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Strengthening of reinforced concrete beams with epoxy-bonded perforated steel plates

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Abstract. Although being one of the most popular strengthening techniques in reinforced concrete beams, the use of steel plates bonded to the soffit raises problems of ductility. This study aims at investigating the influence of the use of perforated steel plates instead of solid steel plates on the ductility of reinforced concrete beams. A total of nine reinforced concrete beams were tested. In addition to an unplated beam, eight beams with perforated steel plates of two different thicknesses (3 mm and 6 mm) were subjected to monotonic loading. Effect of bonding the plates to the beams with anchor bolts and with additional side plates bonded to the sides of the beam with and without anchors is also investigated. The use of bolts in addition to epoxy was found to greatly contribute to the ductility and energy absorption capacity of the beams, particularly in specimens with thick plates (6 mm) and the use side plates in addition to the bottom plate was found to be ineffective in increasing the ductility of a concrete beam unless the side plates are attached to the beam with anchors bolts. The thickness of the plate was found to have little effect on the bending rigidity of the beam.

Keywords: perforated steel plate; beam strengthening; beam repair; plated beam; reinforced concrete beam; bending stiffness; ductility; modulus of toughness; flexural behavior

1. Introduction

The load-carrying capacity of a structure may need to be increased or restored due to various reasons including the change in the intended use, deficiencies in the design and construction, corrosion of reinforcement, changes in the load-bearing system, compliance with the new code requirements, and improvement of the earthquake resistance. There are various methods used in strengthening and repair of concrete beams. Adding new reinforcement and new concrete layer to the bottom or top of a beam is a strengthening method that offers a significant improvement to the load capacity by also preserving the ductile behavior of a concrete beam. Nevertheless, the implementation of this strengthening method is rather cumbersome due to difficulties in the in-place welding of the additional reinforcing bars to the existing ones and compaction of the new concrete

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layer. Wrapping a concrete member with FRP sheets is another common strengthening and repair method. Although this method provides improvement to the load-carrying capacity of the member due to the high tensile strength of FRP composites, the brittle stress-strain behavior of FRP results in a decrease in the overall ductility of the member. Strengthening concrete beams with externally bonded steel plates is also a common practice due to the relatively low prices and availability of steel and the easy implementation of this method. Previous studies have indicated that reinforced concrete beams with external steel plates have a more ductile behavior compared to the reinforced concrete beams with external FRP laminates but this ductility is limited (Arslan 1991).

It is the primary aim of the present study to investigate the enhancement in the ductility of externally plated reinforced concrete beams as a result of the use of perforated steel plates instead of solid (nonperforated) steel plates. The elliptical form gained by the perforations as a result of the elongations in the steel plate is expected to enhance the deformation capacity of the plate and therefore the ductility of a concrete beam. To investigate the improvement in the ductility, a total of nine rectangular reinforced concrete beams were tested. In addition to an unplated reference beam, eight beams with perforated steel plates of two types were subjected to four-point bending. The thickness of the strengthening plate and the type of connection of the plate to the beam (epoxy-bonded, epoxy-bonded and bolted, epoxy-bonded and connected to the sides through collars with and without anchor bolts) are the parameters whose effects on the ductility and strength of the plated beams were investigated in the present study.

2. Literature review

Externally plated reinforced concrete beams fail in flexure if the plate remains attached to the beam up to the ultimate bending moment capacity. In the case of flexural failure of under-reinforced concrete beams, the external plate and the internal reinforcement yield followed by the crushing of concrete. If not properly designed, an externally plated beam may fail due to the separation of the plate prior to reaching the flexural capacity, denoted as the premature peeling failure. This failure can be induced by the stresses from flexural moments at plate curtailment, denoted as flexural peeling, or by the stresses from shear forces at plate curtailment, denoted as shear peeling. In flexural peeling, there is a more gradual separation of the plate and increase in the beam curvature compared to shear peeling, which is a sudden mode of failure initiated by the formation of diagonal tension cracks in the vicinity of the supports (Oehlers 1992). Most of the studies in the literature have focused on premature peeling failure of externally plated reinforced concrete beams, which can be so detrimental that a strengthened beam can fail under moments smaller than the bending moment capacity of even the unstrengthened beam (Raoof *et al.* 2000). The research on the externally plated reinforced concrete beams stems back to the pioneering research of L'Hermite and Breson (1967).

Oehlers and Moran (1990) studied flexural peeling of strengthening plates glued to the soffits of reinforced concrete beams and found out that flexural peeling depends on the thickness of the plate, concrete strength, flexural rigidity of the plated section, and initial curvature of the beam and is independent from the loading history of a beam prior to strengthening, i.e. precracking. Oehlers and Moran (1990) termed the moment at which peeling starts as the serviceability peeling strength (M_{sp}) and the moment at which complete plate separation takes place as the ultimate peeling strength (M_{up}) and proposed the following expressions

$$M_{sp} = \frac{EI_{cp} \cdot f_t}{0.827 \cdot E_s \cdot t} \tag{1}$$

$$M_{up} = \frac{EI_{cp} \cdot f_t}{0.474 \cdot E_s \cdot t} \tag{2}$$

where t is the plate thickness; E_s the elastic modulus of steel; EI_{cp} the flexural rigidity of cracked plated beam; and f_t the tensile strength of concrete. The constants 0.827 and 0.474 in the denominator were determined from the available experimental data. Later, Oehlers (1992) investigated shear peeling of externally plated reinforced concrete beams and found out that this type of peeling is not affected from the lateral reinforcement in the beam and solely depends on the shear strength of the unplated beam with no stirrups. The experiments indicated that there is a strong interaction between flexural and shear peeling modes, which was expressed by the following interaction equation

$$\frac{M_p}{M_{up}} + \frac{V_p}{V_{uc}} \geq 1.17 \tag{3}$$

where M_p and V_p are the moment and shear at plate curtailment location at the instant of peeling; V_{uc} is the shear strength of a reinforced concrete beam without stirrups and external plate. Eq. (3) suggests that a plated RC beam is prone to failure due to plate peeling when the summation of the nondimensional moment term (M_p/M_{up}) and the nondimensional shear term (V_p/V_{uc}) exceeds 1.17. The term “failure” corresponds to the complete separation of the plate from the beam.

Ziraba *et al.* (1994) established a design method for externally plated reinforced concrete beams. Accordingly, a beam is first designed for flexure in a way that the yielding of the external plate and internal reinforcement precedes the crushing of concrete and later checked for shear failure and plate debonding to prevent the premature plate peeling failure which decreases the ductility of the beam to a major extent.

Hussain *et al*. (1995), who studied the flexural behavior of externally plated precracked reinforced concrete beams, found out that the failure mode of an externally plated beam shifts from the ductile flexural failure to the brittle plate peeling failure and the ductility of the beam decreases as the reinforcement ratio of the beam including the external plate increases. The anchorage of the plate end to the beam was found to contribute to the ductility of the beam. However, this contribution was observed to decrease as the plate thickness increases. The ultimate moment estimates calculated on the basis of the ACI Code (ACI 1989) were shown to be in good agreement with the ultimate moments of the beams with thin plates, while the beams with thick plates failed at moments significantly smaller than the estimated values. Finally, externally plated concrete beams were observed to develop diagonal tension cracks at load levels below the shear capacities calculated on the basis of ACI recommendations (ACI 1989).

Raoof *et al.* (2000) carried out an extensive parametric study on the premature plate peeling failure mode of externally plated reinforced concrete beams. The study uses a previously developed theoretical model (Raoof and Zhang 1997) which assumes that the steel plate and the concrete tooth formed by the cover concrete between two adjacent stabilized cracks separates from the rest of the beam when the tensile stress at concrete fibers surrounding the extreme layer of tension reinforcement reaches the tensile strength of concrete. The theoretical solution (Raoof *et al.* 2000) provides upper and lower bound values to the axial stress in the steel plate at which plate peeling failure takes place. The lower bound value is obtained from the following equation

$$\sigma_{s\,\min} = 0.154 \cdot \frac{L_p \cdot h_s \cdot b^2 \cdot \sqrt{f_{cu}}}{h' \cdot b_s \cdot t \cdot (\Sigma 0_{bars} + b_s)} \tag{4}$$

where b_s is the width of external plate; b the beam width; t the plate thickness; h' the net concrete cover height; h_s half of the total depth of the effective tension area of concrete; f_{cu} the cube strength of concrete; and $\Sigma 0_{bars}$ the total circumference of the tensile reinforcing bars. The upper bound value of the axial plate peeling stress is given as the twice the lower bound value ($\sigma_{s\max} = 2\sigma_{s\min}$) by Raoof *et al*. (2000). L_p is defined by Raoof *et al.* (2000) as the effective length of the steel plate in the shear span over which the stresses in the plate/concrete interface remains constant, which is the lower of the actual plate length in the shear span (L_p^1) and the length L_p^2 obtained from the following equation

$$L_p^2 = \begin{cases} l_{\min} \cdot (21 - 0.25) l_{\min} & l_{\min} \leq 72 \text{ mm} \\ 3 \cdot l_{\min} & l_{\min} > 72 \text{ mm} \end{cases} \tag{5}$$

where $l_{\min}$ is the minimum stabilized crack spacing in externally plated beams, obtained from

$$l_{\min} = \frac{(b \cdot 2h_e) \cdot f_t}{u \cdot (\Sigma 0_{bars} + b_s)} \tag{6}$$

where u is the steel/concrete bond strength. The British BS 8110 (BSI 1997) Standard gives the following formulae for calculating u and f_t

$$u = \beta \cdot \sqrt{f_{cu}} \tag{7}$$

$$f_t = 0.36 \cdot \sqrt{f_{cu}} \tag{8}$$

where β is a coefficient dependent on the bar type, taken 0.28 and 0.50 for plain and deformed bars in uniaxial tension, respectively.

The parametric study carried out by Raoof *et al.* (2000) and the comparison of the analytical estimates from the theoretical solution with the experimental results of Oehlers and Moran (1990) indicated that the ratio of the ultimate plate peeling moment (M_{up}) to the ultimate flexural moment (M_{ult}) of an externally plated reinforced concrete beam slightly increases with increasing concrete cube strength, while increasing the beam width and decreasing the plate thickness result in a more pronounced increase in the moment ratio. Furthermore, it was found out that for a constant amount of embedded tension reinforcement, beams reinforced with fewer but larger bars have higher ultimate peeling moments compared to the beams reinforced with more and smaller bars and the ultimate peeling moment increases as the number of layers of tension reinforcement increases. Reducing the ratio of the plate width to the beam width was found to increase the M_{up}/M_{ult} ratio and the presence of flexural cracks in the strengthened beam was shown to have little influence on the peeling moment.

Sallam *et al.* (2004) carried out an extensive study on the flexural behavior of externally plated reinforced concrete beams. It was found out that replacement of the cover concrete with grout increased the resistance of the beams to crack initiation and crack propagation at plate curtailment, offering an effective solution to peeling problem. Anchorage of the bottom plate to the beam by bolts or clamps proved to be an effective method for improving the ductility and load-carrying capacity of the beam. Finally, the use of side plates bolted to the beam was observed to greatly improve the strength of the beam by preventing peeling of the bottom plate.

Altin *et al.* (2005) studied strengthening of shear deficient reinforced concrete beams with external steel plates epoxy-bonded to the sides of the beam along the shear span. Two reference (unstrengthened) and nine strengthened T-shaped beams with three different strengthening arrangements were tested. It was found that all of the strengthened beams had ultimate strengths, rigidities, and ductilities superior to the reference beams. The increase in the concrete surface area covered by the external plate was found to decrease the amount and extent of shear cracking in the beam and the use of several separate external plates along the shear span instead of a continuous single piece was shown to increase the ductility of an externally plated beam.

Su and Zhu (2005) investigated structural performance of reinforced concrete coupling beams strengthened with bolted steel side plates. The strengthened beams were observed to have higher load-carrying, deformation, and energy dissipation capacities compared to the unstrengthened control beam. A parametric study using nonlinear finite element analysis indicated that even a small slip between the bolt and the beam can cause significant reductions in the load-carrying capacity of the connection, which affects the behavior of the beam considerably. Later Zhu and Su (2010) developed two different theoretical analysis models for estimating ultimate strengths of steel side plate strengthened reinforced concrete coupling beams. The model considering the longitudinal and vertical slips and rotation of the bolt connection group, connecting side plates to the beam, was found to provide estimates in a closer agreement with the experimental results obtained by Zhu *et al.* (2007).

Barnes and Mays (2006a) studied the influence of externally bonded steel side plates on the shear resistance of reinforced concrete beams. They tested 15 rectangular and 15 T-shaped beams having different shear span to depth ratios to investigate the influence of external plating on three different modes of shear failure: diagonal tension, shear compression, and deep beam failure. The experiments indicated that the side plates were successful in preventing the underlying shear cracks from opening until bond failures occurred in concrete. Side plates provided considerable increase in the shear capacities of both rectangular and T-shaped beams, designed in accordance with the shear requirements of the BS 8110 (BSI 1997) Code. Finally, the extension of the side plate over the full depth of the web of a T-beam was shown to be an efficient method for increasing the shear resistance of the beam. Barnes and Mays (2006b) were able to develop a design method for rectangular and T-shaped reinforced concrete beams with continuous external steel side plates by using the strain values measured in the steel plates bonded to a number concrete block specimens tested by Barnes and Mays (2006a).

Arslan *et al.* (2008) conducted experiments and numerical analyses on rectangular reinforced concrete beams repaired with external bottom and side plates epoxy-bonded to the beam. The tests indicated that the side plates significantly contribute to the flexural performance of retrofitted beams by preventing propagation of the cracks forming at the ends of the bottom plates. The best flexural performance was attained in beams with side plates of equal length with the bottom plate.

Aykac and Ozbek (2011) tested full-scale T-shaped reinforced concrete beams strengthened with steel plates bonded to the bottom (tension face), top (compression face), and sides of the beam. The use of top plate in addition to the bottom plate and the use of bolts and/or side plates for the plate end anchorage were found to improve the flexural behavior of an externally plated concrete beam and increase its ductility.

3. Experimental pogram

The present experimental program was carried out to investigate the influence of the use of perforated steel plates on the ductility and deformability of externally plated reinforced concrete beams. The plates were epoxy-bonded to the beams. Penetration of the adhesive into the perforations during the application of epoxy was considered to provide a better bonding between the external plate and the beam. Additionally, the influence of the plate thickness, use of bolts and/or side plates (collars) to anchor the bottom plate to the beam, and length and joint spacing of the side plates was investigated within the scope of the study.

3.1 Specimen details

In the present study, a total of nine full scale beams were tested. The beams were 200 mm wide, 500 mm deep, and had an overall length of 4.5 m. The tension and compression reinforcement in each specimen consisted of 3 Ø14 and 2 Ø10 bars, respectively. To provide adequate shear strength, two-legged Ø8 stirrups at 100 mm c/c spacing were provided along each beam. The perforation pattern of the external plates is illustrated in Fig. 1.

The details of the test beams are given in Table 1. The plated beams are denoted with the capital letters "PS" and two lower-case letters and a number following the capital letters. The first lower-

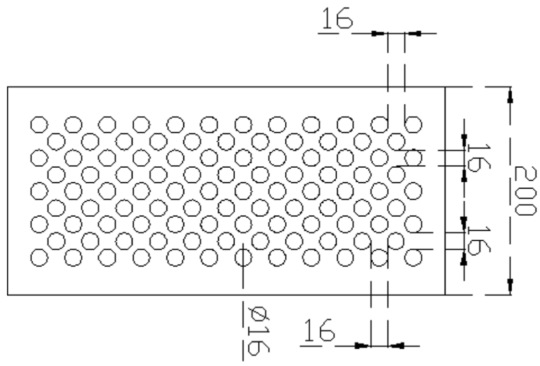

Fig. 1 Plate perforation pattern

Table 1 Test specimens

Steel Plate Location	Bolted Connection
-	-
Bottom	No
Bottom	No
Bottom	Bottom Plate
Bottom	Bottom Plate
Bottom & Sides	No
Bottom & Sides	No
Bottom & Sides	Side Plates
Bottom & Sides	Side Plates

Fig. 2 The external plate in specimen PSbb3

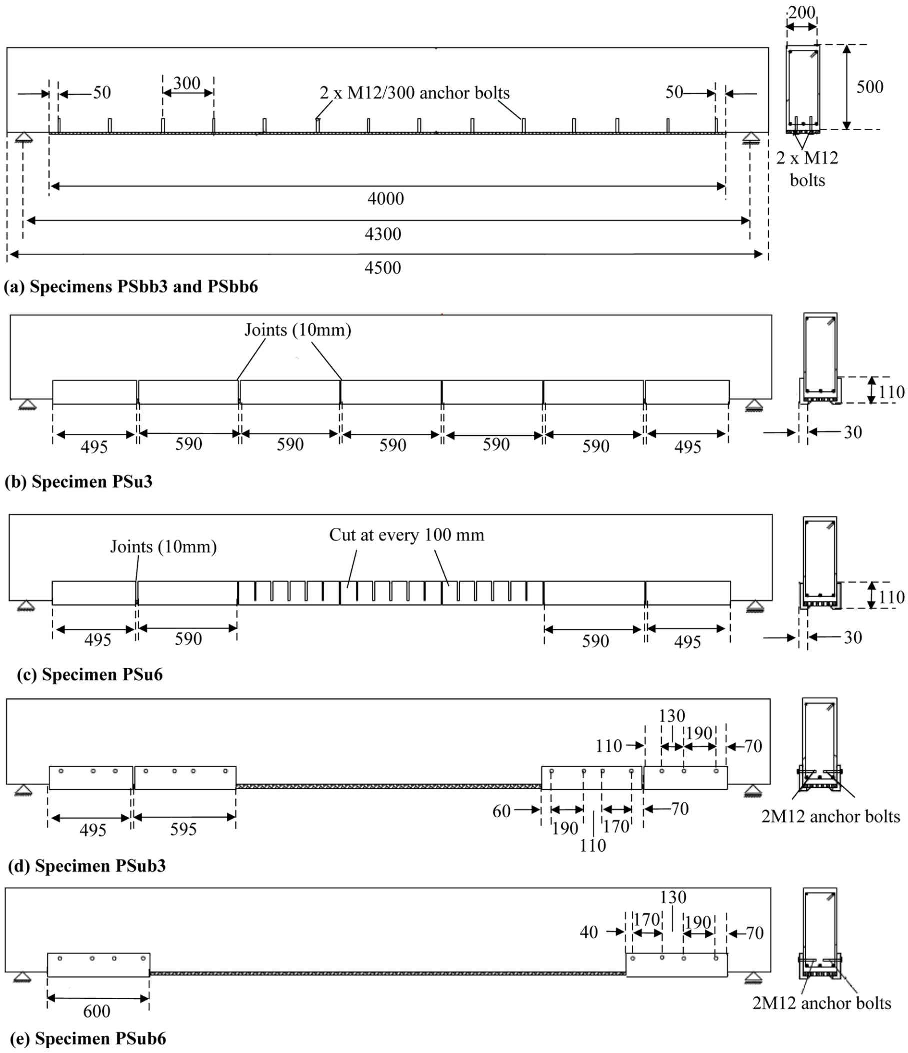

Fig. 3 Specimen details

case letter designates the location of the steel plates. The letter "b" shows that only a bottom plate epoxy-bonded to the soffit of the beam was used while the letter "u" indicates that additional side plates welded to the bottom plate were used to anchor the bottom plate to the beam. The second lower-case letter designates the presence of bolts anchoring the plate to the beam (Table 1). The number in the beam names corresponds to the thickness of the steel plate(s).

In all specimens, the bottom plate covered the full width of the beam (Fig. 2) and extended to a distance of 150 mm from the end supports. The dimensions, reinforcement, and external plate details of specimens PSbb3, PSbb6, PSu3, PSu6, PSub3, and PSub6 are illustrated in Fig. 3. In PSu3, PSu6, PSub3, and PSub6, the bottom legs of the L-shaped side plates were welded to the bottom plate (Fig. 3). To prevent additional tensile strength, the bottom legs of the side plates were cut at every 100-mm interval. The side legs of the side plates were cut at the intervals shown in Fig. 3. Due to the high concentration of flexural cracks around the joints in specimen PSu3, the spacing of the joints was reduced to 100 mm in specimen PSu6 at midspan region.

3.2 Materials and method of plating

The Ø14 tension bars had a yield strength of 450 MPa and the Ø10 compression bars had a yield strength of 500 MPa. The concrete used in the specimens had a characteristic compressive strength of 20 MPa. The M12 anchor bolts used in beams PSbb3, PSbb6, PSub3, and PSub6 had a tensile strength of 800 MPa. The external steel plates had approximately equal yield and ultimate strengths in the order of 280 MPa.

Sikadur-31 epoxy, produced by the Sika-Deteks Company, was used to bond the plates to the beams. The epoxy adhesive had an elastic modulus of 4300 MPa and compressive and tensile strengths of 60 MPa and 15 MPa, respectively. Before the application of epoxy, the surfaces of the plates were sanded, deburred, and cleaned with a degreasing agent and the concrete surfaces were smoothened by grinding. The two-part epoxy was mixed in a ratio of 1:3 and mixed with a low-speed drill. The plate and the beam were pressed against each other after the application of epoxy so that a uniform glueline thickness of 1 mm was provided throughout the bonding surface.

3.3 Test setup, instrumentation, and procedure

The experiments were conducted in a 400-kN capacity test frame. The beams were simply-supported at the ends and subjected to two-point monotonic loading using a steel load-spreader beam. The end supports and the loading points were located at a distance of 2150 mm and 500 mm from midspan, respectively. The distance between the loading points was chosen in a way that the moment diagram created by this loading scheme encloses the diagram created by uniform distributed loading across the beam span. The load was applied by a 600-kN capacity hydraulic jack and measured by a 400-kN capacity load cell. The beam deflections at the loading points and at midspan were measured using LVDT's. Two additional LVDT's were used to detect any possible settlements at the supports. Furthermore, six LVDT's, placed in the central constant moment zone of the beam, were used for measuring the curvatures of the beams during loading. The beams were loaded at 3-6-kN load intervals up to failure. A computerized data acquisition system was used to record the load and deflection measurements.

4. Experimental results and discussion

4.1 Ultimate loads

All of the specimens reached their flexural capacities (Fig. 4) and premature plate peeling mode of failure did not take place in any of the beams. Table 2 tabulates the ultimate loads carried by each beam before failure together with the analytical values calculated from the rectangular stress block analysis of ACI 318 (ACI 2005) and from the concrete stress-strain models proposed by Todeschini *et al*. (1964) and Hognestad *et al*. (1955) (Fig. 5). The experimental ultimate loads are in a very close agreement with the ultimate flexural capacities calculated from the models. The mean and percent coefficient of variation (% COV) of the P_{ex}/P_{an} ratio were calculated to be 1.02 and %7 for the first model (ACI 2005); 1.00 and %6 for the second model (Todeschini *et al.* 1964); and 0.99 and %6 for the third model (Hognestad *et al.* 1955), indicating that no reduction in the load-carrying capacities of the specimens took place due to any possible peeling in the external plates and all the beams reached their flexural capacities.

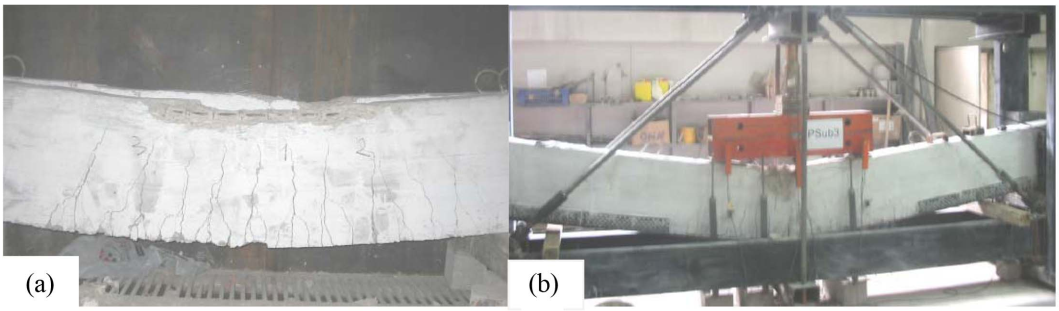

Fig. 4 Flexural failure of specimen (a) PSb3 (b) PSub3

Table 2 Experimental and analytical ultimate loads of the specimens

Beam	Ultimate Load (kN)				P_{ex} / P_{ACI}	P_{ex} / P_{an}	P_{ex} / P_{an2}
	Test, P_{ex}	ACI 318, P_{ACI}	Todeschini, P_{an}	Hognestad, P_{an2}			
SB	152	130	137	138	1.17	1.11	1.10
PSb3	204	188	193	195	1.08	1.06	1.05
PSb6	235	245	247	250	0.96	0.95	0.94
PSbb3	203	188	193	195	1.08	1.05	1.04
PSbb6	241	245	247	250	0.99	0.98	0.96
PSu3	190	188	193	195	1.01	0.99	0.97
PSu6	240	245	247	250	0.98	0.97	0.96
PSub3	185	188	193	195	0.98	0.96	0.95
PSub6	235	245	247	250	0.96	0.95	0.94
				Mean	1.02	1.00	0.99
				St. Dev.	0.07	0.06	0.06
				% COV	7	6	6

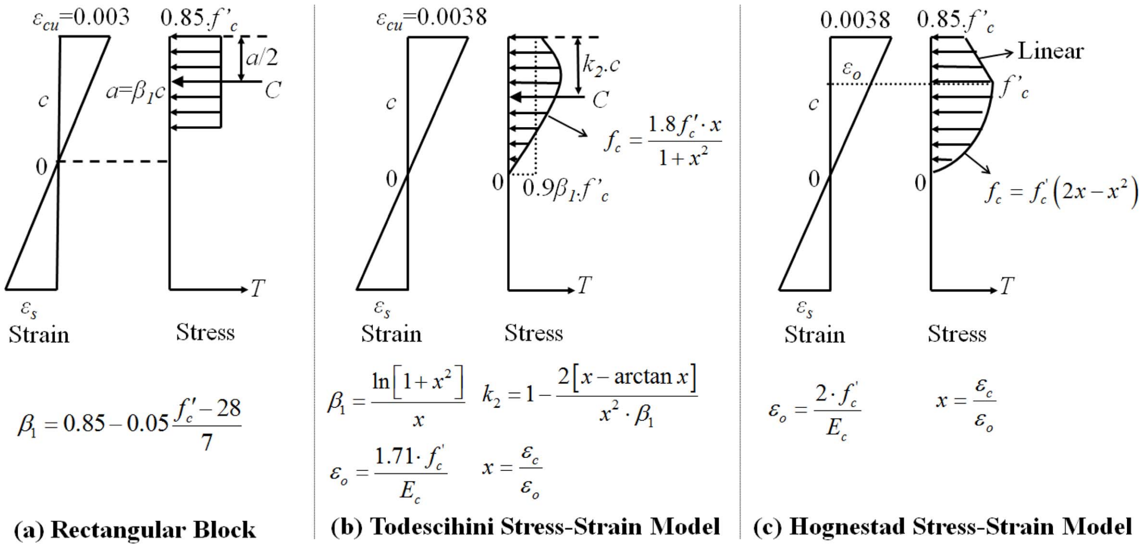

Fig. 5 Concrete stress-strain models used in ultimate load calculations

Table 3 Comparison of the test results with the peeling estimates

Beam	Oehlers and Moran (1990)		Raoof *et al*. (2000)		Plate End Moment at Failure, M_p kN.m	Plate End Shear at Failure, V_p kN	V_p / V_{uc}+ M_p / M_{up} (>1.17)
	M_{sp} kN.m	M_{up} kN.m	$M_{peel,l}$ kN.m	$M_{peel,u}$ kN.m			
PSb3	135	235	34	67	15	102	1.31
PSb6	91	159	24	49	18	118	1.52
PSbb3	135	235	34	67	15	101	1.30
PSbb6	91	159	24	49	18	120	1.56
PSu3	135	235	34	67	14	95	1.22
PSu6	91	159	24	49	18	120	1.56
PSub3	135	235	34	67	14	93	1.18
PSub6	91	159	24	49	18	118	1.52

The same conclusion can be drawn from Table 3, which tabulates the bending moment and shear force at the plate ends of each beam corresponding to the ultimate load measured in the test together with the results obtained from the peeling solutions proposed by Raoof *et al.* (2000), Eq. (4), and Oehlers and Moran (1990), Eqs. (1) and (2), and the peeling failure criterion developed by Oehlers (1992), Eq. (3). The plate end moments at failure are seen to be smaller than the serviceability and ultimate plate peeling moments (M_{sp} and M_{up}), which are the moment values at which plate separation initiates and complete plate separation takes place, respectively. The experimental values are also smaller than the lower and upper bounds for the plate peeling moment ($M_{peel,l}$ and $M_{peel,u}$), calculated from the plate axial stress limits ($\sigma_{s\min}$ and $\sigma_{s\max}$) developed by Raoof *et al.* (2000). Since the experimental plate end moments at failure remained below the M_{sp} and

$M_{peel,l}$ values, it can be concluded that in none of the beams premature plate peeling failure causing reduction in the ultimate load-carrying capacity took place and the external plates remained bonded to the beams up to their flexural moment capacities.

All of the values in the last column of Table 3 are greater than 1.17, implying that the specimens of the present study were expected to experience premature plate peeling failure according to the failure criterion (Eq. (3)) developed by Oehlers (1992). Nonetheless, all beams reached their bending moment capacities and did not fail due to plate peeling. This indicates that Eq. (3) is overconservative for RC beams strengthened with perforated external soffit plates.

4.2 Ductility

Since all of the specimens reached their flexural capacities, the main difference between their flexural behaviors was the deflection range over which this load-carrying capacity could be maintained with no significant decrease. Fig. 6 illustrates the load-midspan deflection curves of all specimens. The load-deflection plots clearly indicate that all of the specimens reached their flexural capacities but the ultimate load was sustained for different deformation ranges depending on the ductility of the beam and the plating and plate end details.

In the present study, two different measures were used for comparing the ductilities of the specimens. The first measure is the deformation ductility index (DDI) which is the ratio of the deflection at the instant when the load-carrying capacity of the beam reduced to %85 of the ultimate load to the deflection at yielding of the outermost layer of tension reinforcement (external plate in plated beams). DDI is more useful in comparing the ductilities of the plated beams with the same plate thickness. Nevertheless, a healthy comparison cannot be made between the ductilities of the beams with different plate thicknesses and between the ductilities of the plated beams and the unplated reference beam by using DDI, since this index is more affected by the reinforcement ratio than the external plating details. For this reason, the modulus of toughness, which is the total area under the load-deflection curve of a beam, was used as a second measure. Different from the modulus of resilience which is energy absorbed by a member within the elastic limits, modulus of toughness indicates the total amount of energy that can be absorbed by a member up to failure, including the elastic and inelastic deformation energies. The deformation capacity and the ductility

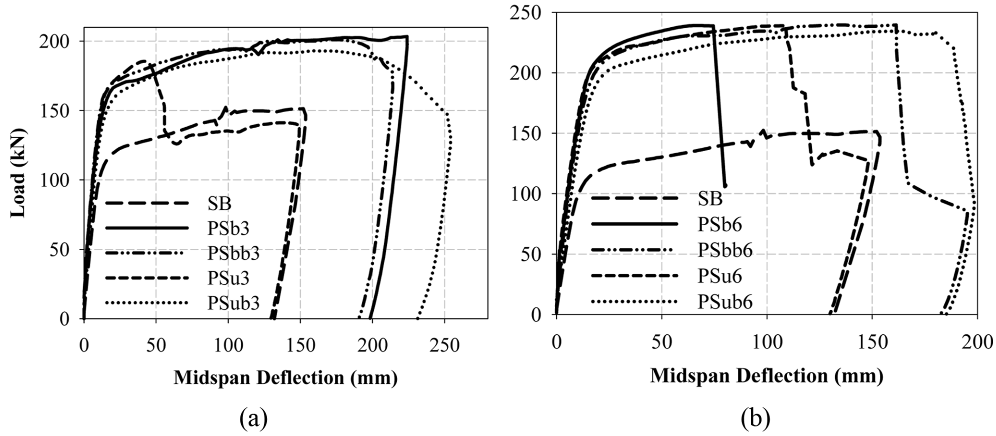

Fig. 6 Load-deflection curves of beams with (a) 3 mm (b) 6 mm external plates

Table 4 Ductility of the specimens

Beam	Modulus of Toughness (Joule)		Deformation Ducti. Index (δ_u/δ_y)		% Deformation in Plate at Failure with respect to the Initial Dimension			
					Plate		Perforation Diameter	
	Absolute	Relative	Absolute	Relative	Length	Width	Lengthwise	Transverse
SB	20536	1.00	16.8	1.00	-	-	-	-
PSb3	41968	2.04	20.9	1.24	2.32	-3.0	12.5	-10.0
PSb6	21244	1.03	7.7	0.46	0.88	-0.5	6.3	-3.1
PSbb3	39997	1.95	21.5	1.28	2.12	-3.5	9.4	-9.4
PSbb6	39293	1.91	13.1	0.78	1.45	-1.5	7.5	-9.4
PSu3	21529	1.05	4.8	0.29	-*	-	-	-
PSu6	29408	1.43	9.7	0.58	-*	-	-	-
PSub3	45089	2.20	20.2	1.20	2.35	-3.0	12.5	-9.4
PSub6	42022	2.05	13.3	0.79	1.00	-3.0	12.5	-12.5

*Could not measured due to experimental difficulties

of a member are dependent not only on the elastic deformations but also on the inelastic deformations that take place before complete failure of the member. Comparison of different strengthening techniques based on modulus of resilience will be misleading, since this comparison will only account for elastic deformations. The inelastic deformations in the structural elements can also contribute to the energy absorption capacity of a structure as long as they do not cause damage to the structural integrity. Therefore, these deformations should also be considered when comparing the ductilities of beams with different strengthening details.

Table 4 tabulates the absolute values of DDI and modulus of toughness for each beam. Furthermore, the relative values of the two measures, which are the ratios of the absolute values corresponding to the plated beams to the absolute value corresponding to the reference unplated beam (SB), are also presented in the table. Finally, the percent deformations in the plates at the end of the test with respect to the initial condition are also included in the table as a measure for the permanent plate deformations and plate deformabilities.

The tabulated modulus of toughness values indicate that the beams with thin (3 mm) plates were capable of absorbing more energy compared to their counterparts (same plating detail) with thick (6 mm) external plates, with the exception of Beams PSu3 and PSu6. As mentioned earlier, the cuts on the side legs of the side plates were not closely spaced enough at midspan region (Fig. 3) of PSu3 causing numerous flexural cracks to form around the cuts, which resulted in the failure of the beam with limited ductility. For this reason, the spacing of the cuts was reduced to 100 mm at midspan region of PSu6, which consequently had a more ductile behavior compared to PSu3. The tabulated values indicate that there is a significant discrepancy between the ductilities of PSb3 and PSb6, while the ductilities of PSbb6 and PSub6 do not significantly differ from the ductilities of PSbb3 and PSub3, respectively. Therefore, it can be established that a ductile flexural behavior can also be achieved in beams with thick external plates if the plates are bolted to the beam or connected to the sides of the beam through collars with bolts. Beams PSub3 and PSub6 have the highest modulus of toughness values among all the specimens, indicating that the highest energy

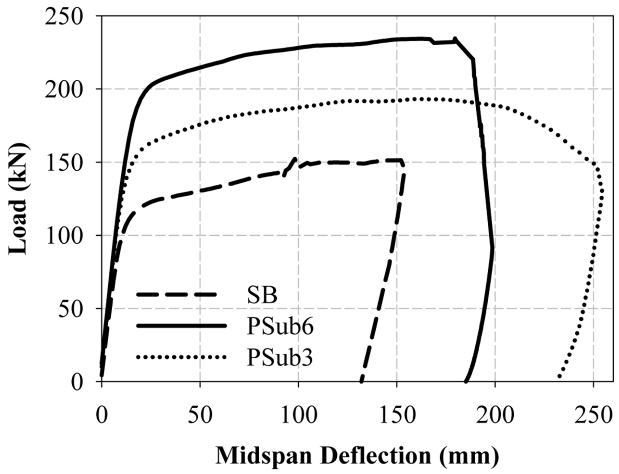

Fig. 7 Effect of plate thickness on flexural behavior

absorption capacity is achieved when the bottom external plate is bonded to the sides through side plates (collars) which are bolted to the sides. Fig. 7 indicates that the increase in the bottom plate thickness did not alter the load-deflection behavior of the beam significantly in the presence of side plates connected to the beam through bolts and PSub6 had ductility comparable to PSub3. Finally, the increase in the plate thickness had the greatest influence on the beam ductility in beams PSb3 and PSb6 and the smallest influence in beams PSbb3 and PSbb6; and PSub3 and PSub6. This implies that the use of bolt anchors in externally plated beams reduces the decrease in the ductility of the beam with an increase in the external plate thickness.

In the present study, reinforced concrete beams with unperforated (solid) external plates have not been tested. To compare the ductilities of beams with perforated plates to the ductilities of beams with solid plates, the results of the tests carried out by Tankut and Arslan (1992) were used. Since the beams tested in the present study and the ones tested by Tankut and Arslan (1992) differed in dimensions, reinforcement and plating details, the comparison was done based on the relative modulus of toughness and DDI values by dividing each result to the result corresponding to the reference beam of the respective study. In this way, the change in the ductility of a beam due to plating could be quantified. Tankut and Arslan (1992) tested 150 × 250 mm rectangular reinforced concrete beams with a clear span of 2.8 m. The tension and compression reinforcement in each beam consisted of 2 Ø14 and 2 Ø10 bars, respectively. All beams were simply supported and subjected to two point loads at 300 mm from midspan. The external plating details of the specimens are presented in Table 5.

The absolute and relative values of the modulus of toughness and DDI of the beams tested by Tankut and Arslan (1992) are presented in Table 6. Among the tested beams, only beams LW, LI, and LV had relative modulus of toughness values comparable to the ones corresponding to the beams tested in the present study, while all beams tested by Tankut and Arslan (1992) had low relative DDI values. In beams LW, LI, and LV, the ends of the bottom plates were held in position by welding to the tension reinforcement, or connecting to the sides of the beam by inclined or vertical collars, respectively. It is noteworthy to mention that the relative modulus of toughness and DDI values of LW, LI, and LV are below the respective values of beam PSb3 of the present study,

Table 5 Specimens tested by Tankut and Arslan (1992)

Beam	External Plate*		Plate End Detail**
	Thickness (mm)	Length (mm)	
BB	No	No	-
SF	2	1200	-
LF	2	2400	-
LW	2	2400	welded to the tension reinforcement
LI	2	2400	connected to the side faces of the beam by inclined collars
LV	2	2400	connected to the side faces of the beam by vertical collars
TV	4	2400	connected to the side faces of the beam by vertical collars

*All plates were 150 mm in width.
**All plates were epoxy-bonded to the beams along their lengths.

Table 6 Ductility of the specimens Tested by Tankut and Arslan (1992)

Beam	Modulus of Toughness (Joule)		Deformation Ductility Index (δ_u/δ_y)	
	Absolute	Relative	Absolute	Relative
BB	398	1.00	5.60	1.00
SF	504	1.27	4.19	0.75
LF	286	0.72	-*	-
LW	689	1.73	2.54	0.45
LI	827	2.08	2.40	0.43
LV	789	1.98	2.60	0.46
TV	554	1.39	-*	-

*Brittle failure

whose bottom plate did not have end anchorage. The contribution of the use of perforated plates instead of solid plates to the ductility of the plated beams can be better understood by considering that the bottom plates used by Tankut and Arslan (1992) were thinner (2 mm) than the bottom plates used in the present study (3 mm and 6 mm). As mentioned before, beams PSu3 and PSu6 are exceptions for these findings due to the limited ductilities of these beams as a result of the severe cracking around the plate cuts in these beams.

4.3 Rigidity

Bending rigidity of a beam is the slope of the initial linear portion of the moment-curvature graph. Evaluation of the curvature of a beam is rather cumbersome and the experimental curvature values are significantly affected by the amount of cracking in the region where the curvature readings are taken. Therefore, the load-midspan deflection data, which is related to the overall rigidity of the beam, provides a more trustworthy measure for the bending rigidity. Table 7 presents the absolute value of the slope of the initial linear portion of the load-deflection curve (Fig. 6) of each beam

Table 7 Rigidities of the specimens

Beam	Rigidity (P_y/δ_y)	
	Absolute (kN/mm)	Relative
SB	9.99	1.00
PSb3	13.11	1.31
PSb6	14.74	1.48
PSbb3	14.90	1.49
PSbb6	13.73	1.37
PSu3	13.36	1.34
PSu6	15.19	1.52
PSub3	12.42	1.24
PSub6	12.20	1.22

together with the relative values with respect to the bending rigidity of the reference beam (SB). The tabulated values indicate that all of the plated beams had bending rigidities much greater than the reference beam. The rigidity of the plated beam with the smallest rigidity exceeded the rigidity of the reference beam by more than 20%. It can also be seen that the beams with thin external plates had rigidity values close to their counterparts with thick plates indicating that the plate thickness had no major influence on the bending rigidities of the plated beams. The same conclusion can also be drawn from Fig. 7 which shows the load-deflection curves of beams SB, PSub3, and PSub6. Although the initial slopes of the plated beams PSub3 and PSub6 differed from the slope of SB, there was no major difference between the initial slopes of PSub3 and PSub6.

5. Conclusions

The present study is dedicated to investigate the flexural behavior of reinforced concrete beams strengthened with perforated steel plates. For this purpose, an unplated reference and eight plated rectangular reinforced concrete beams with 1:1 scale and various plating details were tested and the results were analyzed in terms of the load-carrying capacity, ductility, and stiffness. Furthermore, the results were compared to the results of the tests on reinforced concrete beam strengthened with nonperforated external plates in the literature (Tankut and Arslan 1992). Based on the experimental and analytical studies presented in this paper, the following conclusions were drawn:

- The use of perforated steel plates instead of solid (nonperforated) steel plates in strengthening reinforced concrete beams significantly contributes to the ductility and energy absorption capacity of the beam. The axial deformations in the external plate cause the initially circular holes to evolve into an elliptical shape improving the deformability of the plate and the ductility of the plated beam. The improvement in the deformation capacity of the perforated external plate was also attributed to the better bonding between the external plate and the beam due to the penetration of the adhesive into the perforations during the application of epoxy.
- The rectangular stress block analysis of ACI 318 (ACI 2005) and the stress-strain models developed by Hognestad *et al.* (1955) and Todeschini *et al.* (1964) provide accurate estimates for the ultimate flexural moment capacities of externally plated beams.

• Plating of reinforced concrete beams improves the bending rigidity of the beams. However, the thickness of the external plate has no major influence on the bending rigidity.
• An increase in the thickness of the external plate significantly reduces the ductility of a plated reinforced concrete beam when the external plate is only epoxy-bonded to the beam. Anchoring the external plate to the beam with additional bolts or connecting the plate to the sides of the beam through bolt-anchored side plates reduces the amount of decrease in the ductility of a beam with an increase in the plate thickness.
• Anchoring the bottom plate to the sides of the beam with additional side plates (collars) and anchoring these side plates to the beam with the help of bolts proves to be the best method for improving the ductility of an externally plated reinforced concrete beam.
• When the bottom external plate is fixed to the sides of a beam with the help of side plates, the side plates need to have closely spaced cuts along their lengths to prevent the formation of numerous flexural cracks around the cuts, which in turn cause the failure of the beam before showing sufficient ductility.

References

ACI Committee 318 (1989), Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318-89R), American Concrete Institute, Detroit, USA.
ACI Committee 318 (2005), Building Code Requirements for Structural Concrete and Commentary (ACI 318M-05), American Concrete Institute, Farmington Hills, Mich., USA.
Altin, S., Anil, O. and Kara, M.E. (2005), “Improving shear capacity of existing RC beams using external bonding of steel plates”, *Eng. Struct.*, **27**(5), 781-791.
Arslan, M. (1991), “Strengthening of reinforced concrete beams by epoxy glued steel plates”, M.Sc. Thesis, Gazi University, Inst. of Science and Technology, Ankara, Turkey.
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.
Aykac, S. and Ozbek, E. (2011), “Strengthening of reinforced concrete t-beams with steel plates (in Turkish)”, *Tech. J. Turk. Chamber Civ. Eng.*, **22**(1), 5319-5334.
Barnes, R.A. and Mays, G.C. (2006a), “Strengthening of reinforced concrete beams in shear by the use of externally bonded steel plates: Part 1-Experimental programme”, *Constr. Build. Mater.*, **20**(6), 396-402.
Barnes, R.A. and Mays, G.C. (2006b), “Strengthening of reinforced concrete beams in shear by the use of externally bonded steel plates: Part 2-Design guidelines”, *Constr. Build. Mater.*, **20**(6), 403-411.
British Standards Institution, BSI (1997), “BS 8110-1: 1997, Structural Use of Concrete – Part 1: Code of Practice for Design and Construction”, London, UK.
Hognestad, E., Hanson, N.W. and McHenry, D. (1955), “Concrete stress distribution in ultimate strength design”, *ACI J. Proc.*, **52**(4), 475-479.
Hussain, M., Sharif, A., Basunbul, I.A., Baluch, M.H. and Al-Sulaimani, G.J. (1995), “Flexural behavior of precracked reinforced concrete beams strengthened externally by steel plates”, *ACI Struct. J.*, **92**(1), 14-22.
L’Hermite, R. and Bresson, J. (1967), “Concrete reinforced with glued plates”, *RILEM International Symposium. Synthetic Resins in Building*, Paris, France.
Oehlers, D.J. (1992), “Reinforced concrete beams with plates glued to their soffits”, *J. Struct. Eng.*, *ASCE*, **118**(8), 2023-2038.
Oehlers, D.J. and Moran, J.P. (1990), “Premature failure of externally plated reinforced concrete beams”, *J. Struct. Eng.*, *ASCE*, **116**(4), 978-995.
Raoof, M., El-Rimawi, J.A. and Hassanen, M.A.H. (2000), “Theoretical and experimental study on externally plated R.C. beams”, *Eng. Struct.*, **22**(1), 85-101.
Raoof, M. and Zhang, S. (1997), “An insight into the structural behavior of R.C. beams with externally bonded

plates", *Proc. Inst. Civ. Eng. Civ. Eng. Struct. Build.*, **122**(4), 477-492.
Sallam, H.E.M., Saba, A.M., Shahin, H.H. and Abdel-Raouf, H. (2004), "Prevention of peeling failure in plated beams", *J. Adv. Concr. Technol.*, **2**(3), 419-429.
Su, R.K.L. and Zhu, Y. (2005), "Experimental and numerical studies of external steel plate strengthened reinforced concrete coupling beams", *Eng. Struct.*, **27**(10), 1537-1550.
Tankut, T. and Arslan, M. (1992), "Behaviour of reinforced concrete beams strengthened by epoxy-bonded steel plates", *Tech. J. Turk. Chamber Civ. Eng.*, **92**, 176-185.
Todeschini, C.E., Bianchini A.C. and Kesler C.E. (1964), "Behavior of concrete columns reinforced with high strength steels", *ACI J., Proc.*, **61**(6), 701-716.
Ziraba, Y.N., Baluch, M.H., Basunbul, I.A., Sharif, A.M., Azad, A.K. and Al-Sulaimani, G.J. (1994), "Guidelines toward the design of reinforced concrete beams with external plates", *ACI Struct. J.*, **91**(6), 639-646.
Zhu, Y. and Su, R.K.L. (2010), "Behavior of strengthened reinforced concrete coupling beams by bolted steel plates, Part 2: Evaluation of theoretical strength", *Struct. Eng. Mech.*, **34**(5), 563-580.
Zhu, Y., Su, R.K.L. and Zhou, F.L. (2007), "Seismic behavior of strengthened reinforced concrete coupling beams by bolted steel plates, Part 1: Experimental study", *Struct. Eng. Mech.*, **27**(2), 149-172.