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IMPROVING STRENGTH AND DUCTILITY BY USING STEEL PLATE WRAPPING

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ABSTRACT

The use of a thin steel plate wrapping to strengthen reinforced concrete members in critical sections may considerably improve the ductility and the strength of members. Depending on the type of failure, two procedures were used to quantify the steel plate effects. By using a modified stress-strain relationship or a truss analogy mechanism, strength and ductility were estimated and then compared with test results.

KEYWORDS

Confinement; ductility; stress-strain relationship; truss analogy model ; load reversals.

INTRODUCTION

In recent earthquakes it was clear that the lack of confinement may result in a poor ductile behavior, when inelastic deformations are reached. The ductile behavior of reinforced concrete structures depends on many factors but is highly influenced by the behavior of the confined concrete. A number of experimental and analytical investigations have been carried out in the last decades to investigate the ductile behavior of reinforced concrete members using ties and spirals (Park et al., 1982, Shah et al., 1983, Sheik et al., 1982, Samra, 1990).

On the other hand, in repair and design, techniques of strengthening have been applied by means of using FRP or GFRP plates, steel plates, resins, fiberglass, etc., which give good results by means of improving the strength and ductility of members.

In the past, to strengthen present pier structures, many methods have been proposed, some of which have actually been applied on the site. Regarding the improvement of ductility, the minimum requirement is to confine the cracked concrete. Miyamoto et al., 1987 have proposed to use steel plates for repairing the possible defects of section. Wrapping the member , the plates were attached to the required portion with anchor bolts. This method was reported effective to improve the strength and ductility of members.

As far as the basic function is concerned both the hoops and the steel plate play the same role as confinement of concrete. The difference lies only in how effectively they work.

It is mainly discussed in this paper how the transverse reinforcement (hoops), the steel plate wrapping or the combination of steel plate wrapping with hoops affect the strength and ductility of members. Using a truss analogy model and a modified stress-strain relationship the effects of both hoops and steel plate were quantitatively evaluated.

TEST PROGRAM

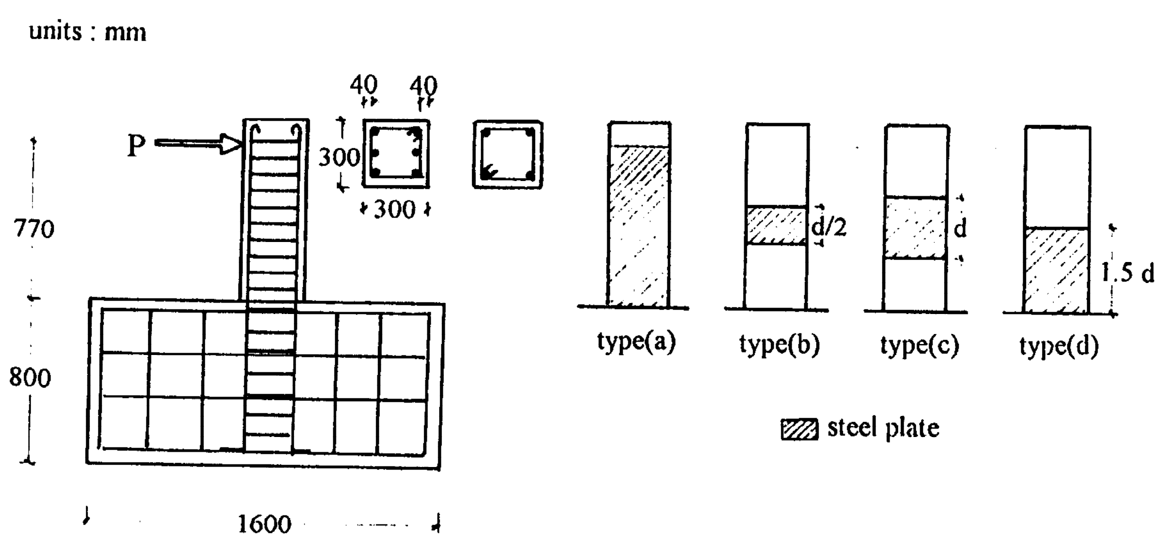
In the experimental program 18 column specimens were tested under incrementally increased load reversals, having as parameters longitudinal reinforcement and the type and spacing of transverse reinforcement. The longitudinal reinforcement consisted of two and four steel bars. At the middle of a span two of four longitudinal reinforcement bars were cut off to provide a weak zone in the column. The transverse reinforcement consisted of hoops, a steel cover plate and anchor bolts. The hoop spacings were 50, 80 , 100 and 210 mm.

There were 4 types of steel plate coverings for the columns. Type (a) whole column span wrapped ; type (b) the covered length was equal to the effective depth of the column. One third of the plate width should match the cut-off point of the reinforcement bars; type (c) similar to type (b) but having a cover length of twice the effective depth, and type (d) the cover zone is limited between the fixed end and the section located at a distance of 1.5 times the effective depth.

The steel grade was SD-30 for the steel bars, G3101-SS31 for the steel cover plate, and the anchor bolts were commercial bolts of $\phi 6$ mm anchored 50 mm into the concrete, and spaced at 80 mm each. The steel plate thickness was 1.2 mm. Both the steel plate thickness and the anchor bolts were selected based on a previous paper(Aviles et al., 1992).

Fig. 1 shows a typical test specimen and types of steel plate strengthening. Table 1 indicates the details of all the specimens.

The load was applied by an actuator of 300 kN capacity. The unit of increment of deflection was determined by the first yield deflection of column (δ_y). The load was controlled by increasing the deflection incrementally by δ_y with 3 cycles of load reversal at each level. All specimens showed the first yielding of longitudinal reinforcement bars at the fixed end region.



(a) Typical test specimen

(b) Types of steel plate strengthening

Fig. 1. Test specimens

Table 1. Details of specimens

Column No.	Section bxh (mmxmm)	Longitudinal bars and (cut-off bars)	Shear span a/d	hoops (D6) s (mm) at wrapped zone	steel cover plate type	t (mm)	bolts	f'c MPa
1	250x250	4-D16 (2-D16)	4	-	-	-	-	36.0
2	250x250	4-D16 (2-D16)	4	-	(a)	1.2	No	24.2
3	250x250	4-D16 (2-D16)	4	-	(a)	1.2	Yes	29.9
4	250x250	4-D16 (2-D16)	4	-	(b)	1.2	No	27.8
5	250x250	4-D16 (2-D16)	4	-	(b)	1.2	Yes	26.9
6	250x250	4-D16 (2-D16)	4	50	-	-	-	32.0
7	250x250	4-D16 (2-D16)	4	80	-	-	-	29.9
8	250x250	4-D16 (2-D16)	4	100	-	-	-	32.0
9	250x250	4-D13 (2-D13)	4	80	-	-	-	33.4
10	250x250	4-D13 (2-D13)	4	100	(a)	1.2	Yes	32.8
11	250x250	4-D13 (2-D13)	4	210	(b)	1.2	Yes	32.8
12	250x250	4-D16	4	-	-	-	-	32.4
13	300x300	2-D19	3	-	-	-	-	28.7
14	300x300	2-D19	3	100	-	-	-	29.5
15	300x300	2-D19	3	-	(c)	1.2	Yes	28.0
16	300x300	2-D19	3	100	(c)	1.2	Yes	29.0
17	250x250	4-D16 (2D16)	4	-	-	-	-	29.9
18	250x250	4-D16 (2D16)	4	80	(d)	1.2	Yes	32.3

TEST RESULTS AND DISCUSSION

Crack Pattern

Fig. 4 shows the crack pattern of ten columns strengthened by the steel plate using different methods. The cracking was observed stripping the steel plate after finishing the loading test.

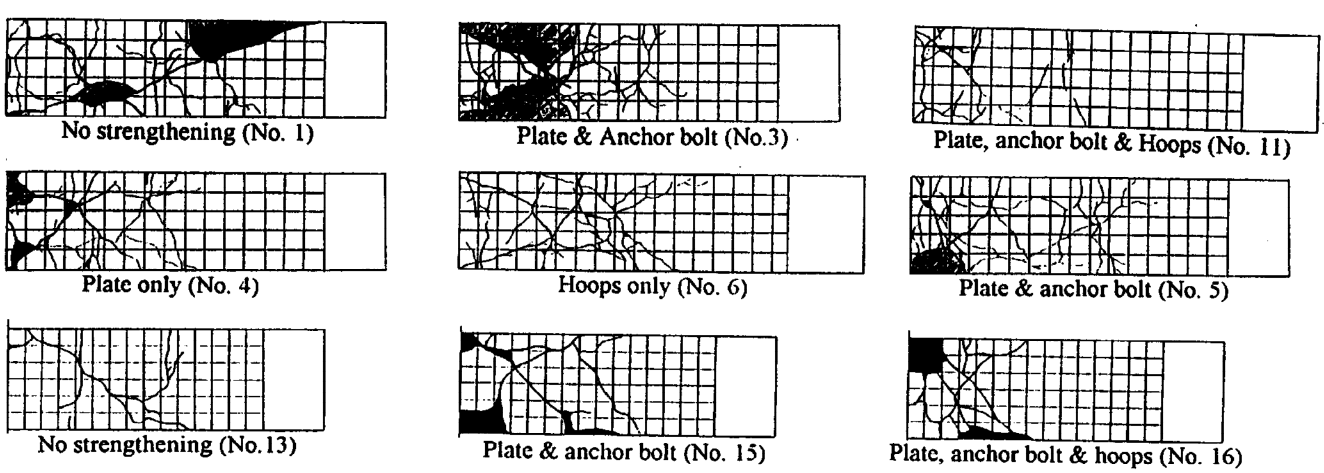
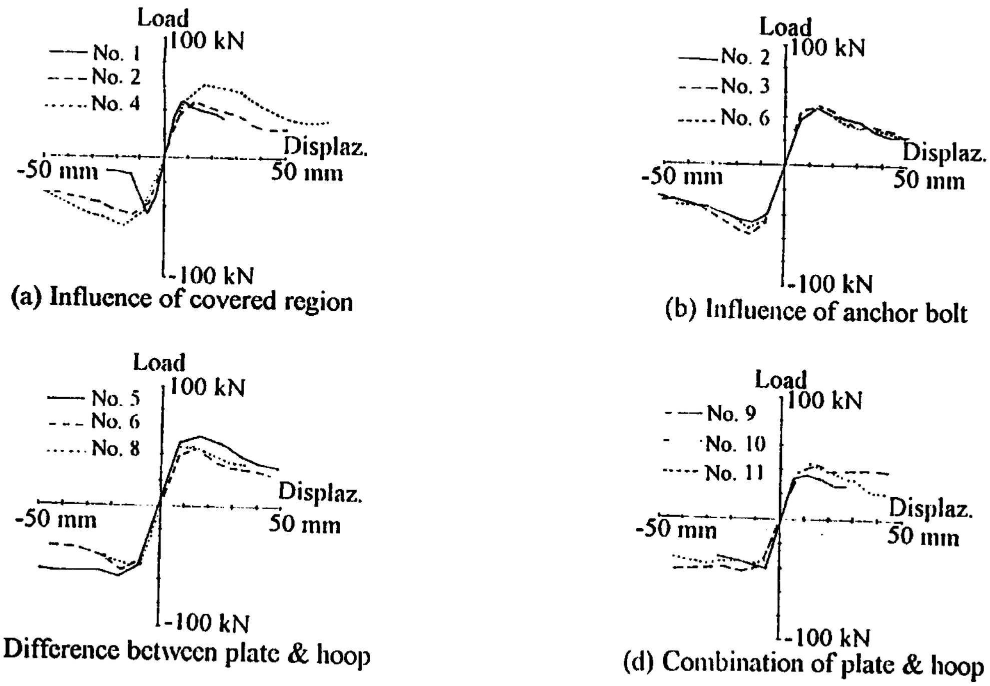


Fig. 5. Crack patterns

In the case of cut-off bars it was observed that without strengthening (No.1) or insufficient strengthening (No.6), the cracks initiated near the cut-off point of the longitudinal reinforcement, however an adequate strengthening prevented the propagation of cracks. The influence of the region covered by the steel plate was recognized in such a manner that the crack developing was shifted and concentrated to the uncovered region. Certainly, the use of anchor bolts to attach the steel plate (No.5) provided much better crack control.

Strengthening near the hinge zone (Nos. 15, 16) proved effective in collaborating to confine the cracked concrete permitting also a good distribution of cracks.



(a) Influence of covered region

(b) Influence of anchor bolt

(c) Difference between plate & hoop

(d) Combination of plate & hoop

Fig. 5. Envelope curves of load-deflection (specimens with cut-off bars)

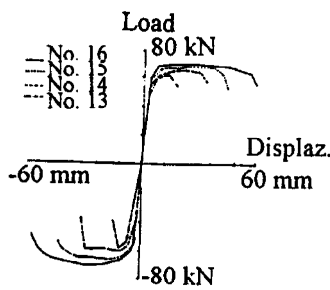


Fig. 6. Envelope curves of load-deflection (specimens without cut-off bars)

<u>Load Deflection Curves</u>

Fig. 5 shows the envelope curves of load-deflection of specimens having cut-off bars. Fig. 5(a) shows the influence of the covered length. In the reference specimen for this type of strengthening (No. 1, without reinforcement), the cracks that initiated at the cut-off point of the longitudinal reinforcing bars propagated drastically after reaching the deflection level of $2\delta_y$, and resulted in shear failure. Strengthening the cut off portion with the steel plate prevented a drastic reduction of capacity (No.2), and enlarging the covered length up to twice the effective depth (No.4) provided about a 20% increase of the maximum capacity.
Fig. 5(b) shows the influence of the use of anchor bolts. The strengthened zone length was equal to the effective depth (d). The column strengthened by the steel plate without anchor bolts (No. 2) showed similar

behavior to the column strengthened by hoops (No. 6). When the steel plate was fixed by anchor bolts (No.3), the maximum load carrying capacity increased a little.

Fig. 5(c) shows the comparison of the effect of hoops with that of steel plate. In the cases where the cut-off portion was strengthened by hoops only (Nos. 6, 8), the spacings of 50 mm (No. 6) and 100 mm (No.8) did not make any difference on the behavior of the columns.

On the other hand, the steel plate fixed by anchor bolts in the zone of twice the effective depth (2d) (No. 5) increased both the maximum capacity and the deflection capacity.

Fig. 5(d) shows the combined effect of steel plate and hoop on the behavior of the columns. Among the three specimens, No. 10 which had steel plate and hoops with 100 mm spacing showed the most ductile behavior.

Fig. 6 shows the envelope curves of load-deflection of the specimens without cut-off bars but strengthened in the zone limited by the fixed end and the section located at 1.5 times the effective depth. Here it was observed that the steel cover plate increased the load carrying capacity a little (Nos. 15, 16). However in this type of specimen it was clear that the steel plate role was remarkable for prevention against the drastic reduction of capacity at large deflection levels.

This paper defined the yield deflection as the deflection when the longitudinal reinforcement bars at the fixed end came to the first yield. The ultimate deflection was determined as the deflection at which the applied load decreased below the first yield capacity. The ductility ratio was then determined by the ratio of the critical deflection to the first yield deflection ($\mu = \delta_u/\delta_y$). The specimen ductility ratios are shown in Table 2.

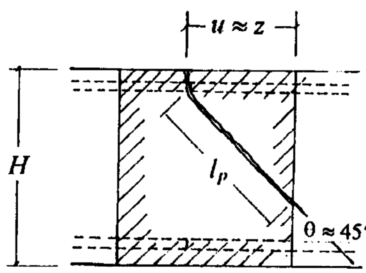


Fig. 7. Truss mechanism of steel plate

STRENGTH AND DUCTILITY EVALUATION

Specimens Failing in Shear

These specimens (Nos. 1 to 13, and No.17) were designed so that the shear capacity was critical at the section where longitudinal reinforcement bars were cut off. If the critical section for the ultimate capacity would remain in the strengthened area, the effect of strengthening by steel plates could be evaluated by comparing the test results. In fact the critical section shifted to the other portion that was not covered by plates. Based on the observation that the steel plates confined the development of diagonal cracks, the effect of the steel plate was assumed similar to that of ordinary shear reinforcement. Taking into account the inclination of diagonal cracks at 45 degrees (Fig. 7) the additional shear capacity by the steel plate was evaluated as :

$$V_{sp} = t \cdot z \cdot f_{sy} \tag{1}$$

where t= the thickness of the steel plate; $z=j \cdot d$; f_{sy}= the yield strength of the steel plate.

The expression for the equivalence between steel plate and hoops is given by

$$\frac{A_w}{s} = 2 \cdot t \cdot f_y \tag{2}$$

where s = hoop spacing; A_w = transverse reinforcement area (hoops); f_y = the yield strength of transverse steel bars (hoops).

Eq.(2) shows that the steel plate was equivalent to the hoops (D6) at approximately 60 mm spacing. Since the shear capacity of the beam was designed lower than the flexural capacity at the fixed end the shear capacity was computed performing a sectional analysis based on the Compression field theory as established on CSA Standard (CPCA, 1985)). Table 2 shows both the test and the computed results for this type of specimens.

An estimation of the ductility index was carried following the procedure given by Aviles et al. 1992, 1994, but omitting any modification of the stress-strain characteristics (i.e. to consider confinement effects given by only hoops). The calculated ductility indexes were obtained from stopping the $P-\delta$ calculation at the level when shear ultimate capacity (obtained in the previous calculation) was achieved.These values are shown in Table 2.

Specimens Developing its Flexural Capacity

Owing to the effect of steel plate which resulted basically in an increase in displacement capacity rather than in load carrying capacity, it proved more effective to base all calculation on the procedure given by Aviles et al. 1992, 1994, where the confinement conditions given by hoops, steel plate and anchor bolts are represented by the confinement index α and the modified stress-strain relationships. The ductility ratios obtained for these specimens (Nos. 14 to 16, and No. 18) are also given in Table 2.

Table 2. Experimental and computed values of strength and ductility

Specimen	Test values		Computed values	
No.	Ultim. capac. [kN]	Ductility ratio	Ultim. capac. [kN]	Ductility ratio
1	46.4	2.8	51.4	1.8
2	48.0	4.2	45.9	2.9
3	49.6	2.7	49.2	1.8
4	57.1	3.9	49.5	2.8
5	58.1	2.8	65.1	1.8
6	48.5	2.0	48.1	1.3
7	57.7	5.6	50.3	3.7
8	52.5	2.7	48.1	1.7
9	35.7	3.2	33.1	2.2
10	41.9	4.2	40.5	2.9
11	42.5	4.4	40.5	3.2
12	60.0	3.0	64.1	1.8
13	65.2	4.2	62.2	3.1
14	63.8	8.0	65.4	6.4
15	54.1	12.5	55.5	9.8
16	54.4	13.6	56.3	11.5
17	57.7	3.0	52.0	2.5
18	76.4	8.9	71.0	7.8

CONCLUSIONS

The following was concluded from this study:

(1) Strengthening weak zones of reinforced concrete members makes it possible to shift the critical section for the ultimate capacity to other zones.

(2) The use of steel plate improves both ductility and strength, specially if it combined with anchor bolts. However, in members failing in shear, the ultimate capacity is increased rather than ductility. On the contrary, in members failing in flexure, the ductility is considerably increased rather than strength.

(3) Using both methods of evaluation: the truss mechanism and the modified stres-strain relationship, gave satisfactory results in estimating strength and ductility of members.

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