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RC beam strengthening using precast RC plate bonding technique

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Scientific paper - Preliminary note

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RC beam strengthening using precast RC plate bonding technique

The behaviour of RC beams strengthened with precast RC plates of rectangular and U cross-section is studied in the paper. An experimental investigation was conducted to assess effectiveness of strengthening with precast plates. The investigation was supported with the 2-D nonlinear finite element analysis (NLFEA). The NLFEA results showed a good level of correlation with test results. The proposed technique is considered to be a good alternative to RC jacketing and the technique is practical, economical and reliable for buildings or bridges with a greater number sisof similar beams.

Key words:

RC beam, precast RC plate, bonding, strengthening, nonlinear finite element analy

Prethodno priopćenje

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Ojačanje ab greda tehnikom povezivanja s predgotovljenom ab pločom

U radu se analizira ponašanje AB greda ojačanih predgotovljenim AB pločama pravokutnog i U-presjeka. Provedeno je eksperimentalno istraživanje s ciljem ocjenjivanja učinkovitosti ojačanja predgotovljenim pločama. To je istraživanje popraćeno i 2-D nelinearnom analizom uz primjenu metode konačnih elemenata (NLFEA). Rezultati NLFEA analize su pokazali dobro podudaranje s rezultatima ispitivanja. Smatra se da je predloženi postupak dobra alternativa ojačanju pomoću AB obloga te da je taj postupak praktičan, ekonomičan i pouzdan za zgrade ili mostove s većim brojem jednakih greda.

Ključne riječi:

AB greda, predgotovljena AB ploča, povezivanje, ojačanje, nelinearna analiza metodom konačnih elemenata

Vorherige Mitteilung

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Verstärkung von Balkenträgern durch Verbundtechniken mit vorgefertigten Stahlbetonplatten

In dieser Arbeit wird das Verhalten von Stahlbetonbalken, die mittels vorgefertigter Stahlbetonplatten rechteckigen oder U-förmigen Querschnitts verstärkt sind, analysiert. Um die Wirksamkeit der Verstärkung durch vorgefertigte Platten zu beurteilen, wurden experimentelle Versuche durchgeführt. Die Untersuchungen wurden auch durch 2-D nichtlineare Finite-Elemente-Analysen (NLFEA) begleitet. Die Rezultate der NLFEA haben eine gute Übereinstimmung mit den Versuchsergebnissen gezeigt. Das vorgeschlagene Verfahren kann als gute Alternative zur Verstärkung durch Ummantelung mit Stahlbeton angesehen werden und hat sich bei Gebäuden oder Brücken mit mehreren identischen Balkenträgern als praktisch, ökonomisch und zuverlässig erwiesen.

Schlüsselwörter:

Stahlbetonbalken, vorgefertigte Stahlbetonplatte, Verbund, Verstärkung, nichtlineare Finite-Elemente-Analyse

1. Introduction

In earthquake engineering, a growing interest is nowadays shown in the assessment and retrofit of existing RC buildings. Different approaches can be adopted in the seismic rehabilitation of RC structures. Possible solutions include: retrofit of existing RC frames, use of resisting shear walls (either by strengthening existing RC walls or inserting new shear walls), or adoption of dissipative devices [1]. The maintenance, strengthening, and upgrading of structural members are perhaps the most crucial problems in civil engineering applications. One of difficulties in the process of strengthening reinforced concrete structures is the selection of a strengthening method that will enhance the strength and serviceability of the structure. The externally bonded steel plates [2-6], and Fibre Reinforced Polymer (FRP) plates [7-9], or RC jacketing [10-15], are generally preferred by designers for the strengthening of structural elements.

Altun [16] determined and experimentally compared mechanical properties of RC beams under simple bending, before and after jacketing with RC. It was noted that the mechanical behavior of jacketed RC beams is similar to and slightly better than that of ordinary RC beams of the same dimensions, despite the fact that the core parts of the jacketed RC beams were in a yielded state. Adhikary and Mutsuyoshi [17] presented results of a parametric study showing the effects of the steel plate depth/beam depth ratio, steel plate thickness, concrete strength, and internal shear reinforcement ratio. Then a design formula for computing the shear strength of beams with web-bonded continuous steel plates was presented. A good agreement was established between the shear strengths computed using the proposed formula and FEM, and experimental results. Arslan et al. [18] investigated effectiveness of flexural strengthening with continuous horizontal steel plates and load-deflection behaviour of rectangular section RC beams after retrofitting. These retrofitted RC beams were tested under the same conditions and the contribution of the repair and strengthening techniques to the load-carrying capacity was investigated. Results obtained by a three-dimensional nonlinear finite element analysis (NLFEA) were compared with the results of experiments, and an equation was proposed for the calculation of ultimate load capacities. Su et al. [6] experimentally determined the effectiveness of bolted side-steel plated concrete beams under different bolt plate arrangements. It was observed that bolt plate arrangements have a dominant effect on the ductility performance of beams in terms of both post-elastic strength enhancement and displacement ductility. Ceroni [19] presented results of an experimental program involving RC beams equipped 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, the end or distributed U-shaped anchoring devices were applied when the strengthening involved FRP carbon sheets. Comparisons between experimental and theoretical failure loads were discussed. Raval and Dave [15] strengthened RC beams using various jacketing methods. As a result of tests on smooth surface jacketed beams, the highest load carrying capacity was observed using the jacketing technique of combined dowel connectors and bonding agent with micro-concrete as compared to other jacketing techniques used.

The objective of this study was to strengthen reference beams with precast RC rectangular and U section plates. Additionally, the effectiveness of flexural and shear strengthening made of precast plates and load-deflection behaviors of RC beams were investigated. Finally, test results were compared with NLFEA for the calculation of first cracking loads, ultimate loads, and modes of failure. It is thought that the proposed technique can be a good alternative to methods shown in the literature, and especially to traditional RC jacketing.

Some methods shown in the literature have shortcomings such as fire, corrosion, debonding under load, and complex implementation. On the contrary, the fire and corrosion problems are not in question for this technique as it is easily applicable, economic and sufficient with regard to capacity increase. In addition, the mould, reinforcement and concrete workmanships do not require in situ position. But the proposed technique increases dead weight of structures. It is a disadvantage for this technique and all jacketing methods. However, this problem can be minimized using thin plates and lightweight concrete in the proposed technique.

2. Experimental Program

Nineteen simply supported RC beams were subjected to a four point load test. Thirteen of them were strengthened with precast plates, and others were reference beams. Mechanical properties of concrete and reinforcement steel used for production of beams and plates were determined using various tests according to Turkish Standards [20, 21]. The beams were scaled in accordance with the laboratory environment and capacity of the hydraulic jack used. Only flexural and shear capacities of beams were improved in this study. However, the compressive strength might be exceeded in case of more reinforced beams. This case should be taken into account in future experiments.

2.1. Materials

A concrete mix containing Portland cement (PC 42.5) and exhibiting a maximum aggregate size of 12 mm in diameter was used in this study. Two types of concrete mixes were prepared for beams and precast plates. Before the production, compressive strengths of concrete beams and plates were intended to be 16 MPa and 35 MPa at 28 days, respectively. As a result of preliminary tests, the water/cement (w/c) ratio of existing beams was selected to be 0.93. The w/c ratio of strengthening plates was selected to be 0.57 to prevent plate damage during the testing. Concrete properties were determined using twelve 150 x 300 mm cylinder samples taken from beams and plates. The constituents and the corresponding proportions of the beam and plate concrete mixes are detailed in Table 1. The sieve analyses of coarse and fine aggregates were conducted as per TS3530-EN933-1 [22] specifications.

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

Material	**Quantity** [kg/m^3]	
	Beam	**Plate**
Water	200	220
Cement	215	380
Fine aggregate	1000	900
Coarse aggregate	950	875
Superplasticizer	2.15	3.80

Mean concrete properties obtained from uniaxial compressive tests of cylinder specimens produced for beams and strengthening plates are given in Table 2. The tensile strength of concrete was estimated from compressive strength (f'_c), according to TS500-2000 [21], and it amounts to $0{,}35\sqrt{f'_c}$ (in MPa).

Table 2. Concrete properties

Specimen	**Compressive strength** [MPa]	**Strain corresponding to top stress** [mm/mm]	**Tensile strength** [MPa]	**Modulus of elasticity** [MPa]
Beam	16.3	0.0026	1.33	22000
Plate	34.4	0.0031	1.94	29500

Three samples were taken from each type of reinforcement and tensile tests were carried out. The yield strength (f_y), ultimate strength (f_u), and modulus of elasticity obtained from tests are shown in Table 3.

Table 3. Properties of reinforcements

Bar size [mm]	E_s [MPa]	f_y [MPa]	f_u [MPa]
6	213000	505	615
8	210000	470	575
10	211000	450	550
12	207000	440	530
10*	211000	455	560

10* - Rods with epoxy

The type of epoxy adhesive including two components is a chemical anchor. This chemical adhesive is used to repair cracked concrete or bond anchorage bolts. Its tensile strength amounts to 30 MPa. The moduli of elasticity of epoxy under flexural and tensile loading are 3800 and 4500 MPa, respectively.

2.2. Experimental procedure and instrumentation

All beams were monotonically loaded up to maximum load capacities in order to define the load-displacement relationship. Nineteen RC beams, i.e. six un-strengthened reference beams and thirteen strengthened beams, were subjected to loading from two points. This load case was chosen because it gives constant maximum moment and zero shear in the section between the loads, and constant maximum shear force between the support and load. The steel transferring beam (2U160) was used for loading from two points. The beams were simply supported with the clear distance of 1350 mm between the supports, and loads were applied at 1/3 of the net length of beam with a 250 kN hydraulic jack in vertical direction. Mid-span beam displacements were measured using a linear voltage displacement transducer (LVDT). The beams were loaded in a controlled way up to the failure point. For each load increment, the displacements were measured using LVDTs placed at mid-span. The test set-up is shown in Figure 1.

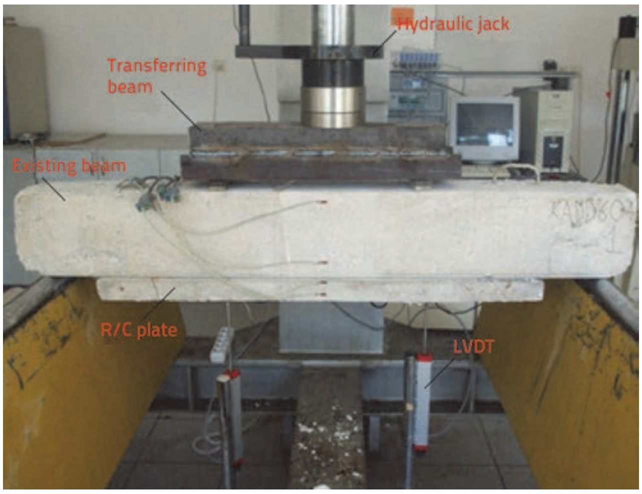

Figure 1. Test set-up

2.3. Details of test beams

2.3.1. Reference beams

Nineteen reference beams with two different stirrups were produced, and thirteen of these were strengthened with precast plates. Although the shear capacity of the first type beams named REFA was sufficient, cf. Figure 2.a, the shear capacity of the second type beams named REFB was designed insufficient, cf. Figure 2(b). The reference beam size used was 150 mm (b) x 220 mm (h) x 1500 mm (l). All reference beams had tension reinforcement (2Ø12) and compression reinforcement (2Ø10). REFAs were tied with Ø8/100 mm stirrups along the beam as shown in Figure 2(a). REFBs were tied with Ø6/350 mm stirrups along the beam as shown in Figure 2(b). Three of the produced REFAs and REFBs were subjected to loading without strengthening. In all beams, the clear concrete cover of the main flexural reinforcement was set to 20 mm.

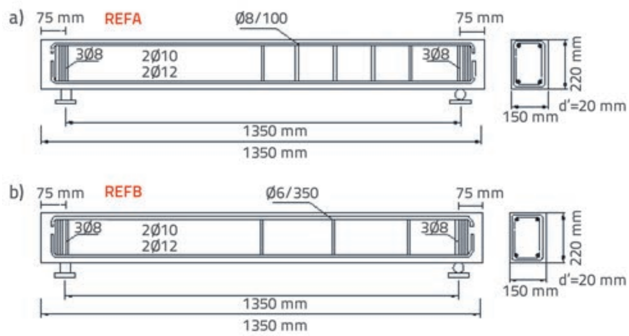

Figure 2. REFA and REFB geometry and reinforcement

2.3.2. Precast RC strengthening plates with rectangular and U sections

Thirteen strengthening plates had been produced before they were bonded to the beams. Twelve rectangular section plates, 60 mm and 120 mm in thickness and 1100 mm in length, were reinforced with 2Ø12 steel in tension zone, as shown in Figure 3. They were named SPR60 and SPR120. SPR60 and SPR120 plates were 150 mm in width. In addition, one U section plate 50 mm in thickness was reinforced with 2Ø12 steel in the tension and compression zones, as shown in Figure 3. This U section plate was named SPU50. The rectangular and U section plates were tied with Ø8/100 stirrups mm along the plate, as shown in Figure 3.

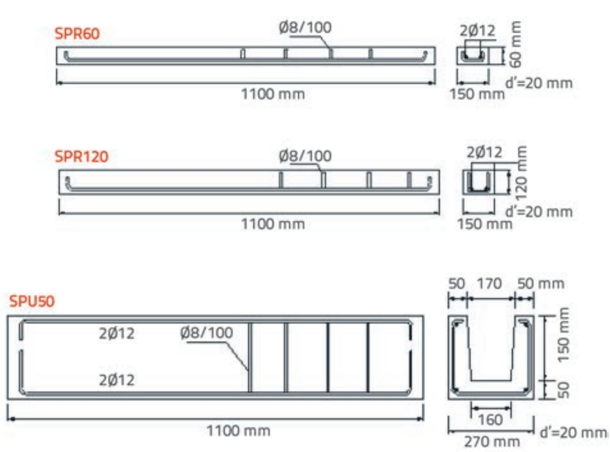

Figure 3. Geometry and reinforcement of precast RC strengthening plates

2.3.3. Bonding and jacketing applications with precast plates

Precast plates and beams were combined using various methods. SPR60 and SPR120 were bonded to the underside of the REFAs for flexural strengthening. REFAs were strengthened with externally bonded SPR60 and SPR120, and named KASPR60 and KASPR120 (Figures 4(a)-(b)). Before application of 10 mm anchor rods, the underside of the REFAs and strengthening plates was drilled to the depth of 150 mm. 12 mm diameter holes were cleaned with pressurized air. A chemical adhesive was injected into the holes, and anchor rods were inserted. Thus KASPR60 (three pieces) and KASPR120 (three pieces) were constructed.

In addition, SPR60 and SPR120 were bonded to REFAs using nuts and washers, as well as chemical adhesive and anchor rods, and named KASPR60-N and KASPR120-N. These (three pieces) were constructed in the same way as KASPR60 and KASPR120 (Figures 4(c)-(d)). Thus strengthened beams were cured under laboratory conditions for at least 28 days before loading.

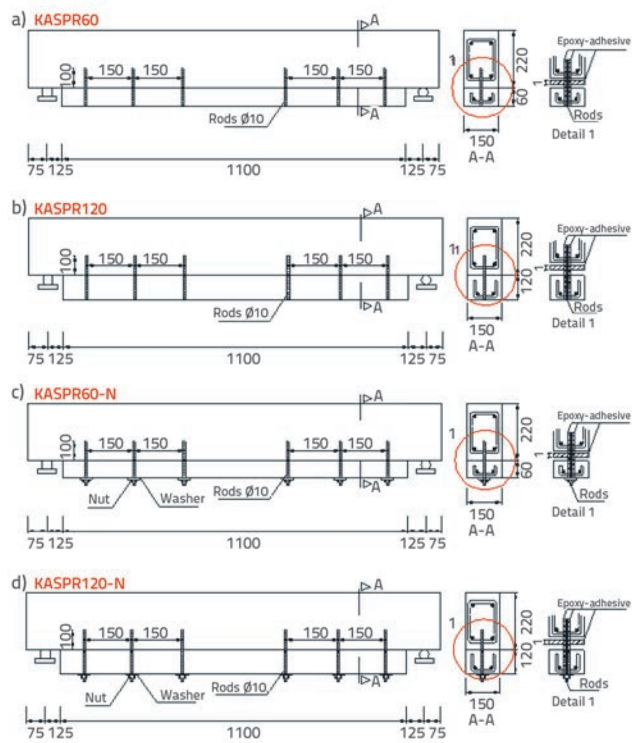

Figure 4. Flexural strengthening of beams with rectangular section plates

One SPU50 was jacketed to REFB for shear and flexural strengthening. SPU50 was bonded to the underside and sides of beams with chemical adhesive and anchor rods as shown in Figure 5. A reference beam was strengthened with externally jacketed SPU50 and named KBSPU50. Firstly, 12 mm diameter

Table 4. Properties of beams

Beams	No.	Height [mm]	Plate thickness [mm]	Bottom reinforcement	Beam stirrup reinforcement [%]	Plate stirrup reinforcement [%]	Nut	a/d
REFA	3	220	-	2Ø12	0.66	-	-	
REFB	3	220	-	2Ø12	0.10	-	-	
KASPR60	3	280	60	4Ø12	0.66	0.66	-	
KASPR120	3	340	120	4Ø12	0.66	0.66	-	2.25
KASPR60-N	3	280	60	4Ø12	0.66	0.66	Yes	
KASPR120-N	3	340	120	4Ø12	0.66	0.66	Yes	
KBSPU50	1	270	50	4Ø12	0.10	0.37	-	

Table 5. Mean values of test results

Tests / Beams	Cracking	Yielding		Ultimate		Ductility	Energy dissipation capacity [kN-mm]	Mode of failure
	Load [kN]	Load [kN]	Displacement [mm]	Load [kN]	Displacement [mm]			
REFA	49.6	77.8	7.5	85	43.9	5.89	3383	F
KASPR60	72	105	8.4	122	82.3	9.88	9327	F
KASPR120	77	101	7.8	119	69.0	8.90	7530	F
KASPR60-N	80	132.3	11.5	141.7	85.7	7.49	11272	S-F
KASPR120-N	91	121.3	9.5	174.7	43.0	4.60	6148	S-F
REFB	41.5	66.3	6.0	74.6	16.3	2.72	943	S
KBSPU50	98.7	-	-	125.3	5.1	-	550	S

S: Shear failure, F: Flexural failure, S-F: Shear+Flexural failure

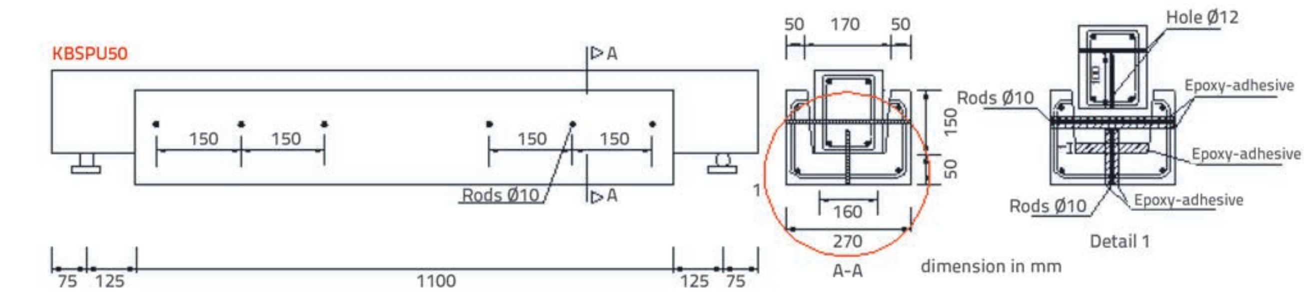

Figure 5. Shear and flexural strengthening of beam with U section plate

holes were drilled at the undersides and sides of the REFB and strengthening plate. The holes drilled from the undersides of the beams were about 150 mm in depth. The sides of the REFB were drilled across. The holes were cleaned with pressurized air. Chemical adhesive was injected into the holes and then 10 mm anchor rods were inserted (Figure 5). Properties of reference and strengthened beams are detailed in Table 4.

3 Test results and discussions

This study investigates the effects of externally bonded precast RC plates on the behaviour of RC beams under flexural and shear load. The proposed technique enhanced considerably the cracking load, ultimate load and displacement as compared to reference beams. The ultimate load and displacement were taken into account as independent of each other. Test results of reference and strengthened beams are summarized in Table 5. Abbreviations S and F stand for the shear load and flexural load, respectively.

3.1. REFA1, KASPR60 and KASPR120

REFA1, REFA2 and REFA3 failed in flexure after the crushing of concrete in the compression zone and rupture of tensile reinforcements. REFAs cracked at 49.6 kN and started yielding at about 77.8 kN and 7.5 mm (mean values). Mean ultimate load and displacement values of REFAs amounted to 85 kN and 44 mm, respectively. REFAs failed due to ductile fracture, and exhibited flexural cracks. The mean values of ductility and energy dissipation capacity of REFAs were 5.89 and 3383 kN-mm, respectively. The first cracking loads of KASPR60s and KASPR120s, strengthened with precast plates measuring 60 mm and 120 mm in thickness, increased on an average by 45 % and 55 %, respectively, as compared to REFAs. The yielding loads of KASPR60s and KASPR120s were enhanced by 35 % and 30 % on an average, and ultimate loads for these increased on an average by 44 % and 40 % in relation to REFAs, respectively. KASPR60s and KASPR120s exhibited a high ductile behaviour, and their ductilities were on an

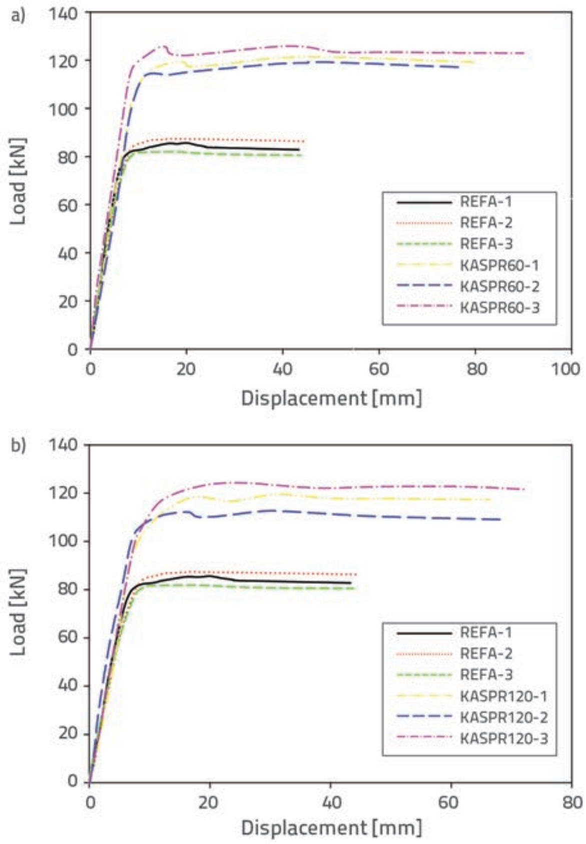

Figure 6. Load-displacement curves of REFA, KASPR60 and KASPR120

average by 68 % and 51 % higher than REFAs, respectively. The energy dissipation capacities of KASPR60s and KASPR120s increased 2.76 and 2.23 times compared to those of REFAs, respectively. Although the failure modes of KASPR60s and KASPR120s were similar, KASPR60 exhibited a more ductile behaviour. It can be observed that first cracks of KASPR60s and KASPR120s formed at the ends of plates and at plates left from beams, which increased the load level. Minor damage to strengthening plates occurred when beams failed at ultimate displacement. On the other hand, KASPR120 could not exhibit a much better behaviour in contrast to KASPR60. Figures 6(a)-(b) show comparison of load-displacement responses of REFAs, KASPR60s and KASPR120s. Cracking patterns of these beams are presented in Figure 7.

3.2. KASPR60-N and KASPR120-N

KASPR60-N and KASPR120-N, strengthened with precast plates, washers, and nuts, behaved more rigidly when compared to KASPR60 and KASPR120. The first cracking loads of KASPR60-Ns were on an average by 11 % higher than those exhibited at KASPR60. The yielding and ultimate loads of KASPR60-Ns were on an average by 26 % and 16 % higher compared to these loads for KASPR-60s. Though an average ductility of KASPR60s was by 32 % higher than KASPR60-Ns, the energy dissipation capacities of KASPR60-Ns were on an average higher by 21 % compared to KASPR60s. It can be observed that first cracks of KASPR60-Ns were in the form of flexural cracks. At the high loading levels, diagonal cracks occurred at the ends of plates due to shear stresses.

The first cracking, yielding and ultimate loads of KASPR120-Ns were on an average by 18 %, 20 % and 47 % higher than the corresponding values for KASPR120. Contrary to this, an average ductility of KASPR120s was by 94 % higher than that of KASPR120-Ns. However, the energy dissipation capacities of KASPR120-Ns decreased on an average by 18 % in comparison to KASPR120s. Although first cracks of KASPR120-Ns were flexural cracks, diagonal shear cracks occurred at the ends of plates at high load levels due to major shear stresses.

During the testing, KASPR60-Ns exhibited a more ductile behaviour compared to KASPR120-Ns. The usage of washers and nuts increased the load capacity of beams, and also

Figure 7. Crack patterns of: a) REFA-1; b) REFA-2; c) KASPR60; d) KASPR120

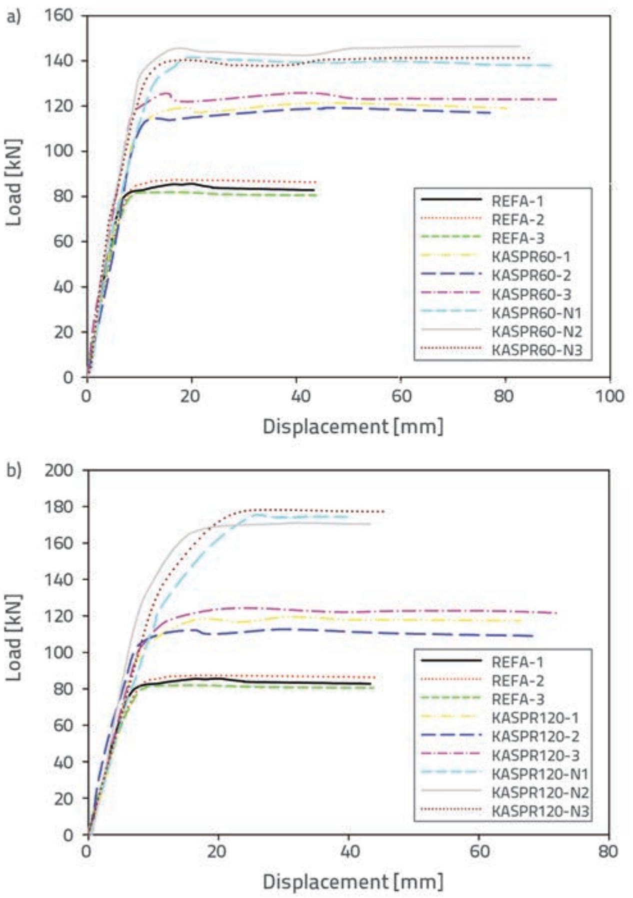

Figure 8. Load-displacement curves of REFA, KASPR60, KASPR120, KASPR60-N and KASPR120-N

decreased the displacement capacity of these beams. As the loading increased, a considerable tilting and constriction of anchor rods was observed (Figure 9.d). Figures 8.a and 8.b show comparison of load-displacement responses of REFAs, KASPR60s, KASPR120s, KASPR60-Ns, and KASPR120-Ns. The cracking patterns of KASPR60-N and KASPR120-N are presented in Figure 9. Additionally, anchor rods on beams with nuts subjected to deformations such as tilting and constriction at high load levels, are shown in Figures 9.c and 9.d.

3.3. REFBs and KBSPU50

In REFBs the first cracks occurred in form of flexural cracks at the load level of about 41.5 kN. REFB1, REFB2 and REFB3 failed in shear, as designed, after considerable diagonal cracking. Although REFBs started yielding at the levels of about 66 kN and at 6 mm (mean values), they failed quite suddenly due to shear cracks. Mean ultimate loads and displacements of REFBs amounted to about 74.6 kN and 16 mm, respectively. Mean values of ductility and energy dissipation capacity of REFBs were 2.67 and 943 kN-mm, respectively. Although the first cracking and ultimate loads of KBSPU50 increased significantly, it exhibited an excessively brittle behaviour. The first cracking load of the KBSPU50 strengthened with a precast U section plate 50 mm in thickness, increased on an average by 2.38 times compared to REFBs. The ultimate load of KBSPU50 was on an average by 68 % higher than that exhibited by REFBs. The energy dissipation capacity of KBSPU50 decreased by 0.58 times compared to that of REFBs. Although the shear strength of KBSPU50 was increased, it failed once again in shear. The first diagonal crack occurred at a very high load level when the effect of U plate was present. After the first crack occurred in beam, the crack propagated quickly, and the strength exhibited by the existing beam was found to be insufficient. The failure modes of REFBs and KBSPU50 were found to be similar. The comparison of load-displacement curves of REFBs and KBSPU50 is given in Figure 10. Cracking patterns of these beams are shown in Figure 11.

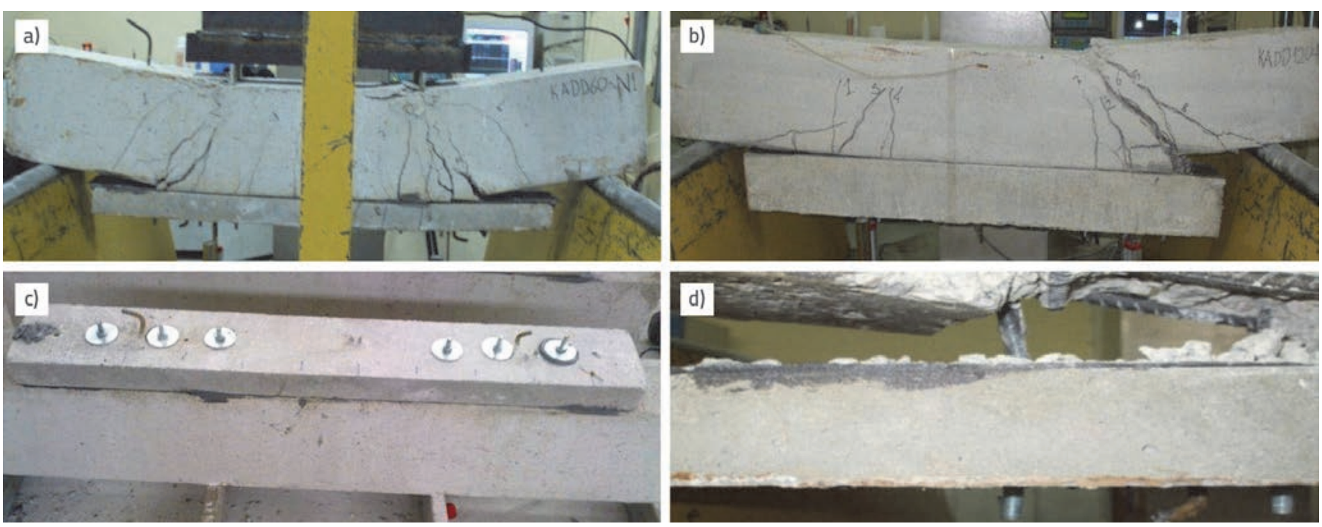

Figure 9. Crack patterns of: a) KASPR60-N; b) KASPR120-N; c) anchor rods with nuts; d) tilting of anchor rod with nut

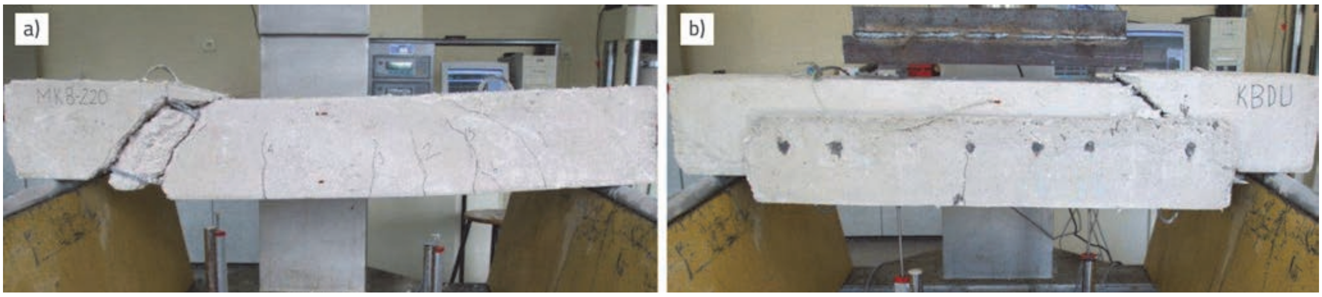

Figure 11. Cracking patterns of: a) REFB; b) KBSPU50

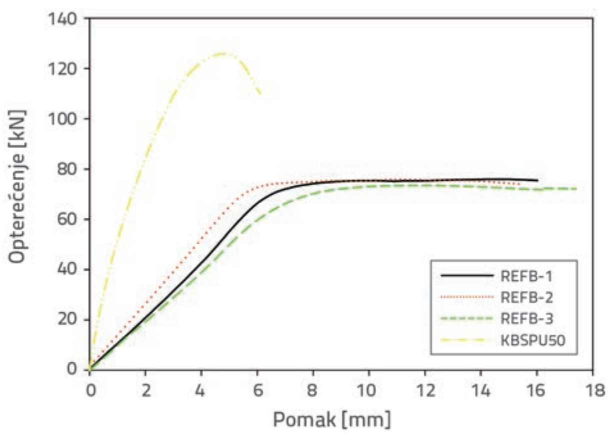

Figure 10. Load-displacement curves of REFB and KBSPU50

3.4. Discussion

The efficiency of strengthening applied with precast plates was investigated in this study. The first cracking, yield, and ultimate load capacities of beams strengthened with rectangular plates, were considerably enhanced with regard to reference beams. However, ductilities and energy dissipation capacities of these beams were increased. High shear stresses caused debonding at the ends of plates. Nuts were used to prevent debonding at the ends and they positively affected behaviour of these beams. An increase in plate thickness decreased ductility of beams. On the other hand, the use of nuts with thinner plates considerably enhanced the load carrying and energy dissipation capacities of beams. Though the use of nuts with thicker plates increased load carrying capacities, ductilities were decreased and diagonal cracks were observed at high load levels. The testing revealed that beams strengthened with thinner plates and nuts exhibited better behaviour compared to other beams. The beams with an insufficient shear capacity were strengthened with precast U-section plates, and they reached a high loading level. Although U plates enhanced shear and flexural capacities of reference beams, a ductile behaviour could not be observed due to an insufficient shear capacity of existing beams at high loads. These beams proved to be excessive brittle and failed with major diagonal shear cracks. It is thought that the proposed method, which enlarges the width and depth of beams, can be used as an alternative to the traditional jacketing method. Hence, it is supposed that the beams strengthened with the proposed method can exhibit a good behaviour under seismic loads.

4. Nonlinear finite element analyses

A nonlinear finite element model [23] was used in order to support the test results. Two-dimensional nonlinear finite element analyses (NLFEA) were carried out for each of the REFA, REFB, KASPR60-N, KASPR120-N and KBSPU50 beams. Comparisons with NLFEA were made according to KASPR60-N and KASPR120-N, since bonding problems for KASPR60 and KASPR120 were observed during the tests. The NLFEA were carried out using the program VecTor2 developed at the University of Toronto. The Modified Compression Field Theory (MCFT) [24] and the Disturbed Stress Field Model (DSFM) [25] are the theoretical bases of the VecTor2 software. The DSFM is a refinement of the MCFT and, hence, it is a smeared rotating crack model. The consideration of compression softening effects in the concrete due to transverse cracking, and tension stiffening effects due to bond mechanisms between the concrete and the reinforcement, are of principal significance for the formulation. Unlike the MCFT, the DSFM also considers divergence of the principal stress and principal strain directions, and takes into account slip deformations on crack surfaces [26].

The eight-degree-of-freedom rectangular mesh sizes of 20x20 mm were selected for REFA, REFB and KASPR60-N. Mesh sizes of KASPR120-N and KBSPU50 were 25x25 mm. A perfect bond between beams and plates was assumed in NLFEA. All longitudinal reinforcements were modelled using truss bar elements; all stirrup steel was modelled as smeared reinforcement. In the NLFEA, the relationship between concrete and reinforcement was determined with the Eligehausen model [23]. Mohr-Coulomb (stress) cracking criterion was used in the analyses. Actual properties obtained by testing concrete and reinforcement specimens are presented in Tables 2-3. The tensile strength of concrete was estimated from the compressive strength (in MPa) according to TS500-2000 [21] and it amounts to $0{,}35\sqrt{f'_c}$. All constitutive modelling was done according to default DSFM models. The load was applied from two points in a displacement-control mode with a typical step size of 0.25 mm for all beams. The first cracking loads,

Table 6. Comparison of test and NLFEA results

Beams \ Tests	Cracking load [kN]			Ultimate load			Mode of failure	
	Test	NLFEA	$P_{C,NLFEA}/P_{C,test}$	Test	NLFEA	$P_{U,NLFEA}/P_{U,test}$	Test	NLFEA
REFA	49.6	53.2	1.07	85	85.5	1.01	F	F
KASPR60-N	80	82.2	1.03	141.7	157.7	1.11	S-F	S-F
KASPR120-N	91	98.4	1.08	174.7	187.1	1.07	S-F	S-F
REFB	41.5	45.2	1.09	74.6	70.6	0.95	S	S
KBSPU50	98.7	99.8	1.01	125.3	129.6	1.03	S	S
		Mean value	1.06		Mean value	1.03		
		Standard deviation	0.03		Standard deviation	0.055		

S: Shear failure, F: Flexural failure, S-F: Shear+Flexural failure

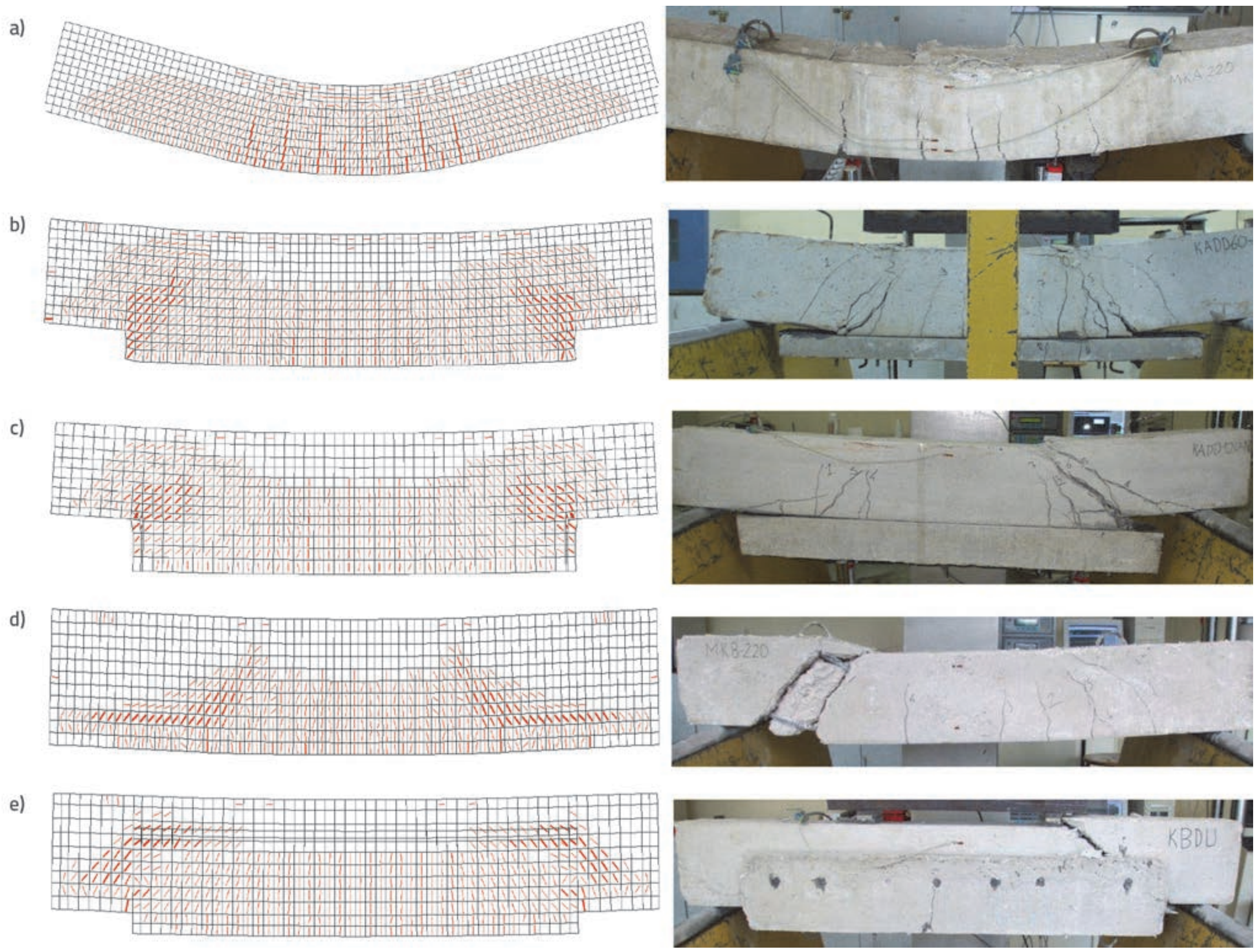

Figure 12. Comparison of test and NLFEA crack patterns

ultimate loads and failure modes of REFA, REFB, KASPR60-N, KASPR120-N and KBSPU50 were obtained from NLFEA, and the corresponding results are comparatively presented in Table 6. The comparison of failure modes obtained by testing and NLFEA is given in Figure 12.

Mean differences between the first cracking and ultimate loads in the test and NLFEA amounted to 6 % and 3 %, respectively. However, standard deviations (SD) at the first cracking and ultimate loads in the test and NLFEA were 3 % and 5.5 %, respectively. Test and NLFEA results showed a good level of correlation with one another in terms of the first cracking, ultimate loads, and modes of failure. REFAs failed with flexural cracks at 85 kN in the tests, and at 85.5 kN in the NLFEA. REFBs failed with diagonal shear cracks at 74.6 kN in the tests, and at 70.6 kN in the NLFEA. In the tests, major shear stresses and debonding were observed at the ends of

KASPR60-Ns. Flexural cracks occurred, its slope decreased at high loads, and KASPR60-Ns failed at 141.7 kN with shear-flexural cracks. Similarly, KASPR60-Ns failed with shear-flexural cracks at 157.7 kN in the NLFEA, and shear stresses occurred at the ends of plates. KASPR120-Ns failed with diagonal cracks at 174.7 kN in the tests, and at 187.1 kN in NLFEA, Its plate was not damaged but major shear stresses occurred at the ends of the plate in both analyses. KBSPU50 failed with major shear cracks at 125.3 kN and 129.6 kN in the tests and NLFEA, respectively, and the plate was hardly damaged in both analyses.

5. Conclusion

The main objective of this study was to investigate the behaviour of RC beams strengthened using precast RC plates. The following conclusions can be drawn:

- Precast RC plates that had rectangular and U sections were bonded to RC beams using different methods. In the first method, beams and plates of rectangular sections were attached using chemical adhesive and anchor rods. In the second method, beams and plates of rectangular section were attached using nuts and washers as well as adhesive and anchor rods. In the third method, beams and plates of a U section were connected using adhesive and anchor rods. Plates of rectangular section measured 60 mm and 120 mm in thickness. U-section plates measured 50 mm in thickness.
- The behaviour of beams was significantly affected by anchor rods and their positions. As the length and section of the beams were small, the beams could be damaged if many rods were used. Thus, the optimum number of rods was determined after several trials.
- All beams were subjected to load from two points. The load carrying capacities and displacement capacities of strengthened beams were enhanced significantly.
- In comparison with REFAs, the ultimate loads of KASPR60s and KASPR120s increased by 44 % and 40 %, respectively. Mean ultimate load values of KASPR60-Ns were by 16 % higher than the corresponding values for KASPR60s. Mean ultimate load values of KASPR120-Ns were by 47 % higher than the corresponding values for KASPR120s. The KASPR60 behaviour was more ductile compared to other units. On the other hand, the energy dissipation capacity of KASPR60-N was higher compared to other units.
- It was observed that the beam behaviour during the testing was significantly affected by washers and nuts.
- The ultimate load of KBSPU50 increased mean 68 % to REFBs. But KBSPU50 exhibited an excessively brittle behaviour. Although beams failed with diagonal shear cracks, precast U-section plates did not suffer any damage.
- The first cracking and ultimate loads of the beams strengthened with precast plates were directly related to the yield and ultimate strength of longitudinal bars, tensile reinforcement ratio, stirrup reinforcement ratio, use of nuts, and concrete strength. However, it can be seen that yielding and ultimate displacements of the beams particularly depend on the RC plate thickness and use of nuts.
- NLFEA were conducted with using the VecTor2 program. A perfect bond was assumed between the beams and plates. Test and NLFEA results showed a good correlation in terms of the cracking, ultimate load, and mode of failures.
- The proposed method appears to be a good alternative to strengthening with FRP, steel plates, and especially traditional RC jacketing. There are no fire and corrosion problems when this method is applied. The method is easily applicable, economic, and sufficient with regard to the increase of capacity and, additionally, the moulding, reinforcement and concrete works do not require in situ positioning.

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