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DESIGN OF FRP STRENGTHENED RC BEAM-COLUMN JOINTS USING STRENGTH HIERARCHY ASSESSMENT METHOD

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Abstract

In this paper, the Fiber Reinforced Polymer (FRP) beam-column joint strengthening layout adopted by the researchers at the University of Canterbury (UC) has been studied. A simplified analysis and design procedure is proposed that can be used to quantify the provided capacity. The proposed procedure can facilitate the use of this strengthening scheme in real life engineering applications. The procedure is validated using the previous experimental work utilizing the same FRP strengthening scheme. The capacity given by the FRPs is assessed using the hierarchy of strength assessment method. In this method, the expected member failure sequences of as-built and strengthened reinforced concrete (RC) beam-column joints as well as the global base shear demands corresponding to each failure mode can be determined. It has been shown that the proposed design and analysis method for the FRP joint strengthening is in good agreement with the experimental test results with similar FRP strengthening schemes.

Keywords: FRP, retrofit, joint shear, hierarchy of strength

1. Introduction

Fibre reinforced polymers (FRPs) are light weight reinforcing fabrics with linear elastic properties. These materials have very high tensile capacity and they have been used in seismic strengthening applications of various structural elements in reinforced concrete buildings (RC). FRPs are adhered to concrete surface typically by using two component epoxy resin. In practice, there are two popular/common types of FRPs. These are carbon fibre reinforced polymers (CFRP) and glass fibre reinforced polymers (GFRP). The properties of these two types of FRPs are summarized in Table 1, where ρ is the density, t_f is the effective thickness of a single sheet, f_{fu} is the tensile strength, ε_{fu} is the rupture strain and E_f is the modulus of elasticity [1]. Although the application of these materials are not limited to strengthening applications, this paper focuses on their use for strengthening vulnerable beam-column joints for shear.

Table 1 – Properties of common FRP types

Fibre	**ρ (g/cm^3)**	**t_f (mm)**	**f_{fu} (MPa)**	**ε_{fu}**	**E_f (MPa)**
GFRP-Sikawrap 100G	2.56	0.36	2300	0.028	76000
CFRP-MBrace C5-30	1.90	0.165	3000	0.008	390000

Over the past decades, researchers around the world investigated various FRP application layouts for strengthening vulnerable RC beam-column joints for shear. Gergely et. al. investigated the influence of CFRP curing process, surface preparation and CFRP layout on beam-column joint shear strength [2]. Ghobarah and Said studied the effects of joint panel zone wrapping and the GFRP dowel anchors on joint shear strength [3, 4]. Liu investigated the effects of GFRP confinement of the ends of the columns below and above the considered beam-column joint [5]. Clyde and Pantelides tested the effect of diagonally aligned CFRP wrapping on joint shear strength [6]. Antonopoulos and Triantafillou tested numerous configurations of FRP applications for increasing joint shear strength [7]. Following these researchers, Pampanin et. al. studied one of these FRP arrangements in detail [8]. The same FRP layout (Fig. 1) was investigated considering the effects of axial load variation and bi-axial lateral load demand by Akguzel [1, 9, 10], outcomes of which are used in the development and validation of the simplified procedure summarized in this paper. In the reported procedure, the gained capacity due to the FRP joint shear strengthening layout is quantified in terms of principal tensile strength. This result can be directly used in the strength hierarchy assessment method. This method is used to identify the effectiveness of the applied strengthening in changing the strength hierarchy (i.e. from joint failure to beam hinging). Moreover, the calculated capacities in this method are directly expressed as the corresponding global base shear [11, 12], which makes it possible to quantify the effect of this local strengthening strategy in terms of the global base shear of the structure as a whole.

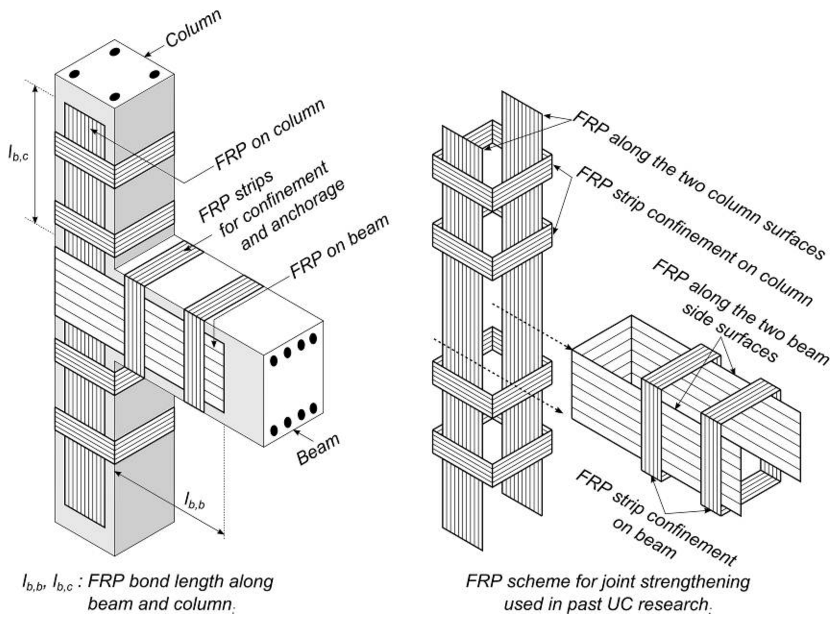


Fig. 1 – Considered FRP layout in studies at UC

2. Summary of Strength Hierarchy Assessment Method

According to the capacity design principles [13], strength hierarchy (or internal hierarchy of strength) represents the order of the structural member capacities where the weakest one is expected to yield/fail first under applied lateral forces. For seismic strengthening applications, the identification of the weakest element and its failure type in a beam-column joint is crucial for selecting the most appropriate strengthening approach. Detailed information on the latest concept of the strength hierarchy and the sequence of events can be found elsewhere [11, 12] as well as its background literature [1, 8, 9]. The method is a comparison of the imposed demand and the member capacities at an RC beam-column joint. This comparison is used in order to determine the expected sequence of structural member capacities under the variation of loading, which is shortly referred as strength hierarchy. In this method, the lateral demand on the structural system is expressed as axial forces (N_i) and bending moments (M_i) on beam-column joints using portal frame analysis method [14]. Using increasing levels of lateral forces, the demand on beam-column joints can be expressed as variations of axial forces and bending moments. The resulting demand can be plotted on *N-M* interaction diagram of the column at the considered beam-column joint. Similarly, the bending capacity of the beam and the shear capacity of the joint panel zone can be represented on the *N-M* interaction diagram for the considered beam-column joint. This process facilitates the comparison of the associated member capacities to the imposed variations of demand. The summary of this method is shown in Fig. 2 for beam-column joints lacking shear reinforcement.

In this method, calculation of the demand, the beam capacity and the column *N-M* diagram are relatively easy. The complication arises when the joint shear capacity needs to be quantified on the *N-M* diagram. The method summarized in Fig. 2 includes an approximate formulation for the joint shear capacity using Mohr's circle for state of stress within a beam-column joint. This is complemented with empirically calculated principal tensile strength values (P_t) for a few vulnerable configurations of joint shear reinforcement detailing, i.e. pre

1970s [1, 15]. It should be noted that the same method is applicable to modern RC structures with a different calculation method for the joint shear capacity [11, 12]. Using these values and the equations of equilibrium at beam-column joints, the resulting capacity can be represented in terms of equivalent column moment as shown in the same figure as M_{ci}. The resulting strength hierarchy is shown by red dashed arrow on *N-M* diagram in Fig. 2. Corresponding base shear values can also be identified using demand curve data directly, which is plotted using increasing values of F and $V_{Base}=6F$ in Fig. 2. Considering FRP strengthening of vulnerable beam-column joints, the representation of the provided shear capacity by the FRP application can be expressed as an equivalent principal tensile strength (P_{tf}). Using the sum of this value with the principal tensile strength of the as-built beam-column joint ($P_{tt}=P_t+P_{tf}$), the retrofitted shear strength can be calculated.

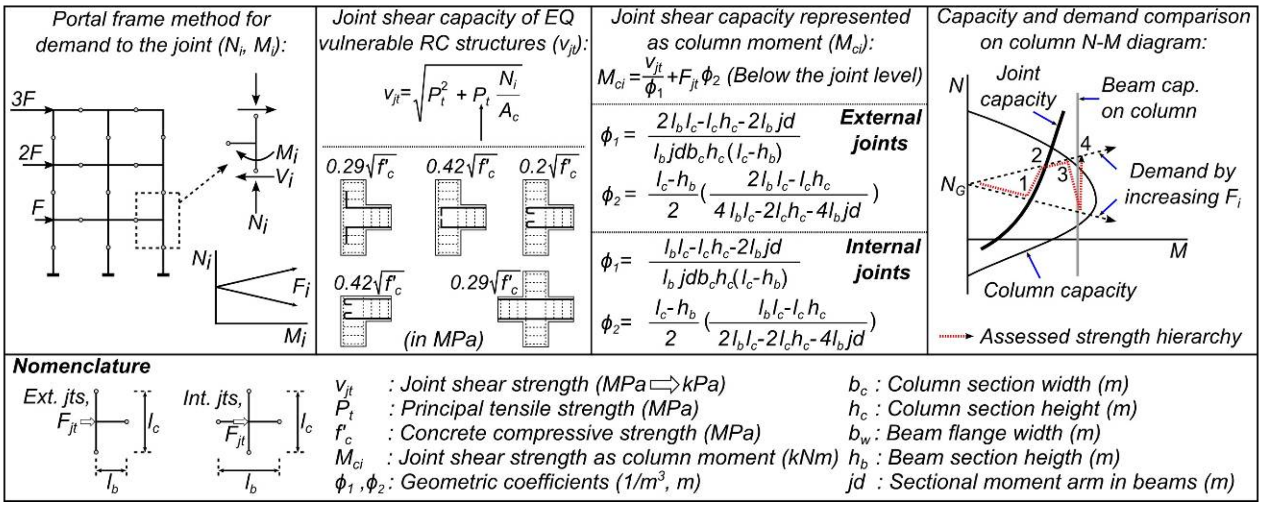


Fig. 2 – Summary of the strength hierarchy assessment

3. FRP Bond Properties and Maximum Usable Strength

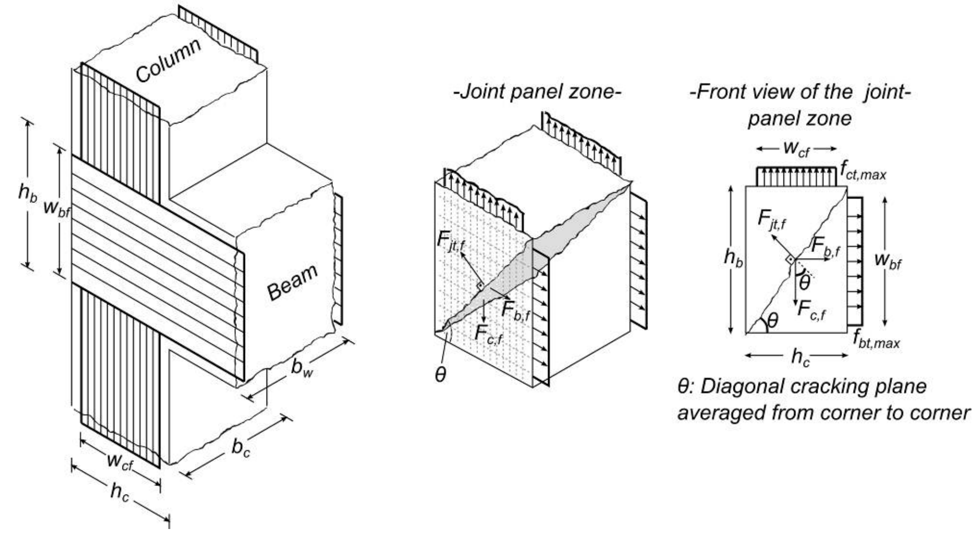
Even though commercially available FRPs have exceptionally high tensile capacity, in reality the capacity of the FRP application significantly depends on the bond strength between the concrete surface and the FRP layer. In most cases, all of the tensile capacity provided by the FRP itself cannot be utilized and de-bonding failures occur at lower stress levels. Typically, the bond length and the resulting maximum usable strength (or de-bonding strength) can be expressed as given in Eq. (1) and Eq. (2) where l_b is the bond length, $f_{t,max}$ is the de-bonding strength, E_f is the modulus of elasticity of FRP, t_f is the thickness of a single FRP sheet, n_f is the number of FRP layers, f_{ct} is the tensile strength of concrete, c_1 is an empirical coefficient reported as 0.64 [16], c_2 is an empirical coefficient reported as 2.0 [17]. It should be noted that the bond strength expressions can only describe the de-bonding failure approximately and the results should be used/evaluated with caution [18].

$$l_b = \sqrt{\frac{E_f \cdot n_f \cdot t_f}{c_2 \cdot f_{ct}}} \tag{1}$$

$$f_{t,\max} = c_1 \cdot \sqrt{\frac{E_f \cdot f_{ct}}{t_f \cdot n_f}} \tag{2}$$

4. Analysis and Capacity Quantification of FRP Strengthened RC Beam-Column Joints

In strength hierarchy assessment of a vulnerable RC beam-column joint, the joint shear capacity is expressed in terms of principal tensile strength (P_t). At such joints, FRP can be provided for the required tensile strength. As a result of this, the provided capacity should be expressed in terms of the principal tensile strength due to the FRP application (P_{tf}). Using this value, the total principal tensile strength (P_{tt}) at the joint panel zone can be used in strength hierarchy assessment in order to assess and express the retrofitted capacity.



a) b) c)

Fig. 3 – The capacity contribution due to FRP at the beam-column joint panel zone represented by $F_{jt,f}$: a) FRP layout at the joint panel zone; b-c) Joint panel zone and the additional capacity due to the FRP sheets on the beam and column using an approximate diagonal cracking angle of θ

The capacity provided by the FRP layout shown in Fig. 3a can be calculated using the diagonal force capacity, $F_{jt,f}$, which can be carried out by assuming an approximate diagonal cracking angle of θ from corner to corner at the joint panel zone. The resulting value can be expressed in terms of principal tensile strength using the area of the cracking plane (as shown in Fig. 3b). The assumption for the diagonal cracking angle is more or less in accordance with the typical cracking patterns that can form at such joint typologies (Fig. 4).

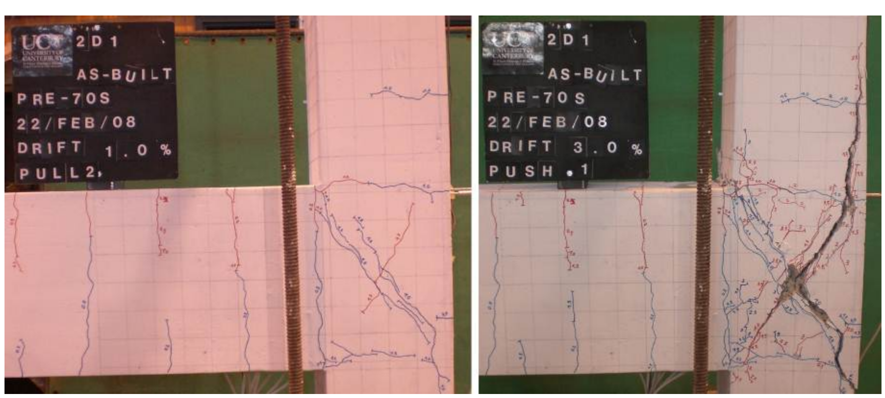


Fig. 4 – Vulnerable RC beam-column joint crack formations in a sub-assembly (Courtesy of Umut Akguzel)

Considering such a cracking pattern, the provided FRP layout forms a mesh reinforcement at these crack locations that utilizes the uniformly distributed tensile capacities of the FRP sheets on the beam and the column (Fig. 3b-c). Accordingly, additional capacity due to the FRP installed on the side surfaces of the beam can be expressed as $F_{b,f}=n_{bf}\cdot n_{bs}\cdot t_{bf}\cdot w_{bf}\cdot f_{bt,max}$ and for the column, the additional capacity can be expressed as $F_{c,f}=n_{cf}\cdot n_{cs}\cdot t_{cf}\cdot w_{cf}\cdot f_{ct,max}$. In these expressions, subscripts b and c represent beam and column, n_f is the number of FRP sheets, n_s is the number of side surfaces where the FRP sheets are adhered, t_f is the thickness of a single FRP sheet, w_f is the width of the FRP, $f_{t,max}$ is the de-bonding strength of the FRP. Using these expressions, the equilibrium perpendicular to the cracking plane can be written as given in Eq. (3).

$$F_{jt,f} = F_{b,f} \cdot \sin\theta + F_{c,f} \cdot \cos\theta \tag{3}$$

Using the calculated $F_{jt,f}$ value and the cross sectional area of the approximated diagonal cracking plane, the capacity gain by the FRP layout can be expressed in terms of principal tensile strength as given in Eq. (4).

$$P_{tf} = \frac{F_{jt,f}}{b_c \cdot \sqrt{h_c^2 + h_b^2}} \tag{4}$$

Using the FRP contribution to the principal tensile strength, P_{tf}, and the principal tensile strength of the as-built joint, P_t (Fig. 2), the total principal tensile strength, P_{tt}, can be expressed as in Eq. (5). The resulting value can be directly used in strength hierarchy assessment in order to assess and compare the shear capacity of the FRP strengthened beam-column joint to the previously calculated as-built shear capacity.

$$P_{tt} = P_t + P_{tf} \rightarrow \nu_{jt} = \sqrt{P_{tt}^2 + P_{tt} \cdot \frac{N_i}{A_c}} \tag{5}$$

Considering the design application of the proposed methodology, the only parameter to be determined is the number of FRP sheets on the beam (n_{bf}) and the column (n_{cf}). The remaining design information are the material properties of the selected FRP type (i.e. modulus of elasticity, E_f, and sheet thickness, t_f) and the geometrical sizing of the FRP sheets (i.e. FRP width on beams, w_{bf}, and columns, w_{cf}). Moreover, the de-bonding strength of the FRPs ($f_{t,max}$) are also dependent on the number of FRP layers used, which is in a square root expression in the denominator of Eq. (2). This suggests that FRP strengthening has diminishing returns for the gained capacity with each layer of additional FRP sheet. According to this, it can be stated that if the strengthening requires more than 2-3 layers of FRP in order to change the strength hierarchy, FRP strengthening is not efficient and feasible. Thus, alternative strengthening options may need to be considered in such cases. The summary of this design and assessment procedure is shown in Fig. 5. It should be noted that strength hierarchy assessment is a major element in this procedure and it forms the basis for determining the strength hierarchy of both as-built and the strengthened beam-column joints. Moreover, by strength hierarchy method, the global base shear values corresponding to each calculated member capacity can be observed directly.

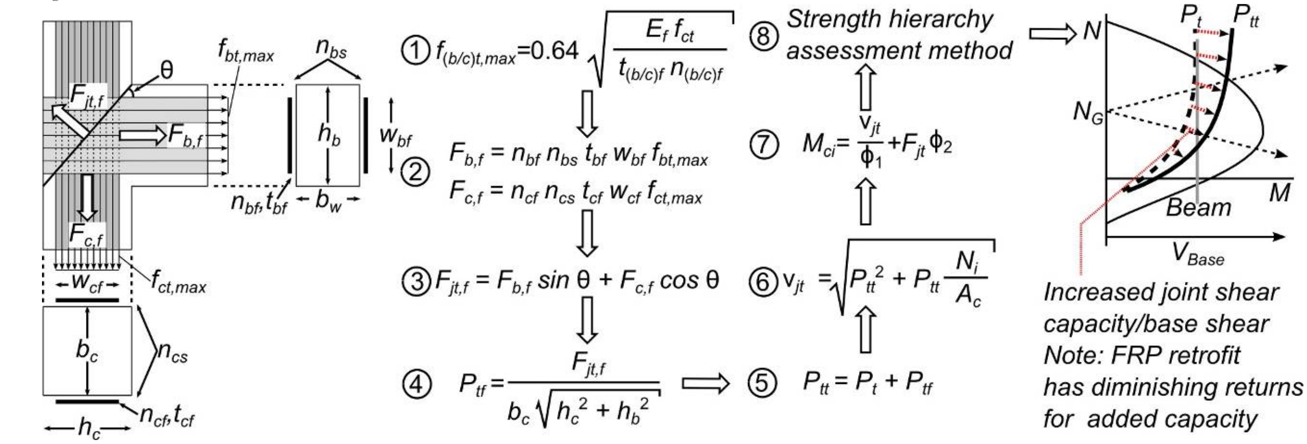
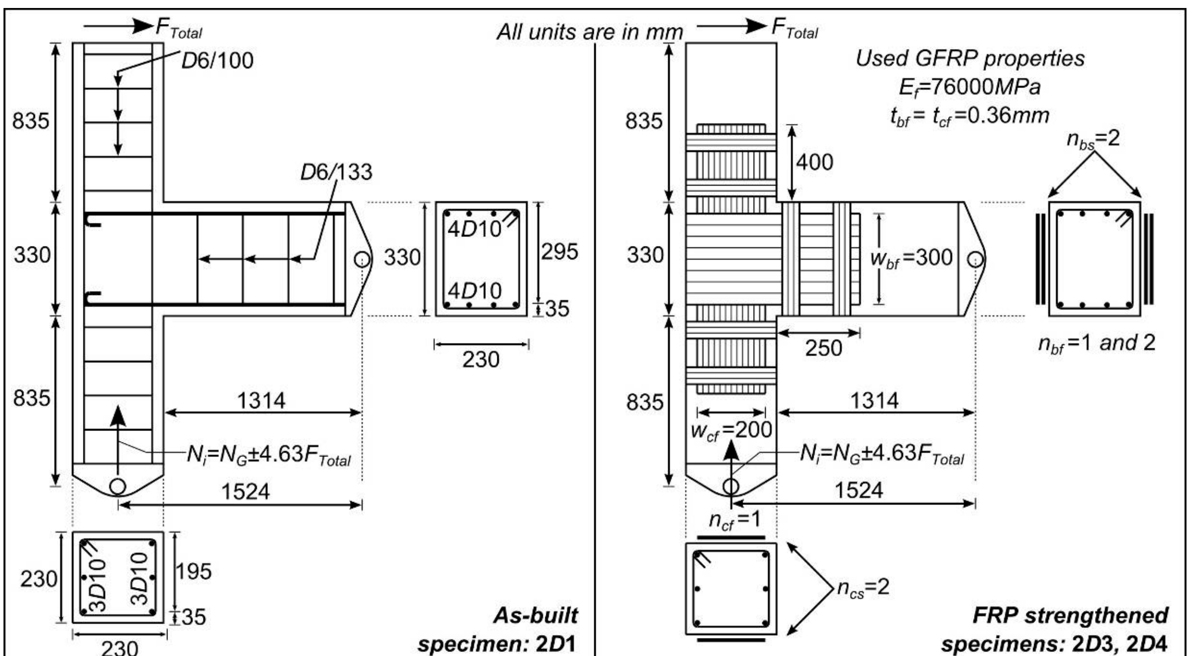


Fig. 5 – Summary for the capacity quantification and design of FRP joint strengthening and the assessment of the design by strength hierarchy assessment

5. Example Application: Beam-Column Joint Sub-Assembly

The validity of the proposed procedure is examined by using previous experimental studies utilizing the FRP strengthening layout given in this paper. In the considered study, vulnerable as-built beam-column joint sub-assemblies and GFRP strengthened beam-column joint sub-assemblies were tested [1]. These test results are compared to the estimated capacity values obtained by the procedure reported herein. The details of the considered test specimens are summarized in Fig. 6.



Properties of the used example test specimens	Specimen	Number of FRP Layers: On beam (n_{bf})	Number of FRP Layers: On column (n_{cf})	f_c' (MPa)	$f_{ct} \approx 0.35(f_c')^{0.5}$ (MPa)	f_y (MPa)	$M_{u,beam}$ (kNm)	$\varphi_{u,beam}$ (rad/m)
	2D1	As-built (No retrofit)		17.9	1.481	430	37.0	0.077
	2D3	1	1	18.0	1.485	340	30.0	0.085
	2D4	2	1	18.7	1.514	340	30.0	0.086

Fig. 6 – Test specimen details

The specimens were tested under quasi-static loading with an axial force variation of N_i=N_G±4.63F_{Total}. The 1st test was carried on the as-built benchmark specimen 2*D*1. The specimen 2*D*3 had the same detailing in the structural elements with a strengthening arrangement using single layers of FRP on the beam and the column, which is a minimum strengthening scheme. The specimen 2*D*4 employed an FRP arrangement with a single layer of FRP on the column and two layers of FRP on the beam. The specimen 2*D*1 showed joint shear failure as expected at about 12.5*kN* lateral force (Fig. 7a). Following this test, the specimen 2*D*3 was tested, but the strengthening applied to this specimen was not adequate. At the end of the test, an underlying joint shear failure was revealed after the removal of the FRP sheets, which might have formed at about 17*kN* lateral force level (Fig. 7b). On the other hand, the testing of the specimen 2*D*4 showed that the strengthening applied to this specimen was adequate to prevent the joint shear failure and caused beam hinging at about 22.5*kN* lateral force level (Fig. 7c). The force deformation hysteresis of these tests are shown in Fig. 7 as well as their end of test photos. It should be noted that the capacity gain with each added layer of FRP gets less efficient (i.e. diminishing returns of shear strength where the lateral capacity gain from 2*D*1 to 2*D*3 is 63% while from 2*D*3 to 2*D*4, it is 11%).

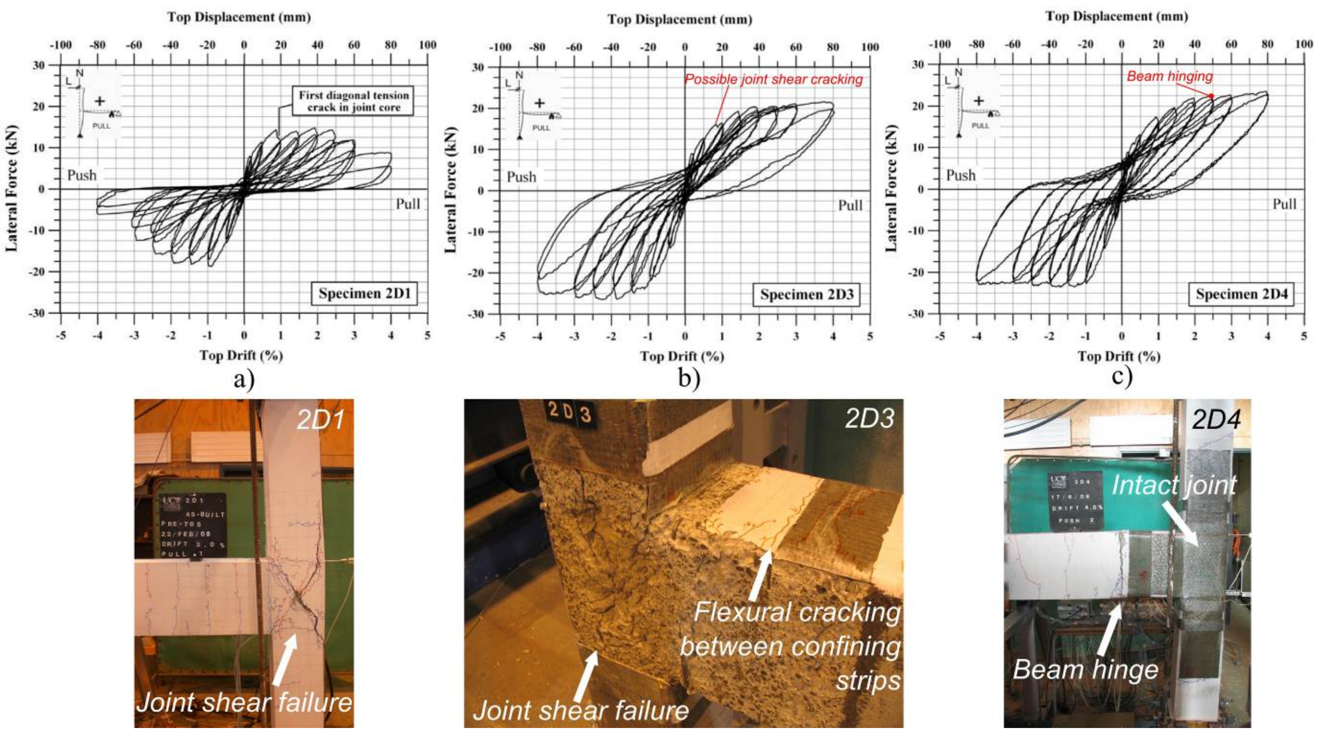


Fig. 7 – Test results of the example specimens

Using the procedure shown in Fig. 5, calculations can be carried out in order to quantify the effects of the applied FRP strengthening arrangements as summarized in Table 2. Then, the resulting values can be directly implemented in the strength hierarchy assessment (Fig. 2) in order to determine the resulting sequence of the member capacities due to the applied strengthening layout.

According to the results of the strength hierarchy assessment of the beam-column joints, joint shear failure of the as-built specimen 2*D*1 can be estimated to occur at 11.5-15.9*kN* lateral force levels (Fig. 8a). This observation is in good agreement with the experimental observations taken during the test of this specimen, which occurred at about 12.5*kN* (Fig. 7a). The assessment of the specimen 2*D*3 showed that the provided strengthening causes an increase in joint shear strength (18.8*kN* lateral force level). However, the achieved capacity is almost equal to the capacity of the beam (18*kN* in Fig. 8b). According to these estimations, there is a high likelihood of joint shear failure occurring before the beam hinging can occur. The testing of the specimen 2*D*3 confirmed this estimation and joint shear failure occurred at approximately 17*kN* lateral force level (Fig. 7b). Considering the last test specimen 2*D*4, the strength hierarchy assessment estimated a joint shear capacity higher than the beam capacity, ensuring a beam plastic hinge mechanism at approximately 18*kN* lateral force level (Fig. 8c). This estimation is in good agreement with the experimental results of this specimen where beam hinging was observed without any joint shear failure at around 20-22.5*kN* lateral force level (Fig. 7c). The strength hierarchy assessment plots for all the test specimens are shown in Fig. 8.

Table 2 – Calculation of the total principal tensile strength (P_{tt}) for the as-built and strengthened beam-column joints

Spec.	n_{bf}	n_{cf}	$f_{bt,max}$ (MPa)	$f_{ct,max}$ (MPa)	$F_{b,f}$ (N)	$F_{c,f}$ (N)	$F_{jt,f}$ (N)	P_{tf} (MPa)	P_t (MPa)	P_{tt} (MPa)
2D1	0	0	0	0	0	0	0	0	0.846	0.846
2D3	1	1	358.33	358.33	77399.28	51599.52	93002.63	1.005	0.849	1.854
2D4	2	1	255.81	361.77	110509.92	52094.88	120451.51	1.302	0.865	2.167

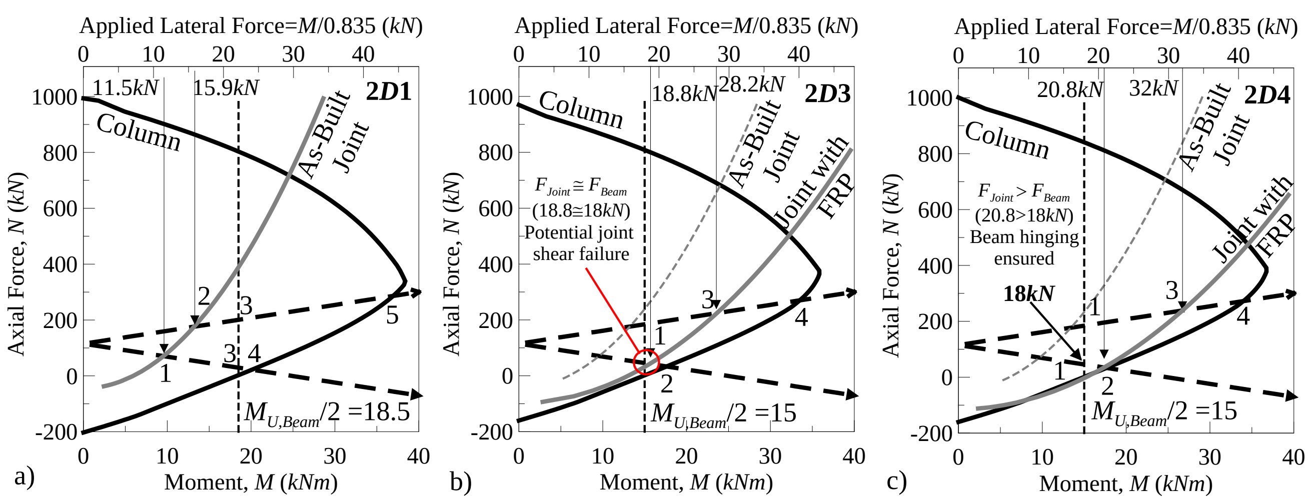


Fig. 8 – Strength hierarchy assessment of the test specimens: a) As-built RC beam-column joint specimen 2*D*1; b) RC beam-column joint strengthened with 1 layer of FRP on the beam and 1 layer of FRP on the column (inadequate strengthening); c) RC beam-column joint strengthened with 2 layers of FRP on the beam and 1 layer of FRP on the column (adequate strengthening)

6. Example Application: 3 Storey RC Frame

In this example, the application of the procedure is reported using a 3 storey RC test frame studied by researchers in Pavia University [8]. This past study focused on testing of a vulnerable as-built specimen and a CFRP strengthened specimen using the FRP layout given in this paper. The details of this test specimen are shown in Fig. 9. According to the test results of the as-built specimen, external beam-column joints failed in shear at 1st and 2nd floor level while the column hinging was observed at the internal beam-column joints at the same levels. The damage to the structural elements at 3rd floor level joints were negligible and most of the observed damage concentrated to the 1st and 2nd levels. The results for the strength hierarchy assessment of the as-built specimen and its comparison to experimental results is published elsewhere [11, 12]. The second test specimen was strengthened with CFRP layout shown in Fig. 9c. The CFRP strengthening layout shown in the figure is quantified using the procedure summarized in this paper. The resulting principal tensile strength values are given in Table 3 in order to be utilized in strength hierarchy assessment of the CFRP strengthened joints.

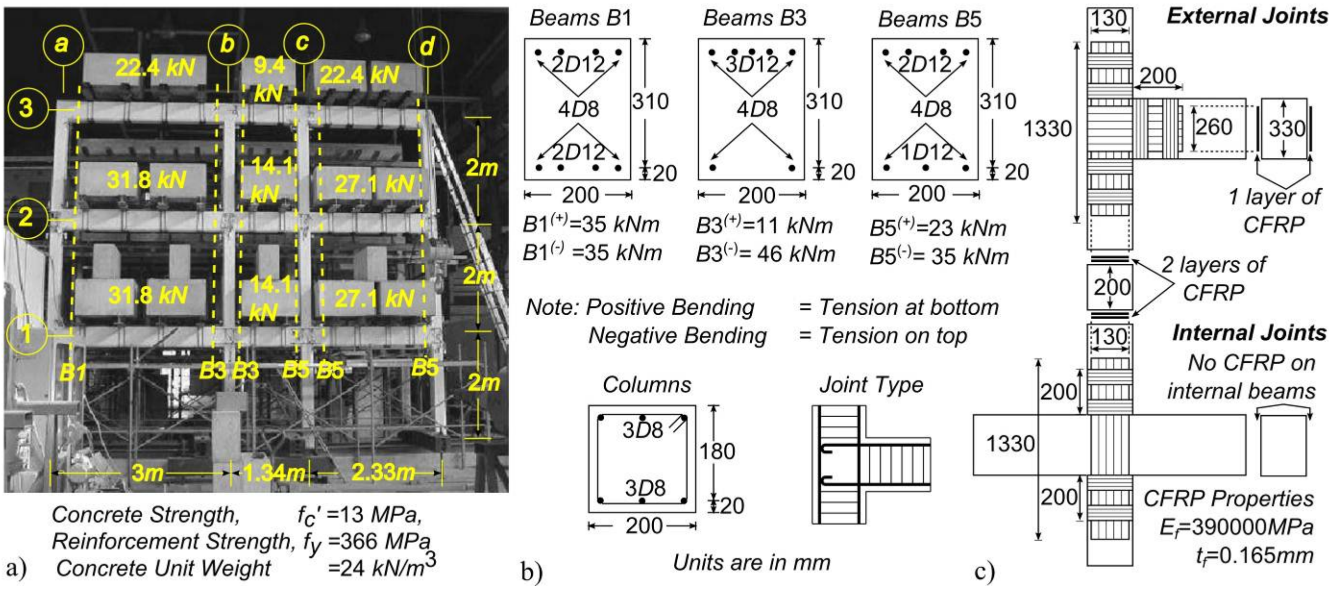


Fig. 9 – Vulnerable RC frame example: a) Structural dimensions; b) Section details; c) Description of the FRP retrofit applied to the specimen

Table 3 – Calculation of the total principal tensile strength (P_{tt}) to be used in strength hierarchy assessment of the strengthened specimen

	n_{bf}	n_{cf}	$f_{bt,max}$ (MPa)	$f_{ct,max}$ (MPa)	$F_{b,f}$ (N)	$F_{c,f}$ (N)	$F_{jt,f}$ (N)	P_{tf} (MPa)	P_t (MPa)	P_{tt} (MPa)
External Joints LVL1-2	1	2	1105.33	781.58	94837.31	67059.56	115862	1.501	0.721	2.222
Internal Joints LVL1-2	0	2	NA	781.58	0	67059.56	34759	0.45	1.046	1.496

Using the P_t and P_{tt} values, the strength hierarch assessment showed that the joint shear capacity of the vulnerable beam-column joints can be improved with the application of the given FRP scheme. As it can be seen in Fig. 10, the application of the FRP increased the joint shear capacity when compared to the as-built joints. Considering the external beam-column joint *a*1 (leftmost at 1st floor level), as-built specimen previously experienced joint shear failure at 31-38*kN* total base shear level and the strengthening application increased the estimated capacity to 64-84*kN* total base shear. Considering the applied variation of demand, the sequence of failures at joint *a*1 are: 1-Column hinge at 51*kN* base shear in push direction; 2-Joint shear at 64*kN* base shear in push direction; 3-Column hinge at 71*kN* in pull direction; 4-Beam hinge at 71*kN* in push/pull direction; 5-Joint shear at 84*kN* base shear in pull direction. Inspecting the strength hierarchy assessment of the remaining joints, similar conclusions can be drawn for each beam-column joint. All of the calculated strength hierarchy assessment plots are shown in Fig. 10.

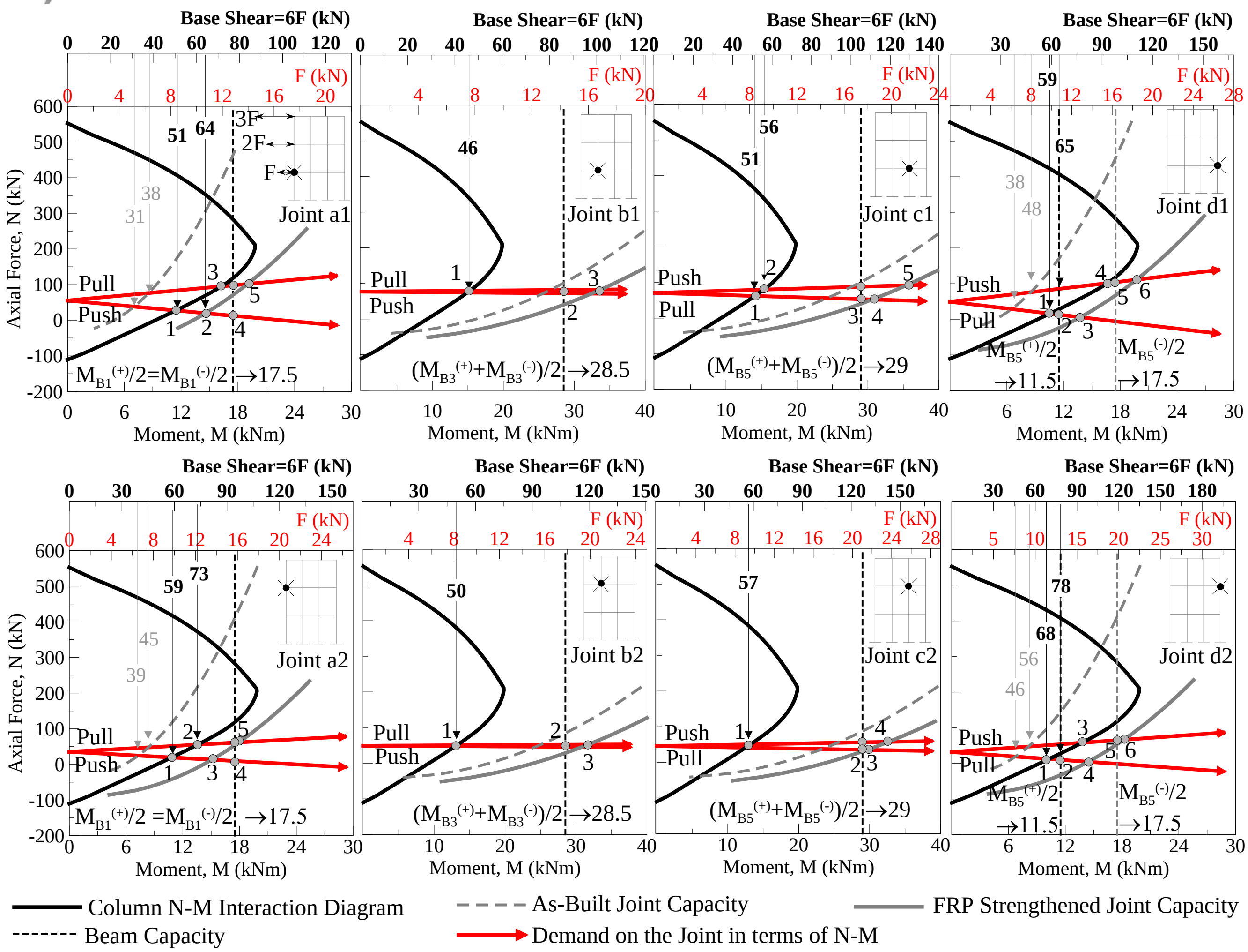
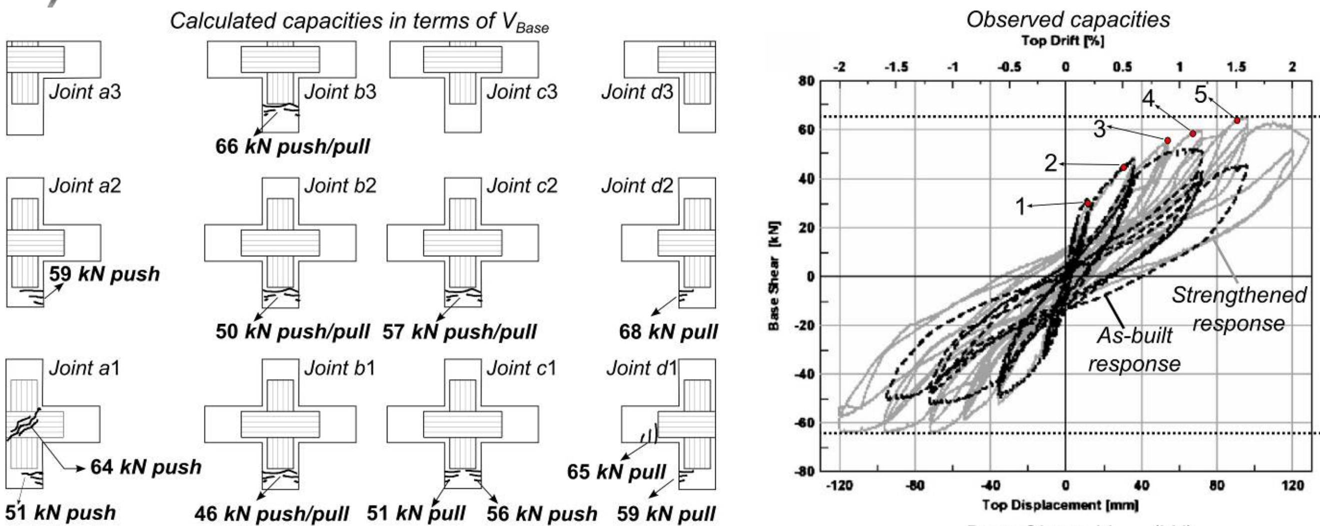


Fig. 10 – Strength hierarchy assessment of the FRP strengthened beam-column joints compared to the as-built joints at 1st and 2nd floor levels and the base shear values corresponding to the failure of the weakest elements (shown in bold)

When the results of the strength hierarchy assessment are inspected for all beam-column joints, a global summary of failure modes can be made (Fig. 11). According to these estimations, the column bending capacities are utilized at 1st and 2nd storeys until approximately 68*kN* base shear level. Up to this force level, joint shear failure at *a*1 and beam hinging at *d*1 are expected to occur at about 64*kN* and 66*kN* lateral force levels, which practically utilizes the structural redundancy of the specimen.

Considering the results of the experiments, the observed peak lateral force level during the test is about 64-65*kN*. This level of force is very close to the estimated joint shear failure and beam hinging at the external beam-column joints of the 1st level (i.e. 64-66*kN*), validating the procedure reported herein. Moreover, the estimated capacities for all beam-column joints can be related to the experimental results of the strengthened specimen by inspecting the points where apparent stiffness changes are observed in the hysteresis curve. These points approximately match the calculated estimations as shown in Fig. 11. More detailed photographic summary of the specimen at the end of the test can be found in the related publication [8].



Possible Failure Type	Base Shear, V_{Base} (kN) Observed	Calculated
Event 1: Potential joint crack at a1	**28-30kN**	**31kN**
Event 2: Column hinging at b1, b2	**45kN**	**46, 50kN**
Event 3: Column hinging at a1, c1	**55kN**	**51, 56kN**
Event 4: Column hinging at d1, a2, c2	**58kN**	**59, 59, 57kN**
Event 5: Joint failure at a1, beam hinging at d1, column hinging at b3, d2	**64kN**	**64, 65, 66, 68kN**

Fig. 11 – Expected failure patterns as a result of the strength hierarchy assessment and comparison to the identified apparent stiffness changes in the experimental response

7. Conclusions

A particular FRP joint strengthening scheme previously used by various researchers at the University of Pavia (Italy) and the University of Canterbury (New Zealand) is studied in detail. A procedure is reported for the quantification and assessment of the provided capacity as a result of the given FRP joint shear strengthening layout. The procedure can be directly used in conjunction with the strength hierarchy assessment method so that the resulting strength hierarchies can be expressed in terms of global base shear demand, which is part of the method. The procedure is validated by two example applications: strengthened beam-column joint sub-assemblies and a strengthened 3 storey RC frame. The obtained results are shown to be in good agreement with the experimental results of the test specimens. The strength hierarchy assessment method and the reported analysis/design procedure are practical tools to assess vulnerable RC beam-column joints and to design FRP strengthening schemes for such joints. The methods do not require complicated computer models. The proposed procedure can be conveniently implemented by the practitioner engineers using only a spreadsheet software and fundamental knowledge of reinforced concrete structures.

8. References

[1] Akguzel, U. (2011). Seismic Performance of FRP Retrofitted Exterior RC Beam-Column Joints under Varying Axial Load and Bidirectional Loading, in Civil and Natural Resources Engineering Department, University of Canterbury: Christchurch, New Zealand. p. 394.

[2] Gergely, J., Pantelides, C.P., and Reaveley, L.D. (2000). Shear Strengthening of RCT-Joints Using CFRP Composites. Journal of Composites for Construction. 4(2): p. 56-64.

[3] Ghobarah, A. and Said, A. (2001). Seismic Rehabilitation of Beam-Column Joints Using FRP Laminates. Journal of Earthquake Engineering. 5(1): p. 113-129.

[4] Ghobarah, A. and Said, A. (2002). Shear Strengthening of Beam-Column Joints. Engineering Structures. 24(7): p. 881-888.

[5] Liu, A. (2001). Seismic Assessment and Retrofit of Pre-1970s Reinforced Concrete Frame Structures, in Civil and Natural Resources Engineering Department, University of Canterbury: Christchurch, New Zealand.

[6] Clyde, C. and Pantelides, C.P. (2002). Seismic Evaluation and Rehabilitation of R/C Exterior Building Joints. in 7th National Conference on Earthquake Engineering, 7NCEE, Earthquake Engineering Research Institute. Oakland, CA.

[7] Antonopoulos, C.P. and Triantafillou, T.C. (2003). Experimental Investigation of FRP-Strengthened RC Beam-Column Joints. Journal of Composites for Construction. 7(1): p. 39-49.

[8] Pampanin, S., Bolognini, D., and Pavese, A. (2007). Performance-Based Seismic Retrofit Strategy for Existing Reinforced Concrete Frame Systems Using Fiber-Reinforced Polymer Composites. Journal of Composites for Construction 11(2): p. 211-226.

[9] Akguzel, U. and Pampanin, S. (2011). Assessment and Design Procedure for the Seismic Retrofit of Reinforced Concrete Beam-Column Joints Using FRP Composite Materials. ASCE Journal of Composites for Construction. 16(1): p. 21-34.

[10] Akguzel, U. and Pampanin, S. (2010). Effects of Variation of Axial Load and Bidirectional Loading on Seismic Performance of GFRP Retrofitted Reinforced Concrete Exterior Beam-Column Joints. ASCE Journal of Composites for Construction. 14(1): p. 94-104.

[11] Tasligedik, A.S. (2015). Calculation of Strength Hierarchy at Reinforced Concrete Beam-Column Joints of Buildings. in 10th Pacific Conference on Earthquake Engineering. Sydney, Australia.

[12] Tasligedik, A.S., Akguzel, U., Kam, W.Y., and Pampanin, S. (2015). Strength Hierarchy at Reinforced Concrete Beam Column Joints and Global Capacity. Journal of Earthquake Engineering. Manuscript under review.

[13] Park, R. and Paulay, T. (1975). Reinforced Concrete Structures. John Wiley and Sons, Inc.

[14] Smith, A. and Wilson, W.M. (1915). Wind Stresses in Frames of Office Buildings. Journal of Western Society of Engineers: p. 341-365.

[15] Priestley, M.J.N. (1997). Displacement-Based Seismic Assessment of Reinforced Concrete Buildings. Journal of Earthquake Engineering. 1(1): p. 157-192, DOI: 10.1080/13632469708962365.

[16] Holzenkaempfer, P. (1994). Ingenieur-modell des Verbunds Geklebter Bewehrung fuer Betonbauteile, TU Braunschweig.

[17] FIB (2001). Bulletin 14: Externally Bonded FRP (Fibre Reinforced Polymer) Reinforcement for RC Structures. International Federation for Structural Concrete (fib).

[18] Saxena, P., Toutanji, H., and Noumowe, A. (2008). Failure Analysis of FRP-Strengthened RC beams. ASCE Journal of Composites for Construction. 12(1): p. 2-14.