

Primljen / Received: 7.9.2020.

Ispravljen / Corrected: 7.3.2021.

Prihvaćen / Accepted: 9.5.2021.

Dostupno online / Available online: 10.7.2021.

# Experimental and numerical investigation of RC beams strengthened with CFRP composites

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Research Paper

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## Experimental and numerical investigation of RC beams strengthened with CFRP composites

This study aims to strengthen reinforced concrete (RC) beams having insufficient shear capacity using Carbon Fibre Reinforced Polymer (CFRP) members and innovative anchorages. An innovative method is also proposed for strengthening beams in interaction site of adjacent structures. Test results show that the behaviour of beams has been improved with CFRP elements. The nonlinear finite element (FE) method, as well as American and Italian guidelines, are used to estimate theoretical capacity of the beams. Test results are compared with theoretical results. It can be concluded that proposed methods can be used reliably, and that the design of RC beams strengthened with these methods can be performed by design engineers based on simple calculations.

### Key words:

RC beam, strengthening, shear, CFRP, anchorages, debonding

Prethodno priopćenje

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## Eksperimentalno i numeričko ispitivanje armiranobetonskih greda pojačanih CFRP kompozitom

U ovom radu naglasak je na pojačanju armiranobetonskih greda nedovoljnog posmičnog kapaciteta uporabom polimera armiranih ugljičnim vlaknima (Carbon Fibre Reinforced Polymer - CFRP) i inovativnih načina sidrenja. Predlaže se, također, i inovativna metoda za pojačanje greda na kontaktu dviju konstrukcija. Rezultati ispitivanja pokazali su da se dodavanjem CFRP-a poboljšava ponašanje greda. Nelinearni proračun proveden je primjenom metode konačnih elemenata (FE) te američkih i talijanskih smjernica s ciljem procijene teoretskog kapaciteta greda. Zaključak je da se predložene metode mogu pouzdano primjenjivati, a projektiranje armiranobetonskih greda pojačanih na taj način može se provesti uz pomoć jednostavnih metoda proračuna.

### Ključne riječi:

armiranobetonska greda, pojačanje, posmik, CFRP, sidrenje, proklizavanje

## 1. Introduction

RC structures and structural elements can be damaged as a result of earthquakes, winds, ground settlement, insufficient workmanship, and insufficient material, changing the intended purpose of the structure. These structures should be strengthened using suitable methods in accordance with appropriate seismic codes. In the case of horizontal stiffness problems, structural performance is improved by RC shear walls, steel bracing, base isolators, or dampers. In many existing structures, RC members are made of low strength concrete, inappropriate longitudinal and stirrup reinforcement, and insufficient anchorage of reinforcement bars. These members are strengthened with methods that generally involve externally bonded reinforcements. These methods are RC/steel jacketing or CFRP wrapping [1-3]. Besides, external steel clamps are used to improve shear behaviour of structural members [4, 5]. However, all these methods have some disadvantages such as the weight, corrosion, fire, and debonding [6]. The RC jacketing method, which is difficult, laborious and takes a long time, increases the self-weight of the structure. In terms of strengthening and repair, CFRP elements have proven to be one of the most effective and widely used methods in construction industry over the last three decades [7-13]. Although CFRP is a lightweight and easy to install material that has high strength, stiffness and durability, it also has some disadvantages. One of the negative GFRP characteristics is its linear behaviour until failure (brittle fracture), i.e. its non-ductile behaviour. Problems relating to the debonding and delamination of CFRP elements still remain quite significant. Past studies reveal that the effectiveness of this method depends on how well the reinforcement is bonded to the concrete surface [14]. Due to the debonding, RC beams can be damaged in shear [15, 16]. In the shear strengthening of RC structures, it is quite significant to delay or avoid debonding and delamination problems [17]. In many studies, various methods have been developed to prevent the debonding of CFRP elements from beam surface [18]. The debonding results from high superficial stresses, which depend on the Young's modulus, thickness, width, and length of CFRP element. If the surface stresses exceed the bond strength of concrete or adhesive, the element is debonded from the surface. Generally, the debonding occurs on the concrete surface because the bond strength of concrete is smaller than that of the adhesive. To solve this problem, mechanical anchoring methods (i.e. bolts or nails) have been proposed [19]. However, these anchors cannot generally prevent premature peeling of the laminate. Connectors can minimize the debonding problem by transferring surface stresses into concrete [18]. Premature debonding of CFRP laminates was prevented in [18] using steel rods and adhesive connectors. The use of connectors improved both load-bearing capacity and failure behaviour of beams. Razaqpur et al. [20] attempted to delay premature debonding with  $\pi$ -CFRP anchors, and obtain higher strength in beams strengthened with CFRP

laminate. The results demonstrated the  $\pi$ -anchoring system's effectiveness, and showed that it is a feasible way for efficiently utilizing strong and thick laminates in strengthening members. Mohamed et al. [21] developed a bore-epoxy anchorage system to delay the debonding problem. It was concluded that the bore-epoxy application delayed the debonding of CFRP plates and increased the shear capacity of beams. Mohamed et al. [22] investigated strengthening applied with externally bonded CFRP sheets and plates on grooves of RC beams deficient in shear. They obtained an increase in the shear-strength over that of the control beam by up to 112 and 141 % for plates and sheets, respectively. Shear strength prediction models, based on conditions of the ACI440.2R-17 shear model, were also developed. Chalioris et. al. [14] examined behaviour of T beams strengthened with CFRP sheets and mechanical anchoring. The load-bearing capacity of beams in which the mechanical anchor was used increased by 72 % compared to the reference beam. Test results were compared with three different code standards. It was concluded that the Codes provided safe estimations. Bocciarelli et al. [23] tested RC elements strengthened in shear with different configurations of CFRP laminates. The test results were compared with the results obtained based on Italian and ACI guidelines. They concluded that the Italian guideline needs revision of shear strength calculations of completely wrapped members. The FE method was adopted to simulate and predict the load-displacement relation of RC beams. Besides, the applicability of proposed experimental methods can be verified by the FE method. Nistico et al. [24] focused on numerical studies of RC beams strengthened with different CFRP applications in shear. They used the microplane model for the concrete and polymer matrix. Stress distribution after the concrete crack appearance and spreading was investigated. Test results confirmed validity of the numerical study. Salih et al. [25] investigated both experimental and numerical behaviour of RC beams whose openings were strengthened using CFRP. FE results were verified by experiments. Kaya and Yaman [26] investigated the effect of the number of anchorage bolts on the glass-fibre reinforced polymer plates bonded to the surfaces of RC T-beams using the FE method. They concluded that the FE method based software was inadequate for modelling RC specimens in shear.

The aim of this study is to strengthen RC beams deficient in shear by using CFRP elements and to delay debonding and delamination of the CFRP fabric with fan shape CFRP anchorages. The fan shape CFRP anchorage is an easily and rapidly applicable and inexpensive method, unlike other anchorage techniques. CFRP fabrics were used to increase the shear capacity of the beams. In addition to CFRP fabrics, CFRP laminates were applied to enhance both the shear and flexural capacities of the beams. Test results were verified with those obtained from American [27] and Italian [28] guidelines, and with nonlinear FE analyses. Thus, simple approaches have been proposed for design engineers and researchers.

Table 1. Mechanical properties of steel bars

Sample	Number	Yielding strength [MPa]	Tensile strength [MPa]	Young's modulus [MPa]
Ø6	3	480	590	212000
Ø10	3	470	575	210000
Ø16	3	435	540	202000

## 2. Material and method

### 2.1. Material

At the design stage of the beams, the concrete compressive strength was aimed at 16 MPa. During the production phase of the beams, 3 cylindrical concrete samples were taken for each beam element. These samples were then subjected to uniaxial compression tests and an average cylinder compressive strength of 16 MPa was determined. According to TS708 [29] conditions, 3 sample sets were produced for the axial tension test for each steel bar measuring 6, 10 and 16 mm in diameter. As a result of the tests, yield strength ( $f_y$ ), tensile strength ( $f_u$ ), and Young's modulus ( $E_s$ ) of the steel bars were determined as presented in Table 1.

A special epoxy resin was used to bond CFRP elements to the surfaces of RC beams. The 7-day compressive strength of epoxy resin is higher than 60 MPa, tensile strength is higher than 50 MPa, and adhesive strength is higher than 3 MPa. The weight per unit volume is 1080 kg/m<sup>3</sup>. The thickness of epoxy resin applied on beam surfaces was approximately 1 mm, in accordance with requirements for the type and technique used in the resin application process. The uniaxial CFRP fabric and CFRP laminate, made of carbon fibres, are frequently used to increase the shear and flexural capacities of beams, respectively. The CFRP fabric used in this study is 500 mm in width, 0.111 mm in thickness, 230 g/m<sup>2</sup> in weight per unit volume, and 4900 MPa in tensile strength. The elongation at failure is 2.1 %. The Young's modulus is 230000 MPa. The CFRP laminates used in this study are 100 mm in width, 1.2 mm in thickness, and 3000 MPa in tensile strength. The elongation at failure is 1.5 %. The Young's modulus is 165000 MPa.

### 2.2. Specifications of specimens

To determine the behaviour of the beams under load, a total of 6 RC beams with insufficient shear capacity

were produced. One of the beams having insufficient stirrups (reference beam) was tested without any strengthening method. Other beams were strengthened with CFRP fabric and laminate by different application techniques. All beams and their properties are presented in Table 2. REF represents the reference beam, while GK1, GK2, GK3, GK4, and GK5 represent beams strengthened with CFRP fabric and laminate.

All beams were subjected to a 4-point bending test under the monotonically increasing load. The loading was carried out using a hydraulic piston and a steel apparatus at a distance of 585 mm from both supports. All beams used in this study had the same geometrical characteristics, their length was 2000 mm, the cross-section height was 300 mm, and the width was 150 mm. All beams were reinforced with 2Ø16 steel bars in the tension zone and 2Ø10 steel bars in the compression zone. Stirrups were Ø6/300 mm. The distance between the two beam supports was 1750 mm. The cross-

Table 2. General properties of beams

Beam	CFRP fabric	CFRP laminate	CFRP anchorage
REF	-	-	-
GK1	+	-	-
GK2	+	+	-
GK3	-	+	-
GK4	+	+	+
GK5	+	+	+

-: not existing. +: existing

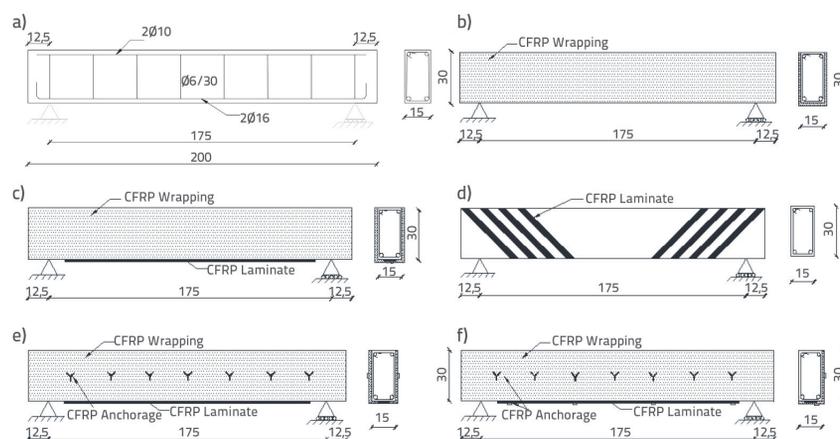


Figure 1. Details of all beams: a) REF b) GK1; c) GK2 d) GK3; e) GK4; f) GK5



Figure 2. Preparation of specimens: a) GK1 b) GK2 c) GK3 d) GK4 e) GK5



Figure 3. Handmade CFRP fan anchorages

section, geometry, and reinforcement properties of the REF beam are presented in Figure 1.a. CFRP configurations differed from each other in strengthened beams. GK1 shown in Figure 1.b was strengthened with U wrapped CFRP fabrics. GK2 was strengthened by both U wrapped CFRP fabrics and CFRP laminate, as shown in Figure 1.c. In GK3, CFRP laminates 5 cm in width and 40 cm in length adhered to both side surfaces of the beam (in Figure 1.d). One of the most important problems for strengthening performed with CFRP members is debonding [16]. Fan anchorages, produced from CFRP fabrics, were used in order to delay or prevent these problems in GK4 and GK5. GK4 was strengthened with U wrapped CFRP fabrics and laminate (in Figure 1.e). GK5 represents the beam in the interaction site of adjacent RC structures. So, GK5 was strengthened with L shaped CFRP fabrics and laminate attached to the bottom of the beam (in Figure 1.f).

GK1, GK2 and GK4 were strengthened by U wrapping, as shown in Figure 2.a-b-d. Beam surfaces were cleaned and repaired before CFRP application. After that, CFRP fabrics were bonded to beam surfaces and saturated with epoxy. In GK4, the CFRP laminate also adhered to the bottom side of

the beam. In GK3, side surfaces were diagonally roughened as shown in Figure 2.c and laminates were attached to these surfaces with epoxy. GK5 were strengthened with L-shaped CFRP fabrics, and laminate was attached to the bottom surface, as shown in Figure 2.e.

GK4 and GK5 were strengthened using CFRP fabric and laminate, as well as CFRP fan anchorages, as shown in Figure 3. Through anchorages, tensile stresses are transferred from the CFRP fabric into the concrete surface. The fan anchorages were produced from CFRP fabric (in Figure 3.a). Then, they were saturated with epoxy resin (in Figure 3.b). Before inserting the CFRP fans, the holes were filled with epoxy resin and these anchorages were inserted into predrilled holes. The epoxy resin was once again applied to beam surfaces, as shown in Figure 3.c.

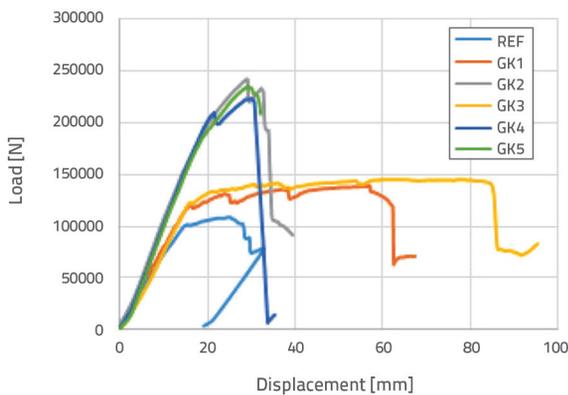
### 2.3. Test results and discussion

The load-displacement relation, energy dissipation capacity, initial stiffness and ductility values of all beams were obtained by testing. The yield load and displacement, ultimate load, failure displacement, and initial stiffness of beams, are

**Table 3. Initial stiffness, load and displacement of beams in tests**

Beam	Yielding load [kN]	Yielding displacement [mm]	Ultimate load [kN]	Failure displacement [mm]	Initial stiffness [kN/m]
REF	105	14	109	33	7.8
GK1	119	16	140	62	7.7
GK2	190	19	242	32	10.1
GK3	122	16	146	85	7.4
GK4	210	21	224	35	9.8
GK5	185	19	235	32	8.1

presented in Table 3. Load-displacement relations are shown in Figure 4. Variations in load and displacement capacities of beams are comparatively presented in Table 4.



**Figure 4. Load-displacement relationships of all beams**

**Table 4. Variations of load and displacement capacities of strengthened beams compared to REF beam**

Beam	Yielding load [%]	Yielding displacement [%]	Ultimate load [%]	Failure displacement [%]
REF	-	-	-	-
GK1	14	16	28	88
GK2	82	40	122	-3
GK3	17	22	34	159
GK4	101	59	105	7
GK5	77	37	115	-2

GK2, GK4 and GK5, which were strengthened with both CFRP fabric and laminate, reached the highest load capacities compared to REF beam. The highest load increase occurred in GK2. GK3 reached the highest displacement of 159 %. Diagonal CFRP laminates used in GK3 considerably enhanced the displacement capacity. The failure displacements of GK2 and GK5 could not be improved compared to the REF beam. In GK4, failure displacement increased by 7 %. The failure displacement of GK1 increased by 88 % and GK3 by 159 %.

**Table 5. Ductility values of beams**

Beam	Yielding displacement [mm]	Failure displacement [mm]	Ductility ( $\delta_u/\delta_y$ )
REF	14	33	2.4
GK1	16	62	3.9
GK2	19	32	1.7
GK3	16	85	5.2
GK4	21	35	1.6
GK5	19	32	1.7

Ductility values of all beams are presented in Table 5. Ductility values of GK1 and GK3 increased considerably as related to the REF beam and the ratios were 62 % and 113 %, respectively. Although CFRP laminates improved the load-bearing capacity of the beam, their ductility reduced. Compared to REF beam, ductility values of GK2, GK4 and GK5 decreased by 31 %, 33 % and 29 %, respectively. The energy dissipation capacities of beams, shown in Table 6, are determined by calculating the area under the load-displacement curves.

**Table 6. Energy dissipation capacities of beams**

Beam	Energy dissipation capacity [Nm]	Increase [%]
REF	2290	-
GK1	6341	177
GK2	3532	54
GK3	9562	317
GK4	3400	48
GK5	3350	46

The energy dissipation capacities of all strengthened beams improved when compared to the REF beam. The highest increase was observed in GK3. Compared to the REF beam, the energy dissipation capacity of GK1, GK2, GK3, and GK4 increased by 177 %, 54 %, 317 %, and 48 %, respectively. It can be concluded that diagonal CFRP laminates have a considerable effect on the beams' ductility.

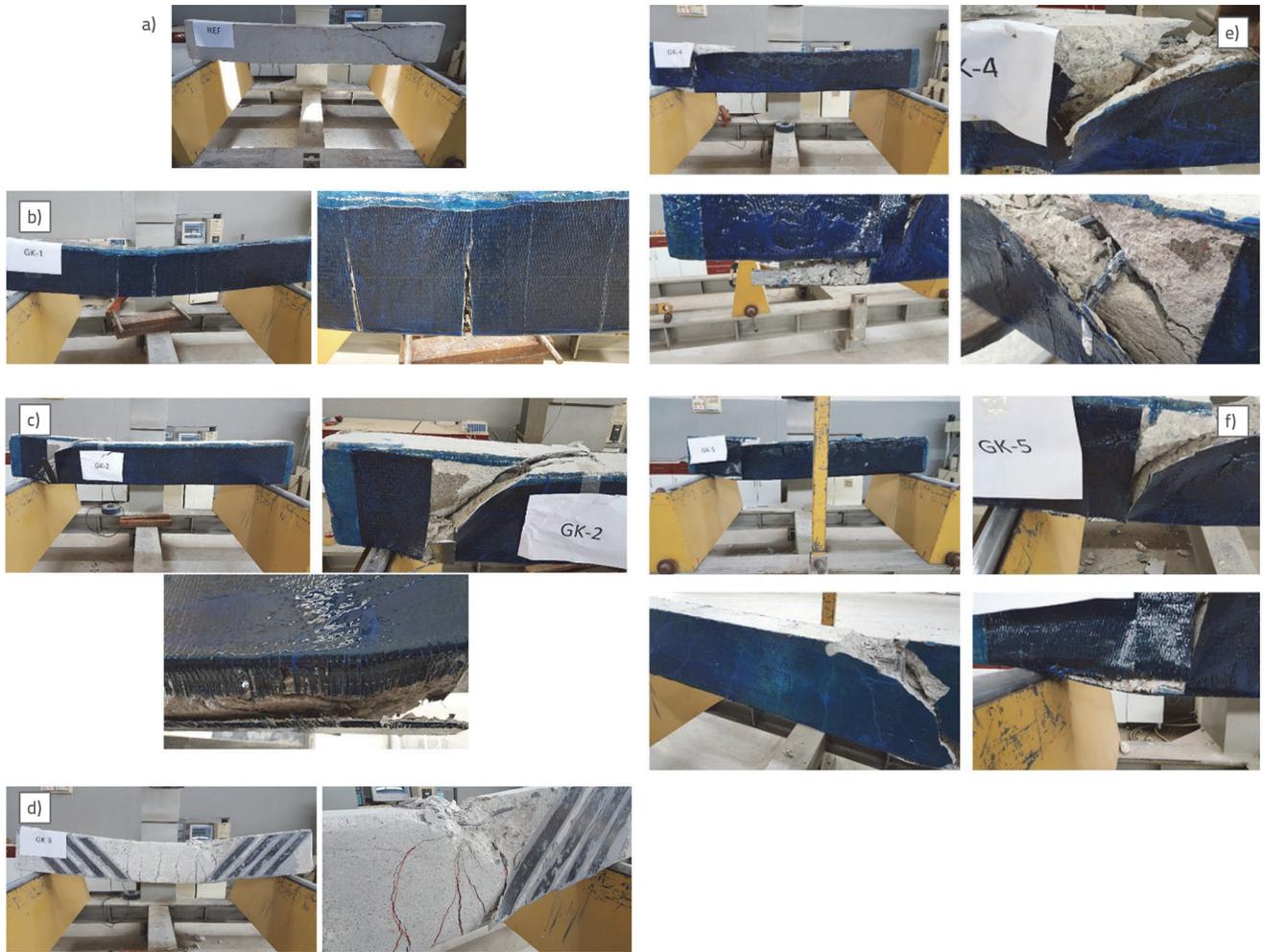


Figure 5. Damage situations of all beams after the tests: a) REF; b) GK1; c) GK2; d) GK3; e) GK4; f) GK5

### 2.4. Failure modes of beams

As a result of tests, failure modes of all beams are shown in Figure 5. While REF failed in shear, as shown in Figure 5.a, GK1 collapsed in flexure, as shown in Figure 5.b. CFRP fabrics changed the failure mode of GK1 from shear to flexure. In ultimate loads, CFRP fabric rupture occurred. In GK1, debonding was not observed. The GK2 reached high loads, but it failed in shear. In GK2, both debonding and delamination were observed, as shown in Figure 5.c. GK3 failed in flexure, as shown in Figure 5.d. For shear strengthening, the use of CFRP laminates significantly improved the shear behaviour of GK3. In GK3, delamination occurred at high loads. Through anchorages, failure displacement of GK4 is slightly higher than that of GK2 having similar properties except for anchorages. The yielding load of GK4 is by 10.34 % greater in comparison with GK2. In GK4, debonding and delamination problems delayed compared to GK2. GK4 failed in shear, as shown in Figure 5.e. GK5 failed in shear such, just like it was observed for GK2 and GK4 (in Figure 5.f). Although L-shaped CFRP fabric was wrapped on two surfaces of GK5, GK5 exhibited similar behaviour to that of GK4 having U wrapping. This situation showed that fan anchorages exhibited desired behaviour in the test because debonding and delamination

did not occur until high loads. It can therefore be concluded that beams in the interaction site of adjacent structures could be safely strengthened by this method. In tests, CFRP fabrics increased the ductility and energy dissipation capacity of beams. Although CFRP fabric and laminate increased the load-bearing capacity of beams, their combination decreased the ductility of beams as shown in the literature [21, 22, 30].

### 3. Analytical study

#### 3.1. Load-bearing capacities of beams according to ACI 440.2R-17 and CNR-DT 200 R1/2013 guidelines

Full or partial wrapping applied with CFRP fabrics increases the shear capacity of RC beams.

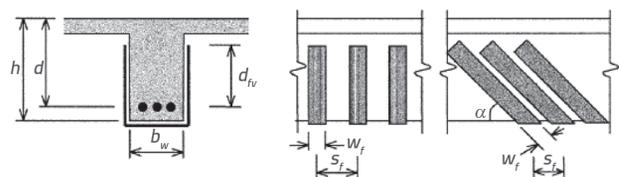


Figure 6. Wrapping with CFRP fabrics of beams [27]

To be able to safely apply loads on beams during the design phase, the design shear forces must not exceed the shear strength of beams strengthened with CFRP fabrics (in Eq.1). Dimensional variables used in beams strengthened with CFRP elements are shown in Figure 6.

In this study, shear capacities of beams were obtained in accordance with ACI 440.2R-17 (2017) and CNR-DT 200 R1/2013 [28]. The design shear strength should be calculated by multiplying the nominal shear strength by the strength reduction factor  $\phi$ , as specified in ACI 318-05 [31] (in Eq. 1).

$$\phi V_n \geq V_u \tag{1}$$

ACI 440.2R-17 [27] is based on the design equations derived by Khalifa et al. [7]. The nominal shear strength of RC beams strengthened with CFRP is calculated by Eq. (2) according to ACI 440.2R-17 [27].

$$\phi V_n = \phi (V_c + V_s + \psi_f V_f) \tag{2}$$

where  $V_c$  and  $V_s$  are calculated according to ACI 318-05 [31]. The reduction factor  $\psi_f$  of 0.85 is recommended for CFRP U-wrapping according to ACI 440.2R-17 [27]. The contribution of CFRP fabrics to the shear strength ( $V_f$ ) is calculated by Eq. (3) according to ACI 440.2R-17 [27].

$$V_f = \frac{A_{fv} f_{fe} (\sin \alpha + \cos \alpha) d_f}{s_f} \tag{3}$$

where  $A_{fv} = 2nt_f w_f$  is the area of the CFRP;  $d_f$  is the effective depth of the CFRP;  $w_f$  and  $s_f$  are the width and spacing of CFRP strips, respectively. In the case of continuous wraps  $s_f = w_f$ ;  $\alpha$  is the angle between the fibre direction and the longitudinal axis of the element;  $f_{fe} = E_f \varepsilon_{fe}$  is the effective design stress of the CFRP;  $\varepsilon_{fe} = k_v \varepsilon_{fu} \leq 0.4\%$  is the CFRP effective strain;  $\varepsilon_{fu}$  is the ultimate rupture strain of the CFRP;  $k_v$  is the bond-reduction coefficient that depends on the specified concrete compressive strength,  $f_c$ , the type of the retrofitting scheme and the stiffness of the CFRP:

$$k_v = \frac{k_1 k_2 L_e}{11910 \varepsilon_{fu}} \leq 0.75 \tag{4}$$

where  $k_1 = (f_c/254)^{2/3}$ ;  $k_2 = (d_f - L_e)/d_f$  (for U-wraps) and  $L_e = 23300/(nt_f E_f)^{0.58}$ . Thus, Eq. (3) can be written as follows:

$$V_f = d_f f_{fe} 2(nt_f) \left( \frac{w_f}{s_f} \right) (\sin \alpha + \cos \alpha) \tag{5}$$

The shear strength of beams is also calculated according to CNR-DT 200 R1/2013 [28]. The shear strength of the beam strengthened with CFRP reinforcement ( $V_{Rd,r}$ ) can be estimated by Eq.(6) given in CNR-DT 200 R1/2013 [28].

$$V_{Rd,r} = \min(V_{Rd,s} + V_{Rd,f} + V_{rd,max}) \tag{6}$$

where  $V_{Rd,f}$  is the CFRP U-wrapping contribution to shear strength. The contribution of CFRP shear reinforcement ( $V_{Rd,f}$ ) is calculated by Eq.(7) based on the Mörsh's truss analogy model.

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} \frac{0.9d}{p_f} f_{fed} \times 2t_f \times b_f \times (\cot \theta + \cot \beta) \tag{7}$$

where  $\theta$  is the inclination of concrete crack to the beam axis,  $\beta$  is the inclination of CFRP strips to the beam axis,  $d$  is the distance from the extreme compression fibre to the centroid of tension steel reinforcement, and  $\gamma_{Rd} = 1.2$  is the partial factor.  $p_f$  is the spacing, and  $f_{fed}$  is the effective strength of CFRP strips ( $f_{fed}$ ) instead of the steel cross-section area ( $A_{sw}$ ) as shown in Eq.(8).

$$f_{fed} = f_{fd} \left[ 1 - \frac{1}{3} \frac{l_{ed} \times \sin \beta}{\min\{0.9d, h_w\}} \right] \tag{8}$$

where  $f_{fd}$  is the design debonding strength of CFRP,  $h_w$  is the web depth completely U-wrapped and  $l_{ed}$  is the design effective bond length.

For strengthening in shear, beams were wrapped with CFRP fabrics in a single layer. For flexural strengthening, CFRP laminates were bonded to the bottom surface of the beam with epoxy resin. For shear strengthening in the GK3 beam, CFRP laminates diagonally adhered to shear spans. The calculation method used for CFRP fabrics was adopted to obtain the shear capacity of GK3, but mechanical properties of CFRP laminates were used in this calculation. As shown in Table 7, test results for beams strengthened with CFRP fabrics only are quite close to analytical results. Test results for beams strengthened with both CFRP fabrics and laminates were by 57.2 % higher in GK2 and by 45.4 %

Table 7. Comparison of test and analytical results

Beam	Test [kN]	ACI440.2R-17 (in shear) [kN]	CNR-DT 200 R1/2013 (in shear) [kN]	$P_{test}/P_{ACI}$	$P_{test}/P_{CNR}$
REF	109	105	97	4.1	12.7
GK1	140	154	124	-9.4	12.2
GK2	242	154	124	57.2	94.8
GK3	146	152	179	-4.1	-18.7
GK4	224	154	124	45.4	80.1

higher in GK4 compared to the value predicted in ACI 440.2R-17 [27] shear equation. These rates were 94.8 % in GK2 and 80.1 % in GK4 compared to CNR-DT 200 R1/2013 [28] shear equation. Although these differences were high, safe design estimations were obtained. It can be seen that failure modes obtained with test and ACI440.2R-17 methods are incompatible with each other. It can be concluded that design engineers can use conditions given in ACI 440.2R-17 [27] and CNR-DT 200 R1/2013 [28] to strengthen RC beams with the proposed method during the design phase.

## 4. FE analyses

### 4.1. Modelling

Nonlinear finite element (FE) analyses of beams were carried out using ABAQUS software [32]. FE analyses were adopted in this study to simulate and predict the load-displacement relation of the strengthened RC beams. Many investigations related to the modelling of concrete cracking, crushing and damage mechanisms have been performed using ABAQUS [11, 33, 34]. The nonlinear static analysis was used in the models. The maximum time increment for each step was 0.001 sec. The classical Full-Newton solving method was used in analyses. Concrete was modelled by using solid elements with eight nodes with three degrees of freedom, which is called C3D8R. Concrete behaviour was considered as plastic and homogeneous. The longitudinal steel and stirrups were embedded in the concrete mesh by using embedded region constraints. The steel bar was defined as truss element with 2 nodes and 3 translation degrees of freedom for each node, which is called T3D2. The steel behaviour was considered as elastic-perfect plastic. The bonding between the steel bar and concrete was assumed to be perfect. Four-node shell elements, called S4R, having six degrees of freedom per node, were used to model CFRP fabric and CFRP laminate. The CFRP behaviour was regarded as linear elastic. The CFRP rupture was controlled by the CFRP strength in tension. The stress-slip between CFRP and concrete was modelled using the approach presented in [35]. An eight-node interface element for transferring shear in nodal forces was applied between the concrete and CFRP elements [36, 37]. Figure 7 shows the three-dimensional FE model containing the geometry, steel, CFRP fabric and laminates and the FE mesh used in this study.

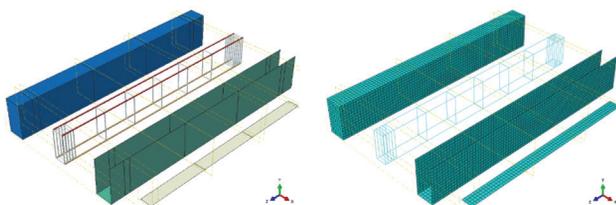


Figure 7. Three-dimensional FE model of beams

### 4.1.1. Concrete damage plasticity

The modelling and FE analyses of RC elements are quite complex since concrete exhibits a nonlinear behaviour in compression and tension. For concrete, the damage situation is determined with strain hardening/softening behaviour, and it is quite difficult and complex. For this purpose, the Concrete Damage Plasticity (CDP) model based on the theory of plasticity and damage mechanics is frequently used in damage analyses [11, 34, 38]. In this model, damage in concrete is defined with equivalent plastic strains in tension ( $\epsilon_t^{ckh}$ ) and compression ( $\epsilon_c^{pl/h}$ ). For the CDP model, the default values of eccentricity, dilation angle, viscosity parameter,  $f_{bo}/f_{co}$  and K are 0.1, 36, 0, 1.16 and 0.667, respectively.

### 4.1.2. Uniaxial compressive and tensile behaviour of concrete

In CDP models, the relation between the damage parameters and the compressive strength of concrete is determined with the plastic hardening strain ( $\epsilon_c^{pl/h}$ ) in compression (in Figure 8.a). In this study, the stress-strain relationship of RC elements is considered according to Eurocode 2 [39], and equations proposed by [40] are used to model the uniaxial behaviour of concrete in compression. The compressive damage parameter ( $d_c$ ) can be expressed as shown in Eq. 9.

$$d_c = 1 - \frac{\sigma_c}{\sigma_{cu}} \tag{9}$$

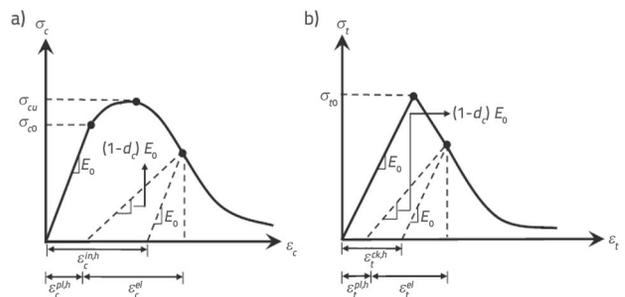


Figure 8. Behaviour of concrete under uniaxial loading [32]: a) Compression; b) Tension

Tensile strength of concrete ( $f_{ctm} = \sigma_{t0}$ ) is obtained as shown in Eq. 10.  $\epsilon_{ot}^{el}$  is the strain at ultimate tensile strength (in Eq. 11)..

$$f_{ctm} = \sigma_{t0} = 0.30 (f_{cm})^{2/3} \tag{10}$$

$$\epsilon_{ot}^{el} = \frac{\sigma_{t0}}{E_0} \tag{11}$$

In CDP models,  $\epsilon_t^{ckh}$  is the cracking strain in tension (Figure 8.b) and is obtained as shown in Eq. 12.

Table 8. Material properties of concrete with CDP model

Concrete compressive behaviour			Concrete tensile behaviour		
Stress [MPa]	Inelastic strain	Damage parameter	Stress [MPa]	Cracking strain	Damage parameter
10.2	0.00049	0	<b>1.90</b>	0	0
14	0.00092	0	<b>1.35</b>	0.00012	0.287
15	0.00114	0	<b>1.12</b>	0.00023	0.405
16	0.00149	0	<b>0.84</b>	0.00054	0.557
15	0.00192	0.060	<b>0.63</b>	0.00115	0.667
14	0.00206	0.108	<b>0.44</b>	0.00285	0.767
12	0.00235	0.248	<b>0.33</b>	0.00566	0.822
10.5	0.00249	0.339	<b>0.26</b>	0.01006	0.859
7.9	0.00270	0.503	<b>0.19</b>	0.02106	0.895
5.9	0.00285	0.630	<b>0.18</b>	0.02606	0.903
2.4	0.00306	0.855	<b>0.17</b>	0.03106	0.910

$$\varepsilon_t^{ck,h} = \varepsilon_t - \frac{\sigma_{t0}}{E_0} = \varepsilon_t - \varepsilon_{0t}^{el} \tag{12}$$

The tensile damage parameter ( $d_t$ ) can be expressed as follows:

$$d_t = 1 - \frac{\sigma_t}{\sigma_{t0}} \tag{13}$$

Wang and Hsu [41] defined the relation between stress and strain of concrete in tension, as shown in Eq. 14.

$$\sigma_t = \sigma_{t0} \left( \frac{\varepsilon_{0t}^{el}}{\varepsilon_t} \right)^n \quad \varepsilon_t > \varepsilon_{0t}^{el} \tag{14}$$

The value  $n$ , which represents the rate of weakening, is proposed to be 0.4, according to [41]. The concrete compressive and tensile behaviour was determined based on damage plasticity formulations mentioned in this section, while CDP parameters are presented in Table 8.

#### 4.1.3. Modelling Interaction between Concrete and CFRP

The epoxy resin used to attach the CFRP fabrics and laminates to the RC beam is a thin viscous film. The surface-to-surface contact between the CFRP and RC beam is obtained with this epoxy resin. This behaviour is modelled as hard contact. The linear elastic traction–separation behaviour is expressed in Eq. 15. According to ABAQUS [32], the interface adhesive layer has a finite thickness and is generally modelled by the surface-based cohesive behaviour.

$$K_n (K_s, K_t) = \frac{E_{resin}}{t_{adhesion}} \tag{15}$$

The damage initiation criterion and damage evolution law are quite significant for obtaining the failure mechanism. The damage initiation of this epoxy film model is governed by the amount of  $t_n(t_s, t_t)$  and the final separation is governed by the damage evolution law, which is the amount of energy according to the curve shown in Figure 9.

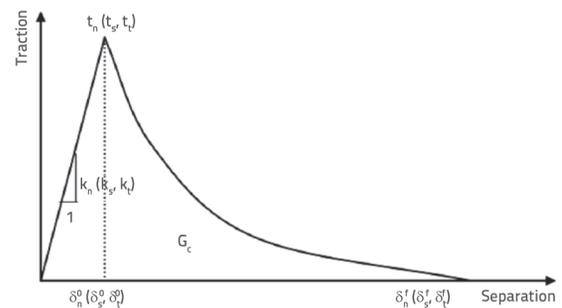


Figure 9. Typical traction–separation response with exponential damage evolution [32]

#### 4.2. FE results and discussion

The damage and stress distributions for concrete, steel, CFRP fabric and laminate are shown in Figs. 10-12. Figs. 10.c, 11.c, and 12.c show the damage contour for tensile stress, which corresponds to the damage exceeding the crack strain of concrete. Figure 10 shows the test and FE analysis results for the REF beam. Test results of the REF beam show a good correlation with FE results. The beam failed with diagonal shear cracks in the test. Similarly, the damage parameter in tension reached 0.85, and the beam collapsed due to shear stress in FE analysis. Figure

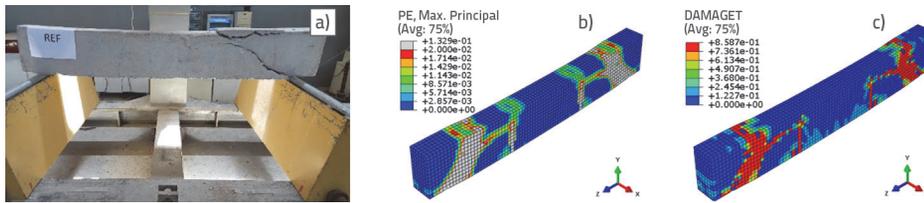


Figure 10. REF beam analysis results: a) experimental result; b) stress distribution in beam only (MPa); c) tensile damage

11 shows the test and FE analysis results, tensile stress, and damage parameter in concrete and CFRP elements of GK-1. The beam failed as ductile with flexural cracks in the test. In FE analysis, the damage parameter in tension for GK-1 reached 0.85, and significant flexural cracks occurred. Results obtained from test and FE analysis of GK-1 exhibit a good correlation. Figure 12 shows the test and FE analysis results of GK-2. The beam reached high loads with the effect of CFRP laminate and fabric in the test, and it failed with shear stress. The tensile damage parameter of GK-2 reached the maximum value, and the beam failed due to shear stress in FE analysis, as shown in Figure 12.c. The test and FE analysis results of GK-2 show a good correlation. Figure 13 compares the load and mid-span displacement curves obtained from the test and FE analyses for these beams. In the test and FE analyses, the beams exhibit a good correlation in terms of the load-displacement relation. Table 9 summarizes load values obtained by both methods. The maximum loads obtained with FE analyses were quite close to the test results.  $P_{max, FE} / P_{max, test}$  values of REF, GK-1 and GK-2 were 0.97, 1.07 and 1.02, respectively.

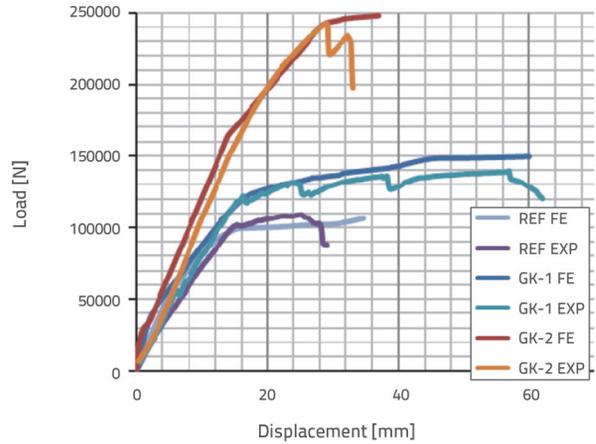


Figure 13. Load displacement curves obtained from test and FE results

Table 9. Comparison of test and FE results

Beam	$P_{max, FE}$ [kN]	$P_{max, test}$ [kN]	$P_{max, FE} / P_{max, test}$
REF	106	109	0.97
GK-1	149	140	1.07
GK-2	248	242	1.02

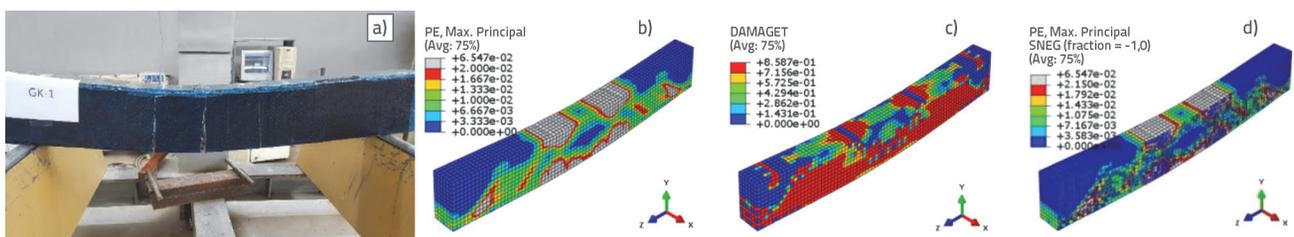


Figure 11. GK1 beam analysis results: a) experimental result; b) stress distribution in beam only (MPa); c) tensile damage; d) stress distribution in beam with CFRP fabric (MPa)

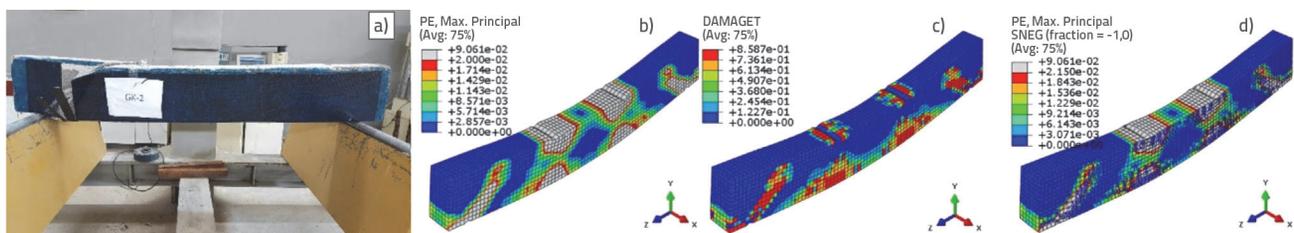


Figure 12. GK2 beam analysis results: a) experimental result; b) stress distribution in beam concrete only (MPa); c) tensile damage; d) stress distribution in beam with CFRP fabric (MPa)

## 5. Conclusions

In this study, RC beams of insufficient shear capacity were strengthened with CFRP fabric and laminate, and their behaviour was investigated. While the ultimate load of the beams wrapped with CFRP fabric only slightly increased, their ductility values were considerably enhanced. The debonding and delamination, in cases when the highest loads were reached in tests, were observed in CFRP elements. In beams, CFRP fan anchorages contributed to the yielding load and ultimate displacement. However, the ultimate load decreased. The diagonal application of CFRP laminates on side surfaces of beams enhanced ductility of beams considerably, and the beam failure mode changed from shear to flexure. This method is cheaper and easier than the application of CFRP fabric in terms of workmanship. It can therefore be concluded that the proposed method can easily be applied for strengthening inadequate RC beams in shear. The beam wrapped with L-section CFRP fabrics exhibited the behaviour that was almost similar to the beam having U wrapping. The beam's ultimate load increased considerably. For

this reason, it can be concluded that beams in the interaction sites of adjacent RC structures can be reliably strengthened with the proposed method.

Test results were found to be in good agreement with the results obtained by American [27] and Italian [28] guidelines. Experimental and analytical failure modes were compatible with each other. Design engineers can reliably use these guidelines for the proposed methods. In this study, FE analyses were performed to simulate and predict the load-displacement relation of the strengthened RC beams. The ultimate load, load-displacement relation, and damage of beams were determined in FE analyses. The FE analysis results are in good agreement with test results. In tests, beam damage situations were successfully predicted with nonlinear FE analyses.

Consequently, it can be concluded that insufficient RC beams can reliably be strengthened with proposed methods against shear failure. Design engineers can define the most suitable method for strengthening by identifying deficiencies in the RC structure members.

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