USE OF FRP FABRIC FOR STRENGTHENING OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

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ABSTRACT

Devastating earthquakes in the last 3 years have shown that non-engineered concrete frames are particularly vulnerable to seismic action and are a major cause of loss of lives. This structural type constitutes a large share of the building stock, both in developed and developing countries, and hence represents a substantial exposure. Direct observation of damaged structures, following the Kocaely, Turkey 1999 earthquake, has shown that damage occurs usually at the beam-column joints, with failure in bending or shear, depending on geometry and reinforcement distribution and type.

While substantial literature exist for the design of concrete frame joints to withstand this type of failure, after the earthquake many structures were classified as slightly damaged and, being uneconomic to replace them, at least in the short term, suitable means of repairs of the beam column joint area are being studied. Furthermore there exist a large number of buildings that need retrofitting of the joints before the next earthquake.

The paper reports the results of cyclic tests carried out on cruciform beam-column joint specimens, with two different configurations of geometry and various configuration of strengthening by externally bonded FRP fabric. The specimens were designed to comply with gravity load design codes, but no seismic design was considered. In the design of the FRP wrapping, two different type of fabric were considered and three layout of the wrapping strips.

INTRODUCTION

In the last decade, the effect of external application of fibre-reinforced polymers (FRP) plates and wraps to reinforced concrete beam-column joints to increase their performance has been investigated theoretically and experimentally. Previous to this, steel jackets were used to reinforce the joint area, as well as the use of R.C. jackets (Baraka and Prion, 1995). The use of flat and corrugated steel plates, have also been investigated (Beres et al, 1992; Ghoborah et al, 1997). However, these options have been found to be labour intensive, requiring a high level of workmanship, and often add considerable weight to the elements. Additionally, steel plates need corrosion protection and their attachment requires either the use of epoxy adhesives combined with bolts, or special grouting (Antonopoulos, 2001).

External application of FRP material provides a practical solution to improve the overall performance of an R.C. frame structure without the necessity of a radical alteration to the original structure. Externally bonded FRP may be used in a repair capacity for structures that have undergone moderate earthquakes damage or to reinforce structures that are considered to be vulnerable or substandard. The use of FRP offers several advantages, related to its high strength-to-weight ratio, resistance to corrosion, fast and relatively simple application (Karbhari, 2001). However, FRP is to date still rather expensive so its use must be optimised to minimise material wastage. One disadvantage of FRP is its dependence on bond to the concrete it is to strengthen, which is a function of the tensile capacity of the concrete and the type of surface preparation used.

Various authors (Bakoss et al., 1999, Elsanadedy and Mosallam, 2000, Ghobarah and Said, 2001, Granata and Parvin, 2001) have conducted tests on different layout of FRP fabric and sheets bonded to R.C. beam-column connections. The tests all concord on the effectiveness of the strengthening procedure to increase stiffness and ductility while increases in shear and flexural strength and in energy dissipation are highly dependent on proper confinement of the concrete and anchorage of the wrapping. Extensive comparative testing carried out by Antonopoulos et al (2001) allows drawing a few general conclusions on previous work:

- Debonding dominates the behaviour of external FRP reinforcement unless low area fractions (relates amount of EFRP to amount of concrete) were used or mechanical anchorages provided
- Flexible fabrics were more effective than plates (for the same area fraction).
- The effect of column EFRP reinforcement is rather limited.
- Wrapping of longitudinal FRP sheets with transverse layers is a highly effective anchorage mechanism.
- The effect of high axial column load has a positive effect on FRP-strengthened joints.
- Effectiveness of the FRP increases as the amount of transverse steel in the joint decreases

One other important issue, when studying the seismic behaviour of low-engineered concrete frames, is the effect of use of plain round bars. Liu and Park (2001) have proven that there are important reduction in strength, up to 25% as compared with specimens using deformed bars, while the flexibility is up to twice greater. Both phenomena are associated with bond degradation and hence the assumption of plane section does not hold any longer.

In comparison to the amount of experimental research undergone in recent years, there has been relatively little in the way of analytical models of FRP reinforced beam-column joints. This is mainly due to the complexity of behaviour of beam-column joints under cyclic load. Behaviour of the joint core exhibits shear and bond stress, as well as the phenomenon of confinement. Fatigue effects under cyclic loading cause the behaviour of the joints to be dependent upon loading pattern. The lack of a standard testing procedure has made direct comparison between different tests highly difficult.

Analytical work on beam-column joint behaviour was first developed by Bonacci and Pantazopoulou (1992) who proposed a formulation based on compatibility of strain and stress equilibrium within the core. This model showed that shear strength of a joint depends on usable capacity of concrete as well as the presence of shear hoops and increased vertical load. Based on this work, Triantafillou (1999) extended the model to include the presence of externally bonded reinforcement, either plate or fabrics. Good agreement with experimental results was obtained.

Despite the amount of research that has been carried out in recent years on FRP reinforced beam-column joints, there has been no research on the effectiveness of FRP strengthening of beam-column joints typical of building construction in Europe before the introduction of seismic codes.

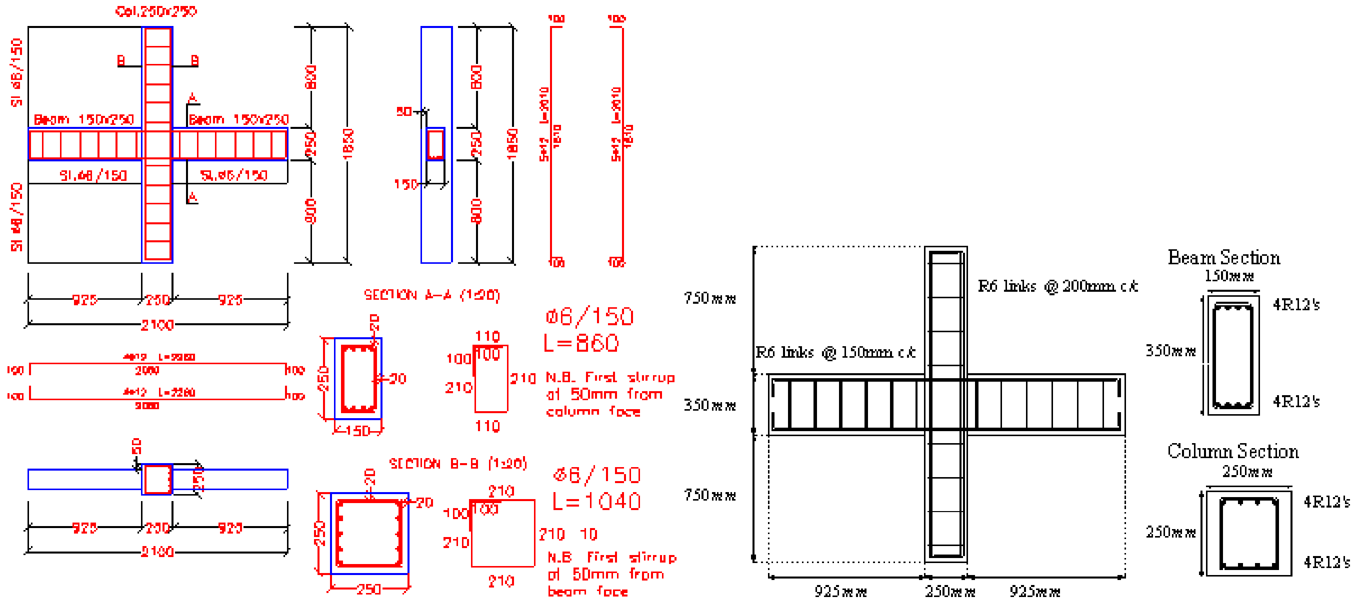
These frames were typically designed to gravity loads only, detailed with mild steel smooth bars and often providing strong beam/weak column arrangements. Also usually frame are complete in one direction only of the two orthogonal plan directions. Buildings designed to these specifications have dramatically failed to withstand earthquakes in the recent events in Turkey and India. Development of a feasible means of strengthening would be invaluable in terms of reducing vulnerability of these buildings and saving lives. In particular, this work focuses on a possible worse case scenario for a pre-seismic code building combining the following structural deficiencies:

- Plain round bar, mild steel reinforcement
- Inadequate transverse links in the joint core
- Weak concrete
- Strong beam/ weak column set-up for one set of tests.

This work aims to validate the use of FRP for strengthening and repair of such buildings, transforming them from potentially dangerous to ones that will remain stable during an earthquake.

SPECIMENS DESIGN

The specimens were designed following the standards and provisions of the Italian code of practice issued in 1960, based on permissible stresses. The material chosen were concrete R_{ck} =20 Mpa and steel FeB32K mild steel in smooth bars. Two configurations, one governed by beam failure (WB) and one governed by column failure (SB) were considered. The geometric dimensions for the two prototypes and their reinforcement details are summarised and Figure 1. For each configuration six specimens were cast.



a) b)

Figure 1 Geometry and construction details for the specimens: a) WB configuration; b) SB configuration

Concrete cubes were tested for each casting, comprising 3 specimens. 'C20' concrete was used for the first two batches and 'C15' and 'C10' concrete for the second two, all mixes delivered by a ready mix lorry. The three classes of concrete chosen reflect observation in situ of damaged and collapsed buildings in Turkey (D'Ayala 2003).

Tensile tests were also carried out on the steel used for the reinforcement, on 4 specimens of bars of diameter 8, 12, 16 mm. The average yielding stress is 353 Mpa, the failure stress is 480 Mpa, and strain at yield is $\varepsilon_{s,y} = 0.0022$. No transverse links were provided in the joint core, and shear links in the column and the beams started at 50mm from the faces of the core. All steel reinforcement used was plain round mild steel bars.

DESIGN OF THE FRP STRENGTHENING

In designing the FRP reinforcement the various failure modes of the reinforced concrete cross section /FRP wrapping need to be considered and provided against in the detailed design. FRP are usually applied to concrete either in the form of rigid plates or flexible fabric. According to studies conducted in the last ten years (Chajes et al. 1994) the following failure modes can be identified: flexural failure due to either compressive failure of the concrete or tensile failure of the FRP, usually accompanied by yielding of the longitudinal steel bars; shear failure if the FRP application shifts the critical condition of the element to shear capacity; failure of the bond connection between the plate or fabric and the cover concrete or failure of the bond between the cover concrete and the internal steel reinforcement. Either of these can occur at the end of the plate or along the plate at the location of flexural or shear cracks. The design should avoid premature bond failure and non-ductile shear failure of the structural elements. Hence the layout of the wrapping should not only increase the bending capacity of the cross section but also ensure proper anchorage of the strips and control the final shear capacity.

Three different layout of the strengthening have been considered in relation to the pre-existing weakness of the assembly. Hence for the first batch of specimens, characterised by a weak-beam configuration (WB), strengthening layout W1 was implemented, with two different FRP materials, Mbrace C1-30 and Mbrace C5-30, which differ in stiffness and strength. For the second batch of specimens characterised by strong beam- weak column configuration (SB), two further layouts W2 and

W3 were considered along W1. The design philosophy behind each strengthening layout is briefly discussed in the following.

Layout W1 consists of corners strips positioned at the beam-column junctions to increase the moment capacity of the beam section continuous over the support (Granata and Parvin, 2001). For these to be effective, it is essential to include beam and column wraps, laid on top, which will prevent the debonding of the longitudinal strip or the failure of the cover concrete in tension. The effectiveness of this type of reinforcement is dependent on how well the corner strips at the corner of the beam-column junction are placed during application, as well as the relative strains required to mobilise delamination at this point and tension in the wraps, respectively. Placing corner strips on either side of the column and beam also provides a mechanism for shear transfer between the column elements. This allows shear to still be transferred between beam and column even after degradation of the joint core. The vertical and horizontal strips placed on column and beam face, respectively, are in place to increase the strength and stiffness of the joint core. Again, premature delamination is prevented by the beam and column wraps, which are laid on top. The orthogonal beam and column strips work by restraining joint deformations, acting in tension to resist the opening of tensile cracks on the face of the joint, as well as in shear to prevent shear distortion of the joint core.

Type W2 is designed for the weak column- strong beam typology, and is aimed at improving the bending capacity of the column and the shear capacity of the joint. It consists of thin rectangular strips, wrapped diagonally around the joint core, with narrow strips positioned at the beam-column junctions. The portion stuck to the column is wider than that stuck to the beam, so as to give a greater increase in moment capacity for the column than the beam. Overlaid on top of the diagonal and corner strips full beam and column wraps are included. The diagonally wrapped rectangular strips reinforce the joint core directly by confinement of the concrete, allowing a diagonal compressive strut to form and transfer shear across the joint. In particular they confine the concrete at the compressive corners of the joint as well as preventing the formation of tensile cracks. These strips are laid beneath the beam and column wraps to help reduce premature debonding.

Type W3 is similar to type 'W2' as far as the diagonal strips are concerned. However, the strips at the beam-column junction are omitted, and instead a 300mm wide vertical strip is stuck to the front and back column faces before the diagonal wrapping is applied. The vertical strip is principally in place to further increase the shear capacity of the joint as well as confining the concrete at the corners of the joint core in conjunction with the diagonal wrapping.

For the first batch of specimens, with the geometric dimensions and the reinforcement chosen, and the FRP reinforcement consisting of strips of dimensions 100mm width and 0.165mm depth, the ultimate bending moment capacity of the beam, assuming failure of the FRP in tension, are 40.2 kNm and 50kNm, corresponding to either C1-30 or C5-30 FRP, being used, respectively. These represent increments of 47% and 82% of the corresponding unstrengthened cross sections.

In the second batch of specimens, only product C1-30 was used. The bending moment capacities of the strengthened cross-section are 33.9KNm for the column and 47KNm for the beam, with an increment of 9.35 and 1.3%, respectively. However this is not sufficient to shift the behaviour from weak to strong column.

The FRP area used is smaller than the critical area causing the crushing of the concrete and hence the strengthen cross section can still be considered under reinforced and the failure will actually occur by tensile failure of the FRP as assumed in the design calculations.

Having increased the bending moment capacity in both series of specimens the shear failure becomes critical and specific strengthening is introduced to prevent this. In the strengthening configuration W1 this is achieved by inserting face strips on the beam and column, in configuration W2 this role is played by the diagonal strips and in configuration W3 by both the diagonal strips and the vertical strip on the face of the column.

The possible failure mechanisms are similar to what seen above, crushing of concrete, and/or failure of the FRP in tension or debonding. The design criterion used in this case is the failure of the concrete.

Once the reinforcement has been designed for bending and shear requirements, the issue of the end debonding of the FRP strips needs to be addressed and provided for. On the basis of new experimental evidence, which seems to disprove the previous assumption of interaction between bending moment and shear force in the end peeling phenomenon, Smith & Teng (2000) have recently proposed a new model which also accounts for the observation that the shear force causing debonding is greater than the shear capacity of the concrete:

$V_{db} = 1.2 V_c$, with V_{db}, shear force causing debonding, and V_c calculated according to EC2 provisions. The limit of validity of this formula is $\frac{M_{end}}{M_u} \leq 0.72$, where M_{end} is the value of bending moment at the cross section coinciding with the end of the strip and M_u is the ultimate bending moment capacity of the beam. In the present case this condition is met for both groups of specimens and hence the limitation on the shear force is complied with. The limiting value is substantially lower than the capacity calculated on the basis of the design provisions, and perhaps over conservative, as it does not take into proper account the various phenomena contributing to the bond strength between cover concrete and FRP. It is worth noting that the above condition is defined on the basis of work done with FRP plates rather than strip and hence the bonding mechanisms might be rather different.

TESTING EQUIPMENT AND PROCEDURE

The test rig used, shown in Figure 2, makes use of the strong walls and floor of the Structures laboratory facilities at Bath University. The base of the column is connected to the floor using a steel hinge connection, which allowed the column to rotate about its base during testing. The ends of the beams are connected to the ground by 30mm threaded bars, with hinge connections at their base to allow moderate lateral movement. Load cells were incorporated along the length of the threaded bars to measure the vertical reactions in the beams. This arrangement reproduces in the specimen the type of deformation set in frames by seismic action.

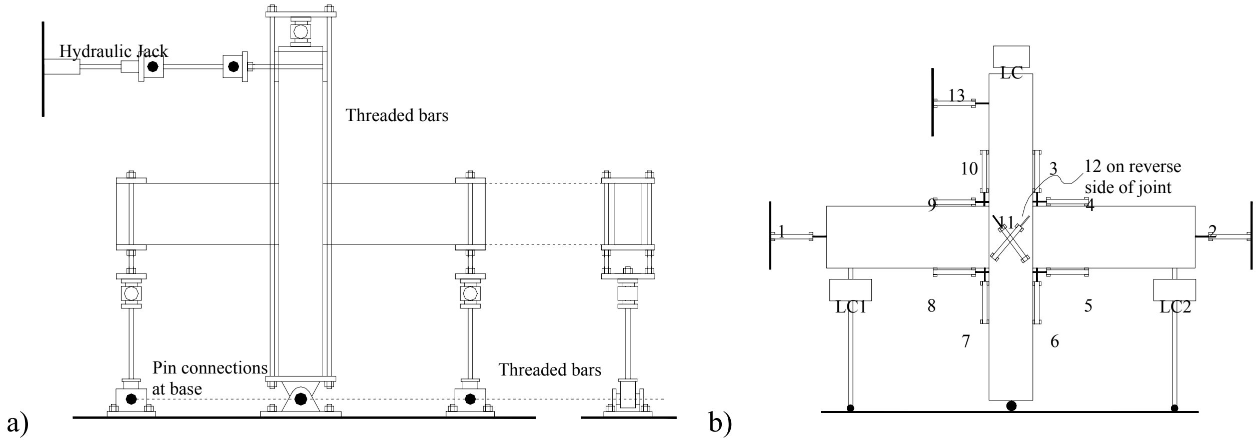


Figure 2: a) Test apparatus and b) Transducers position

Before loading the specimen, a 10kN downwards load was applied to both beams by tightening of bolts at the end of the threaded bars. This represents floor loading. A constant vertical load of 70kN, also representing gravity loads, was applied to the column by tightening bolts on four vertical threaded rods attached to steel plates at the top and bottom of the column. A load cell at the top of the column and strain gauges along the bars measured maximum variation of this load of about 12% during the largest cycles.

Horizontal loading was applied to the top of the column using a 50kN two-way hydraulic jack, with in-built load cell, which allowed cyclical load to be applied. The jack was connected to the top of the column via a bar with two hinges at its extremities so as to let the top of the column free to rotate during the lateral displacement.

A total of 13 displacement transducers were used (see Figure 2b) to monitor movement around the joint core, at the ends of the beams and at the top of the column. All transducers, load cells and the loading jack were connected to a data acquisition system, which recorded the data in the form of a spreadsheet.

The horizontal loading sequence is applied in a series of displacement controlled cycles to predetermined column top displacements in multiples of 6mm starting from 6mm up to 48mm, in both the push and pull directions. For each displacement three loading cycles were performed. Displacement was controlled manually at a quasi-static rate. In all 16 specimen were tested, 8 for each batch, of which two unstrengthened, two repaired and strengthened and 4 with different layout or strengthening material.

OBSERVED BEHAVIOUR

Unstrengthened specimens

As expected, for the weak beam configuration, in the two control specimens initial cracks form at the beam –column connection on the upper face of the beam, for 6mm displacement of the head of the column. As the displacement increases, for 12 mm the beams show a second series of cracks corresponding to stirrups position, while the initial crack at the beam-column face progresses along the beam cross section and, for 18 mm displacement, travels horizontally to the column, just above the joint. For 24 mm the mid span cracks open further and assume an inclined direction, as the shear force in the beam increases. Also at this stage the initial crack reaches the joint panel with a horizontal profile on the face of the column. At 36 mm the concrete starts spalling from the bottom and top edge of the beam at the location of the initial crack. No loss of material is observed in the joint panel. The maximum displacement reached is 45 mm for WB-S1-U and 44mm for WB-S2-U. The maximum values of stress resultants in bending and shear for each specimen are summarised in Table 1. It can be noted that while in the case of WB-S1-U only 70% of the ultimate bending moment design value was achieved, in the case of WB-S2-U the ultimate bending moment capacity was fully developed.

a)  b) 

Figure 3.a) Detail of bending failure:crack opening at beam column interface in specimenWB-S1-U
b) Detail of bending failure in the column and shear failure of the joint in specimen SB-S2-U

For the strong beam configuration, the crack pattern was also initiated at the beam column interface, corresponding to 6 mm of horizontal displacement. However, at 12mm displacement the beam-column interface cracks started extending horizontally into the column at the top and bottom of the joint core. Also, more hairline flexural cracks appeared on the top half of both beams at 200mm or more distance from the joint. At 18mm displacement, spalling occurred at all four corners of the joint core, increasing in severity with increasing displacement. At 24mm, diagonal shear cracks in the joint core were observed. At 36mm horizontal (flexural) cracks appeared in the column approximately 100mm above and below the beam top and bottom beam faces respectively. Diagonal shear cracks in the joint core continued to extend as the test progressed, with additional diagonal cracks appearing, accompanied by further spalling in both the joint core itself and at the corners of the joint core. The column fails prematurely in bending, with only 63% of its bending capacity having been mobilised for specimen SB-

S1-U and 44% for SB-S2-U, where failure is concentrated in the joint. This is due to the poor quality of the concrete of the second batch of the second series.

Repaired specimens

All repaired specimen show stiffness sensibly lower than the corresponding control specimen, notwithstanding the repair of the cracks. This is due to the lack of bond between the longitudinal reinforcement and the concrete. In the WB configuration for 6 mm of displacement the cracks in the column-beam joints reopened readily, and substantially increase for the next increment of lateral displacement. At 30 mm of displacement the cracks run horizontally onto the face of the column and then diagonally into the joint, shown by significant readings in the diagonal transducers. The longitudinal strip on the upper face of the beam tends to debond forming a bulge. For 42 mm a horizontal crack cross completely the column face just above the joint and a plastic hinge forms at this location, its rotation controlled by the presence of the FRP. The inspection of the specimens after removal of the FRP reinforcement confirms the presence of a diagonal crack in the joint and a horizontal crack in the column.

Table 1: Summary of stress resultant in beam and column

Specimen	Maximum displacement (mm)	Maximum vertical load (KN)	Shear in beam (KN)	Bending moment in beam (KNm)	Horizontal force (KN)	Bending moment in column (KNm)
WB-S1-U	45	78	22.2	18.9	28.6	25.7
WB-S1-R-C1	42	83	29.3	27.3	30.4	27.4
WB-S3-C1	42	85	28	25.9	33.6	30.2
WB-S5-C1	50	85	35.8	33.2	29.1	26.2
WB-S2-U	44	80	29.6	27.4	30	27
WB-S2-R-C5	42	73	39.8	36.9	37.7	28.27
WB-S4-C5	42	73	44.5	41.2	40	30
WB-S6-C5	37	80	43.5	40.3	38.7	29.03
SB-S1-U	36	73	43.1	37.8	28.9	19.7
SB-S1-R-W1	48	75	44.5	38.9	27.4	18.7
SB-S3-W1	36	76	43.2	37.8	33.9	23.1
SB-S2-U	36	73	29.7	26.0	20.4	13.9
SB-S2-R-W2	48	77	35.2	30.8	20.8	14.2
SB-S4-W2	48	78	43.9	38.4	39.3	26.7
SB-S5-W3	48	76	39.2	34.3	29.8	20.3
SB-S6-W2	42	77	33.5	29.3	26.9	18.3

A similar pattern is observed in the SB specimen with strengthening configuration W1. However the magnitude of the cracks and spalling is heightened by the poorer quality of the concrete. Delamination of the FRP, on the face of the joint, occurred at 18mm displacement, and this increased with increasing displacement, along with delamination of the corner strips at the beam-column junction. At the end of the test, bulging was observed in the column wraps at the sides of the column, suggesting spalling of the concrete underneath, and almost total delamination of the FRP on the joint core. The W2 strengthening configuration proved more effective for small displacements, however the stiffness of the specimen was very low. At 18mm cracks at the beam-column interface were observed, followed by some delamination of the diagonal strips of FRP at the corners of the beam-column joint on the side of the column. Bulging of the FRP occurred at 36mm displacement, accompanied by some delamination on the front and back faces of the joint core

Strengthened specimens configuration W1

For the WB specimens strengthened with C1-30, the first fissures occur at 6 mm at the beam –column interface in the concrete exposed zone, while further hairline cracks open at a distance of 80 to 100mm from the outer edge of the FRP wrapping. For 24 mm of displacement horizontal cracks appear on the column sides and face in the space between the FRP strips. The cover concrete starts spalling from the column corners for a displacement of 30mm. For 36mm the horizontal cracks on the column start opening and propagate vertically toward the centre of the joint. At 42 mm the FRP delaminates from the upper beam-column face. After removal of the wrapping the joint core shows airline diagonal cracks. A similar behaviour has been observed for the specimens strengthened with C5-30. However this delivers even higher capacity for the beam while undergoing smaller deformations.

For the SB configuration specimen the initial behaviour was remarkably similar to the one of the WB configuration, however at 36mm displacement, horizontal splitting occurred in the column wraps at the point where the vertical column strip underneath finished. Other horizontal cracks were observed in the column just above and below the upper and lower column wraps, respectively. At this point major delamination of the FRP was observed on the joint core face (probably initiated by shear cracking in the joint itself). Spalling was observed at the corners of the joint core (although not as severe as in the unstrengthened or repaired case). Removal of the FRP revealed diagonal shear cracks in the joint and loose concrete at the corners of the joint core.

Strengthened specimens configuration W2 and W3

Flexural hairline cracks were observed in the top half of both beams between 250mm and 400mm from the beam-column interface, at 12mm displacement, with some additional cracking at 18mm displacement. Delamination was first detected, at 24mm displacement, in the corner strip at the beam-column interface. Further cycles caused noticeable separation between beam and column in this region. However, further delamination of the corner strip was prevented by the presence of the beam and column wrapping. At 30mm displacement, horizontal cracks were also observed below the lower column wrapping and above the upper column wrapping, which opened up with increasing displacement cycles. At 36mm displacement, horizontal cracks at the corners of the column (level with the top and bottom of the beams) appeared, with some delamination of the FRP on the joint core face. In the final stages of the test, splitting of the diagonal FRP strips occurred parallel to the direction of the fibres. During the test, no bulging of the FRP was detected at the corners of the joint core with only minimal delamination on the face of the joint core. Once the FRP had been removed, no sign of cracking in the joint core or the column was visible, however extensive cracks were clearly visible at the beam-column interface. In the specimen with worse concrete quality also spalling of the corners was observed.

The configuration W3 had an initial behaviour very similar to W2, however delamination of the FRP was less apparent than in the previous case for the diagonal strips and No delamination was detected on either face of the joint core throughout the test. Removal of the FRP after testing revealed a minor diagonal shear crack on the front face of the joint core, a horizontal crack around the perimeter of the column about 200mm above the joint core, and substantial cracking at the beam-column interface.

DISCUSSION OF RESULTS

In order to compare the relative performance of the specimens, the strength, stiffness, and cumulative energy dissipated was calculated for each load cycle. Strength is defined as the envelope curve of the displacement load cycles for each specimen. Stiffness is defined as the gradient of the peak-to-peak slope on a load-deflection plot. The cumulative energy dissipated by each specimen is defined as the area enclosed by the hysteretic loop on a load-deflection plot for each cycle. Note that in order to make comparisons between the specimens as clear as possible only the stiffness and cumulative energy dissipated for the first of three load cycles for each displacement.

Strength

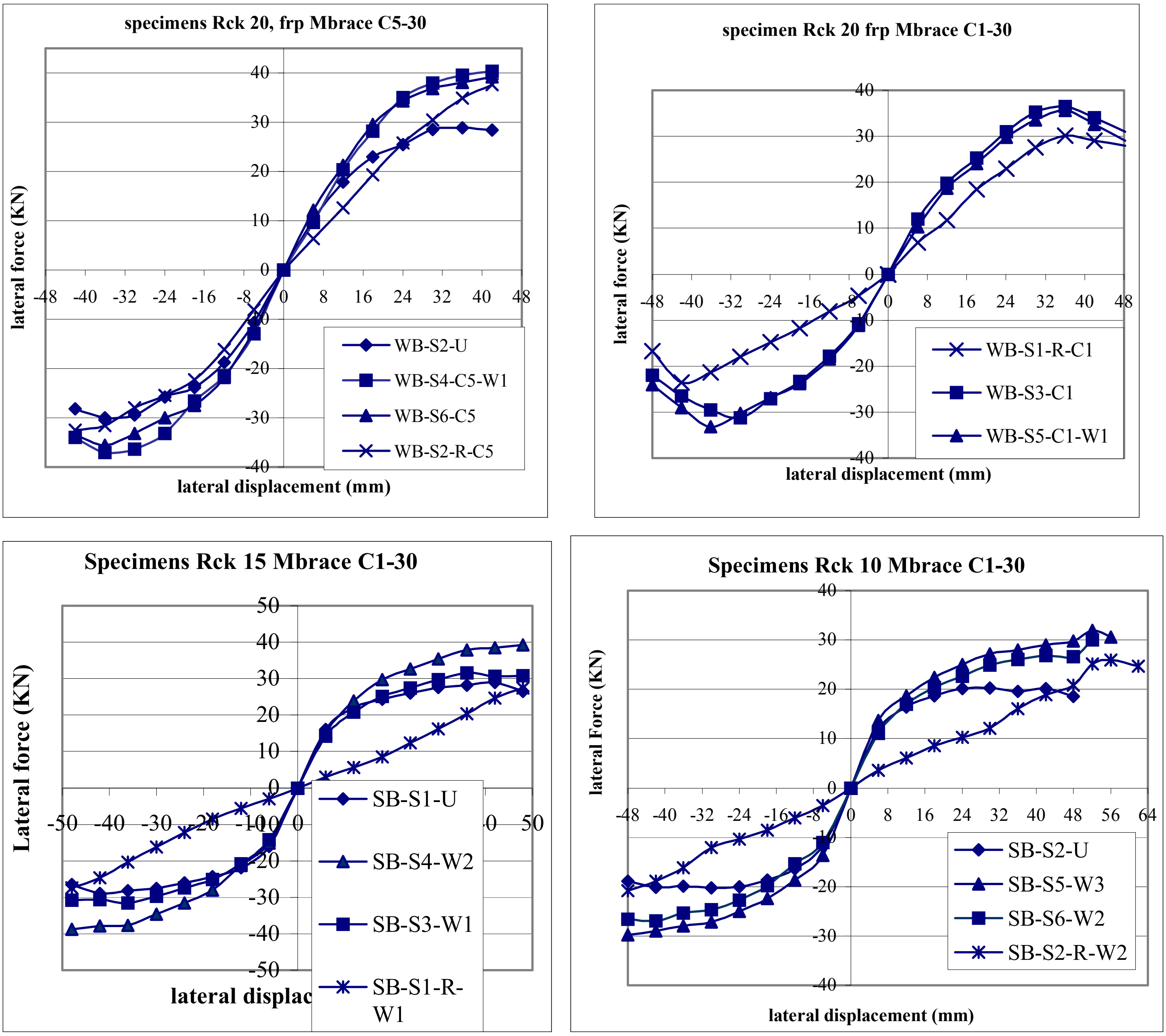


Figure 4: Envelope load displacement curves for the 4 batches of tests

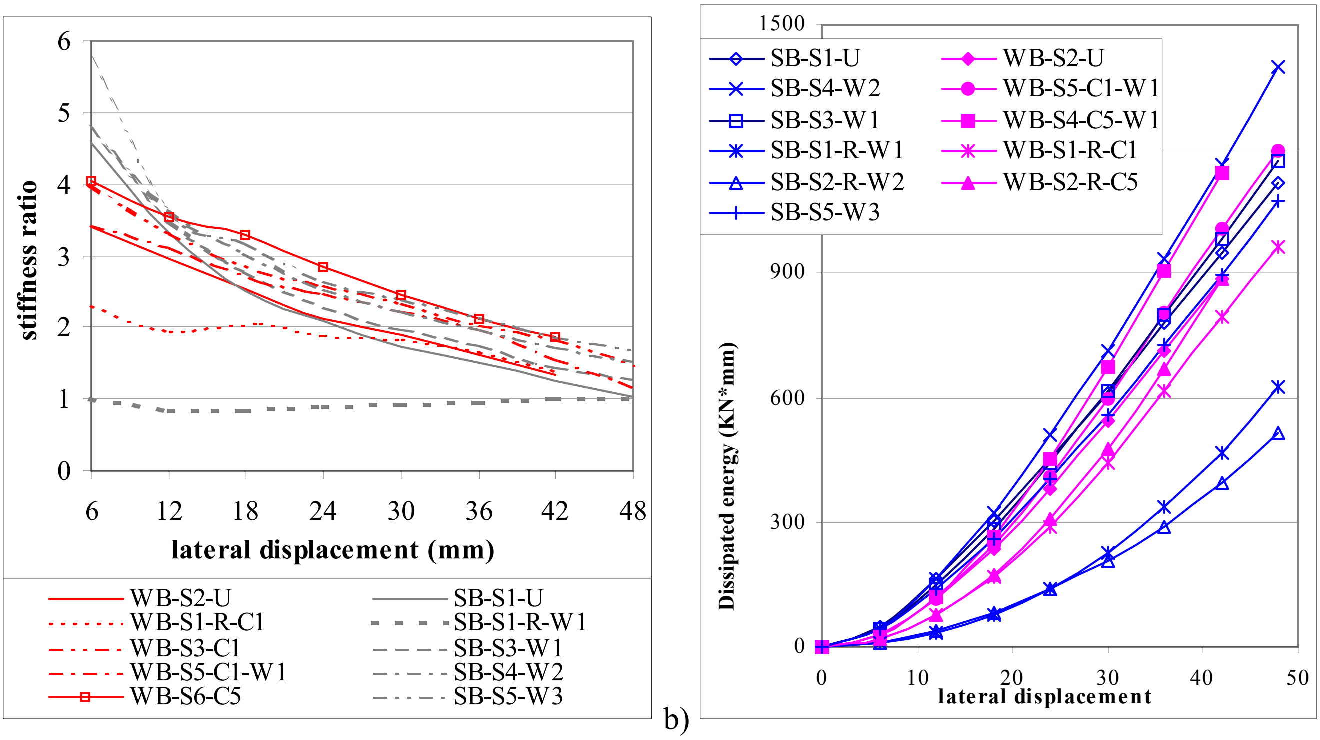
The maximum bending moments of the column and beam for both WB and SB control specimens were approximately 35% and 20% less, respectively, than the section analysis predicted. This was as expected and is due to the following:

- The section analysis is based on a 'plane sections remain plane' assumption, which is violated for plain round longitudinal bars as they experience bond degradation (Liu and Park, 2001). Hence, flexural capacity will be overestimated.
- The full capacity of the beam and column may not have been reached due to the premature failure of the joint core in the case of the SB configuration.

For the WB configuration the increase in strength associated with the strengthening was 19% for specimen strengthened with C1-30 and 33% for specimen strengthened with C5-30. The specimen repaired with C5-30 also showed increased strength for displacements greater than 30mm. However specimens strengthened with C5-30 were not able to sustain the load beyond 42 mm of displacement. Of the SB configuration the best performance in term of strength was associated with strengthening configuration W3. This may be due to a greater area fraction of FRP on the joint core with respect to W2. However W3 was associated with the worst quality of concrete, so absolute values favour configuration

W2. Strengthening configuration W1 performed poorly and it appears evident that is not satisfactory in shifting the behaviour from strong beam to strong column, as aimed.

Configuration W3 showed minor cracking in the joint core, with horizontal cracks in the column. suggesting that it was not entirely successful in shifting the failure mode to flexural hinging in the beam. However, given the poor quality of the concrete, these results imply that this type of strengthening is viable and effective in improving joint core behaviour in low-engineered structures.



a) b)

Figure 5. a) stiffness degradation diagram; b) cumulative energy dissipated diagram

<u>Stiffness degradation and Energy dissipation</u>

The degradation in stiffness is sharper in the SB series than in the WB configuration, and this is due to both the greater geometric stiffness and the poorer quality of the concrete in the SB series. As it was already apparent from the strength plots, the stiffness of the repaired specimen is lower than the control specimens and showing modest variability throughout the test. This is to be ascribed to the lack of bond between the longitudinal reinforcement and the concrete in all repaired specimens. Beyond 30 mm displacement the curve of the repaired specimen of the WB configuration coincides with the unstrengthened one. This is due to the relatively modest level of damage observed in the WB unstrengthened specimens as compared with the SB configuration equivalents. It should also be noted than for displacement greater than 18mm there is substantially little difference between the two series. The strengthening proves in general beneficial in terms of increasing stiffness throughout the test (up to 30% greater than the control specimens), however the rate of degradation is constant.

As the diagrams in Figure 5 show, the repaired specimens generally performed inadequately in comparison to the control specimens. This suggests that the wrapping configuration W1 is not a suitable design for repair of beam-column joints with a strong beam/weak column set-up and with smooth mild steel bars. The poor performance of repaired specimens could be attributed to the lack of confinement the wrapping provided to the joint core, leaving exposed corners where previous spalling had occurred. Hence, shear transfer in the joint (which would have predominately been by diagonal compressive strut formation) received little contribution from the wrapping- only from the grout. Conversely, specimens strengthened with wrapping configuration W1 show increased performance in terms of stiffness and energy dissipation, with respect to control specimens. It may be concluded that wrapping configuration W1 is more suitable for use in strengthening of undamaged structures than for repair. However, the strengthened specimens experienced the same type of failure as the control ones, substantial cracking in column and joint core, which makes it unsuitable for retrofit of structures with a strong beam/ weak column configuration.

From the diagram of dissipated energy it can be seen that, with the exception of the repaired specimens, the SB configuration specimens all had larger hysteretic loops than their WB counterparts. As expected the strengthened specimens had better dissipation rate than the unstrengthened. It is worth noticing that the specimens with C5-30 experienced less stiffness degradation and greater energy dissipation than the specimens strengthened with C1-30 and that the specimens strengthened with the configuration W2 performed better in terms of dissipated energy than the ones with configuration W1. In the diagram of Figure 5b the performance in terms of energy dissipation of the specimen with configurationW3 appears to be rather poor, but this is to be ascribed, as already mentioned to the very poor strength characteristics and quality of the concrete.

CONCLUSIONS

The results suggest that CFRP strengthening of beam-column joints with a strong beam/weak column arrangement can shift failure from brittle failure of the joint core towards flexural hinging at the beam-column interface, as well as upgrading joint core capacity.

CFRP strengthening was more effective as a means of strengthening than as a means of repair of the damaged specimens. However, it is likely that the method of repair of the joint (i.e. removal of only loose concrete and replacement with grout) was inadequate rather than the FRP wrapping.

CFRP wrapping of the beam-column joints was more successful with diagonal FRP wrapping, compared to the orthogonal wrapping. This is due to the fact that the orientation of the diagonal strips was closer to being parallel to the principal stresses in the joint core concrete than the principal fibres in the orthogonal wrapping, making more effective use of the FRP. It is also due to the enhanced confining action of the concrete at the joint corners by the diagonal FRP strips.

The application of the strengthening does not alter the initial failure mode in the configuration characterised by a weak beam. The final mode of failure is still in bending of the beam with partial failure of the FRP in tension.

Experimental values of ultimate bending moments obtained are lower than the ones calculated with the theoretical models, assuming plane sections, and this corresponds with the observation of Liu and Park, besides the fact that the FRP probably had a strength lower than the nominal one. The difference between observed and predicted values ranges between 17% and 35%. The increase in ultimate bending moment capacity between unstrengthened and strengthened specimens ranges from 26.3% for the repaired specimen to 76%.Analysis of the unstrengthened specimens had proven that joint failure was not an issue for the first configuration and this is proven by lack of cracks in the joint area of the two tested reference specimens. However as a result of the increased bending capacity provided by the FRP, it was noted that it was necessary to increase the shear capacity of the joint by adding vertical strips on the face of the columns. The FRP strips aimed at improving the shear capacity have proved effective and neither shear failure of the elements or debonding of the strips occurred in any specimen.

However the design of wrapping configurations did not include the presence of floor slabs. This restricts the way in which the joint and its connected elements can be wrapped. For example, complete wrapping of the adjoining beams would not be possible without perforation of the floor slab.

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