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# Experimental Comparison of Novel CFRP Retrofit Schemes for Realistic Full-Scale RC Beam–Column Joints

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## Abstract

Existing reinforced concrete (RC) moment-resisting frames (MRF) built with inadequate detailing or before the introduction of detailed seismic design codes (pre-1970s) are highly vulnerable to brittle failure mechanisms under earthquake loading. To prevent potentially catastrophic failures and consequent human and economic losses in future earthquakes, efficient and practical retrofit solutions are required for these buildings. This paper presents an experimental study focused on the development of retrofit solutions that adopt carbon fibre reinforced polymers (CFRP) to improve the seismic performance of existing RC MRF at their beam-column connections. It is highlighted that to date, most experimental studies in this field have used simplified test specimens that have ignored the presence of slabs and secondary beams at beam-column connections. This may lead to an unrealistic assessment of FRP retrofit schemes. Hence, in this study, results from six full-scale cyclic tests on typical pre-1970's interior beam-column joints with slab and transverse beams are presented. The tests are used to assess three proposed CFRP schemes composed of a combination of FRP strengthening methods and selective slab weakening. Each scheme is designed to meet a distinct retrofit objective: (1) enhancement of the lateral load capacity (2) enhancement of ductility and (3). enhancement of the lateral load capacity, ductility, as well as changing the dominant failure mode of the joint from a column hinging mechanism to one where the plasticity is mainly concentrated in the beams.  A comparison of the retrofitted specimens to the behaviour of a deficient specimen and a specimen designed to modern guidelines (EC8), highlights the successful achievement of the respective retrofit objectives and the necessity to weaken the slab to achieve a favourable failure mechanism that will allow compliance to be achieved with current retrofit codes.

## Introduction

Reinforced concrete (RC) moment resisting frame (MRF) structures built before the introduction of modern seismic codes (pre-1970's or 80's), are not designed with adequate seismic resistance and ductility (Hoffman et al. 1992). These are hence found to constitute a disproportionally large fraction of damaged or collapsed buildings in post-earthquake field reports, as shown in Christchurch, New Zealand (Kam et al. 2011), Kocaeli, Turkey (Sezen et al. 2003) or Viña del Mar, Chile (Aranda et al. 2014). Inadequate detailing of beam-column joints and an inadequate hierarchy of strengths between framing members play a critical role in the poor seismic behaviour of pre-1970 RC MRFs.

Vulnerable pre-1970's RC frames constitute a large proportion of the existing building stock in many earthquake-prone countries. They tend to have high occupancy and comprise commercial, residential properties, and critical structures, including schools and hospitals (Ghosh and Sheikh, 2007). Overall this leads to a significant total risk composed of high exposure (population density), vulnerability (structural deficiencies) and hazard (high seismicity). With the large scale of existing structures not presenting adequate seismic resistance, demolition and rebuilding of deficient structures at scale is neither feasible, nor is it economical or sustainable. Instead, a number of studies have highlighted the lifetime economic and human benefits of retrofitting RC structures through cost-benefit analyses (Chiu et al. 2013; Smyth et al. 2004).

Traditional retrofit, including steel or RC-jacketing, are common, however often require significant intrusive work that can leave buildings unoccupied for longer periods of time. The added weight of such retrofits and their susceptibility to corrosion are further shortcomings. Base-isolation has also been adopted (e.g.: Luca Trombetta et al. 2014), however the current cost of the intervention on existing buildings at foundation level prohibits the use of this intervention in many cases. Fibre reinforced polymers (FRP) have been used extensively for the repair and retrofit of pre-1970 RC MRF in the aftermath of the 2009 L'Aquila and 2011 Christchurch earthquakes, and often prove more cost efficient then alternative retrofits (e.g.: Del Vecchio et al. 2016). Their high strength-to-weight ratio and corrosion resistance make FRP particularly attractive as retrofit material (Ghosh and Sheikh 2007). In addition, their application can be performed rapidly and without disrupting building occupancy, which can reduce down-time in businesses and the need of relocating inhabitants in residential properties (Bousselham 2010).

When applied in the field, FRP retrofits are most commonly implemented at component-level (beam or column) or are applied to exterior face of exterior beam-column joints. Their application at interior beam-column joints or to the joint itself is not often seen, mainly due to the practical difficulties to the placement of FRP around joints due to the presence of slabs and secondary beams.

Significant experimental research to improve the behaviour of beam-column joint sub-assemblies can however be found in the literature. For instance, tests on two retrofitted one-third scale interior joints indicate that the use of fan-shaped carbon-fibre anchors may be a viable solution to achieve continuous retrofit of columns through joints for an improved strength hierarchy (Shiohara et al. 2009). The work of Akguzel and Pampanin (2012), highlights the potential of combining FRP retrofits with selective weakening of slabs to promote a more ductile and dissipative beam hinging failure mode in 2/3-scaled corner joint specimens with slabs. This technique is a counter-intuitive seismic retrofit strategy in which a structure is initially weakened in a specific location to provide a more ductile behaviour, e.g. by cutting the slab to promote a beam-hinging mechanism. This is followed by a strengthening intervention to increase the capacity of the structure.

A recent study by Eslami and Ronagh (2014), uses CFRP to strengthen beam ends to relocate the plastic hinge away from the joint interface, in order to increase ductility and protect the joint from unwanted damage. These studies highlight the effectiveness of FRP as retrofit material in terms of enhancing strength and ductility of RC beam-column joints and are building the foundation for future uses of FRP for global structural retrofit interventions.

The results from over 30 experimental investigations on the cyclic behaviour of FRP retrofitted non-seismically designed RC joints have been systematically compiled in a database of over 200 specimens (Pohoryles and Rossetto 2014). Review of this database reveals that a majority of tests in the literature do not reflect real conditions, as specimens have scaled dimensions (61%) and do not contain slabs or transverse beams (88%). These factors however influence the failure mechanism of joints and affect the effectiveness of FRP retrofits (Choudhury et al. 2013; Park and Mosalam 2013). Moreover, ignoring the presence of slabs and transverse beams significantly impacts the requirements and practical retrofit layout, including the location of anchors, continuous flexural strengthening of weak columns and the shear strengthening of inaccessible interior joints. This may become an important aspect in a comprehensive retrofit in which exterior joints, as well as beams and columns around interior joints are retrofitted, as highlighted by limited experimental (Gallo et al. 2012) and numerical (Pohoryles et al. 2015) evidence.

To ensure the wider acceptance and practical implementation of suitable, fast, and efficient retrofits to address the constant threat of inadequate seismic behaviour of existing RC frames, there is therefore a significant need for developing, and testing, FRP retrofits for realistic RC structures. In this paper, the results from six full-scale cyclic tests that focus on realistic specimens with slabs and transverse beams, are presented. Three CFRP retrofit schemes of increasing complexity are proposed and compared. These interventions are designed to achieve different aims, including increased lateral load capacity, enhanced ductility, and a change in failure mechanism from a column hinging mechanism to a ductile weak-beam/strong-column failure mechanism. The cyclic performance of the retrofitted specimens is compared with that of a deficient, pre-1970's design specimen and a specimen designed to modern seismic guidelines (Eurocode 8). It is shown that the retrofit schemes are successful in meeting their respective performance objectives, in terms of increasing strength and displacement ductility, as well as reducing post-peak strength degradation. The paper concludes with a discussion on whether the proposed schemes present an effective retrofit solution and further work needed to achieve their practical implementation.

## Experimental program

### ***Specimen details and material properties***

Shortfalls identified from the existing literature (Pohoryles and Rossetto 2014), in terms of joint geometry, scale and loading arrangements, are used to design an experimental study on realistic joints with slabs and transverse beams. The test specimens are designed to represent real-scale interior beam-column joints in a four-storey RC moment resisting frame (MRF) structure. The geometry and reinforcement detailing of the specimens is shown in Fig. 1.

The superior and inferior columns represent a half-storey 1.50 m column with a square cross-section of 300 mm by 300 mm. Similarly, each main beam represents a 2.05 m half-span of a beam with a rectangular cross-section of 450 mm deep to 300 mm wide. For the specimens with slab, the slab is 1.95

m wide, with a depth of 150 mm, and the transverse beams are 825 mm long and have the same cross-sectional dimensions and reinforcement detailing as the main beams.

The six specimens tested in this study are three control specimens, C1, C1-sw and C-EC8, and three retrofit specimens of increasing complexity, C1-RT-A, C1-RT-A-sw and C1-RT-B-sw. The label 'sw' refers to specimens with selective weakening cuts in the slab. Apart from specimen C-EC8, the steel reinforcement detailing of the specimens aims to reflect the common design deficiencies of a beam-column joint in a pre-1970's four storey reinforced concrete residential building in Southern Europe. The design of beams, columns and joint is based on the flexure and shear demand computed from the REBA (1967) Portuguese RC code. The limits given within are followed and a normalised base shear factor for lateral load of 0.05 of building weight (273 kN) is chosen accordingly. Specimen C-EC8 is designed to Eurocode 8 (CEN 2004) for the high ductility class (DCH), with a pga of 0.36g (zone 3 in many Southern European countries) and ground type D (soft-to-firm cohesive soil). As expected, a much larger base shear of 1360.9 kN (normalised base shear of 0.18) is obtained for the same four storey building. The reinforcement detailing adopted in all specimens is summarised in Table 1.

For the specimens designed to REBA (1967), the detailing adopted leads to a number of seismic deficiencies typical of pre-1970's designs. These deficiencies lead to brittle failure mechanisms that are related to the specimen's non-compliance with capacity design principles. The specimens present an inappropriate hierarchy of strengths with a lower flexural capacity of the columns than the beams (weak-column/strong-beam mechanism) and a low shear capacity of the joint. The latter is due to the lack of shear reinforcement in the joint, as well as a lack of confinement in the columns due to inadequate transverse reinforcement spacing.

The mean values and standard deviation of the material properties from three compressive cylinder tests (Ø150 x 300 mm$^2$) for each of the six test specimens are summarised in Table 2. The results from tensile strength tests on steel and CFRP are reported in Table 3 and Table 4, respectively. For all FRP retrofits, CFRP sheet is used. The tensile strength is evaluated using characterisation tests performed according to the testing method in ISO/DIS 10406-2:2013. The parameters reported in Table 3 are $f_{u,FRP}$, $\varepsilon_{u,FRP}$, $E_f$ and $t_f$, the ultimate strength, strain, elastic modulus, and thickness of FRP, respectively.

***FRP retrofit design and application***

In this study, three different retrofit schemes of increasing complexity are evaluated, as shown in Fig. 2. The schemes are designed to consider practical limitations, and hence do not involve excessive removal of concrete or full wrapping of inaccessible members. In addition, the presence of slabs and transverse beams framing into the joint are explicitly accounted for. Retrofit scheme A is the simplest design, using FRP strands for strengthening of the columns through the slab. It aims to improve the strength of the specimen by increasing the moment capacity of the deficient columns, and to increase the global displacement capacity of the specimen by connecting the flexural strengthening of the superior and inferior columns. Scheme A-sw, adds selective weakening (sw) of the slab with the aim of promoting a more ductile beam hinging failure mechanism. Finally, scheme B-sw aims to increase strength and ductility of the specimen to achieve a behaviour comparable to specimens designed to modern design codes. This is the most intricate scheme including a combination of FRP strengthening of columns, beams and joint, as well as selective weakening of the slab.

The design of the FRP retrofit of the individual members is carried out using a step-wise design methodology described in more detail elsewhere (Pohoryles 2017). This procedure uses equations based on the Italian CNR-DT-200.R1/2013 (CNR 2013) guidelines to evaluate the number of FRP layers required in the respective retrofit locations. For each retrofit scheme, first, the relative bending and shear capacities of columns, joint and beams are evaluated using Eurocodes 2 (CEN 2008) and 8 (CEN 2004). Based on capacity design principles, the need for retrofit of the individual members is established. Following the design logic of Eurocode 8, the beams are designed to fail in bending and the columns are designed based on the retrofitted capacity of the beams. For details of the design procedure, please refer to Pohoryles (2017).

***Scheme RT-A***

The aim of scheme RT-A is to offer a simple and realistic solution to delay a brittle undesired column failure mechanism and increase the ductility of the specimen. To achieve this, it is necessary to increase the shear capacity of the column, provide confinement to the column to avoid buckling of the column bars, and to increase the flexural capacity of the columns to delay the occurrence of the weak-column/strong-beam mechanism. To achieve flexural strengthening, continuity through the joint in the vertical FRP is required, as recommended by the latest draft ACI 440-F guidelines (ACI 2014).

By means of continuous strengthening, the columns above and below the joint are activated, which results in more symmetric behaviour of the two columns. This delays and potentially prevents single-storey failure mechanisms, enabling higher levels of drift to be achieved in the full structure. Another advantage of providing continuous flexural strengthening is that a better development length can be achieved as well as better anchorage of the FRP sheets (Vrettos et al. 2013). However, practically achieving the vertical continuity of the FRP is complicated for interior joints by the presence of the slab, and has only been addressed by very few experimental campaigns (e.g. Shiohara et al. 2009).

After surface preparation (roughening) of the concrete and rounding of the edges to a radius of 25 mm, a first layer of 250 mm wide CFRP sheet as column flexural strengthening is applied, extending 750 mm onto both columns. To provide continuity of the longitudinal column strengthening sheets through the joint, vertical FRP strands are used. These are inspired by previous efforts by Shiohara (2009), who used FRP anchor ropes at the corners of columns in beam-column joints. Rather than proprietary FRP anchors available in Japan, in this scheme 750 mm wide CFRP sheets, rolled into strands and glued together using epoxy are used (see Fig. 3). As shown in Fig. 4, the strands are passed through plastic tubes and then through holes in the slab at the corners of the columns and the ends are splayed out and bonded onto the columns to serve as fan-type anchors. They are then anchored on the inferior column by a steel plate and pre-stressed at the superior column.

Following CNR-DT-200.R1/2013, the amount of flexural strengthening required is evaluated as six layers of vertically applied CFRP sheet. As shown in Fig. 5(a), this is achieved by applying one layer of vertical FRP sheet on the concrete as base layer and two splayed-out FRP strands (six layers at an angle of about 10° from vertical axis). Next, as indicated in Fig. 5(b), horizontal column confinement wraps are applied to anchor the splayed-out strands, as well as to provide confinement and shear strengthening of the columns. Close to the joint, three layers of 250 mm wide confinement CFRP are applied to the column, these are reduced to two layers of 500 mm wide CFRP wrap 250mm from the column-slab face.

***Scheme RT-A-sw***

The second scheme (RT-A-sw) aims to improve the displacement ductility of the specimen. This is done by selectively weakening the slab in addition to strengthening the column as in RT-A, to ensure a ductile beam failure mechanism that follows capacity design principles. RT-A-sw hence aims to prevent the column-hinging mechanism expected for retrofit A by reducing the strong contribution of the slab and allowing for a more symmetric rotation of the beams. The addition of selective-weakening to the retrofit scheme is inspired by previous research (Akguzel and Pampanin 2012).

For retrofit RT-A-sw, first, the slab concrete and reinforcement are cut along a length of 600 mm (i.e. two column depths, as suggested by Akguzel and Pampanin, 2012) using a circular saw (Fig. 6 (a)). After surface preparation (roughening) of the concrete and rounding of the edges to a radius of 25 mm, the same procedure as for retrofit RT-A is followed, as shown in Fig. 6 (b). However, only 4 layers of flexural strengthening are used in RT-A-sw (two 500 mm wide sheets) due to the weakening of the slab.

***Scheme RT-B-sw***

The objective of retrofit scheme B-sw is to achieve a cyclic performance similar to that of a structure designed to modern guidelines (C-EC8). The retrofit aims to increase the strength of the sub-assembly to reach a level close to 80% of C-EC8. Following the design philosophy in Eurocode 8, FRP flexural strengthening of columns, as well as confinement and shear strengthening are applied to attain a strong-column/weak-beam mechanism. As in retrofit A, to achieve continuous flexural strengthening through the slab and joint, vertical FRP strands are used to connect the bottom and superior column retrofit.

To attain a more ductile and dissipative behaviour, the retrofit scheme promotes the formation of a beam-sway mechanism with a plastic hinge (PH) forming in the beams. In order to maintain joint integrity and avoid yield penetration into the joint core, the scheme ensures the plastic hinge forms at some distance away from the beam-joint interface i.e. at a distance of one beam-depth (450 mm). This PH relocation distance is based on previous work by Eslami and Ronagh (2014).

This is achieved by designing the beam strengthening and slab weakening to achieve a weak section in zone (2) of Fig. 7. As shown conceptually by the three moment diagrams the beam capacity in zone (2) is designed to be low enough to reach yield at this location before beam sections at the beam/joint interface and column sections at the column/joint interface.

The retrofit steps for B-sw are illustrated in Fig. 9. As in retrofit A-sw, selective weakening cuts are first applied in the slab (Step 1). The vertical FRP is then applied, starting with a base layer (Step 2) and FRP strands to connect both columns (Step 3&4). The strands are mechanically anchored using steel anchors. Confinement and shear strengthening is applied as horizontal FRP wraps in Step 5. As for C1-RT-A, for the columns to achieve their target strength a total of six layers of vertical FRP over a length of 750 mm from the column-joint interface and three layers of horizontal wrapping are needed.

The dimensions for the applied FRP in the beams and joint are shown in Fig. 8. In Step 6, two 100 mm wide strips are applied as FRP strands at the top and bottom faces of the beams, through the joint area and along a length of 450 mm (zone 1) for PH relocation to zone 2. The continuous strengthening through the joint area also provides the required anchorage to develop PH relocation capability, similar to the anchorage grooves used by Eslami and Ronagh (2014). The transverse strengthening of the beams (Step 7) consists of 50 mm wide strips spaced at 75 mm and is applied as full wraps through holes in the slabs.

Finally, as the specimen is designed to pre-seismic design codes, the joint shear capacity is very low. Whilst many joint shear strengthening schemes with horizontal (e.g.: Ghobarah and Said 2002) or X-wrapping (e.g.: Pantelides et al. 2008) have been presented in the literature, none have considered the presence of transverse beams explicitly. Here, strengthening of the joint is provided by means of horizontal FRP strands through its core. These consist of two 150 mm wide strips rolled-up and passed through holes at the transverse beam/joint interface (shown as part of Step 6). These strands are then splayed out and extended for 300 mm onto the beams for anchorage. The joint FRP strands are bonded to the concrete as they are passed through the pre-drilled holes. All FRP sheets are additionally anchored using bolted steel plates to avoid end-debonding.

Appropriate development lengths for longitudinal FRP and anchorage by transverse FRP wrapping are ensured according to both, section 4.2.2.5 of the CNR guidelines (CNR 2013) and section 13 in ACI-440 (ACI 2008). Still, based on a thorough literature review (Pohoryles 2017), mechanical anchorage is seen as an important additional component in any flexural FRP retrofit. Steel anchors are hence provided to add a further degree of redundancy and ensure slippage of the longitudinal FRP and brittle debonding failure are prevented at all cost.

***Test set-up and loading***

The loading set-up for the quasi-static cyclic tests is shown in Fig. 10 and pictures of the set-up and specimen in the laboratory of the University of Aveiro are shown in Fig. 11. A constant axial load (N1) of 425 kN is applied through external pre-stress rods, which are pin-jointed at the hydraulic actuator at the top of the superior column and the bottom support of the inferior column. This axial load is applied before the beam supports are fastened. The value of N1 is calculated for a second storey column in a typical residential four-storey RC frame in Europe. To induce a higher axial load in the first storey column, an additional axial load (N2) of 25 kN is applied at the inferior column. The second axial load is applied after beam supports are fastened to induce reaction forces in the beam supports, simulating moments from gravity loading. The beams and slab are simply supported by means of roller supports at their ends, which prevent any vertical displacement, but allow for rotation and lateral movements.

Using a hydraulic actuator, a lateral cyclic displacement ($d_c$) or drift (Δ) protocol with three cycles per increment is applied at the top of the superior column, 1.5 m from the centre of the joint core. The drift values (in ± %) at each increment are: 0.1, 0.2, 0.3, then 0.5 up to 6.0 % with 0.5 % increments. The maximum lateral displacement at 6.0 % drift is 180 mm. The rate of displacement application ranges from 0.1 in the first cycles up to 1.5 mm/second in the last cycles.

The general arrangement of the experimental monitoring equipment is shown in Fig. 12. The specimens are monitored using eight strain gauges (±0.6% accuracy) on the reinforcement (four on the superior column, one on the inferior column, two on the bottom beam bars and one on the top beam bars) and one strain gauge located on one FRP strand. Overall 16 LVDT's (error < 0.025 mm), 28 rectilinear displacement transducers (error < 0.05 mm), four draw-wire position transducers (error < 0.5 mm), and four inductive linear position sensors (error < 0.4 mm) are used to provide data on the deformation and damage evolution in the sub-assemblage.

**Experimental results**

The results of the six full-scale tests are presented in this section and are summarised in Table 5. For all specimens, the global lateral force–displacement behaviour is presented in Fig. 13, where the occurrence of cracking, spalling, buckling, and yielding observed during testing is also indicated. Yield drift, $\Delta_y$, is defined as the drift at which the first strain gauge reading exceeds the steel yield strain (0.2%), while ultimate drift, $\Delta_u$ is defined as that occurring at a strength reduction of 20% from the maximum force ($F_{max}$) (Park et al. 1987). The ultimate displacement ductility, $\mu_{\Delta u}$, is defined as the ratio between $\Delta_u$ and $\Delta_y$. The post-peak softening slope ($S$) is defined as the slope between $F_{max}$ and $F_u$, and is an indirect measure of residual strength of a structure. The cumulative energy dissipation ($E_d$) is defined as the integral of the force-displacement curve, while the energy dissipated by individual members (columns and beams) is calculated from the moment-rotation curves along the length of the members. $K_i$ represents the initial value of peak-to-peak lateral stiffness, $K_p$, expressed in units of kN/mm, which is defined as the slope between the maximum positive and negative force at the first cycle of each displacement level. Finally, the specimen's behaviour upon repeated cyclic loading is evaluated in terms of inter-cycle strength degradation ($F_{deg,1\text{-}2}$), i.e. the reduction in strength at the end of the 1$^{st}$ and 2$^{nd}$ cycles at each level of drift.

***Behaviour of control specimens C1, C1-sw and C-EC8***

The two control specimens designed to pre-1970's guidelines, C1 and C1-sw, show an undesirable brittle failure mechanism characterised by a low displacement ductility and limited energy dissipation (as observed in Table 5 and Fig. 13). The final damage in the superior columns of both specimens are shown in Fig. 14. Their behaviour is dominated by large rotation of the superior column leading to localised plastic hinge formation, followed by concrete crushing, and buckling of the superior column bars just above the column/slab interface. This type of observed failure mechanism can be described as a single-storey column failure, as no significant damage is observed in the rest of the specimen. The experimental observations are in line with predictions from a previously undertaken numerical study (Pohoryles et al. 2015).

For C1, initial cracks are first observed in the superior column during the 0.5% drift cycles. This is followed by yielding of the superior column bars at 0.65%. The beam bars, in turn, do not reach yield due to limited rotation of the primary beams. The relatively low peak lateral force of 63.1 kN is recorded at 1.27% drift. During the associated 1.5% drift cycle, two minor cracks in the beams, as well as in the slab are observed. The cracks spread along the entire width of the slab, perpendicular to the loading direction, with one crack along the transverse beam/slab interface and second parallel one, about 300 mm from the transverse beam.

After plastic hinge formation in the column, concrete crushing and buckling of the column bars just above the column/slab interface is observed. This observation can be attributed to the inadequate spacing of lateral reinforcement, and hence lack of confinement in the columns. The load bearing capacity reduces drastically, and the ultimate state is reached suddenly at 2.3% drift, corresponding to the lowest recorded displacement ductility of 3.6. This inadequate seismic performance of the specimen is also characterised by a low cumulative energy dissipation (32.1 kNm) and significant post-peak softening, hence a low residual strength.

For the control specimen with selective weakening cuts in the slab, C1-sw, the first cracks in the superior and inferior columns also become noticeable during the 0.5% drift cycles. Yielding of the longitudinal column reinforcement above the joint is also observed at a yield drift of 0.48%. Further flexural cracks along the length of the columns are noticed with increasing drift levels. A crack along the transverse beam/slab interface at the end of the weakening cuts is observed at 0.5% drift, confirming the anticipated contribution of the slab bars in the non-weakened region. At higher drift levels (1.0%), the crack extends along the entire weakening cut. For the beam bars, unlike specimen C1, yield is also observed at 0.74% drift for the bottom reinforcement. The slightly larger peak lateral force of 67.5 kN (+7% vs. C1) is recorded at -0.96% drift. After plastic hinge formation in the column, the ultimate state is reached at 1.85% drift ($\mu_{\Delta u}$ = 3.9), with concrete crushing and spalling, as well as buckling of the column bars just above the column/joint interface. Limited energy dissipation (22.7 kNm, -29.2% vs C1) and significant post-peak softening are hence also observed for this specimen.

Overall, selective weakening of the slab alone does not significantly alter the specimen's behaviour, as the same failure mechanism is observed in both control specimens. The weak-column strong-beam strength hierarchy is unaffected by the cuts in the slab, as the columns remain the weakest member. The slight increase in strength (7 %) and ductility (7.5%) can be attributed to the higher concrete strength in C1-sw. The reduction in the total cumulative dissipated energy may be a result of reduced slab contribution.

For the control specimen designed to Eurocode 8, C-EC8, as expected, an improved cyclic performance is observed. As shown in Table 5, it presents a much higher strength of 123.9 kN (+96.4% compared to C1) and ductility of 5.9 (+61.9%). As a result of the improved seismic detailing, a ductile failure mechanism is observed, with damage spread over a larger area in both beams and columns (Fig. 15). Flexural cracks in the columns are noticeable from 0.3% drift, while cracks in the beams are only observed in the 1.0% drift cycles. The high reinforcement ratio of the column and joint ensure that unlike the non-seismically designed specimens, yielding of reinforcement bars is first recorded in the bottom beam bars (0.89%), with yielding of column bars occurring at higher levels of drift (1.6%). For the slab, five parallel cracks are seen in the top and bottom surface, indicating a strong slab contribution. However, the first noticeable crack at 1.5% drift originates at the end of slab away from the joint and then further extends towards the column. The peak force is reached at -3.45% drift, at which point cover spalling is seen at the column/joint interfaces. With increased crushing at the superior column base and the top of the inferior column, strength degradation is observed, and the ultimate force is reached at 5.2% drift, corresponding to a ductility of 5.8.

The final damage state, shown in Fig. 15, reveals that despite adhering to capacity design, the columns govern the ultimate failure, with visible concrete crushing at the column-joint interfaces. This can be attributed to a stronger slab contribution and a higher over-strength of the beam bars than anticipated by the Eurocode 8 design equations, which is discussed in more detail in a separate work (Rossetto et al. 2017). Still, damage in the columns is symmetric in the inferior and superior columns and no buckling is observed, corresponding to a significant improvement in behaviour compared to C1. This improved failure mechanism is also characterized by a more dissipative behaviour (+437.1% vs C1), with reduced post-peak softening (-5.8%) and inter-cycle strength degradation (-54.5%).

***Behaviour of retrofitted specimens C1-RT-A, C1-RT-A-sw and C1-RT-B-sw***

For specimen C1-RT-A, with the simplest column-only retrofit, large cracks at the slab/column and joint/inferior column interfaces are observed (Fig. 16). The failure mechanism ultimately involves both columns, eliminating the single-storey mechanism of C1, as shown conceptually in Fig. 2(b). A higher value of drift is hence necessary to achieve the same curvature in the columns and yield of the column bars is delayed significantly (0.83% drift, +36.4% vs. C1), as shown in Table 5.

Fig. 13 shows that the behaviour of C1-RT-A is characterised by a higher lateral load capacity and reduced strength degradation, but higher pinching of hysteresis loops as compared to control specimen C1. While, no rupture or significant debonding of FRP is observed throughout the test, the increased pinching can be associated to the slippage of the non-bonded FRP strands through the plastic tubes placed along the joint region. This also explains the relatively low maximum FRP strain recorded in the strands (0.11%).

First cracking is observed at the inferior column/joint interface at 0.5% drift. Two initial cracks in the top of the slab, perpendicular to the direction of loading, are also observed at 0.5%. Looking at Fig. 13, the FRP retrofit does not affect the initial stiffness, $K_i$, significantly (+4.6%). At 0.8% drift, yield is observed in the superior column bars, but also in the bottom beam bars. This is rapidly followed by yielding of the inferior column bars at 0.9% drift, highlighting a failure mechanism involving both columns. At 1.0% a first crack in the beam is observed at the beam/joint interface, with a second crack occurring 200 mm away from the joint. The cracks in the top of the slab are now replicated on the bottom of the slab.

At 1.5% drift, a diagonal crack in the bottom of the slab, as well as some torsional cracks in the transverse beams are noticeable. These cracks become wider at larger drift values, with concrete starting to break away at the corner of the transverse beam at 3.0% drift. The cracks at both column/joint interfaces open significantly with increasing levels of drift and at 3.5% drift, the peak force of 87.7 kN (+39.1% vs C1) is reached. After this point the longitudinal FRP on the columns debonds locally in proximity of the column/joint interface. This can be related to concrete crushing underneath the FRP wrap, which is confirmed by post-test inspection. This observation also explains the more pronounced post-peak softening compared to the other retrofit specimens observed for the specimen. At 4.0% drift, the crack along the column/slab interface opens fully, no other flexural cracks underneath the FRP can however be observed along the length of the column, indicating a highly localised failure in the column/joint interface. Between 4.0% and 5.0% the concrete wedges appearing in the transverse beams fully break off.

The strength of the specimen reduces with increased concrete crushing in the column/joint interface. Buckling of the column bars is however successfully prevented due to the FRP confinement wraps, and the post-peak softening behaviour is hence considerably improved (-33.6% vs. C1). With reduced softening, a much larger ultimate drift of -5.25% is consequently obtained, leading to a large ductility of 6.3 (+74.6%). The larger ductility can be attributed to a combination of increased confinement of the column through the FRP wraps and the failure mechanism involving both columns rather than a single column, with quasi-symmetric rotation (+/- 0.44 $m^{-1}$ peak curvature) of the superior and inferior columns throughout the test.

For the specimen retrofitted to scheme A with selective slab weakening, C1-RT-A-sw, a change in hierarchy of strengths is anticipated, resulting in a failure mechanism governed by the beams and joint. Indeed, as shown in Fig. 17 (b), more damage occurs in the beams and hence an improved cyclic behaviour is achieved as compared to C1 and C1-sw.

As the failure of C1-RT-A-sw is governed by the beams, which only have a slightly higher moment capacity then the columns in C1, the increase in lateral load resistance is less significant for C1-RT-A-sw (+13.5 % vs C1) then for C1-RT-A (+39.1%). Compared to the control specimen C1, significant increase in displacement ductility is however observed for C1-RT-A-sw (6.69, +84.9% vs C1), which is larger than that achieved in C1-RT-A (6.32, +74.6%). Due to the prevention of column bar buckling, it can also be observed that the post-peak softening is drastically improved compared to C1 (-66%) and that this improvement is more significant than for C1-RT-A. In terms of dissipated energy C1-RT-A-sw provides the same increase as C1-RT-A (+194.7% vs C1), despite having a lower lateral load resistance.

Cracks in the beam are observed in the bottom face at lower drift levels (0.5% drift) compared to C1-RT-A (1%). Yield is then recorded at 0.78% drift (+27.6% vs C1) in the column bars. This is followed by yield in the bottom beam bars at 1.24 % drift and in the top beam bars at 1.89% drift. The first cracks in the columns are observed at the column/joint interface at 1.5% drift. As a consequence of selective weakening, increased rotation of the beams is observed, leading to a larger number of cracks in the bottom of the beam. At 2.0% drift, five parallel cracks with a spacing of ca. 200 mm are observed in Fig. 17 (b). The crack at the beam/joint interface is observed to open more significantly with increasing drift cycles than for C1-RT-A. This crack is observed to extend along the length of the slab/transverse beam interface indicated by the red circle in Fig. 17 (b). After reaching the maximum force, at 2.5% to 3.0% drift, cracks in the beam top face are also noticed. The observation of cracks in the slab between 2.0% and 3.0% drift indicates slab participation, however not to the extent of specimen C-EC8.

In the transverse beams, torsional cracks are first observed at 1.5% drift, leading to cracks around a wedge at the transverse beam/joint interface. The cracks extend fully at 2.5% and two wedges finally break off at 3.5% on one side of the transverse beams. The observed spalling is more significant than for specimens C-EC8 and C1-RT-A. The torsional cracks are caused by the difference in rotation at the two ends of the transverse beam (joint end and free end). An increased rotation in the main beams, with associated greater joint deformation, explains the importance of the observed wedging.

In the post-peak softening regime, the cracks at the beam/joint interfaces are observed to open significantly. At 4.5% drift, concrete crushing is observed underneath the FRP wraps in the superior and inferior columns, leading up to the ultimate state at 5.2% drift. In the last drift cycles at 6.0% drift, slight debonding in the FRP column wrap near the joint interface is observed.

For the final retrofit specimen, C1-RT-B-sw, a very ductile behaviour with high lateral load bearing capacity is observed. The performance of C1-RT-B-sw presents a significant strength increase of 37.7% with respect to C1, reaching 70.1% of the strength of C-EC8, despite a lower concrete strength than the control specimens. The failure mechanism is again dominated by a large crack in the column/joint interface. However, significant damage and rotation of the beams is also observed, with a plastic hinge forming away from the joint, as anticipated by the design.

The envisaged hierarchy of strengths from the retrofit design is confirmed, with damage spread along the length of the beams and slab as shown in Fig. 18, starting from the envisaged plastic hinge zone, 450 mm away from the joint. This leads to a ductile and dissipative failure mechanism with a very large ductility of 6.9 recorded for C1-RT-B-sw (6.9, +89.6% compared to C1) and a total cumulative energy dissipation of 111.6 kNm (+247.8% vs C1), close to 75% of specimen C-EC8 at its ultimate drift. The lowest inter-cycle strength degradation (-61.8% compared to C1) and a strongly improved post-peak softening (-61.8%) are also observed for C1-RT-B-sw, indicating a significantly improved performance under cyclic loading and improved residual strength, respectively.

In terms of damage evolution, a late onset of yielding (0.95% drift) and cracking (1.0% drift) in the beams is observed. First cracks in the beams occur in the bottom face about 300 mm from the joint, with two further cracks appearing at 450 mm and 600 mm from the joint interface in the next drift cycle (1.5%). At 1.5% drift, three cracks are also visible in the top face of the beam and slab.

At 1.5% drift, a crack at the column/joint interface becomes apparent. With increasing drift levels, a larger opening of the cracks is observed. The maximum recorded strain in the vertical FRP strands remains however significantly lower than the debonding or rupture strain (0.08%). At 2.0% drift, the first crack in the slab bottom face, perpendicular to the main beam axis, is observed about 600 mm from the transverse beams. This is followed by two further parallel cracks at the slab/transverse beam interface and 300 mm from the interface at 2.5% drift. At 2.5% drift, these cracks are seen on the top face of the slab.

In the beams, further cracks at 2.5% drift, about 900 mm from the joint are observed. These cracks extend into the slab at 3.0% drift. At 3.0% and 3.5% drift, diagonal cracks in the slab bottom face are observed, originating from the end of the selective weakening cuts along the main beams. At 4.0% drift, the cracks in the slab extend fully across the width of the slab and a last crack in the beams, around 100 mm from the beam/joint interface is seen.

At 5.0% drift, a partial rupture of the main beam FRP strand is observed. The rupture occurs not due to excessive tensile stress but rather due to a shear mechanism where the strand comes into contact with the transverse beam FRP strand. At 5.5% drift, partial rupture of the transverse beam FRP strand is also observed.

Overall cracking in the beams and slabs extends further than for any other specimen including for C-EC8. Compared to retrofits RT-A and RT-A-sw, due to the strengthening of the transverse beams in RT-B-sw, no torsional cracks in the transverse beams and hence no diagonal cracks at the slab/transverse beam interface are observed. Moreover, due to the joint strengthening, no damage in the joint core is observed after removal of the transverse beams. The FRP strips in the joint reach a maximum strain of 0.12%, indicating that they are indeed activated.

## Analysis of Test Results and Discussion

The analysis and discussion presented in this section seeks to answer whether effective retrofit schemes for realistic beam-column joints can be designed to achieve the initially outlined targets. The three retrofit schemes are compared in terms of several response metrics and assessed in greater detail. This enables a critical comparison on the effectiveness of the three retrofit schemes, evaluating their relative benefits.

### *Lateral load capacity*

The comparison of force-drift envelope in Fig. 19 shows that retrofits RT-A and RT-B-sw are most effective in enhancing the lateral load capacity of the beam-column connection. For both retrofits, an increase in strength close to 40% compared to C1 is observed, leading to a capacity close to 70% of that of the EC8 specimen. This is slightly lower than the strengthening design capacity (80% of EC8), but presents a substantial increase in strength as compared to other efforts in the literature for beam-column connection specimens with slab that show an average strength increase of 26% (Pohoryles 2017). For retrofit RT-A-sw, for which the beams are not strengthened, only a 13.5% increase in strength is observed. This is similar to the 12.8% strength increase reported by Shiohara (2009) for interior joint specimens without slab and transverse beams (i.e. a cruciform specimen) retrofitted with vertical FRP strands. The similarity between the performance of the specimen with selective weakening here and the cruciform specimens of Shiohara (2009) is suggestive that existing retrofit guidance (and implied retrofit effectiveness), which is largely based on experiments with cruciform specimens, could be applicable also to schemes with selective weakening with little modification.

In terms of ductility, there is little difference between retrofit schemes RT-A-sw ($\mu_{\Delta u}$ = 6.7) and RT-A ($\mu_{\Delta u}$ = 6.3). With the plastic hinge relocation in the beams, retrofit RT-B-sw achieves the highest ductility of 6.9.

From Fig. 19 it is also apparent that the post-peak softening behaviour of the retrofits with selective weakening are improved with respect to RT-A. For all retrofits, a better softening behaviour than the control specimens C1 (-490 kN/m) and C-EC8 (-470 kN/m) are obtained. The softening is large for the control specimens as buckling and crushing failures are observed, respectively. For RT-B-sw, a softening stiffness of -190 kN/m is obtained, corresponding to an improvement in softening of 62% compared to the control specimen C1. The softening behaviour is slightly better with retrofit RT-A-sw, with a softening stiffness of -170 kN/m (-66% vs C1). Finally, for retrofit RT-A without selective weakening, the softening is still improved compared to C1 (330 kN/m, -33.6%), but nearly double that of RT-A-sw. This means that for retrofits with selective weakening a larger residual strength is achievable.

***Damage and failure mechanism***

In terms of damage and failure mechanisms, for all three retrofits, compared to the control specimens, buckling of the superior column bars and the single-storey failure mechanism are prevented. From the three retrofit specimens, cracking and yielding are delayed to the highest drift levels for RT-B-sw, making it easier to repair in case of less significant earthquakes.

This observation is particularly evident when looking at the damage evolution for the retrofitted specimens and control specimens C1 and C-EC8, presented in Fig. 20. The damage descriptions accompanying the

HRC damage scale proposed in Rossetto and Elnashai (2003) are used to assign a damage state to the physical observations of the test specimens in the figure. For comparison, the performance levels for concrete frames in ASCE 41 (2017) are also indicated, using the conversion relationship presented between the damage states and the HRC damage scale in Rossetto and Elnashai (2003).

At low drift levels, the retrofitted specimens present a similar performance, with consistently less damage than the control specimens. For all tested specimens, moderate damage is reached for drift levels below 1.0%. Limited yielding is observed for all specimens, which is in line with the definition of the IO (immediate occupancy) performance level of ASCE 41. Moderate to extensive damage with spalling is observed for drift levels above the 2.0% drift, hence complying with the drift limit for the LS (life safety) performance level. Here, all three retrofits outperform the two control specimens. While RT-A displays similar performance to C-EC8, RT-A-sw and RT-B-sw reach moderate to extensive damage at significantly higher drift levels.

Finally, it can be observed that for C1 extensive damage up to partial collapse is reached around 2-2.5% drift, hence not meeting the 4.0% drift limit prescribed for collapse prevention (CP). This is expected for a specimen designed to pre-1970's guidelines. In turn, all retrofitted specimens and C-EC8 reach their ultimate point (partial collapse) after 4.0% drift, and hence present adequate behaviour with respect to the limits in ASCE 41. While retrofits RT-A and RT-A-sw show a similar performance to C-EC8, retrofit RT-B-sw clearly outperforms them, reaching the ultimate drift around 6.5 %.

The three retrofit schemes are also assessed for their ability to move damage from the columns and joint to the beams. As shown in Fig. 21, for retrofit RT-A the lowest curvatures along the beams are observed, while for the retrofits with selective weakening, RT-A-sw and RT-B-sw, much larger and symmetric curvatures in hogging and sagging are recorded together with significant damage along the beams.

The greatest level of damage distribution along the beam is produced by retrofit RT-B-sw, where the initial cracks in the beams are also formed further away from the beam-joint interface (i.e. at 500mm) than for retrofit RT-A-sw (i.e. at 0mm). The success of retrofit RT-B-sw in relocating the plastic hinge along the beam, away from the beam-joint interface, is confirmed by the highest curvatures in hogging and sagging recorded for this specimen at zone 500 mm along the beam, as shown in Fig. 22. The curvatures recorded are about three times higher than for retrofit RT-A-sw.

However, overall the beams are most effectively activated in specimen C1-RT-A-sw, as highlighted by the plots of relative contribution of the different components of the specimens to the total energy dissipation in Fig. 23. These plots are prepared by calculating the energy dissipation of the different members from their moment-rotation curves at different sections along the length of the members (Fernandes et al. 2011; Melo 2014). For the control specimens (C1 and C1-sw) over 80% and for C1-RT-A and C1-RT-B-sw over 70% of the total energy dissipation is dissipated by the columns. This is significantly reduced to 50 % in C1-RT-A-sw, with 26% of the total energy dissipated by the beams and 24% by the joint, slab and transverse beams.

Despite this observation, Fig. 24 shows that C1-RT-B-sw and C-EC8 have very similar energy dissipation plots at component level, suggesting that this retrofit achieves an acceptable energy dissipation compared to modern seismic design. For C1, only 2.4% of the total cumulative energy is dissipated by the beams and slab, while for C-EC8 (12.4%) and C1-RT-B-sw (14.4%) a more significant proportion of the total energy dissipation is due to the beams. It is also noted that, for retrofit RT-A and RT-A-sw, significant damage is observed due to torsional forces in the transverse beams. This is effectively prevented by the FRP placed in the first 450 mm of the beams in RT-B-sw. Similarly, damage in the joint observed for C1-RT-A-sw is effectively prevented by the joint shear strips in retrofit RT-B-sw.

Overall, in terms of damage and failure mechanism, it is concluded that RT-B-sw is the most advantageous, requiring the least repair in moderate earthquakes and performing well in relocating damage away from the joint. Retrofits RT-A and RT-A-sw are still seen to achieve their respective targets and provide improved behaviour compared to the control specimens.

***Dissipated Energy***

All proposed retrofitted specimens show a significant improvement in total cumulative energy dissipation compared to the control specimen C1. For retrofits RT-A (93.8 kNm) and RT-A-sw (94.5 kNm), similar levels of energy dissipation are reached, corresponding to an increase of nearly 200% compared to C1. For retrofit RT-B-sw (111.6 kNm), the increase is even larger (+247.8 %), reaching about 65% of the dissipated energy of the EC8 specimen (172.3 kNm) at the maximum drift level. Looking at the plot of dissipated energy against ductility in Fig. 25, the evolution of energy dissipation follows a similar path for the control specimen and the retrofit specimens at low levels of ductility. It can be observed that for retrofit

RT-B-sw at the same level of ductility, the dissipated energy is about 20% higher than for RT-A and close to 40% larger than for RT-A-sw. The retrofit also reaches similar levels to the targeted 80% at the maximum ductility of the specimen designed to EC 8. From the three retrofits, RT-B-sw is hence also the most dissipative, nearly reaching the target performance of 80% of EC8 at equivalent ductility levels.

***Stiffness degradation***

The improved ductility and dissipative behaviour of the specimen retrofitted to RT-B-sw, with delayed onset of cracking and yielding, is also reflected in the degradation of peak-to-peak stiffness shown in Fig. 26. In order to assess the degradation in stiffness more objectively, the peak-to-peak stiffness values are divided by the initial stiffness, $K_i$, for each specimen. It can be observed that the evolution of stiffness degradation for the retrofit RT-A-sw is similar to the control specimen C1. For RT-A, the improvement is marginal, while for RT-B-sw a better performance, close to C-EC8 is observed.

***Inter-cycle strength degradation***

Finally, the last metric used to assess the performance of the retrofits is the inter-cycle strength degradation ($F_{deg}$) between the first and second, as well as the first and third cycles. The $F_{deg}$ is a diagnostic indicating the resilience of a specimen to repeated loading, which is of crucial relevance in real earthquake events (Ibarra et al. 2005). At increasing levels of drift, the corresponding increased damage reduces the specimens' ability to perform consistently under repeated loading, as shown by the generally increasing $F_{deg}$ values with drift for all specimens.

For the control specimens, relatively low $F_{deg}$ values are observed at low drift values, but at the ultimate drift, due to the observed column bar buckling, very high values of $F_{deg,1\text{-}2}$ (above 25%) and $F_{deg,1\text{-}3}$ (above 65%) are obtained. For the retrofit specimens, instead, no sudden increase to high values of strength degradation is observed. When comparing the three retrofit specimens, it can be observed that for RT-A, similarly to C-EC8, the $F_{deg,1\text{-}2}$ remains constant around 10% after 2% drift, while it increases up to 15% for RT-A-sw. The best performance is again obtained with retrofit RT-B-sw, for which a maximum strength degradation of 8.9% is observed.

After the third cycle, the strength degradation increases for all specimens, with C1-RT-A ($F_{deg,1\text{-}3}$ = 20.3%) and C-EC8 (18.7%) again performing similarly, while C1-RT-A-sw displays the largest strength degradation (24.4%). The performance of retrofit RT-B-sw remains consistently good, with a slowly increasing $F_{deg,1\text{-}3}$ up to 15.7%. Analysis of inter-cycle strength degradation hence shows that retrofit B-sw is the most effective at reducing strength degradation upon repeated cycling, even outperforming the specimen designed to modern guidelines.

**Conclusions**

In this paper, the results from six full-scale interior beam-column joint tests are presented with the aim of assessing the effectiveness of three different CFRP retrofit schemes. To ensure practical applicability to real structures, the retrofit scheme design considers the realistic geometric challenges and the specimens include slab and transverse beams.

The proposed and implemented retrofit solutions provide very satisfactory improvements to the behaviour of as-built specimens. Overall, the combination of fan-shaped splaying of the strands, mechanical anchorage with steel plates and FRP anchorage with horizontal wraps proves successful in avoiding significant debonding.

The simplest, column-only, retrofit scheme A, is shown to improve the specimen's strength and ductility. The FRP confinement wrapping successfully prevents rebar buckling and the FRP strands avoid the single-storey failure observed for the control specimens, hence improving the ductility of the specimens. The retrofit scheme with selective weakening, retrofit A-sw is shown to be most effective in increasing ductility, due to significantly increased beam rotation. In terms of strength, only a 13.5% increase in lateral load capacity is observed.

Overall, retrofit RT-B-sw can be seen to achieve the most significant improvement in seismic performance by all relevant diagnostics, most crucially in terms of lateral load capacity (+38% to C1), ductility (+90%) and post-peak softening (-62%). The load capacity of 70% of the EC8 specimen, is lower than anticipated (80%), but still a substantial improvement to proposed retrofits for specimens with slab in the literature (26% average strength increase). Finally, relocation of damage along the beams is also successful and leads to the most dissipative mechanism (+248%), reaching the target performance of 80% of EC8 in terms of dissipated energy. Full relocation of the ultimate failure to a beam-only mechanism is however not accomplished.

Retrofit B-sw is shown to have great potential for practical implementation. It is anticipated that this scheme can be further improved in future work. As the retrofit effectiveness depends strongly on the strain reached in the FRP strands, anchoring the FRP strands in the joint region to reduce the free, non-bonded length, is suggested.

Finally, the results on joints with selective weakening suggest that current retrofit design guidance, developed for cruciform joints, can be applied when the slab contribution is reduced by sw cuts. This observation however needs to be tested further by means of experimental and numerical work on different joint geometries, e.g. cruciform specimens. Similarly, potential applicability of the retrofit procedure to exterior or corner joints would need to be tested further.

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**Table 1.** Summary of reinforcement details for pre-1970's and EC8 specimens

| Spec. | Beams | | | | | | | | Column | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Main bars | | | | | Shear | | | Main bars | | | Shear | | |
| | $d_{bl}$ | top | $\rho_l$ | bot | $\rho_l'$ | $d_{bw}$ | $s$ | $\rho_w$ | $d_{bl}$ | # | $\rho_{tot}$ | $d_{bw}$ | $s$ | $\rho_w$ |
| | mm | # | % | # | % | mm | mm | % | mm | | | mm | mm | % |
| All | 12 | 4 | 0.34 | 3 | 0.25 | 8 | 200 | 0.17 | 12 | 8 | 0.01 | 6 | 150 | 0.13 |
| C-EC8 | 16 | 2 | 0.30 | 2 | 0.30 | 6 | 100 | 0.19 | 25 | 8 | 0.04 | 8 | 80 | 0.42 |

**Table 2.** Concrete strength and its standard deviation ($\sigma_{fcm}$) for all specimens

| Specimen | $f_{cm}$ (MPa) | $\sigma_{fcm}$ (MPa) |
|---|---|---|
| C1 | 23.4 | 0.4 |
| C1-sw | 26.0 | 0.5 |
| C-EC8 | 32.7 | 0.3 |
| C1-RT-A | 23.8 | 0.6 |
| C1-RT-A-sw | 22.0 | 0.7 |
| C1-RT-B-sw | 19.3 | 0.1 |

**Table 3**. CFRP material properties.

| Property | Value |
|---|---|
| $t_f$ | 0.223 mm |
| $f_{u,FRP}$ | 3232 MPa |
| $\varepsilon_{u,FRP}$ | 1.7% |
| $E_f$ | 194.1 GPa |

**Table 4.** Steel reinforcement material properties.

| Property | $f_y$ / $f_u$ (MPa) |
|---|---|
| Φ25 | 595/695 |
| Φ16 | 585/687 |
| Φ12 | 450/570 |
| Φ10 | 530/620 |
| Φ8 | 540/640 |
| Φ6 | 538/645 |

**Table 5.** Summary of experimental results

| Specimen | $F_{max}$ | Ultimate | $\Delta_y$ | $\mu_{\Delta u}$ | $E_d$ | $S$ | $K_i$ | $F_{deg,1-2}$ |
|---|---|---|---|---|---|---|---|---|
| | (kN) | damage | (%) | $\Delta_u$ / $\Delta_y$ | (kNm) | (kN/mm) | (kN/mm) | (%) |
| C1 | 63.1 | Sup. column | 0.65 | 3.62 | 32.08 | -0.49 | 6.60 | 23.22 |
| C1-sw | 67.5 (7%) | Sup. column | 0.48 (-21.8%) | 3.89 (7.5%) | 22.7 (-29.2%) | -0.4 (-18.4%) | 7.71 (16.7%) | 27.86 (20%) |
| C-EC8 | 123.9 (96%) | Column, Beams | -0.89 (46.4%) | 5.86 (61.9%) | 172.28 (437.1%) | -0.47 (-5.8%) | 7.18 (8.7%) | 10.57 (-54.5%) |
| C1-RT-A | 87.7 (39%) | Both columns | -0.83 (36.4%) | 6.32 (74.6%) | 93.84 (192.5%) | -0.33 (-33.6%) | 6.91 (4.6%) | 10.86 (-53.2%) |
| C1-RT-A-sw | 71.6 (13%) | Beams, Joint | 0.78 (27.6%) | 6.69 (84.9%) | 94.53 (194.7%) | -0.17 (-66%) | 6.39 (-3.2%) | 13.92 (-40.1%) |
| C1-RT-B-sw | 86.9 (38%) | Beams, Column | 0.95 (55.7%) | 6.86 (89.6%) | 111.57 (247.8%) | -0.19 (-61.8%) | 5.65 (-14.5%) | 8.87 (-61.8%) |

Note: Values in parentheses are % difference to C1