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Efficiency of Beam–Column Joint Strengthened by FRP Laminates

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Abstract

The recent earthquakes have shown that the vulnerability and the defects of the concrete joints in beam–column framed structures were the main causes for many building collapses. Such vulnerability and defects are in general the consequences of many factors.

External strengthening with composite materials represents an alternative and a sound and efficient technique to improve the performances and aptitude to withstand seismic action. However, while the use of such strengthening technique offers many advantages, it has some disadvantages, particularly a remarkable loss of ductility.

The present study examines the effects of an external strengthening of reinforced concrete beam–column joints against cyclic loading using CFRP laminates and GFRP sheet.

The experimental program is constituted of three beam–column reinforced concrete joints at a scale of one to three (1/3) tested under the effect of a prestressing axial load acting over the column. The beams were subjected at their ends to a reverse cyclic loading under displacement control to simulate a seismic action.

Strain and cracking fields were monitored with the help a digital recording camera. Following the analysis of the results, a comparison was made concerning the performances in terms of ductility, strength and mode of failure of the different strengthening solutions.

Keywords

FRP, reinforced concrete, cracking, tensile strength, compression strength, shear strength, upgrading, ductility

1. Introduction

During recent earthquakes, failure of beam–column joints is identified as the principal cause of collapse for many moment-resisting frame buildings. Effective and

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economical strengthening techniques to upgrade the joint resistance and the ductility in existing structures are needed.

In the past, a variety of techniques have been used to improve both the capacity and the ductility of reinforced concrete joints, with the most common technique being reinforced concrete or steel jacketing. These techniques produce various difficulties in practice at the joint, namely, availability of qualified workmanship, they are labour intensive, there is need for meticulous detailing and increased dimensions. To overcome the difficulties associated with these techniques, recent research efforts have focused on the use of epoxy-bonded FRP sheets or strips with fibres oriented properly so as to take up tension forces due to shear.

In the last four decades, several research papers have been published about studying the effect of seismic loads on poorly detailed reinforced concrete beam–column joints, designed prior to the publication of the seismic regulations. Among these studies, Hanson and Connor [1], Zerbe and Durrani [2], Paulay [3], Pantazopoulou and Bonacci [4, 5], Cheung *et al.* [6], Hakuto *et al.* [7] and Hwang and Lee [8], are the most important contributions. However, research papers on FRP repaired/strengthened beam–column joints are very limited. Antonopoulous and Triantafillou [9] conducted a comprehensive experimental program through 2/3-scale testing of 18 exterior joints. Their study revealed the role of various parameters, such as the area fraction of FRP and the distribution of FRP on the shear strength of exterior joints. They also highlighted the importance of mechanical anchorages in limiting premature debonding. Ghobarah and Said [10, 11] and El-Amoury and Ghobarah [12] have developed an effective selective rehabilitation scheme for reinforced concrete beam–column joints using advanced composite materials.

Mukherjee and Joshi [13] have studied experimentally the effect of FRP in improving shear strength and ductility of RC beam–column joints under simulated seismic forces; they have proposed an effective way of repair and strengthening for the critical sections of the frames.

Ghobarah and El-Amoury [14] have developed effective rehabilitation systems to upgrade the resistance to bond-slip of the bottom steel bars anchored in the joint zone and to upgrade the shear resistance of beam–column joints. Antonopoulos and Triantafillou [15] and Gergely *et al.* [16] have presented analytical models for the prediction of shear capacity of the FRP strengthened beam–column joints.

The literature reviewed shows that systematic studies to determine the behaviour of the repaired and/or strengthened members under cyclic loading are still limited. Moreover, the behaviour of FRP repaired beam–column joints subjected to seismic actions is not well established at the various successive stages of response, such as before and after yielding of reinforcements, crushing of concrete and finally fibre fracture or debonding. In addition, the effect of FRP configuration scheme in upgrading the joint also needs to be addressed.

The present paper is an additional contribution in this direction and aims at studying the behaviour of beam–column joints under different strengthening configurations.

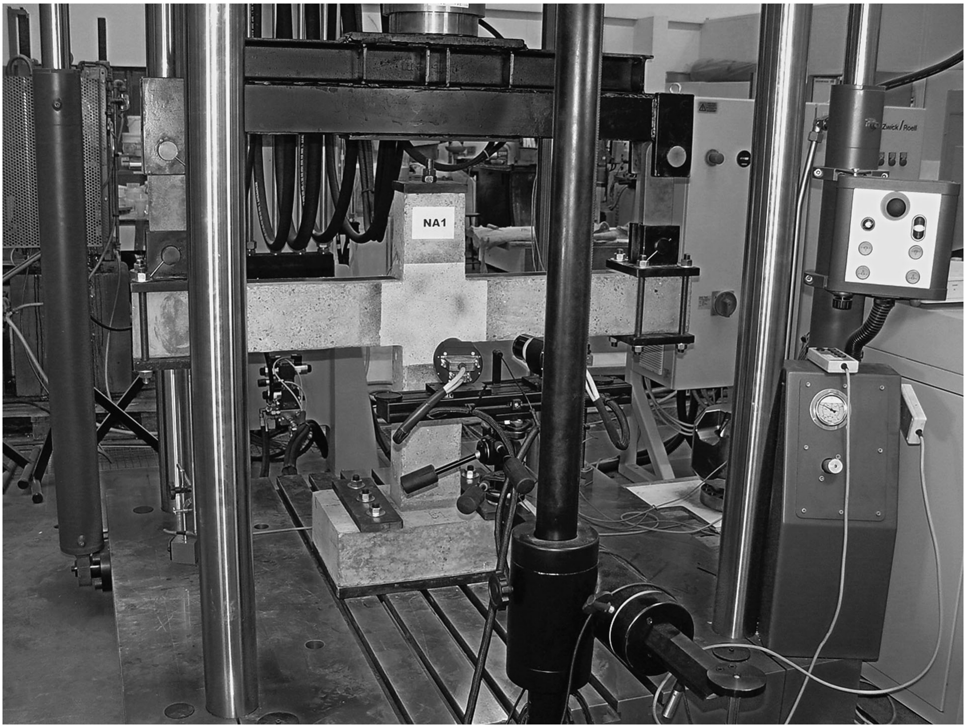


Figure 1. View of the test set up.

The efficiency and the effectiveness of carbon and glass fibre reinforced polymers (CFRP and GFRP) in upgrading the strength and ductility of seismically deficient interior beam–column joints have been studied. For this purpose, three reinforced concrete interior beam–column sub-assemblages were constructed with non-optimal design parameters. These sub-assemblages were subjected to cyclic load histories so as to provide the equivalent of severe earthquake damage (Fig. 1).

2. Research Objectives

The main objective of this research is to investigate the effects of the FRP reinforcement detailing on the behaviour of the beam–column connection, particularly its failure mechanisms and its ductility.

3. Experimental Program

3.1. Tested Specimens

One-to-three scaled reinforced concrete interior beam–column joint specimens were prepared in this study. A schematic sketch of the specimen used is shown in Fig. 2 and the reinforcement details are given in Fig. 3. Four 8 mm diameter

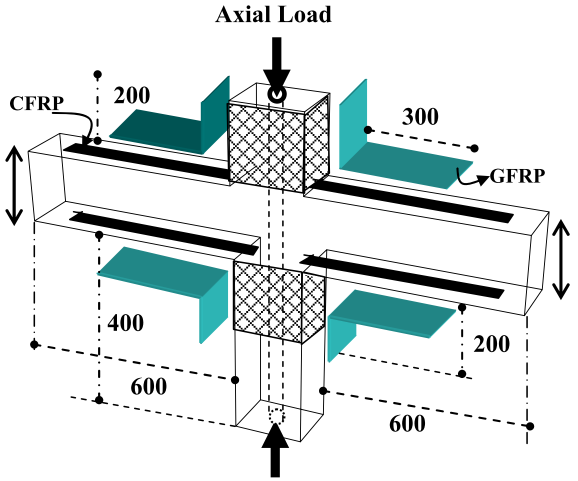


Figure 2. Strengthening configuration. All dimensions in mm. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

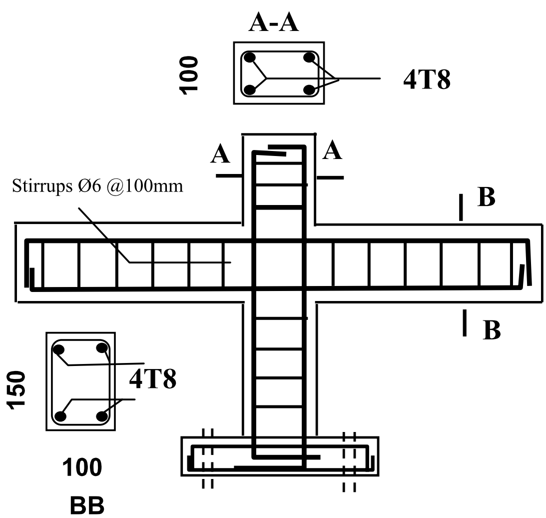


Figure 3. Specimen reinforcement. All dimensions in mm.

longitudinal steel bars were used for both the beam and the column; steel stirrups of 6 mm diameter spaced at 100 mm were used as shear reinforcement. The beams and the column had a rectangular cross-section of 100 mm by 150 mm.

3.2. Material Properties

The properties of the strengthening materials used in the experiments are given in Table 1. The properties given in 'b' are the composite properties and not the properties of the fibres.

Table 1.
Properties of strengthening FRP materials

Material	Modulus of elasticity (GPa)	Tensile strength (MPa)	Fibres orientation	Thickness (mm)	Elongation at failure (%)	Surface mass (g/m^2)
a — SIKA manufacturer's data (properties of the fibres)						
CFRP Sika Carbodur	165	2800	Unidirectional	1.2	1.7	
Fiber GFRP Sika Wrap 430G	76	2200	Unidirectional	0.17	2.8	430
Epoxy Sika 30	12.8	30	–	1	–	–
Epoxy Sika 330	3.8	30	–	1	0.9	~500 g
b — Laboratory test (composite properties)						
GFRP	19.2	325	Unidirectional	–	1.7	–

Table 2.
Details of tested specimens

Specimen	Details	Observation
NC1	Control	–
NA1	With 1 Carbodur plate CFRP on the top and the bottom of beams	Strengthened
NR1	With 1 Carbodur plate CFRP and 2 GFRP layers in L shape on the top and bottom of beams + 2 Wraps GFRP layers on the column	Repaired

Note: NC1 represents the control specimen without any strengthening.

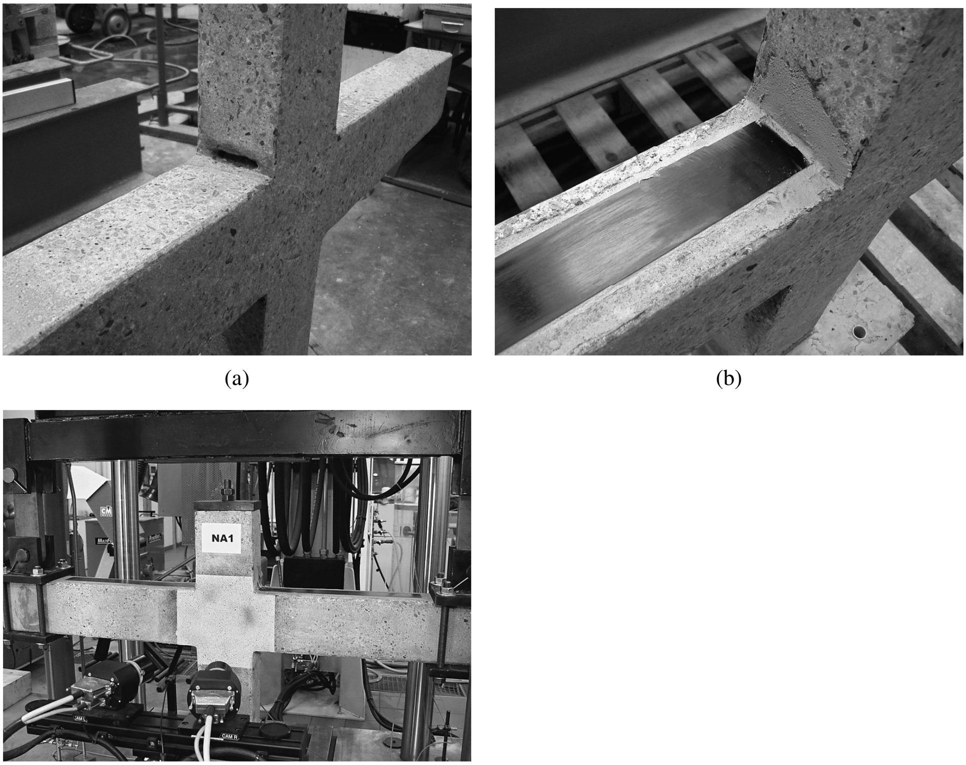
The average concrete strength in compression was 38 MPa; the yield strength of the steel bars used as tensile, shear and compression reinforcement has been determined by standard tensile tests — the average value was 580 MPa.

3.3. Strengthening and Instrumentation

The specimens were strengthened by using carbon and glass FRP materials (Table 2). Prior to the application of the FRP, the concrete substrate was smoothed by grinding and cleaning. The cement paste was removed from the surface and the coarse aggregates were exposed. The corners of all the members were ground to create a flare.

All the beams were subjected to a reversed cyclic load (Fig. 2) up to failure using a hydraulic machine of ±250 kN capacity. The supports were made from hardened steel plates, cut and formed with a suitable thickness to sustain the applied load without any deformation that may affect the test results (Fig. 1).

The bottom of the column was attached on the machine through a slab pad with special bolts. The column was subject to a constant pre-stressed axial load of 100 kN, which is about 25% of the ultimate load carrying capacity of the column.



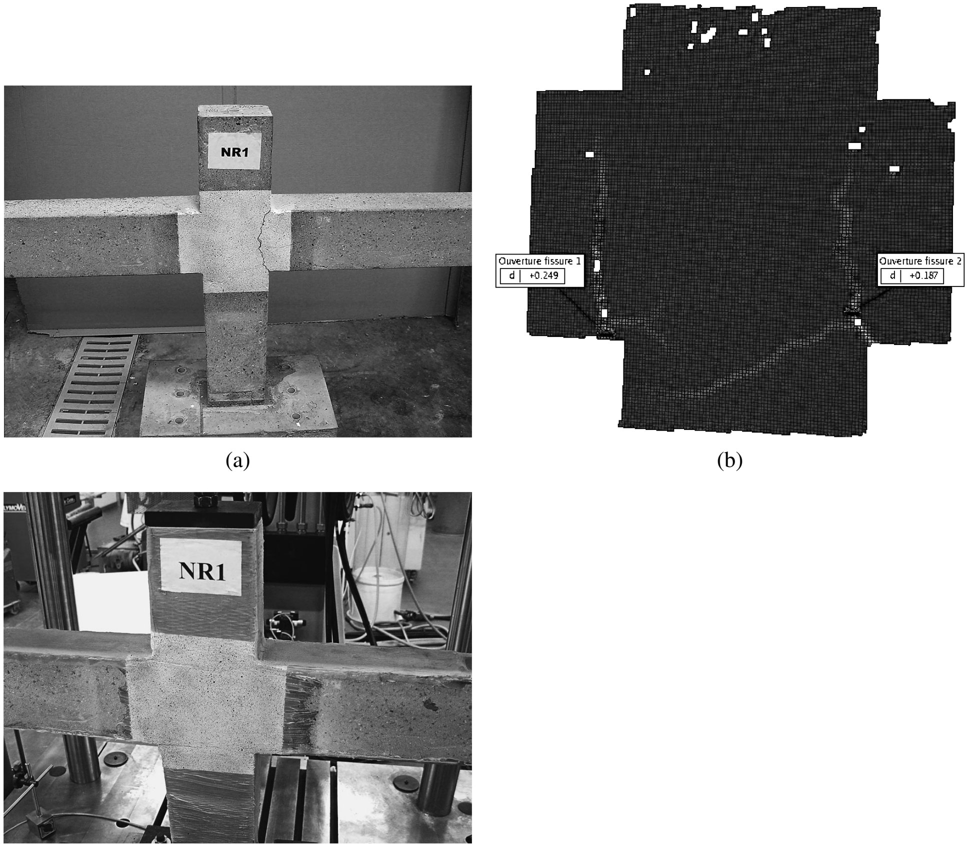
(a) (b) (c)

Figure 4. Specimen NA1 strengthening.

The deflection of the beam specimens was measured at the tip of the beam with the help of displacement transducers (LVDT) placed respectively on the beam specimens and on the loading arm of the testing machine. The test beams were equipped with stain gauges and camera for deformation measurements and monitoring.

Specimen NA1 was strengthened on the top and the bottom with a pultruded CFRP carbon plate (50 mm wide and 1.2 mm thick). To achieve a good bond between the plate and the concrete beam at the joint, a groove of 60 mm wide and 25 mm deep was created inside the concrete joint (Fig. 4). The plates were inserted into the joint. The groove was then filled by injecting epoxy resin (Sikadur 30). The column was not strengthened.

Testing on the specimen NR1 was conducted in two stages. In stage one, the specimen was mechanically damaged. Each beam of the specimen was cracked by applying cyclic loadings. With the help of the camera we could monitor thoroughly and carefully the damage process of the specimen. The test was stopped at the stage where the crack's width, localized at the corner of the node, reached 0.2 mm (in compliance with Eurocode2, Section 7.3.4). The specimen was then repaired (Fig. 5) as follows: the concrete cracks were carefully cleaned and all the voids were filled with epoxy (Sikadur 30). Then, the strengthening scheme of the specimen



(a) (b)

(c)

Figure 5. Specimen NR1 repair.

NA1 is used. To ensure a good anchorage of the carbon plates, two layers of GFRP sheet in L shape were added on the top and the bottom of the beams at each corner of the node. The fibres of the glass wrap were directed along the longitudinal axes of beams. The columns were then wrapped with two layers of GFRP. The fibres were perpendicularly directed to the axes of the column. To evaluate the efficiency of the repair, a second test was carried out.

The load cycle was predefined as shown in Fig. 6; the displacement started from the neutral position and oscillated harmonically about that position until failure of the beam. It increased at a uniform rate 0.25 mm/cycle, each cycle consisting of five full waves of the same amplitude with a frequency of 0.3 Hz.

4. Discussion

For the control specimen, the first crack was observed at a load of 8 kN as shown in Fig. 7. The beams have failed at the joint through the formation of hinges. The

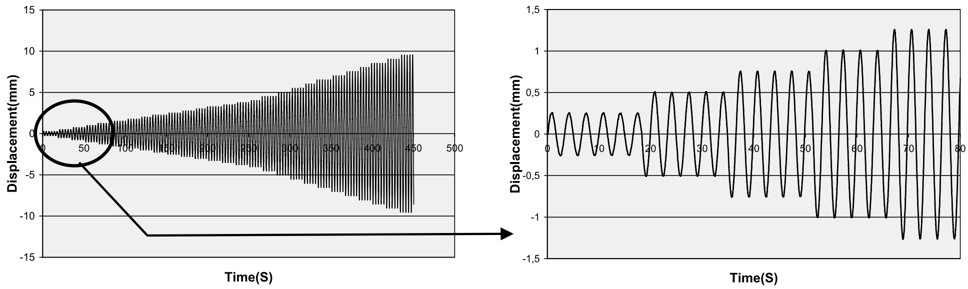


Figure 6. Cyclic-loading history used in this study.

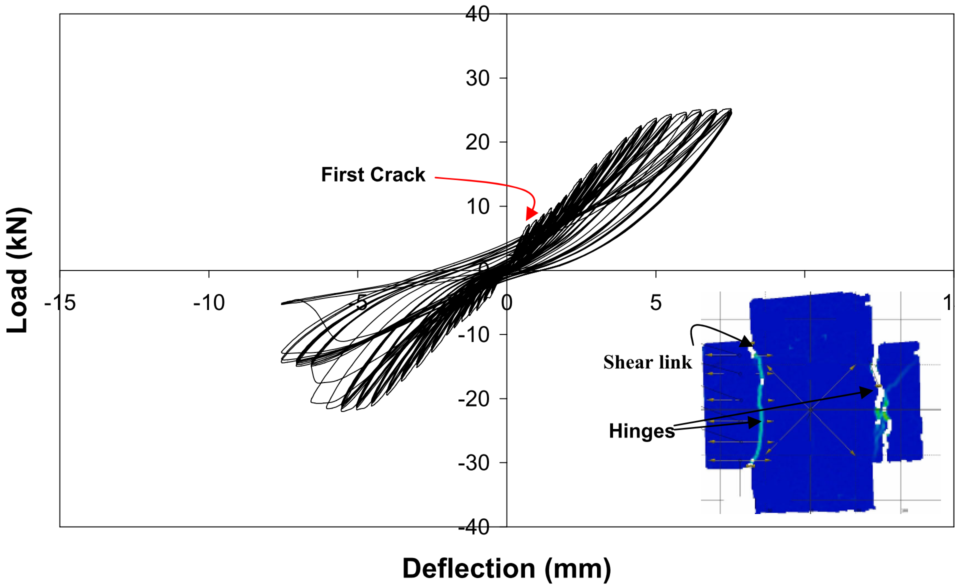


Figure 7. Beam tip load displacement and failure mode for control specimen NC1. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

hinges have formed between the two shear links of the beam. The concrete has spalled-off in such a way that vertical failure planes were created. This has resulted in free rotation of the beam with no transfer of bending moment to the column. The control specimen failed in shear at the joint: it reached a maximum load of 25 kN and an ultimate deflection of 8.5 mm.

The hysteresis behaviour of the control specimen showed considerable pinching with severe strength deterioration and stiffness degradation.

For specimen NA1 (Fig. 8), the displacement levels of the first few cycles remained elastic during the test and a vertical crack appeared. The concrete spalled-off in such a fashion that two semi-circular surfaces were created in the joint, as shown in Fig. 8. For this failure, the carbon plates have pulled out of the joint without any damage to them. Moreover, a thin layer of cement paste was pulled out; this

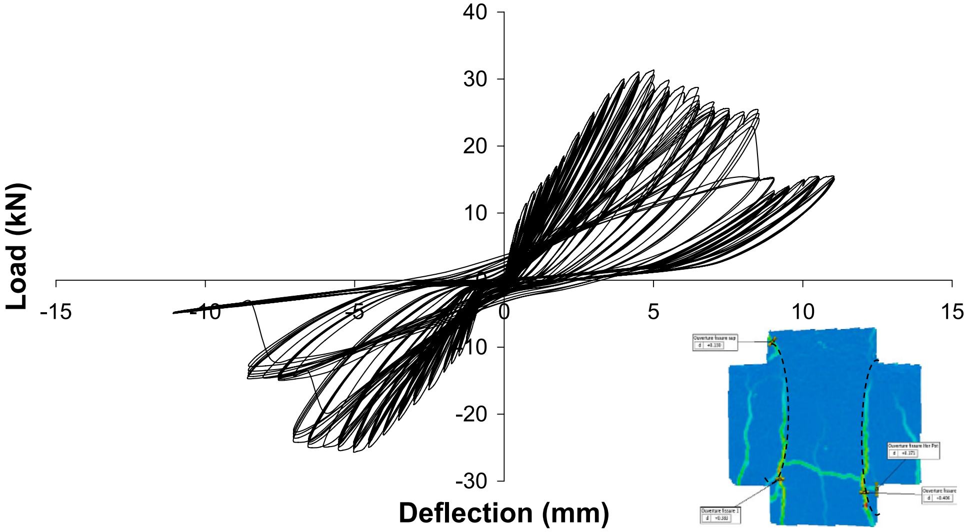


Figure 8. Load *versus* displacement and failure mode for NA1 specimen. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

shows that the carbon plates' pull-out was due to the failure of the cement paste in the groove.

At the post peak cycles, as the tension in the fibres is lost due to pull-out of the CFRP, the existing steel bars started to take up the developed tension force, the tension force in the bars was transferred to the column and a horizontal crack appeared on the top of the lower column under the joint region. After that, a drop in the beam load was observed together with sever pinching and stiffness degradation; this occurred mainly during the last cycles and was followed by the rupture of the steel reinforcement, which resulted in the total failure of the beam. The specimen reached a maximum load of 31 kN and the displacement controlled testing was stopped at a displacement of 11 mm. It was confirmed that the large deformation of the steel provided significant ductility to the joint behaviour.

The repaired specimen NR1 (Fig. 9) exhibited higher stiffness and higher peak load; the failure planes were approximately vertical. This mode of failure is due to the presence of the GFRP sheet and the Wrap of the column which confined the concrete effectively and did not allow it to spall-off. As a result, in cyclic opening and closing of cracks, the crack surfaces could react against each other in compression. Therefore, the moment resistance of the beam was not lost. This has resulted in a higher collapse load and displacement. The specimen reached its ultimate load at 36 kN and a corresponding displacement of 5.3 mm; the load carrying capacity decreased to 16 kN and a corresponding displacement of 11.5 mm when the GFRP sheet broke-up.

From hysteresis loops' envelopes of the tested specimens shown in Fig. 10, the following comments can be made:

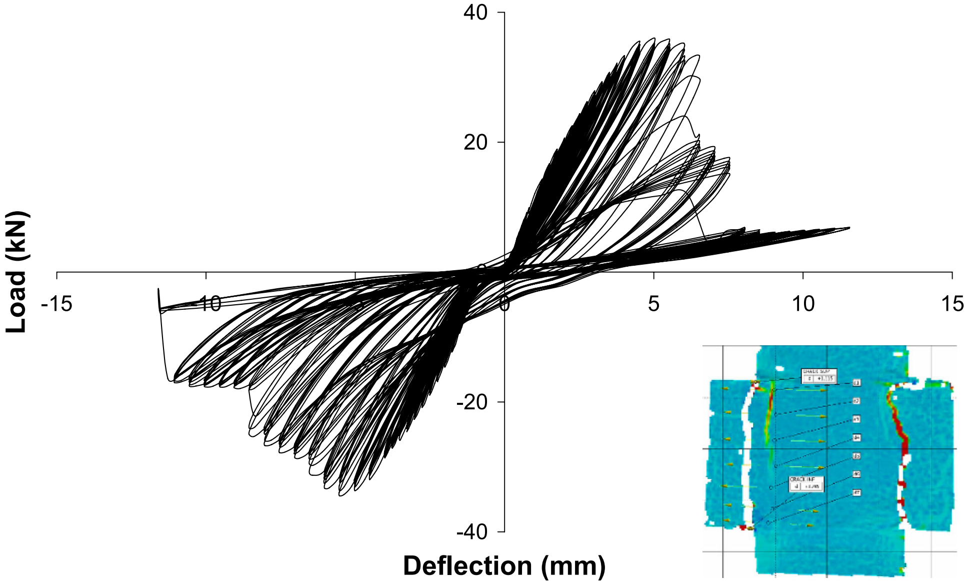


Figure 9. Load *versus* displacement and failure mode for NR1 specimen. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

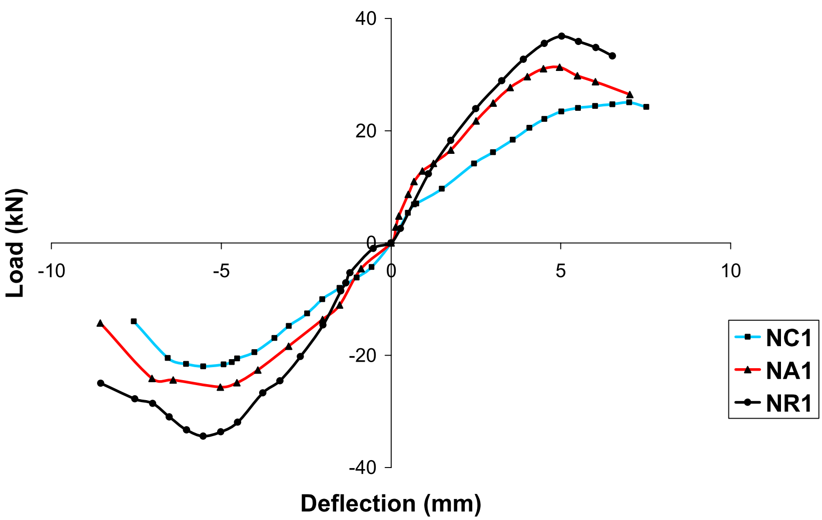


Figure 10. Hysteresis loop envelopes of the test specimens. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

- The control specimen has the lower initial stiffness, and when comparing the peak-to-peak stiffness of the tested joints, the stiffness degradation of the control specimen joint was higher than the specimens NA1 and NR1. The degradation of the stiffness with lateral movement was less in both the CFRP and the

Table 3.
Test results

Specimen	Load max (kN)	Increase (%)	Deflection at yield load (mm)	Deflection at peak load (mm)	Deflection at 10% drop of peak load (mm)	Ductility index	Increase (%)
NC	25	–	5.2	6.86	8.03	1.54	–
NA1	31	24	2.9	4.8	6.5	2.24	45
NR1	36	44	3.5	5.03	6.7	1.9	23

GFRP strengthened joints compared to that in the corresponding unstrengthened control specimens. This is a desirable property in earthquake situations. It was observed in past earthquakes that most of the RC structures failed (or collapsed) due to a sudden loss of stiffness of structural joints with increasing lateral movement of the structure.

- Specimen NA1 showed high initial stiffness compared to specimen NR1 because of pre-cracking of the repaired specimen.
- Ductility is an important parameter for earthquake resistant structures. The ductility factor is computed as the ratio of ultimate displacement to the displacement at first yield of internal steel. According to Paulay [3], for the purpose of computing the ductility factor, the ultimate displacement may be considered as that corresponding to 10% drop in the peak load. Table 3 shows that the application of composite has improved the ductility of repaired and strengthened specimens significantly. For NA1, the ductility has improved by 45% with respect to its respective control specimen. For the repaired specimen NR1, however, the increase in ductility with respect to the corresponding control specimen is 23%. Furthermore, as shown in Fig. 11, the ductility for the strengthened specimen is higher than the corresponding repaired specimen. This is an expected trend.
- The highest load was obtained for the repaired specimen with a hybrid glass/carbon fibres strengthening configuration, giving a strength gain of 44% (Fig. 12).

5. Conclusions

The results of the experimental program, presented in this paper, establish the effectiveness of CFRP laminates in upgrading deficient beam–column joints. The results of CFRP repaired and strengthened specimens were compared with their corresponding control specimens and, in general, it was observed that the combination of CFRP laminates and GFRP sheet improves the shear resistance and the ductility of the RC joints to a great extent.

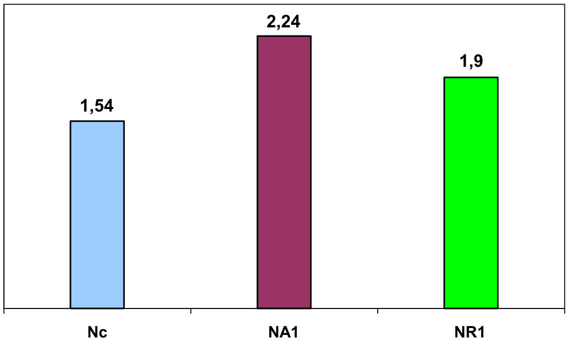


Figure 11. Ductility index. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

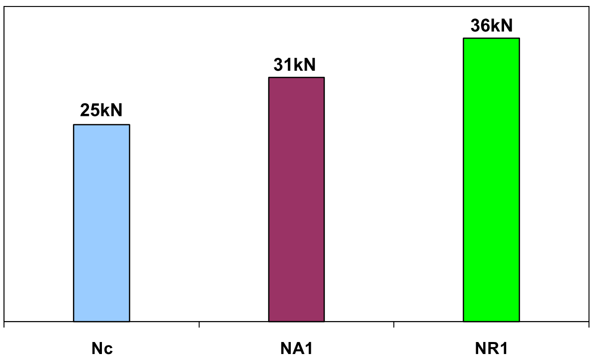


Figure 12. Peak load for the specimens tested. This figure is published in color on http://www.ingentaconnect.com/content/vsp/acm

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