Steel and Composite Structures, *Vol. 27, No. 4 (2018) 453-464*
DOI: https://doi.org/10.12989/scs.2018.27.4.453

Strengthening of reinforced concrete beams using external steel members

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(*Received October 23, 2017, Revised February 6, 2018, Accepted March 22, 2018*)

Abstract. The objective of this study is to devise an alternative strengthening method to the ones available in the literature. So, external steel members were used to enhance both flexural and shear capacities of reinforced concrete (RC) beams having insufficient shear capacity. Two types of RC beams, one without stirrups and one with lacking stirrups, were prepared in the study. These beams were strengthened with external steel clamps devised by the authors and with external longitudinal reinforcements. Although the use of clamps alone didn't have a significant effect on the load carrying capacity of the tested beams, the ductility increased approximately tenfold and the failure behavior changed from brittle to ductile. Although the use of clamps and longitudinal reinforcements together did not significantly increase the ductility of the beams, it approximately doubled their load capacities. The results of the experimental study were compared to the ones obtained from nonlinear finite element analysis (NLFEA) and it was observed that they were compatible. Finally, it can be concluded that the devised method could be applied to structural members as an alternative to methods in application due to lightness, low-cost, easy applicable and reliable.

Keywords: RC beam; strengthening; external clamp; shear; test; finite elements

1. Introduction

Throughout the globe, a huge number of buildings become vulnerable to damage due to aging, rebar corrosion, and natural hazards, particularly earthquakes (Kothandaraman and Vasudevan 2010). Section enlargement (Zhang *et al*. 2012, Chalioris *et al*. 2014), bonded steel plating (Swamy *et al*. 2008, Arslan *et al*. 2008, Su *et al*. 2010, Hamad *et al*. 2011, Yang *et al*. 2015, Osman et al. 2017), FRP composites wrapping (Hawileh *et al*. 2009, Kišiček *et al*. 2007, Wu *et al*. 2013, Mofidi and Chaallal 2014, Qin *et al*. 2015) and external post-tensioning are some of the techniques which are widely used to strengthen various structural elements. Each technique has a number of limitations. High cost, difficulty in execution, reduction in headroom, loss of aesthetic value, increase in self-weight, need of a careful surface preparation, de-bonding failure are some of the problems of these techniques.

Altin et al. (2003) carried out experiments to determine the behavior of beams under flexure by attaching external clamps. The appropriate ratio of clamps applied and the locations of the clamps on this method were investigated. External clamps which improved the ductile behavior of the members controlled the cracks successfully and prevented propagation of cracks. Strengthened beams reached the bending capacities. Adhikary and Mutsuyoshi (2006) presented the results of a parametric study accounting for the effects on RC beams of steel plate depth/beam depth ratio, steel plate thickness, concrete strength and internal shear reinforcement ratio. Finally, a design formula to compute the shear strength of beams with web-bonded continuous steel plates was developed. A comparison between the shear strengths computed using the proposed formula and FEM as well as the experimental results was made. Ceroni (2010) carried out an experimental study of RC beams reinforced with external strengthening made of carbon FRP sheets or Near Surface Mounted FRP carbon bars. Monotonic and cyclic loading histories were applied according to a four-point test scheme. Moreover, end or distributed U-shaped anchoring devices were applied when the strengthening was made of FRP carbon sheets. Comparisons between experimental failure loads and failure loads obtained from theoretical predictions were discussed. Kothandaraman and Vasudevan (2010) devised a new technique by providing external reinforcement at the soffit level of the beam.

The technique of keeping the reinforcement externally at soffit level was found to be viable and the moment carrying capacity of the beam sections was increased considerably. In spite of subjecting the externally reinforced beams to more deflection than reference beams, the extent of recovery of deflection was more. Similarly, although strengthened beams reached higher loads than reference beams, the widths of cracks were less. Finally, experimental results were compared with theoretical results obtained from the Indian code and approximate results were obtained. Chalioris *et al*. (2014) researched using of thin reinforced self-compacting concrete jackets to repair and strengthen concrete members. Specimens were repaired

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http://www.techno-press.org/?journal=scs&subpage=6
ISSN: 1229-9367 (Print), 1598-6233 (Online)

with three-sided jackets having the minimum thickness. It was determined that strength, ductility, deformation capacity and model of failure of repaired members reached those of ideal monolithic member. Osman *et al*. (2017) investigated a repairing technique of the steel plate that effectively strengthen RC members and increased the serviceability of pre-cracked RC beams with openings. Two un-strengthened reference beams, five beams having pre-cracked before the application of the steel plate and non-damaged one beam were tested. The results showed that steel plate strengthening had an important effect on strengthening effectiveness and failure mode at the max strength.

In this study, a new technique is proposed to enhance the flexural and shear capacity of RC beams. The beams were strengthened using external longitudinal bars and clamps. The details of the external clamps were improved by the authors. The effects of stirrup spacing, clamp spacing and longitudinal reinforcements on the behavior of beams were experimentally investigated. The effects of external longitudinal reinforcements and clamps on the flexural and shear capacities of beams were determined with experimental studies and the results were compared with those of NLFEA.

2. Research significance

The aim of this study is to develop a new, reliable, light and economic method in order to strengthen RC beams. RC beams having insufficient shear capacity are strengthened using external steel clamps and longitudinal bars. Detail of clamp is developed by the authors. In addition, an easier method is applied in order to anchor longitudinal bars. Load and displacement capacities of beams are considerably improved with external members. During tests, any debonding problem hasn’t been observed. Besides, proposed method increases lower self-weight of structure when compared to strengthening with traditional steel plating and jacketing in literature. During the application in a RC structure, it is enough to drill holes along slab and embed steel rods to the holes. It is concluded that external members can be applied as jacketing, FRP or steel plating in order to strengthen RC beams.

3. The experimental program

3.1 Materials

Table 1 Concrete mix adopted for producing a cubic meter of concrete

Material	kg/m^3
Water	190
Cement	200
Fine aggregate	1000
Coarse aggregate	1000
Super Plasticizer	2.0

Table 2 Properties of reinforcements

Bar size (mm)	E_s (MPa)	f_y (MPa)	f_u (MPa)
8	210000	470	575
10	211000	450	550
12	207000	440	530
10-Clamp	211000	455	550

A concrete mix consisting of Portland cement (PC 42.5) and maximum aggregate size of 12 mm in diameter were used in this study. Concrete properties were determined with twelve 150×300 mm cylinder samples. The constituents and the corresponding proportions of the beams are detailed in Table 1.

The mean value of cylinder compressive strength determined by uni-axial test was 16.76 MPa. Tensile strength of concrete was estimated from the compressive strength (f_c') as $0.35\sqrt{f_c'}$ (in MPa) according to TS500-2000. Three types of steel reinforcement bars with diameters of 12 mm, 10 mm and 8 mm were used in the beams. Three samples were taken from each type of reinforcement bars and tensile tests were carried out on these samples, conforming to TS708 (2010). The yield strength (f_y), ultimate strength (f_u) and modulus of elasticity obtained from tests are shown in Table 2.

The type of epoxy-adhesive including two components which is used in this study acts as a chemical anchor. This chemical adhesive is used to anchor external longitudinal reinforcements. The tensile strength of this is 30 MPa. The moduli of elasticity of epoxy under flexural and tensile loads are 3800 and 4500 MPa, respectively.

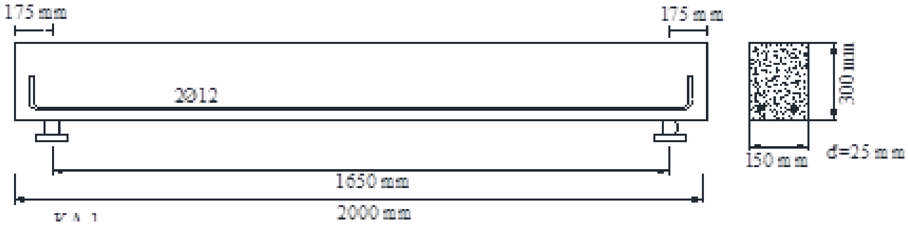
3.2 Geometry of specimens and test set-up

Seven RC beams including two reference and five strengthened beams were tested under four point loading. The beams were divided into two groups denoted by KA and KB. Section, rebar and concrete properties of KA and KB beams were computed by using TS500-2000 requirements. All specimens were 150 mm wide, 300 mm deep. The details of each beam are given in Table 3. The KAs and KBs had tension reinforcement 2Ø12 and KBs had also compression reinforcement 2Ø10. Kas were without stirrups along the beam as shown in Fig. 1(a). KBs were tied with stirrups Ø8/400 mm along the beam as shown in Fig. 1(b). The concrete cover was 25 mm.

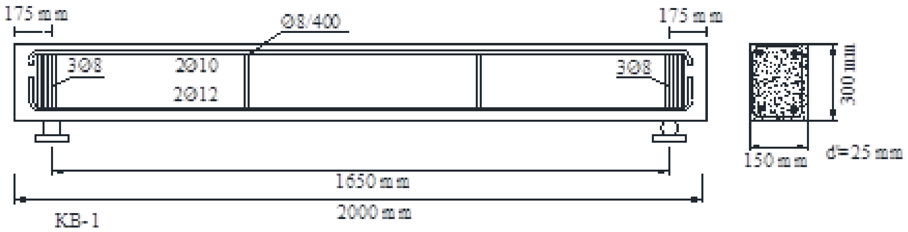
KA1 and KB1 beams were loaded without strengthening in order to determine the effectiveness of the proposed technique. Five beams were strengthened by various methods. KA2 beam was confined with external steel clamps having 10 mm diameter placed along the beam with a spacing of 75 mm. Furthermore, two external longitudinal reinforcements having 12 mm diameter were anchored to the tension region of the beams with epoxy (in Fig. 2(a)).

KB2 and KB3 beams were confined with external clamps but external longitudinal reinforcement was not used for these beams (see Figs. 2(b) and (c)). For KB2 beam, external clamps were placed along the beam with a spacing of 75 mm as shown in Fig. 2(b). For KB3 beam, external clamps were placed between supports and loading points with a spacing of 75 mm and between loading points with a spacing of 150 mm as shown in Fig. 2(c). KB3 beam was designed in order to develop a solution more economic than KB2 beam. Because shear stresses in the middle of beam subjected to vertical loads were lower than that of ends.

Both external clamps and longitudinal reinforcements were used for strengthening of KB4 beam. Clamps were confined along the beam with a spacing of 75 mm.

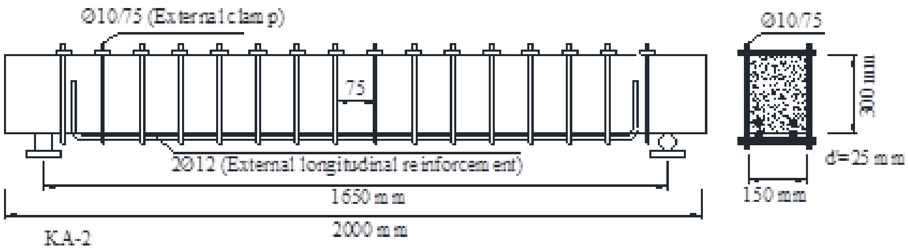


(a) KA1

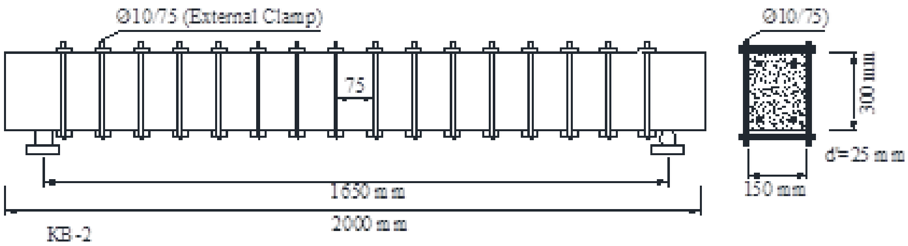


(b) KB1

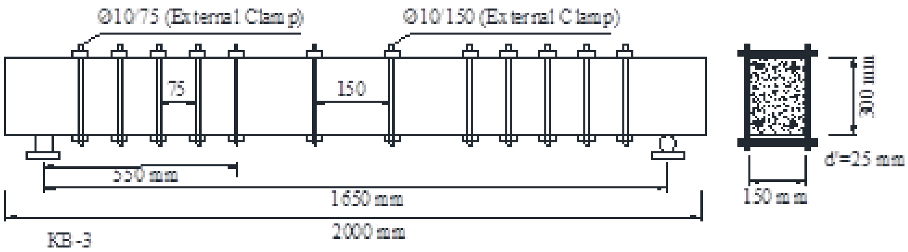
Fig. 1 Reference beams



(a) KA2

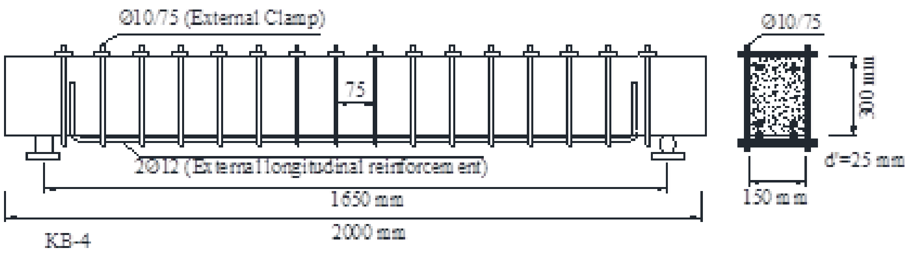


(b) KB2

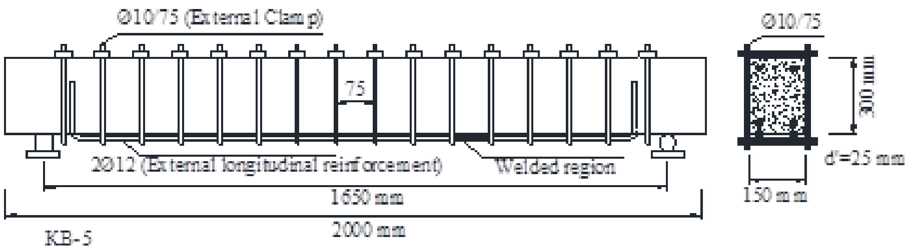


(c) KB3

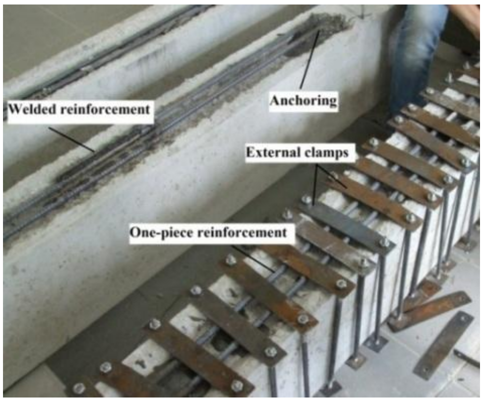
Fig. 2 Strengthened beams



(d) KB4



(e) KB5



(f) KB4 and KB5 beams

Fig. 2 Continued

Table 3 Details of test beams

	Stirrups	Top reinforcement	Bottom reinforcement	External clamp	Anchored longitudinal reinforcement	Welded-anchored reinforcement
KA1	-	-	2Ø 12	-	-	-
KA2	-	-	2Ø 12	Ø 10/75	2Ø 12	-
KB1	Ø 8/400	2Ø 10	2Ø 12	-	-	-
KB2	Ø 8/400	2Ø 10	2Ø 12	Ø 10/75	-	-
KB3	Ø 8/400	2Ø 10	2Ø 12	Ø 10/150/75	-	-
KB4	Ø 8/400	2Ø 10	2Ø 12	Ø 10/75	2Ø 12	-
KB5	Ø 8/400	2Ø 10	2Ø 12	Ø 10/75	2Ø 12	Ye-s

Longitudinal reinforcement was anchored to the tension region of the beams with epoxy as shown in Fig. 2(d) and (f).

KB5 beam was similar to KB4 but external longitudinal bars of KB5 consisted of two segments in order to apply easier according to KB4. Segments were welded together (Fig. 2(f)), after having been anchored to the tension region with epoxy.

KA2 beam was designed to improve both shear and flexure capacities of beams without stirrup. KB2 and KB3 beams were designed to improve only shear capacity of beams with low stirrup. KB4 and KB5 beams were designed to improve both shear and flexure capacities of beams with low stirrup.

Four-point load tests were carried out through steel transfer beam in two points located at 550 mm distance

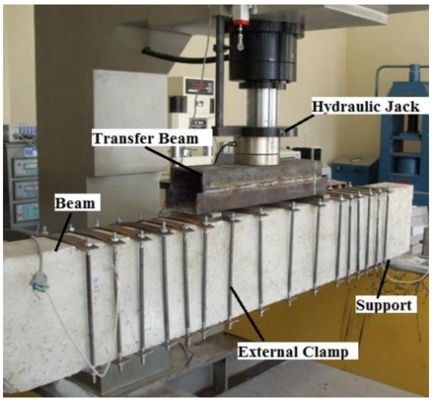


Fig. 3 Test set-up and loading scheme

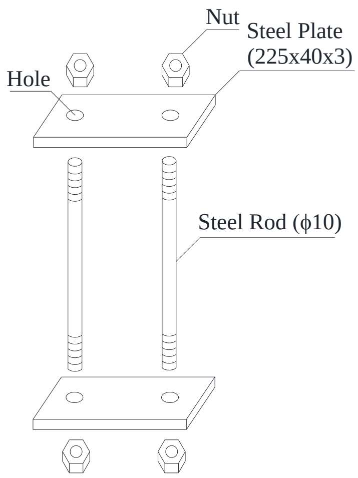


Fig. 4 The devised external clamp

from each support (Fig. 3).

The detail of the external clamp improved by the authors is shown in Fig. 4. The external clamp was consisted of nuts, steel plates and steel rods. Steel rods were 350 mm long and 10 mm in diameter. The dimensions of steel plates were 225×40×3 mm. The diameter of holes on steel plates was 11 mm.

4. Test results and discussion

This study investigates effects of external steel elements on behaviors of RC beams under flexure and shear. The developed technique considerably enhanced the load carrying capacity, ductility and energy dissipation capacity of the beams. The use of only external clamps achieved very high ductility and energy dissipation capacity. Furthermore, modes of failure were changed to flexural from shear. The use of both clamps and longitudinal reinforcements considerably enhanced the load carrying capacities of the beams. On the other hand these beams exhibited a lower ductility than beams strengthened with only external clamps. The test results of reference and strengthened beams are summarized in Table 4. Ductility is to be calculated as $\delta_{ultimate}/\delta_{yield}$ and energy dissipation capacity is to be determined using area below the load-displacement curve.

4.1 KA1 and KA2 beams

The behaviour of KA1 beam without stirrups was excessively brittle with sudden fracture (in Fig. 5(a)). KA2 beam strengthened with both external clamps and longitudinal reinforcements exhibited a more ductile

Table 4 Mean values of test results

Tests	Cracking	Yielding		Ultimate load (kN)	Displacement at failure (mm)	Ductility	Energy dissipation capacity (kN-mm)	Mode of failure
	Load (kN)	Load (kN)	Displacement (mm)					
KA1	40.3	76.5	6.50	81.6	6.7	1.0	379	S
KA2	89.4	161.4	13.2	171.8	21.2	1.6	7882	S-F
KB1	45.6	110.2	11.3	110.5	11.5	1.0	910	S
KB2	64.6	116.6	10.5	130.2	105.6	10.1	12994	F
KB3	61.2	100.7	12.9	118.1	113.9	8.8	11615	F
KB4	110.3	179.1	19.8	201.2	54.6	2.8	9980	S-F
KB5	102.2	171.5	21.4	176.4	38.3	1.8	6112	S-F

(a) KA1 (b) KA2

Fig. 5 Reference beam and beam strengthened with both external clamps and longitudinal reinforcements

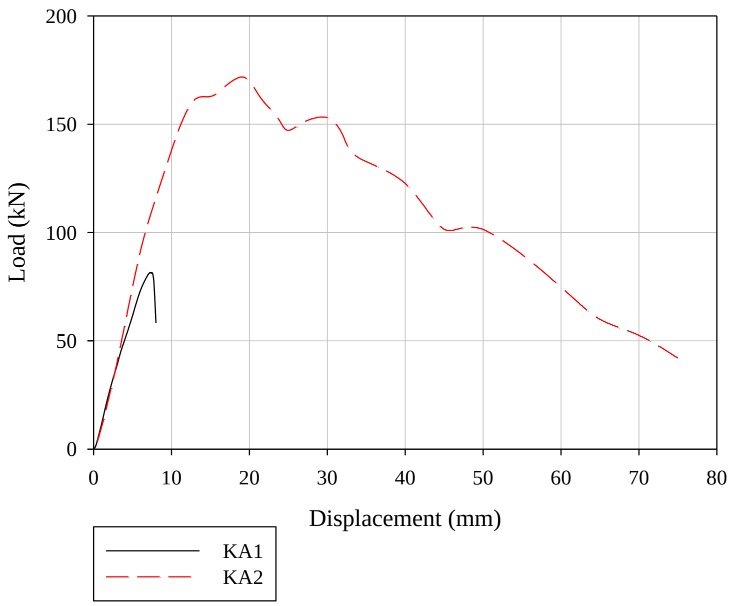
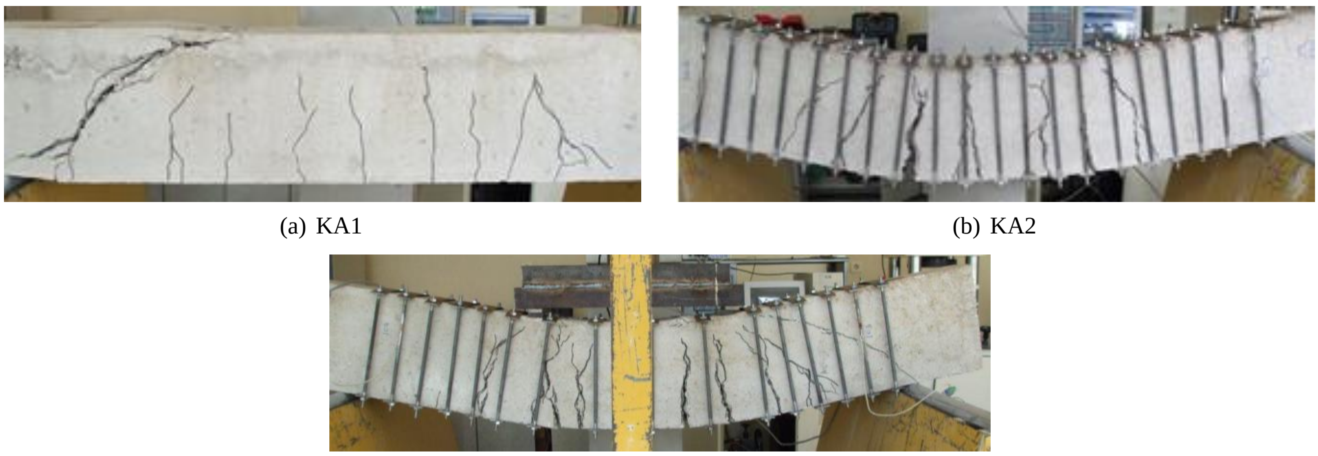


Fig. 6 Load-displacement responses of KA Beams



(a) KA1 (b) KA2

(c) KB3

Fig. 7 Reference beam and beams strengthened with only external clamps

behavior (in Fig. 5(b)). Load-displacement responses of KA Beams are shown in Fig. 6. In test, flexural cracks on KA2 beam occurred and then the slope of cracks was decreased. Fracture of KA2 beam was with shear-flexure effect. The energy dissipation and load carrying capacities of KA2 beam were significantly increased. The first crack was observed at 40.3 kN, for KA1 beam, where on the other hand, the first crack occurred at 89.4 kN, for KA2 beam. The cracking load of KA2 beam was 2.22 times higher than that of KA1. Yielding initiated at a load of 76.5 kN with a displacement of 6.5 mm in KA1, which it failed in shear at 81.6 kN with a displacement of 6.7 mm next yielding. KA2 yielded at 161.4 kN and 13.2 mm displacement. Yielding load of KA2 was 2.11 times greater than that of KA1. However, ultimate load and displacement at failure of KA2 were observed as 171.8 kN and 21.2 mm, respectively. Ultimate load of KA2 was 2.11 times higher than that of KA1. Ductility and energy dissipation capacity of KA2 were enhanced to 1.6 and 20.1 times those of KA1, respectively.

4.2 KB1, KB2 and KB3 beams

Reference beam KB1 having deficient stirrups showed brittle behavior during tests (in Fig. 7(a)). Although KB1 reached higher loads and displacements than KA1 beam, it also failed suddenly. The cracking loads of KB1, KB2 and KB3 beams were observed as 45.6 kN, 64.6 kN and 61.2 kN, respectively. KB1 beam initiated yielding at 110.2 kN and 11.3 mm and when it reached 11.5 mm, it failed suddenly. When KB1, KB2 and KB3 beams were about similar load levels, they initiated yielding. This case showed that load levels of KB2 and KB3 beams strengthened with only external clamps did not increase significantly, but

(a) KB4 (b) KB5

Fig. 8 Beams strengthened with both external clamps and longitudinal reinforcements

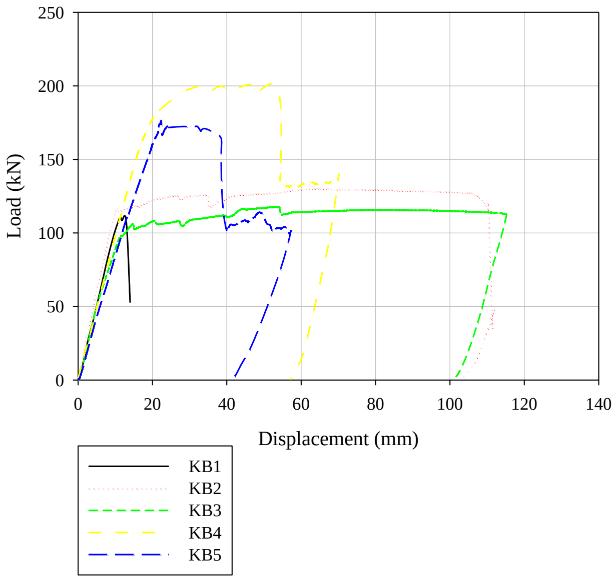


Fig. 9 Load-displacement responses of KB Beams

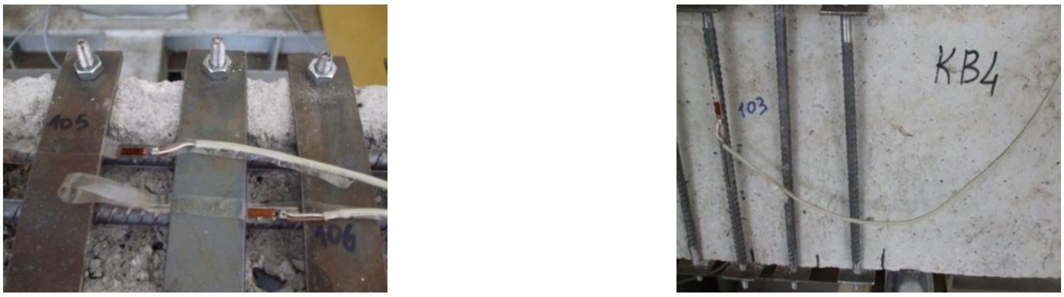
ductility was highly enhanced (in Figs. 7(b)-(c)). KB2 and KB3 beams reached about 130.2 kN and 118.1 kN, and ultimate loads were 1.18 and 1.07 times higher than that of KB1 beam, respectively. However, ductilities of KB2 and KB3 beams were enhanced to 10.1 and 8.8 times with respect to that of KB1 beam. Energy dissipation capacities of KB2 and KB3 beams increased to14.3 and 12.8 times that of KB1 beam, respectively. Although KB2 and KB3 beams strengthened with only external clamps reached the highest displacements, increasing in the load carrying capacity was insignificant. Increasing the spacing of external clamps from 75 mm to 150 mm along the central zone of beam did not make a significant change in the behavior of the beams. So, it was proposed that confinement zone at the ends of beam was applied as seismic code.

4.3 KB4 and KB5 beams

KB4 and KB5 beams were compared with KB2 beam strengthened with only external clamps. KB4 beam strengthened with anchored longitudinal reinforcements as well as clamps did not exhibit high ductility but it did not fail suddenly, either (in Fig. 8(a)). KB5 beam strengthened with anchored and welded longitudinal reinforcements as well as clamps behaved similar to KB4 (in Fig. 8(b)). The cracking loads of KB4 and KB5 beams were observed at 110.3 kN and 102.2 kN, respectively. The cracking loads of KB4 and KB5 beams were 1.71 and 1.58 times higher than that of KB2, respectively. KB4 beam yielded at 179.1 kN and 19.8 mm. KB5 beam initiated yielding at 171.5 kN and 21.4 mm. Yielding loads of KB4 and KB5 beams increased to 1.54 and 1.47 times that of KB2 beam, respectively. Ultimate loads of KB4 and KB5 beams were 201.2 and 176.4 kN respectively, and these increased to 1.55 and 1.36 times that of KB2 beam, respectively. Ductilities of KB4 and KB5 beams were at 2.8 and 1.8, respectively. Energy dissipation capacities of KB4 and KB5 beams were calculated as 9980 and 6112 kN-mm and were 0.77 and 0.47 times smaller than that of KB2 beam, respectively. These ratios were enhanced to 10.97 and 6.72 times in comparison KB1 beam, respectively. Although beams reached higher loads with using of longitudinal reinforcements as well as clamps, they did not show a good ductility. Besides, it was observed that welding of longitudinal reinforcements enhanced the behaviors of beams relative to reference beams. In addition, beam strengthened with one-piece reinforcements (KB4 beam) exhibited much better behavior than KB5 beam with welded reinforcements. Load-displacement responses of KB Beams are shown in Fig. 9.

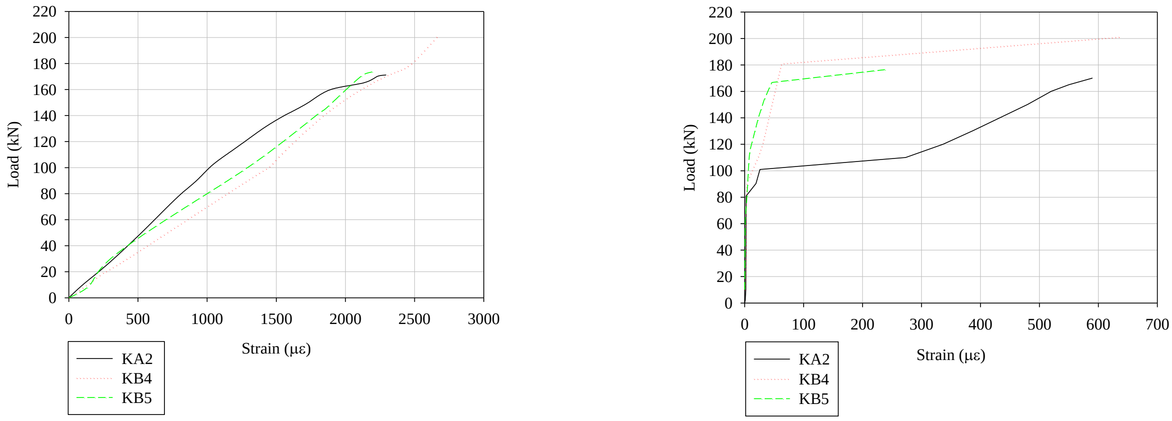
4.4 Load-strain responses

The strains of KA2, KB2, KB3, KB4 and KB5 beams were measured with strain gauges glued on external clamps



(a) Strain gauges on longitudinal bars (b) Strain gauge on clamp

Fig. 10 Strain measurements



(a) The strains in longitudinal reinforcements (b) The strains in clamps

Fig. 11 Load-strain responses

and longitudinal reinforcements (in Fig. 10).

Although an attempt was made to measure the strains of all of the strengthened beams, strains almost never occurred in KB2 and KB3 clamps. Therefore, load-strain responses of KB2 and KB3 couldn't be shown in Fig. 11. On the other hand, KA2, KB4 and KB5 reached the highest loads and the major strains in steels occurred due to diagonal cracks.

It was understood that anchorage of longitudinal reinforcements has significant effects on the behavior of beams during tests. Once beams were started loading, the strains in longitudinal reinforcements increased with a constant slope as shown in Fig. 11(a). Anchored reinforcements moved together with beams and so the strain in reinforcements got close to their yield limit strains. Fig. 11(b) shows the max strains in clamps at right and left shear spans of KA2, KB4 and KB5. When diagonal cracks on beams initiated to occurred, the slopes of the curves had shown a change with effect of shear force. Since diagonal cracks on KB2 and KB3 did not occur during tests, the strains in clamps could not be obtained.

4.5 Discussion

It has been observed that the technique proposed has significant effects on behaviors of beams. The use of external clamps alone significantly enhanced ductility of beams. But the combined of longitudinal reinforcements and clamps which increased cracking, yielding and ultimate loads of beams decreased ductility. In tests, KA2 exhibited much better behavior than reference beam and the flexural rigidity, the energy dissipation and the load carrying capacities of it were considerably enhanced. Cracks on KB2 and KB3 were controlled successfully and clamps prevented propagation of cracks. Shear failures were converted to flexural failures by using only external clamps. However, KB2 and KB3 exhibited highly ductile behavior (about 10 times more). Although ductilities of KB4 and KB5 did not considerably increase, load carrying capacities of these were significantly enhanced. However, it was observed that load-displacement behavior of beam was deteriorated at the least when two piece external reinforcements joined by welding were applied.

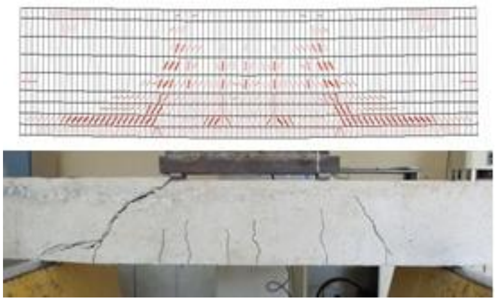
5. Nonlinear finite element analyses

A nonlinear finite element model (Wong and Vecchio 2002) was used in order to support the test results. Two-dimensional nonlinear finite element analyses (NLFEA) were carried out for all of the beams by using a package program (VecTor2) developed at the University of Toronto. The theoretical bases of VecTor2 are the Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986) and the Disturbed Stress Field Model (DSFM) (Vecchio 2000). The DSFM is a refinement of the MCFT and hence, is a smeared rotating crack model. Principals of

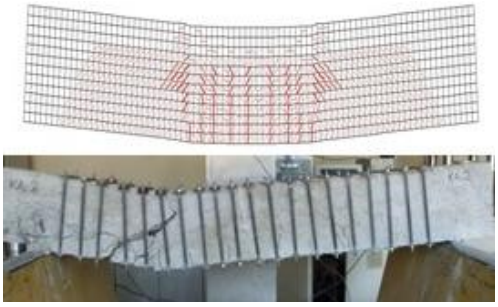
Table 5 Test and NLFEA results

Tests	Cracking load (kN)			Ultimate load (kN)			Mode of failure	
	Test	NLFEA	$P_{C,\text{test}}/P_{C,\text{NLFEA}}$	Test	NLFEA	$P_{U,\text{test}}/P_{U,\text{NLFEA}}$	Test	NLFEA
KA1	40.3	45.2	0.89	81.6	94.2	0.87	S	S
KA2	89.4	107.6	0.83	171.8	198.9	0.86	S-F	S-F
KB1	45.6	48.5	0.94	110.5	113.5	0.97	S	S
KB2	64.6	62.4	1.04	130.2	112.8	1.15	F	F
KB3	61.2	60.1	1.02	118.1	108.5	1.09	F	F
KB4	110.3	107.6	1.03	201.2	198.9	1.01	S-F	S-F
KB5	102.2	107.6	0.95	176.4	198.9	0.89	S-F	S-F
		Mean	0.96		Mean	0.98		
		COV %	7.3		COV %	10.4		

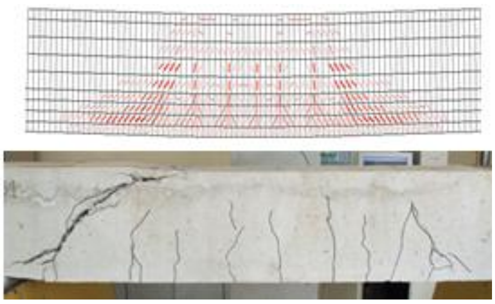
* S: Shear; F: Flexural; S-F: Shear+Flexural



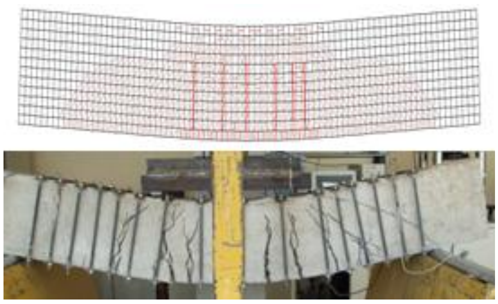
(a) KA1



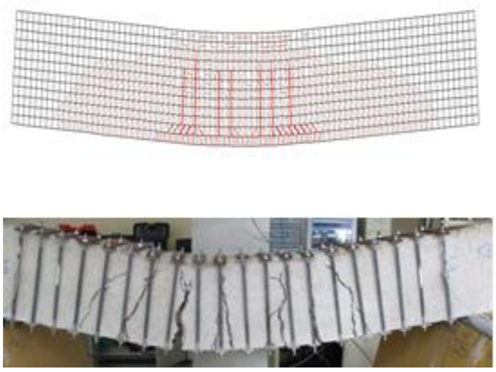
(b) KA2



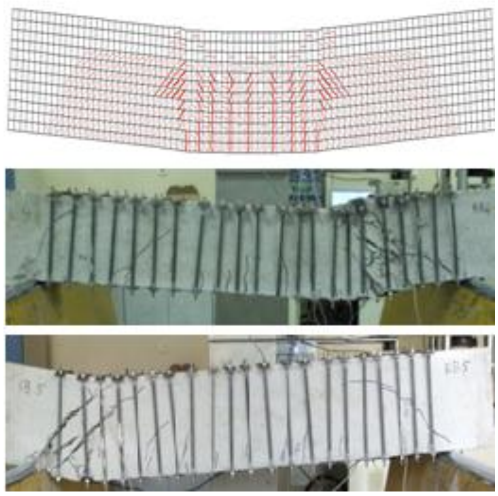
(c) KB1



(d) KB2



(e) KB3



(f) KB4-KB5

Fig. 12 Crack patterns of all beams

the formulation is the consideration of compression softening effects in the concrete, due to transverse cracking, and of tension stiffening effects due to bond mechanisms between the concrete and the reinforcement. The DSFM, unlike the MCFT, also considers divergence of principal stress and principal strain directions, and takes into account slip deformations on crack surfaces (Vecchio and Shim 2004).

The eight-degree-of-freedom rectangular mesh sizes of all beams were selected as 20×20 mm. All longitudinal reinforcements were modeled using truss bar elements; all stirrup steels were modeled in concrete as smeared reinforcement. It was assumed that external clamps could be modeled as smeared reinforcement in order to wrap beam and provide a significant confinement effect in experiments. In NLFEA, relationship of concrete and reinforcement were determined with Eligehausen model (Wong and Vecchio 2002). Mohr-Coulomb (stress) cracking criterion was used in analyses. The actual properties obtained from tests of concrete and reinforcement of specimens were presented in the Table 2. Tensile strength of concrete was estimated from the compressive strength as $0.35\sqrt{f_c'}$ (in MPa) according to TS500-2000. All constitutive modeling was done according to the default models of the DSFM. Loading was applied from two points in a displacement-control mode with a typical step size of 0.25 mm for the all beams. The results obtained from NLFEA and tests are shown in Table 5.

The cracking loads P_C, ultimate loads P_U and failure modes are summarized in Table 5. Due to smeared modeling of external clamps, a single NLFEA analysis for KA2, KB4 and KB5 was carried out. Since welded connection cannot be modeled in NLFEA, results of KB4 and KB5 are same. The values of the first cracking load and maximum load experimentally found and those calculated by NLFEA are quite close to each other. The mean of $P_{C,exp}/P_{C,NLFEA}$ values was calculated as 0.96 with an approximate standard deviation of 7.3%. The mean of the $P_{U,exp}/P_{U,NLFEA}$ values was found to be 0.98 with a standard deviation of 10.4%. Although the cracking and ultimate loads found experimentally are quite close to those by NLFEA, displacement values are not in agreement due to the rigid behavior in NLFEA. In addition, material behavior and support conditions have also contributed to this difference. For comparison the cracking forms obtained experimentally and obtained by NLFEA are shown together in Table 5 and Fig. 12. KA1 and KB1 failed by shear fracture in the experiments as a result of diagonal cracks that were also observed in the NLFEA (Figs. 12(a)-(c)). KB2 and KB3 failed at big displacements due to flexure cracks formed in both experimental analysis and NLFEA (Figs. 12(d)-(e)). KA2, KB4 and KB5 failed due to shear-flexural cracks occurring at big loads in the experiments (Figs. 12(b)-(f)). Similarly, they collapsed due to cracks formed by shear-flexural in the NLFEA.

6. Theoretical model according to TS500-2000

Design engineers can easily computed flexural and shear capacities of beams by using TS500-2000. Design shear capacity of beams strengthened with external steel clamps is determined with Eqs. (1) and (2).

$$V_r = 0.8V_{cr} + V_w + V_{clamp} \tag{1}$$

V_{cr} denotes shear cracking strength of section. V_w and V_{clamp} denote contribution of stirrups and clamps to shear strength, respectively.

$$V_r = 0.8x0.65\, f_{ctk} b_w d + \left(\frac{A_{sw}}{s} f_{ywk} d \right)_{stirrup} + \left(\frac{A_{sw}}{s} f_{ywk} d \right)_{clamp} \tag{2}$$

The nominal moment capacity of strengthened beams can be computed by using Eqs. (3) and (4) with TS500-2000.

$$0.85\, f_{ck} b_w k_1 c = A_s f_y \tag{3}$$

$$M_r = A_s f_y \left(d - \frac{k_1 c}{2} \right) = \frac{P_F L_n}{4} + \frac{P_G L_n^2}{8} \tag{4}$$

where A_s is the cross-sectional area of tensile reinforcement; k_1 is the ratio of the depth of the equivalent rectangular stress block to the depth of the neutral axis and expressed in TS500-2000. P_F, P_G, L_n are the theoretical failure load, uniform distributed load due to weight of beam and clear span of beam, respectively.

If application engineers need to this strengthening method, they can obtain load carrying capacities of beams with Eqs. (5) and (6). Clamps considerably increase shear capacity of beams and so flexural capacity is more critical than shear capacity.

$$M_r^{clamp} = \phi \left[A_s f_y \left(d - \frac{k_1 c}{2} \right) \right] \tag{5}$$

$$M_r^{clamp+bar} = \lambda \left[A_s f_y \left(d - \frac{k_1 c}{2} \right) \right] \tag{6}$$

Ø and λ are correction coefficients for calculation of

Table 6 Test and theoretical results

Specimens	Yielding load (kN)				
	Test	Theoretical	Ø	Corrected theoretical (CT)	P_{Test}/P_{CT}
KB2	116.6	95.7	1.05	100.49	1.16
KB3	100.7	95.7	1.05	100.49	1.00
KB4	179.1	163.6	1.05	171.78	1.04
KB5	171.5	163.6	1.05	171.78	1.00
KA2	161.4	163.6	1.05	171.78	0.94
				Mean	1.028
				COV %	7.5

flexural capacities of beams strengthened with only clamp and clamp + longitudinal bar, respectively. The coefficients have been obtained by using theoretical and experimental results. After design engineers easily compute flexural capacities of strengthened beams with TS500-2000, these flexural capacities should be multiplied with $\emptyset$ and λ.

$\emptyset$ and λ coefficients have been computed as 1.052 and 1.048, respectively. Since these correction factors are closed to each other, they can be rounded up to 1.05 for both cases. So, ultimate moment carrying capacity of beams having clamp and longitudinal bar is obtained by using Eq. (7).

$$M_r = 1.05\left[A_s f_y\left(d - \frac{k_1 c}{2}\right)\right] \tag{7}$$

7. Conclusions

The aim of this study is to propose a new method in order to strengthen RC beams with insufficient shear capacity by means of external clamps and longitudinal reinforcements. The RC beams were tested and the results of these tests were compared to those obtained by NLFEA analyses. In this study, the authors devised new details for external clamps and anchoring of longitudinal reinforcements. The beams strengthened by using only external clamps showed high ductility. The cracks in these beams occurred in the form of flexural and these cracks could not extend beyond the regions restricted by the external clamps. It was observed that clamp spacing had a positive effect on the behavior of the beams. It was observed that using more clamps in confinement zones of the beams where shear was more effective, was appropriate. Although the combined use of both clamps and longitudinal reinforcements considerably increased the load capacity of the beams, it caused a low increasing in ductility. Since it was difficult to anchor the longitudinal reinforcements to the beams in one piece, these reinforcements were anchored in two pieces and welded afterwards. According to the experiments, it was observed that the load carrying and displacement capacities of beams strengthened by welded longitudinal reinforcements decreased when compared to that of beams strengthened by reinforcements in one piece. The results obtained from tests were compared to those obtained from the NLFEA analysis. The cracking loads, ultimate loads and failure forms of beams were investigated by both of the abovementioned methods and very close results were obtained. In summary, the developed method could be a good alternative as it is very easy applicable to structural members, is light, no fire problem and offers a low cost approach. But, corrosion problem of external steel members should be inhibited and some precautions should be taken.

If external clamps are be properly connected to the longitudinal bars and concrete, it is predicted that a truss can be formed (the strut and tie theory) for resisting the external loads. The topic should be investigated for different strengthening schemes in the future.

Acknowledgments

The research described here was supported by the Scientific Research Project Commission of Celal Bayar University (Project No. Muh2012-33). The generous support of the construction materials by ER PREFABRIK Corporation is gratefully acknowledged.

References

Adhikary, B.B. and Mutsuyoshi, H. (2006), "Shear strengthening of RC beams with web-bonded continuous steel plates", *Constr. Build. Mater.*, **20**(5), 296-307.

Altin, S., Tankut, T., Anil, Ö. and Demirel, Y. (2003), "Response of reinforced concrete beams with clamps applied externally: An experimental study", *Eng. Struct.*, **25**(6), 1217-1229.

Arslan, G., Sevuk, F. and Ekiz, I. (2008), "Steel plate contribution to load-carrying capacity of retrofitted RC beams", *Constr. Build. Mater.*, **22**(3), 143-153.

Ceroni, F. (2010), "Experimental performances of RC beams strengthened with FRP materials", *Constr. Build. Mater.*, **24**(9), 1547-1559.

Chalioris, C.E., Thermou, G.E. and Pantazopoulou, S.J. (2014), "Behaviour of rehabilitated RC beams with self-compacting concrete jacketing – Analytical model and test results", *Constr. Build. Mater.*, **55**, 257-273.

Hamad, B., Masri, A., Basha, H. and Baalbaki, O. (2011), "Behavior of T-shaped reinforced concrete beams partially confined by structural steel", *Constr. Build. Mater.*, **25**(2), 1037-1043.

Hawileh, R.A., Naser, M., Zaidan, W. and Rasheed, H.A. (2009), "Modeling of insulated CFRP-strengthened reinforced concrete T-beam exposed to fire", *Eng. Struct.*, **31**(12), 3072-3079.

Kišiček, T., Sorić, Z. and Rak, M. (2007), "Testing beams and slabs reinforced with FRP bars", *Gradev.*, **59**(7), 581-595.

Kothandaraman, S. and Vasudevan, G. (2010), "Flexural retrofitting of RC beams using external bars at soffit level–An experimental study", *Constr. Build. Mater.*, **24**(11), 2208-2216.

Mofidi, A. and Chaallal, O. (2014), "Tests and design provisions for reinforced-concrete beams strengthened in shear using FRP sheets and strips", *Int. J. Concr. Struct. M.*, **8**(2), 117-128.

Osman, B.H., Wu, E., Bohai, J. and Abdallah, M. (2017), "Repair technique of pre-cracked reinforced concrete (RC) beams with transverse openings strengthened with steel plate under sustained load", *J. Adhes. Sci. Technol.*, **31(**21), 2360-2379.

Qin, S., Dirar, S., Yang, J., Chan, A.H.C. and Elshafie, M. (2015), "CFRP shear strengthening of reinforced-concrete T-beams with corroded shear links", *J. Compos. Constr.-ASCE*, **19**(5), 04014081.

Swamy, R.N., Jones, R. and Charif, A. (2008), "The effect of external plate reinforcement on the strengthening of structurally damaged RC beams", *Struct. Eng.*, **67**(3), 45-56.

Su, R.K.L., Siu, W.H. and Smith, S.T. (2010), "Effects of bolt plate arrangements on steel plate strengthened reinforced concrete beams", *Eng. Struct.*, **32**(6), 1769-1778.

TS500 (2000), Requirements for design and construction of reinforced concrete structures; Turkish Standards Institution, Ankara, Turkey. [In Turkish]

TS708 (2010), Steel Bars for Concrete; Turkish Standards Institution, Ankara, Turkey. [In Turkish]

Vecchio, F.J. (2000), "Disturbed stress field model for reinforced concrete: formulation", *J. Struct. Eng.*, **126**(9), 1070-1077.

Vecchio, F.J. and Collins, M.P. (1986), "The modified compression field theory for reinforced concrete elements subject to shear", *ACI Journal*, **83**(2), 219-231.

Vecchio, F.J. and Shim, W. (2004), “Experimental and analytical reexamination of classic concrete beam tests”, *J. Struct. Eng.*, **130**(3), 460-469.

Wong, P.S. and Vecchio, F.J. (2002), VecTor 2, Formworks User’s Manual.

Wu, Y. and Liu, K. (2013), “Characterization of Mechanically Enhanced FRP Bonding System”, *J. Compos. Constr.*, **17**(1), 34-49.

Yang, S. Cao, S. and Gu, R. (2015), “New technique for strengthening reinforced concrete beams with composite bonding steel plates”, *Steel Compos. Struct., Int. J.*, **19**(3), 735-757.

Zhang, D., Ueda, T. and Furuuchi, H. (2012), “Concrete cover separation failure of overlay strengthened reinforced concrete beams”, *Constr. Build. Mater.*, **26**(1), 735-745.

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