely demand, would provide critical information about the expected sequence of events (or damage-failure mechanism, such as beam or column flexural yielding/hinging, joint shear failure etc). In order to achieve this, the retrofit design methodology proposed by Pampanin et al. (2007) and further refined by Akguzel and Pampanin (2009) has been adopted. Accordingly, to achieve the target performance of the retrofit strategy, a simplified step-by-step design procedure following the detailed assessment of each joint component in as-built and retrofitted configuration is performed.

According to the aforementioned procedure, the capacities of the structural elements within the beam-column joint subassembly (e.g., beam, column and joint) can be evaluated by referring to specified limit states. The limit states can be defined referring to beam or column hinging, reinforcement yielding, concrete spalling and cracking, or extensive damage of the joint core as well as FRP debonding or failure. The corresponding limit state of the section or element in question is written in terms of the equivalent moment in the column. This can be easily achieved following equilibrium considerations within the beam-column joint system. At a later stage, in order to evaluate and control the governing mechanisms in the joint subassembly under different demand conditions of a retrofitted joint, the actual sequence of events should be determined by comparing demand and capacities. The nomenclature used to represent the equivalent column moments are given in Figure 3 for as-built and retrofitted corner beam-column joints.

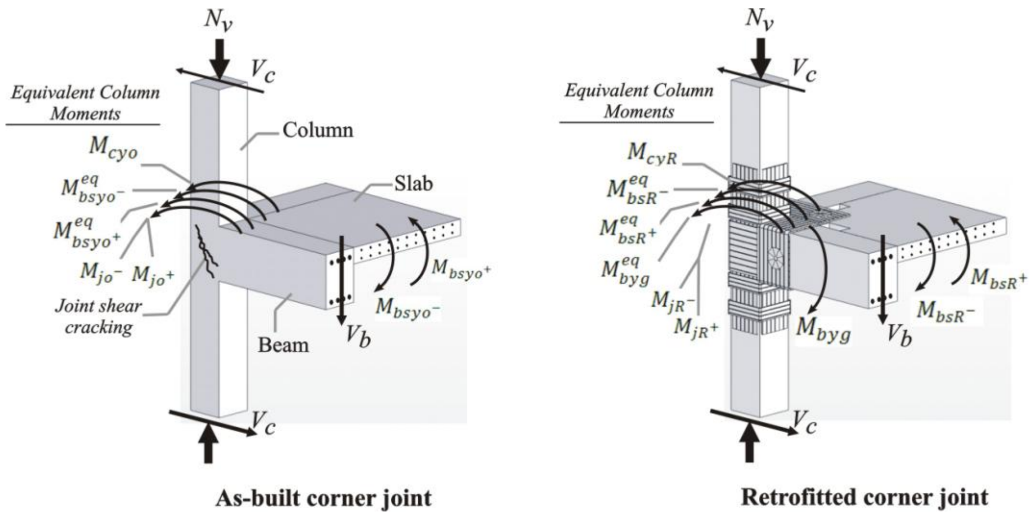

Figure 3. Schematic illustration of external actions and equivalent column moments for as-built and retrofitted corner joint

In the capacity assessment of as-built and retrofitted beam and column sections, traditional section capacity analysis (i.e., fibre-section analysis) are extended to include the effect of FRP flexural and confinement effects. The shear strength of the as-built exterior joints of the RC frame is evaluated according to the proposed procedure, formulated based on experimental and numerical evidences (Pampanin et al., 2002; Akguzel and Pampanin, 2010, Genesio et al., 2010). Principal tensile stresses vs. joint shear deformation rules were used to evaluate the behaviour of the strengthened joint panel zone, based on the experimental and postulated physical models on joint mechanics. According to the proposed model, the total shear strength of the retrofitted joint is regarded as the combination in parallel of the contributions from the as-built (unretrofitted) joint panel zone and that from the composite materials attached to the specimen. More information on the implemented semi-empirical approach for the analysis of FRP strengthened exterior joints can be found in literature (Akguzel and Pampanin, 2009).

In the evaluation of the M-N (Moment-Axial Load) diagram, the effect of asymmetry as a consequence of the presence of the slab is herein accounted for, typically neglected in previous work. In addition, in exterior joints which include a strong transverse beam, torsion is seen to be an important parameter to be considered when evaluating the capacity of the joint. This can be incorporated in different ways, for example adding a bigger area for the resisting joint (Kam et al 2011) or by means of estimating the torsional capacity of the transverse beam and adding that effect directly to the joint resisting capacity (Quintana Gallo et al. 2011b). In Figure 4, an asymmetric M-N diagram which included the action of the slab and transverse beam in introduced. The sequence of events for the As-built and retrofit configurations are indicated as AB1, AB2, AB3 and R1, R2, R3, respectively.

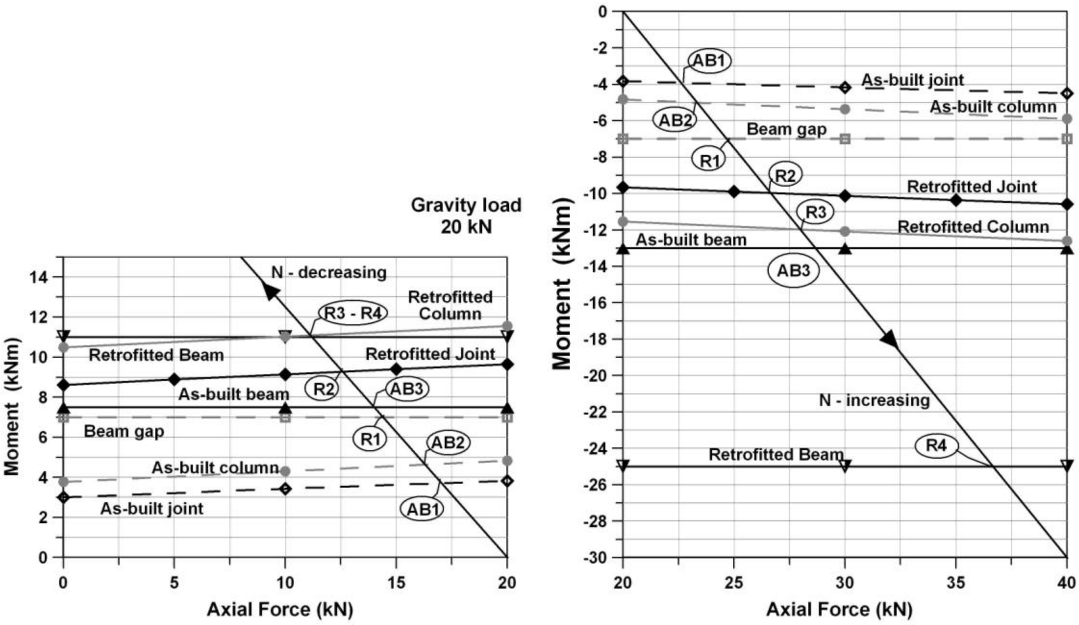

Figure 4. M-N performance domain hierarchy of strength reversal – failure sequence changed
ABi means event 'i' (i = 1, 2, 3) As-Built specimen; Rk means event 'k' (k=1, 2, 3, 4) Retrofitted specimen

Depending on the evaluation of the effective width of the slab acting in tension, or more appropriately on the amount of longitudinal reinforcement in the slab that activate and the resulting level of stress demand, results may vary significantly. In previous investigations (Kam et al 2010) it was suggested to use 1.1 times the beam height (h_b) as effective width of the flange in tension, from centerline to the corresponding side ($2.2h_b$ in total). When considering this effective width in the beam capacity, in this case, negative moment (slab top in tension) increases by means of 60% when compared to the corresponding plane beam. For positive moment (slab in compression) bending moment capacity increases only slightly. Note that axial load decreases from the initial gravity load when the slab acts in compression (positive moment – upwards beam shear action in the joint) and increases when the slab acts in traction (negative moment – downwards beam shear action in the joint).

Table 2. Sequence of events for the as-built and retrofitted corner joint units

Direction	Type	Critical Events Failure Types	Sequence No.	Axial Load (kN)	Equivalent Moment (kNm)
positive moment	as-built corner joint	Joint	AB1	17	3.8
		Column	AB2	16.3	4.7
		Beam	AB3	13.4	8.5
	retrofitted corner joint	Beam Gap	R1	14.5	7
		Joint	R2	12	10.1
		Beam	R3	11.4	11
		Column	R4	11.2	11.3
negative moment	as-built corner joint	Joint	AB1	22.6	-3.8
		Column	AB2	23.3	-5
		Beam	AB3	28.6	-12.8
	retrofitted corner joint	Beam Gap	R1	24.6	-6.9
		Joint	R2	26.7	-9.8
		Column	R3	28	-12
		Beam	R4	36.7	-25

In Figure 4 the sequence of events for positive and negative bending moments is evaluated. Results are summarized in Table 2. It can be seen that for the as-built specimen for positive moment the sequence would be: joint – column – beam, meanwhile for negative bending moments the sequence would be: column – joint – beam. The difference is attributed to the torsional resistance of the transverse beam, which helps supporting the panel zone region when the slab acts in tension (Ehsani and Wight 1985, Di Franco et al. 1995). When the slab acts in compression, on the other hand, this additional resisting mechanism is not activated and the joint is much weaker and is expected to crack before the column hinges. This behavior is in line with the damage observed in the corner beam column joints on the as-built frame test-model specimen (Quintana Gallo et al 2011a, b).

2.2 *Identification of demand curves*

The demand curves accounting for the variation of the axial load can also be plotted in the same domain. Hence, the feasibility of different retrofit techniques can be evaluated by the designer and modified by rearranging the sequence of events based on considerations on the local material limit states (e.g. concrete crushing, steel yielding and FRP debonding) first and then on the associated global mechanisms and expected seismic performance levels (e.g. damage control, collapse prevention, life safety etc.).

Axial load variation is directly related to the shear that the beam is able to transmit to the panel zone region. When the beam experiences positive moments (slab in compression) then uplifting shears are introduced into the joint, decreasing the axial load in the column, and vice versa. This is reflected in Figure 4, where in the positive moment diagram the axial load diminishes, whereas in the negative moment diagram the axial load increases. Estimation of demand curves, represented by a line in the case of static incremental loading, can be obtained from typical equilibrium considerations or by means of a pushover analysis. Also, experimentally, the variation of axial load in bottom columns was captured using vertical accelerometers, results which can be found in Quintana Gallo et al (2011b). In light of the uncertainty of the problem when addressed in the dynamic range, it is suggested that a range of curves is used for assessing the sequence of events instead of using a unique number. In this particular case, the results do not vary significantly in the whole range of loads from ±100% of the initial gravity load corresponding to 20 kN.

2.3 *Shear strength check*

In general, all members of a structure should be assessed to ensure that each member has sufficient capacity to resist the increased shear demand after retrofitting. Increasing the flexural strength may result in shear failures, which are brittle and undesirable failure modes (ACI-440.2R-08). Specifically in the joint subassembly, the flexurally strengthened beam and column elements shall be capable of withstanding the anticipated increase in loads associated with the capacity of the flexurally strengthened members. This is of crucial importance to guarantee the occurrence of anticipated ductile flexural failure before any brittle shear failure in these elements. In case of different demand conditions, further external FRP reinforcement by wrapping or partially wrapping the members needs to be applied accounting for additional shear strength requirements. Accordingly, shear strength assessment has been performed for the retrofitted and unretrofitted members following ACI-440.2R-08 and New Zealand concrete structures standard, NZS 3101 recommendations.

2.4 *FRP dowel anchorages*

It is well known that the full load capacity of the FRP system might not be fully exploited due to premature detachment of the FRP from the concrete surface of the strengthened member. In this case, the complete utilization of the FRP material is limited. Based on the experimental evidence it has been shown that, in case of exterior corner or one-way beam-column joints of a frame structures under high varying axial loads, the problem of debonding is aggravated (Akguzel and Pampanin, 2010a and 2010b).

In order to mitigate this problem, the FRP anchor dowels are used in the FRP intervention of corner beam-column joints to prevent debonding/delaminations, as well as to strengthen the laminates against buckling under compression loads. They are prepared by twisting the strips of CFRP sheets, folding into two and epoxied into pre-drilled holes. 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.

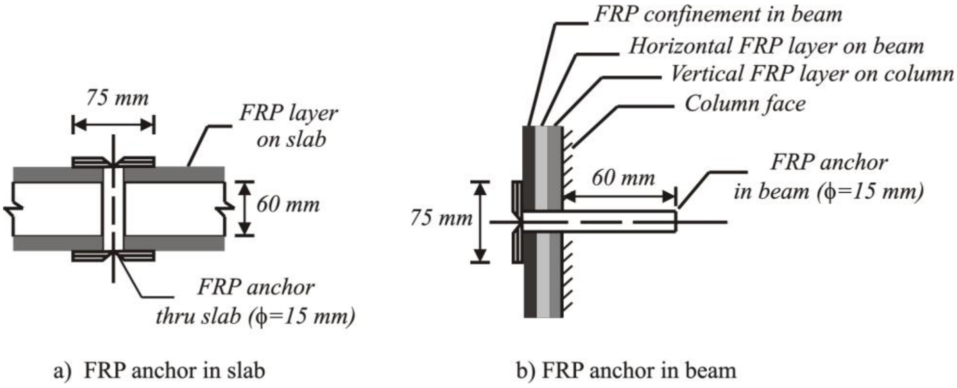

Figure 5. Details of FRP anchor dowels installed in slab and beam elements

2.5 *FRP application layout*

As mentioned previously, the retrofit strategy followed was aimed to protect the beam-column joint regions in RC frame, which are regarded as one of the most vulnerable and critical structural elements in pre-1970s as observed in the recent earthquakes as well as after the test of the as-built RC frame. The reader is refered to the companion paper for more information (Quintana Gallo et al., 2011a). For this purpose, a *partial retrofit* strategy was implemented which consisted of protecting mainly the corner joints and exterior one-way joints of the RC model. In this way, it was deemed to prevent the formation of a soft-storey mechanism providing the plastic hinge development in beams framing in exterior columns and sufficient deformation/rotation capacity is guaranteed within the critical elements ((Pampanin et al., 2007; Pampanin and Akguzel, 2011). In Figure 6., the dimetric view of the retrofitted RC frame model is given. It is important to note that, the intervention is intended to be carried out with minimum invasiveness from the outside of the building, thus allowing for limited disruption of the internal activities and/or relocation of people.

In Figures 7 and 8, the application sequence and dimensions of uni-directional glass fiber sheets (GFRP SikaWrap-100G, see Table 1) are presented in a step-by-step fashion for corner and one-way exterior joints. For all type of joints the first three steps consist of the installation of (1) vertical laminates on the exterior sides of the column faces (Step 1); (2) horizontal laminate on the joint region (Step 2) and (3) vertical laminates inside the column faces (Step 3). The main aim in these applications is to increase the flexural and shear strength of the columns and joint regions as well as to prevent the expulsion of a concrete wedge which was observed in the tests of the as-built frame. This results in a relocated beam hinge as shown in Fig. 4 with the intention of providing a ductile beam side-sway mechanism. 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 are wrapped with FRP sheets which are starting from the bottom face of the slab and extended to the top of the slab (Step 4 and Step 5).

The same application schemes are implemented for the one-way exterior joints (Figure 8) following the installation of horizontal FRP sheets on the beam under the slab (Steps 4, 5 and 6, Figure 8). A horizontal layer of FRP sheet is also attached on the top of the slab inside the column to increase the flexural strength of the slab (Step 7, Figure 8). In the application Step 6 of corner joints (Fig. 7) and Step 8 of one-way exterior joints (Fig. 8), anchorage strips are wrapped around the columns to increase the lateral confinement of the plastic hinge regions, to stabilize and restrain the longitudinal steel reinforcing bars and provide additional shear strength of the flexurally strengthened regions. Lastly, the FRP anchor dowels are installed in the beam and slabs to prevent debonding, as well as to strengthen the laminates against buckling under compression (e.g., Step 7- Fig.7 and Step 9 – Fig. 8, for corner and exterior one-way joint, respectively).

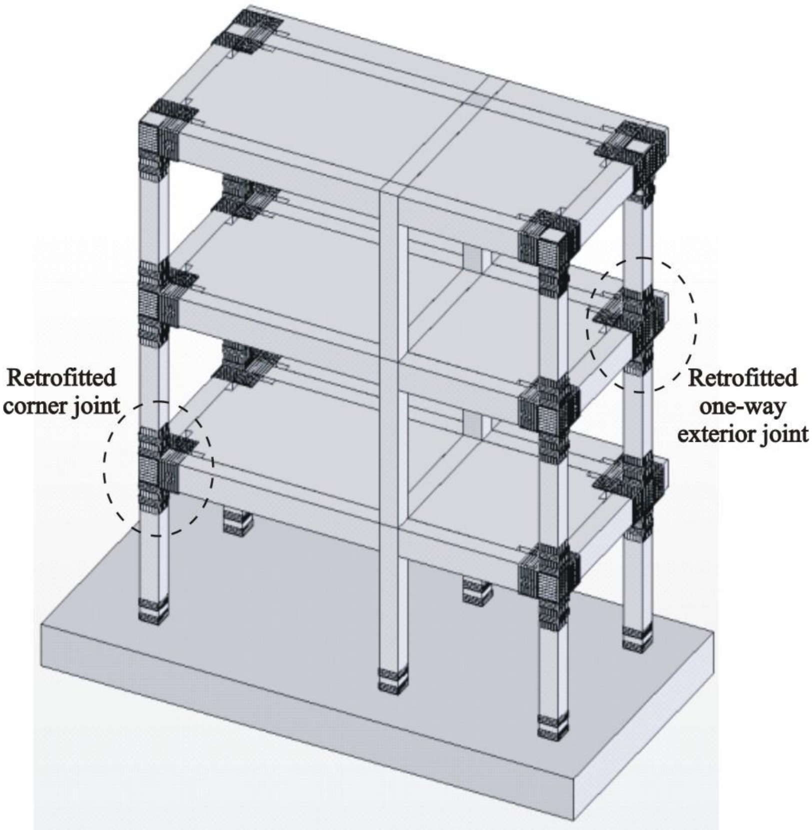

Figure 6. Dimetric view of the retrofitted RC frame model

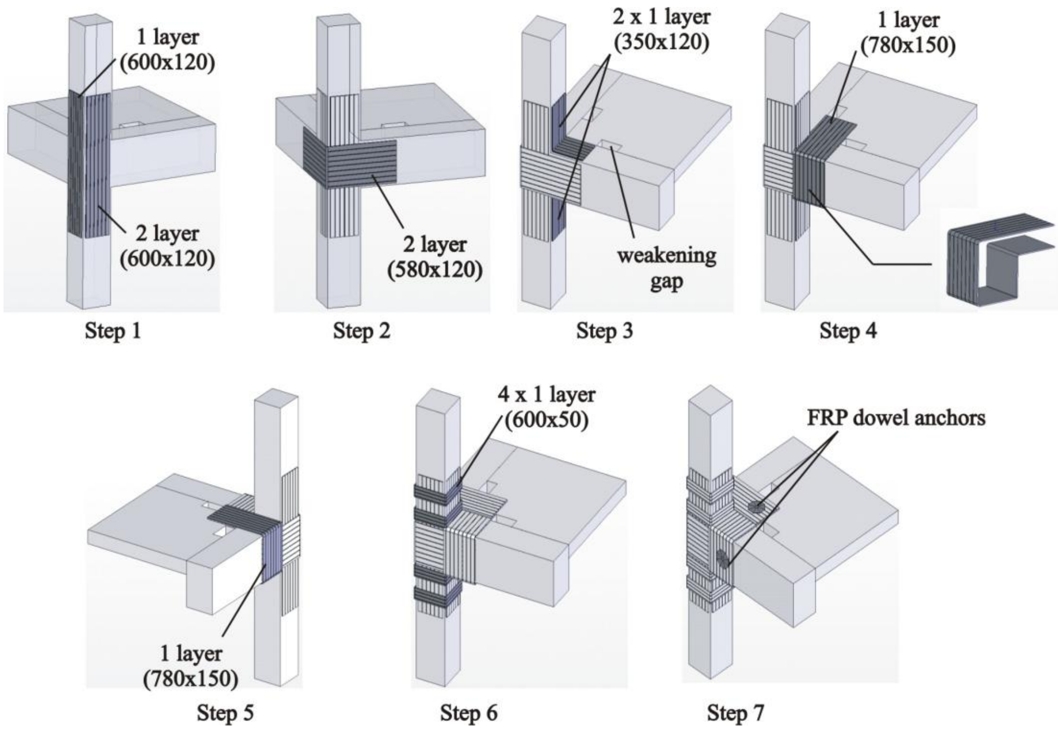

Figure 7. Dimensions and application sequence of FRP sheets for corner joints

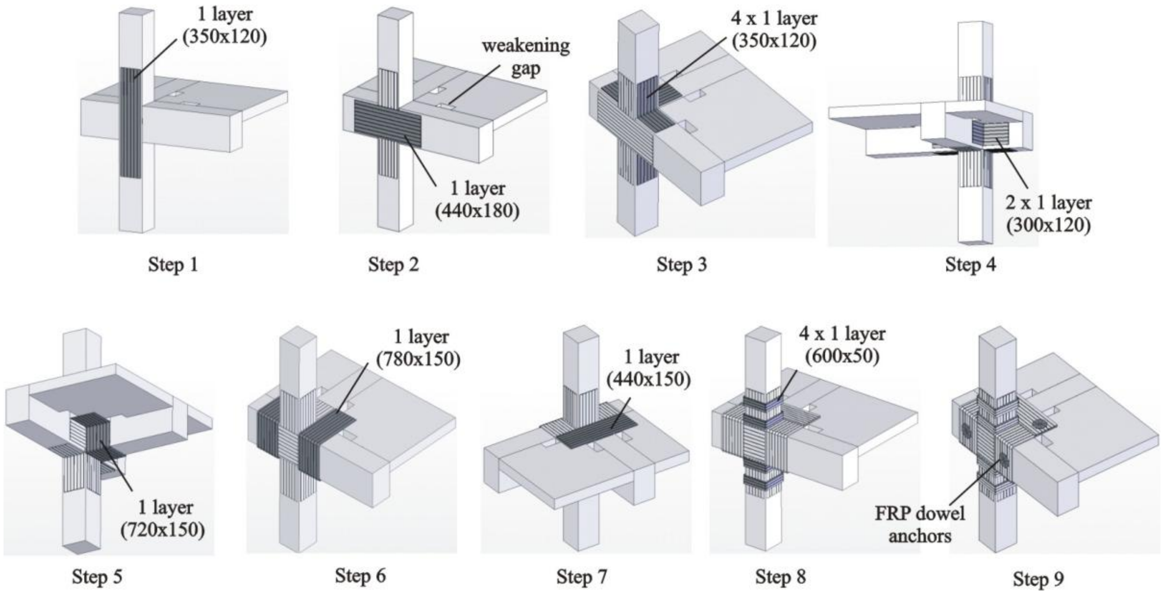

Figure 8. Dimensions and application sequence of FRP sheets for one-way exterior joints

3 CONCLUDING REMARKS

In this paper, a description of the GFRP retrofit strategy and solution for the RC frame building specimen designed and produced according to pre-1970's code design construction practice, in order to be tested on the shake-table at the University of Canterbury has been provided. Within the framework of the performance based-retrofit strategy, a step-by-step methodology for the assessment and retrofit of the critical elements (i.e., exterior joints) based on the use of fiber-reinforced polymers (FRP) has been explained. Particular focus has been given to the evaluation of the hierarchy of strength along with the sequence of events within critical locations in the RC frame such as corner and one-way exterior beam-column joints with slabs in their as-is as well as retrofitted configurations. Lastly, the sequence of application for the adopted FRP design has been explained in detail.

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