Repair and Strengthening of Reinforced Concrete Beams

HONORS THESIS

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By

Yunlan Zhang

The Ohio State University

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ABSTRACT

Repair and strengthening of damaged or vulnerable reinforced concrete structures is important in order to guarantee the safety of residents or users. Beams are important structural elements for withstanding loads, so finding the efficient repair and strengthening methods are necessary in terms of maintaining the safety of the structures.

This research study investigated various repair, retrofit, and strengthening techniques for reinforced concrete beams. The comparison and summary of each repair and strengthening method are provided in this thesis.

The thesis involves the literature review of current experimental test of repair and strengthening techniques for reinforced concrete beams. The experimental studies were summarized by describing the specimens and loading details, All the methods in the research were categorized into five chapters: section enlargement and concrete jacketing, external reinforcement, steel plates, unbonded-type strengthening, and concrete repairs. The installation procedures were summarized and the advantages, shortcomings, and considerations of each method were also discussed in the thesis.

ACKNOWLEDGMENTS

I want to appreciate my research adviser, Halil Sezen, for he continued help and guide me finish my research. Without his supports and teaching my thesis would never be finished. He patiently taught me professional knowledge and guided me to learn how to do research. Though he had a lot of teaching and other research works, he kept improving my thesis.

I want to thank Jayanth. He is a graduate student who under professor Sezen, and he has been doing research on repair and strengthening concrete column. I wish to think him for he taught me how to use the select the useful papers in the research. He also gave suggestions for how to draw the useful information from the papers. I would not have been able to find enough resources for my research without Jayanth's help.

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VITA

March 1, 1989……………………………..Born- Beijing, China

September10, 2012……………………………………B.S. Civil Engineering

FIELDS OF STUDY

Major Field: Civil and Environmental Engineering and Geodetic Engineering

Specialization: Structural Engineering

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CHAPTER 1 INTRODUCTION

1. 1 Introduction

Natural disasters such as earthquakes, tornados, and tsunamis threaten the integrity of civil infrastructure and safety of their users. A large number of reinforced concrete buildings and bridges built in the United States before the1970s typically do not have sufficient capacity to resist the forces during such catastrophes. In order to guarantee the safety of the people; older, existing structures need to be repaired and strengthened to prevent their collapse. Efficient methods need to be developed for structural repair and strengthening. This thesis evaluates the effectiveness of available methods for repair and strengthening of reinforced concrete beams.

The thesis focuses on a research of experimental studies done on repair and strengthening of reinforced concrete beams. The objective of this thesis is to find out the pros and cons of each repair and strengthening methods. The results of the research can help engineers choose the best approach in their projects based on different environmental and economical condition.

1.2 Overview

A detailed literature review is conducted on research studies involving repair and strengthening experiments on reinforced concrete beams. The specimen, loading details, and the type of repair or strengthening method are described. If available, the strength and deformation capacity before and after the repair or retrofit as well as the change in the capacity are reported. Some basic details of the experiments and response parameters are summarized. The thesis is organized to include the following: experiment and specimen details, material properties, specimen geometry, basic test setup and loading, and reported critical data (e.g., measured load-displacement relations). The repair and strengthening methods include:

Section enlargement and concrete jacketing, e.g., reinforced concrete jackets.

External reinforcement, e.g., external tendons.

External steel reinforcement, e.g., web-bonded continuous steel plates.

Unbonded-type, e.g., wire rope units.

Concrete repair, e.g., epoxy injection.

1. 3 Unique feature of the research:

The research includes repairing the damaged specimens or strengthening of undamaged specimens with or without applied loading. Although the thesis tries to contain as much repair and strengthening methods as possible, the scope of this thesis is limited. So some strengthening and repair methods may not be mentioned, though they are also effective and widespread. For instance, strengthening concrete beams with fiber reinforced polymer materials is a widely used method due to its significant advantages. However, it was only slightly mentioned in some chapters, and no details were supplied in the thesis.

1. 4 Scope

This thesis presents the current concrete beam repair and strengthening techniques by summarizing the experimental research results reported in technical journal papers. Chapter 2 is about strengthening of beams by section enlargement and concrete jacket, Chapter 3 describes external reinforcements to strengthen RC beams, Chapter 4 presents strengthening of beams by attaching steel plates, Chapter 5 involves unbonded-type strengthening methods, and Chapter 6 introduces some concrete repair methods. All the repair and strengthening techniques have their advantages and shortcomings. Choosing the optimum method depends on the specific conditions.

CHAPTER 2 Section Enlargement and Concrete Jacketing

2. 1 Introduction

Placing additional layer of concrete surrounding an existing beam is called section enlargement. Concrete jacket is to add reinforced concrete jacket on the existing beam. Jacketing by reinforced concrete could improve resistance against applied loads and enhances the durability at same time. Furthermore, section enlargement and concrete jacketing maybe easier and cheaper compared to other approaches such as steel plate jacketing.

2. 2 Description of Previous Research on Section Enlargement

Sprayed concrete is one of the common enlargement approaches. Diab et al. (1998) conducted experimental research to test the efficiency of using sprayed concrete for strengthening the reinforced concrete beams. Nine specimens were categorized into three test series. The first series included three reference reinforced concrete (RC) beams (P1-P3). In the second series, three beams (PR1-PR3) with same properties and dimension with P1 were loaded to damage them. Then, two reinforcing steel bars and a layer of sprayed concrete strengthened them. The three specimens in the third series have same dimension with P1 and tested in the same manner with series two, however, instead of strengthening specimens with a layer of sprayed concrete, specimens were strengthened with concrete including metallic glass ribbon fibers. All of the specimens were 5000 mm long, and further properties of specimens and strengthening details are exhibited in Table 2.1 and Figures. 2.1-2.3. The ultimate load and deflection are presented in Table 2.1 and Figure. 2.4. The experimental results indicate that using sprayed concrete to strengthen reinforced concrete beams can effectively increase their load carrying capacity or stiffness. Furthermore, additional metallic glass ribbon fibers in sprayed concrete improved the crack pattern and ultimate capacity of RC beams. Adding metallic glass ribbon fibers to reinforced concrete beams improved flexural strength, enhanced cracking pattern, reduced tensile stress and greatly increased the first cracking moment.

Table 2. 1- Dimension and experimental results of specimens.

Specimen	Beam Dimension (mm)		Ultimate Load (kN)
	Width	Depth	
P1	130	330	55
P2	130	330	171
P3	130	330	210
PR1	200	400	141
PR2	200	400	136.5
PR3	200	400	134.8
PR4	200	400	160
PR5	200	400	149.5
PR6	200	400	148

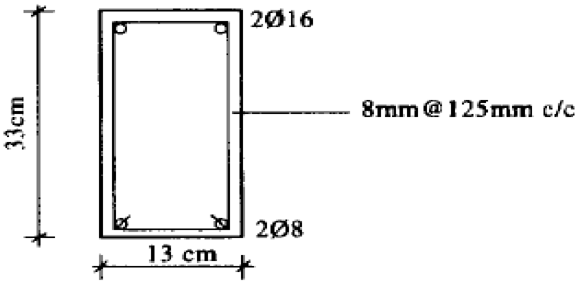


Figure 2. 1- Cross-section of beam P2 and beams PR1 to PR6 (Diab et al. 1998).

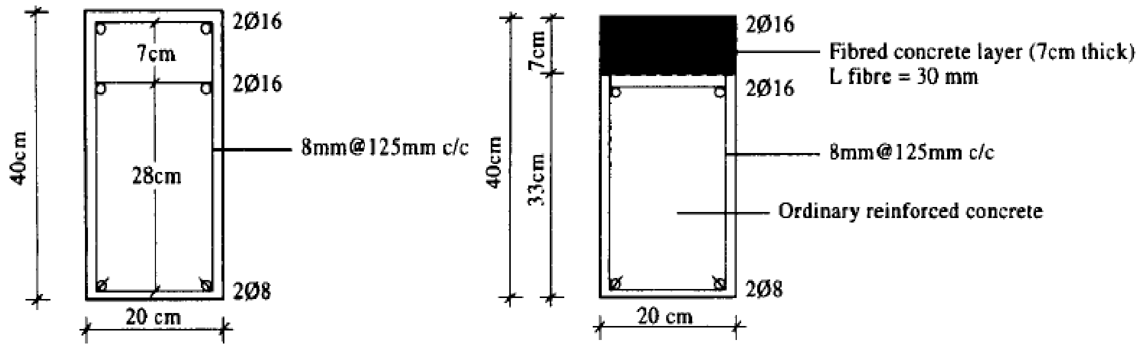


Figure 2. 2- Cross-section of beam P2 and P3 (Diab et al. 1998)

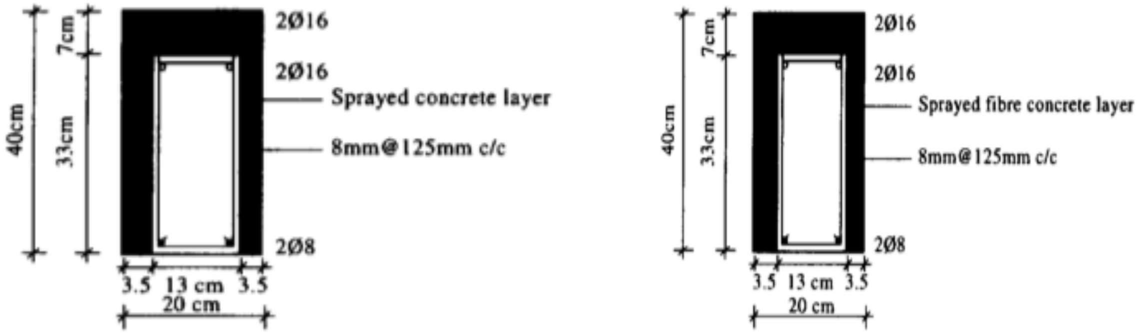


Figure 2. 3- Cross-section of beams PR1 to PR3 after repair (Diab et al. 1998).

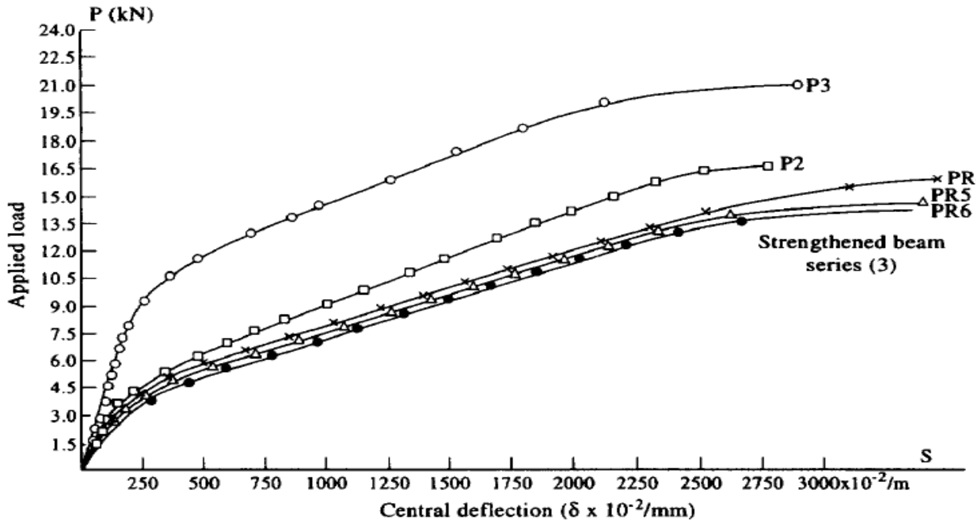


Figure 2. 4- Load-central deflection curve for strengthened beams (Diab et al., 1998).

Adding strain hardening cementitious composites (SHCCs) layer to RC beams is another effective section enlargement strengthening method, SHCC causes strain localization, which limits the ductility. Combining the SHCC and a small amount of steel reinforcement enhances the strain capacity by preventing the stiffness degradation of strengthening layer caused by cracking. Mohamed et al. (2012) conducted experiments to compare the flexural behavior of RC beams strengthened with steel reinforced and unreinforced SHCC layer cast to their soffit. Four-point bending flexural tests were conducted up to failure on two RC control beams, four RC beams strengthened with a steel-reinforced Ultra High Performance Strain Hardening Cementitious Composite (UHP-SHCC) layer, two RC beams strengthened with an unreinforced UHP-SHCC layer and four RC beams strengthened with a steel-reinforced mortar layer (see Table 2.2 and Figure 2.5). All specimens have dimensions: 200 mm × 200 mm×1800 mm and all the strengthened specimens have a 50 mm thick respective strengthening layer. The ultimate loads, the cracking loads and the yielding loads of the specimens are shown in Table 2.3. The load-deflection responses for tested beams are exhibited in Figure. 2.6. Test results revealed that using the small amount of steel reinforcement and UHP-SHCC like BU2 dramatically increase the load carrying of RC beams. Also this combination significantly enhanced the post peak behavior.

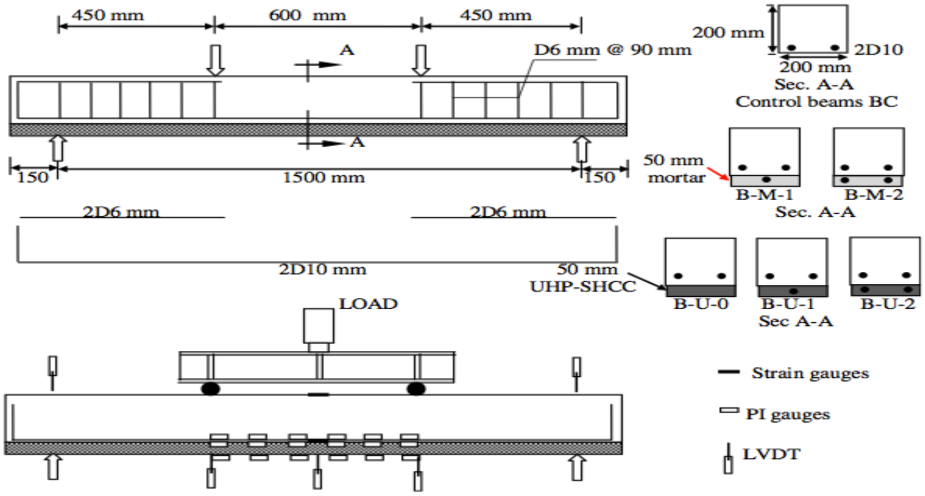


Figure 2. 5- Test setup and specimens' details (Mohamed et al. 2012).

Table 2. 2- Description of test beams.

Specimen	Strengthening layer reinforcement			No. of tested beams
	SHP-SHCC	Reinforced mortar	Reinforcement ratio %	
BC	–	–	–	2
B-U-0	–	–	0	2
B-U-1	1 D6	–	0.3	2
B-U-2	2D6	–	0.6	2
B-M-1	–	1 D6	0.3	2
B-M-2	–	2 D6	0.6	2

Table 2. 3- Cracking, yield, and ultimate load and corresponding displacement values.

Beam	Cracking				Yield		Ultimate load		Maximum crack width developed in the substrate concrete		
	Substrate conc.		UHP-SHCC		Main reinforcement						
	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)	At 32 kN (mm)	At 48 kN (mm)	At ultimate load (mm)
BC	19.0	1.09	–	–	41.0	3.5	49	40.15	0.25	3.5	3.5
B-U-0	35.0	0.52	50.0	0.95	78.3	3.92	82.3	4.77	–	0.05	0.2
B-U-1	35.8	0.48	50.5	1.02	82.2	4	88.9	5.96	–	0.05	0.2
B-U-2	35.9	0.54	51.0	1.04	95.5	4.69	100.7	11.00	–	0.05	0.25

B-M-1	31.0	0.46	−	−	51.0	3.6	62	32.05	0.05	0.30	3.0
B-M-2	31.5	0.47	−	−	60.0	3.5	74.4	37.05	0.05	0.25	3.5

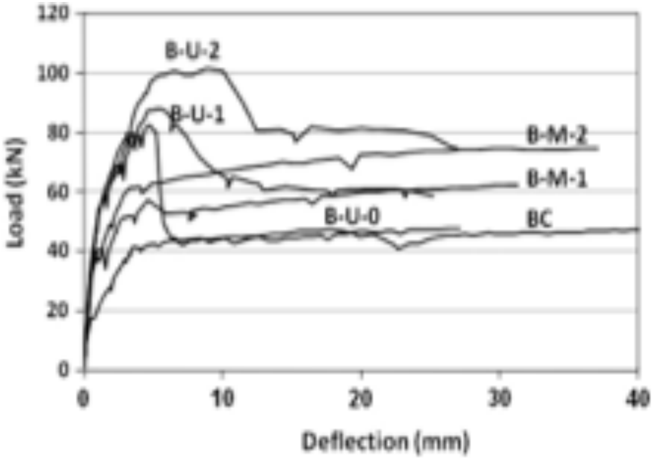


Figure 2. 6- Load-mid-span deflection curves for beams tested by Mohamed et al. (2012).

Ferrocement is the thin layer made of a cement mortar reinforced with a layer of small diameter wire meshes. Strengthening RC beams with ferrocement laminates is also one type of section enlargement. Flexural and shear capacity can be improved by casting ferrocement laminates to the soffits of beams or three sides except top face of beams. Paramasivam et al. (1998) reviewed the previous studies and concluded that ferrocement laminates is the viable material for strengthening concrete structures because it has higher tensile strength to weight ratio, toughness, ductility, durability and cracking resistance that is considerably greater than conventional cement based materials.

Sirju and Sharma (2001) compared the enhancement of different methods of strengthening reinforced concrete members under axial compression and bending. The control or unstrengthened beam is exhibited in Figure 2.7, and two test beams that were strengthened by ferrocement and fibre cement are shown in Figure 2.8. The first concrete beam was strengthened with five layers of 12 mm hexagonal mesh and rendering with 40 mm thick concrete to the surface. The second beam strenghthened with 40 mm thick fibre reinforced concrete. The test results and load deflection relationships were shown in Table 2.4 and Figure 2.9. Experimental results revealed that compared to the unstrengthened control beam, strengthened beams have higher ultimate flexural strength and stiffness. The failure mode for strengthened beams was ductile and gradual. As shown in Figure 2.8, Sirju and Sharma also tested beams strengthened with steel plates. These specimens will be discussed in Chapter 4.

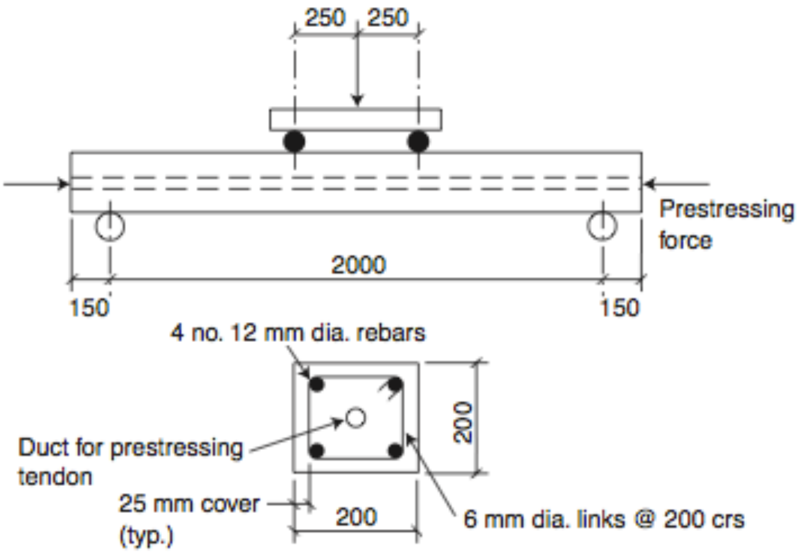


Figure 2. 7- Test Setup and specimen cross section（Mohamed et al. 2012)

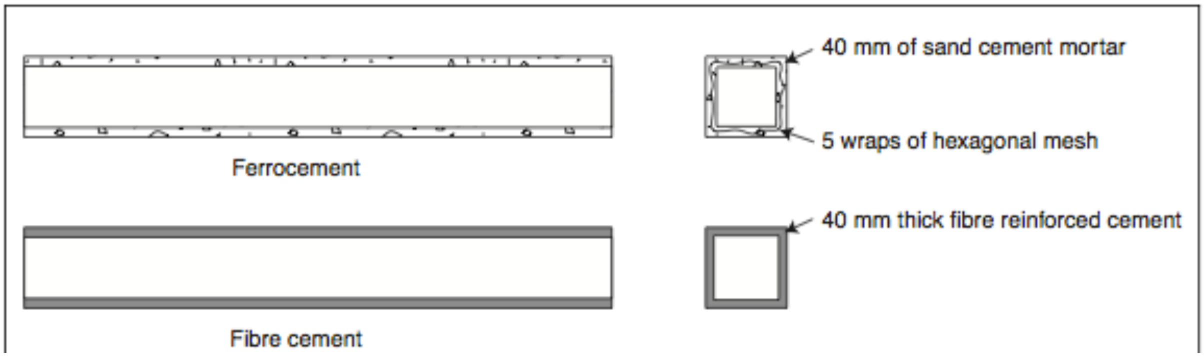


Figure 2. 8- Details of beams (Sirju and Sharma, 2001).

Table 2. 4-Test results and Comparison of flexural strength enhancement after strengthening.

Strengthening technique	Moment at first cracks (kN m)	Increase (%)	Moment at failure (kN m)	Increase (%)	Failure flexural load (kN)	Effective area of reinforcement based on BS 8110 Design Chart (%)
None (control)	12.87		18.02		48.05	1.13
Ferrocement	15.63	21.5	29.24	62.3	77.97	3.4
Fibre cement	17.48	35.7	27.58	53.1	73.58	2.9

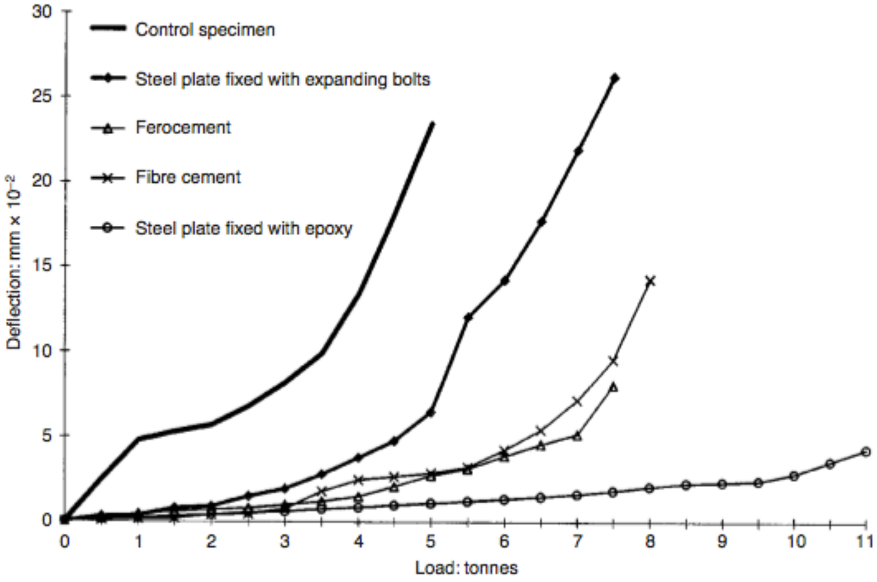


Figure 2. 9- Mid-span deflection of test specimens (Sirju and Sharma. 2001).

Shehata and Shehata (2008) investigated the behavior of RC beams strengthened by partial jacketing using expansion bolts as shear connectors. They categorized eight beams, which were 150 mm wide, 400 mm deep and 4500 mm long, in three groups A, B and reference group C. The three unstrengthened reference beams were in the group C and the other five partially jacketed beams were in group A and group B (Figure. 2.10 and Table 2.5). After two initial loading cycles the beams cracked, applied two lines of expansion bolts to the five beams on each side as showed in Figures 2.10 and 2.11. The holes were close to the inner stirrups and just above the main longitudinal steel. Table 2.8 provides specimen properties and ultimate load, where fcm is average concrete compressive strength, *d* is effective beam depth, *Asb* is area of main steel in the beam, *Asr* is added area of main steel in the jacket, *ρst* is total geometrical ratio of main reinforcement and *Pu*, *exp* is ultimate experimental load. Figures 2.12 through 2.17 provide the measured beam deflections, main steel strains, main steel strains inside the jackets and the maximum relative displacement between the beams and the jackets. The experimental results showed that partial jacketing is an effective strengthening method. In order to get proper anchorage, the inserted depth of the expansion bolts should be greater than five times the bolt diameter and not less than 50 mm. Exposed part of expansion bolts should be left without the extension. Exposed part and holes of expansion bolts should be as close as possible to the original stirrups and original main longitudinal steel of beams.

Table 2. 5- Characteristics of tested beams.

Beam	*fcm* (MPa)	*d* (*mm*)	*Asb* (mm^2)	*Asr* (mm^2)	ρst (%)	*Pu*,exp (kN)	Failure mode
Group A							
V1-A	41.6	382	285	300	1.02	150	Flexural
V2-A	38.6	402	285	600	1.47	205	Flexural/Shear
V3-A	39.2	409	285	800	1.77	229	Flexural/Shear
Group B							
V1-B	36.4	360	600	300	1.67	186	Flexural
V2-B	41.4	377	600	600	2.12	235	Flexural
Group C							
REF1	36.2	386	285	-	0.49	72	Flexural
REF2	41.4	369	600	-	1.08	130	Flexural
REF3	40.8	351	1230	-	2.33	219	Flexural

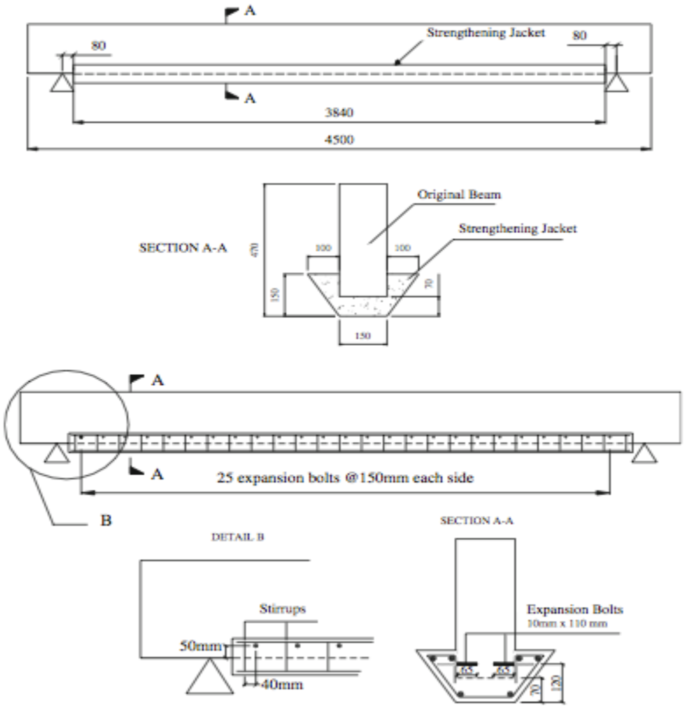


Figure 2. 10-Strengthening details of specimen tested by Shehata and Shehata (2008).

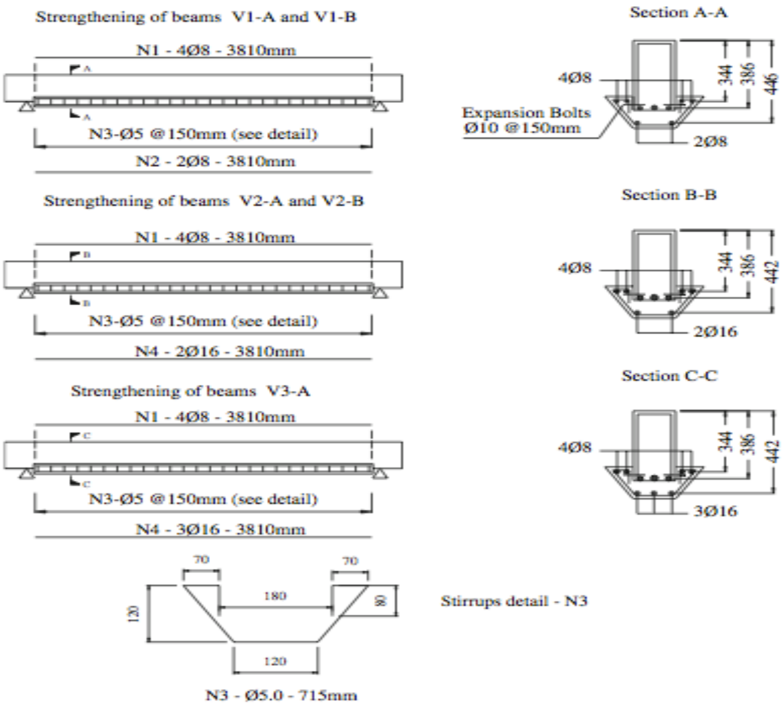


Figure 2. 11- Details of strengthening reinforcement in the jackets (Shehata and Shehata 2008).

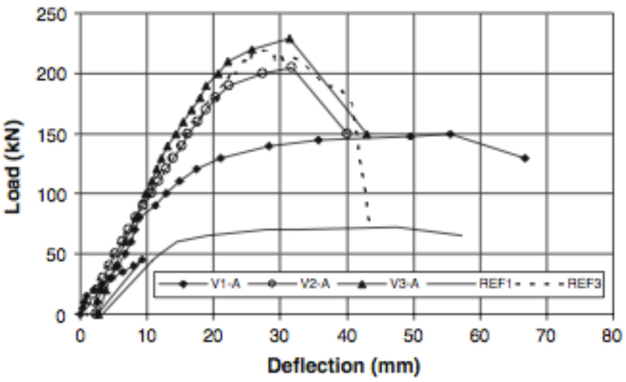


Figure 2. 12- Load-deflection curves for the beams of first group together with reference beams REF1 and REF3 (Shehata and Shehata 2008).

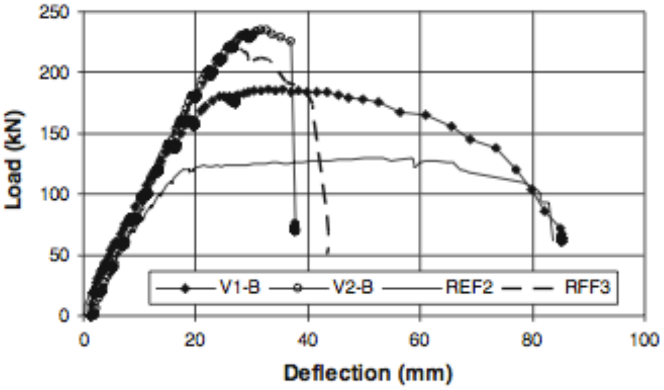


Figure 2. 13- Load-deflection curves for the beams of second group together with reference beams REF2 and REF3 (Shehata and Shehata 2008).

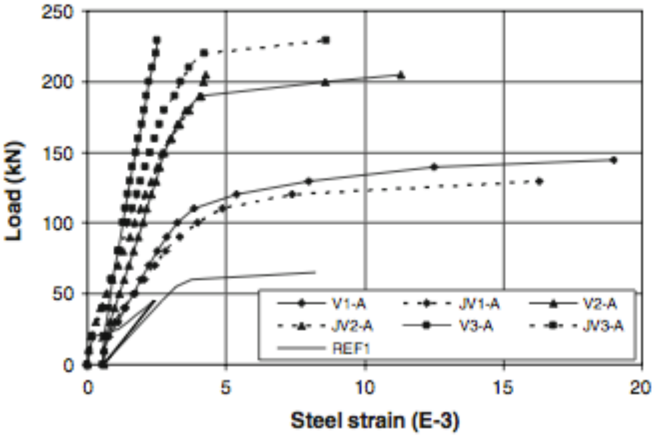


Figure 2. 14- Load-main steel strain at mid-span curves for the beams of first group together with reference beams REF1 (Shehata and Shehata 2008)

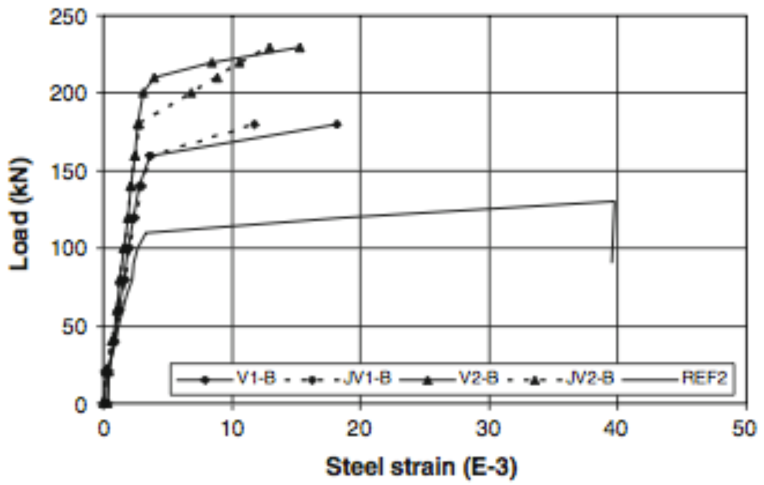


Figure 2. 15- Load-main steel strain at mid-span curves for the beams of first group together with reference beams REF2 (Shehata and Shehata 2008).

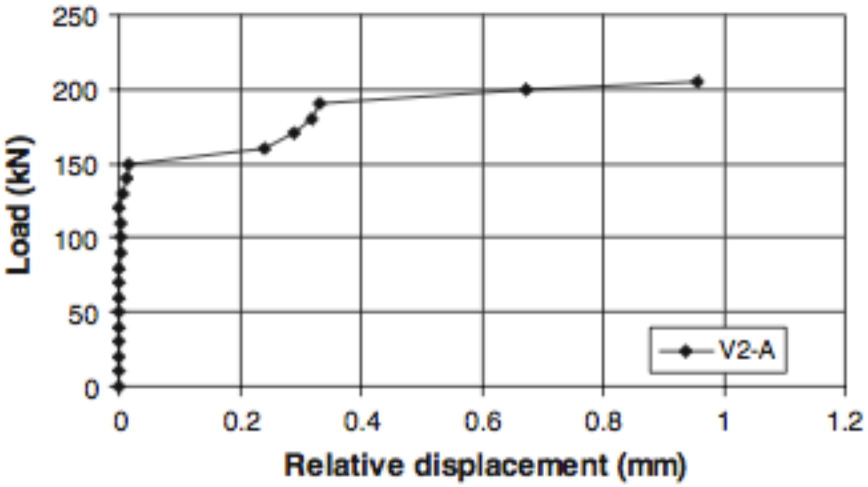


Figure 2. 16- Load-maximum relative displacement curves between the beam V2-A and the jacket (Shehata and Shehata 2008).

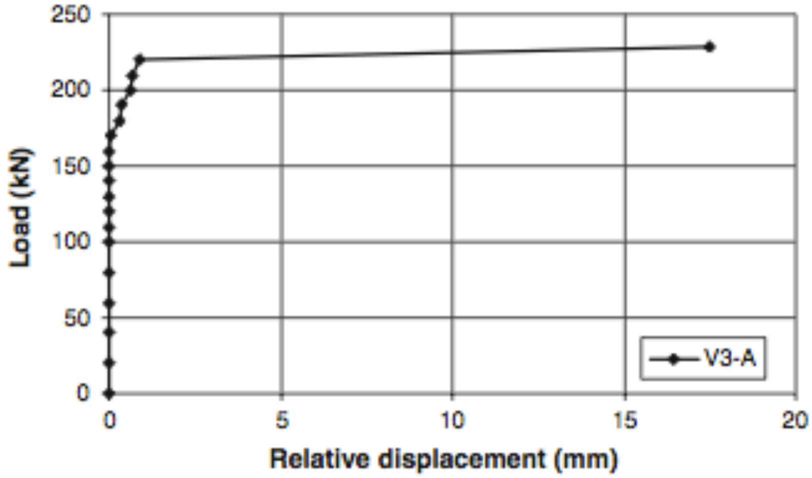


Figure 2. 17- Load-maximum relative displacement curve between the beam V3-A and the jacket (Shehata and Shehata 2008).

2. 3 Previous Research on Concrete Jacketing of Beams

Similar to section enlargement method concrete jacketing can be easy, effective and inexpensive technique to rehabilitate and strengthen concrete structures. Concrete jacket is achieved by adding the reinforced concrete jacket to the existing structure components such as beams and columns.

Altun et al. (2004) compared the mechanical properties of RC beams before and after jacketing under bending test. Altun categorized nine 1800 mm long reinforced concrete beams with 20 MPa concrete strength, 420 MPa steel strength in three groups based on their three different cross sections (Table 2.6) and then loading them until full failure. The other nine beams that have the same dimensions were strengthened with 100 mm thick RC jackets on all four sides (Figure 2.19), loaded them to full plastic yield. Typical test beam is shown in Figure 2.18 and the results of experiment are shown in Figure 2.20 and Table 2.6. The results revealed that damaged RC beams would behave similar to the ordinary RC beams of same dimensions with added RC jackets. However, the beam with highest ductility ratio is the most efficient since the section area is relatively less as compared to the section resisting the maximum ultimate load. This reduces the amount of cost of the jacketing material.

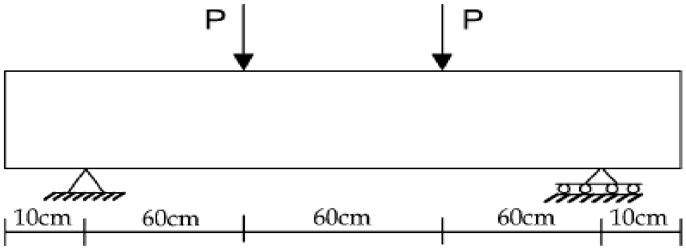


Figure 2. 18- Loading configuration of the jacketed RC beam (Altun et al. 2004).

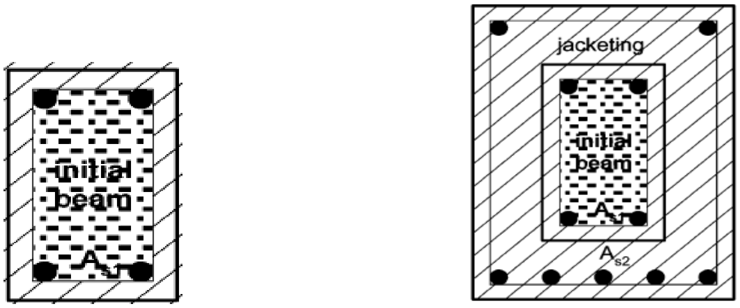
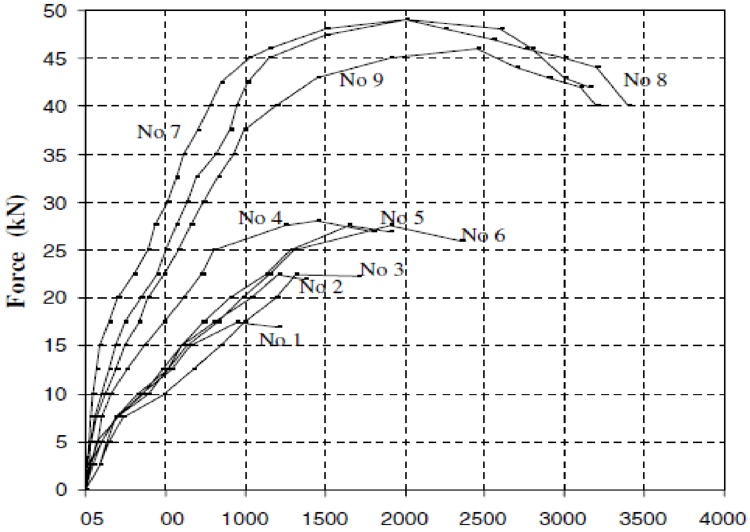


Figure 2. 19- Cross-section of beams before and after jacketing (Altun et al. 2004).

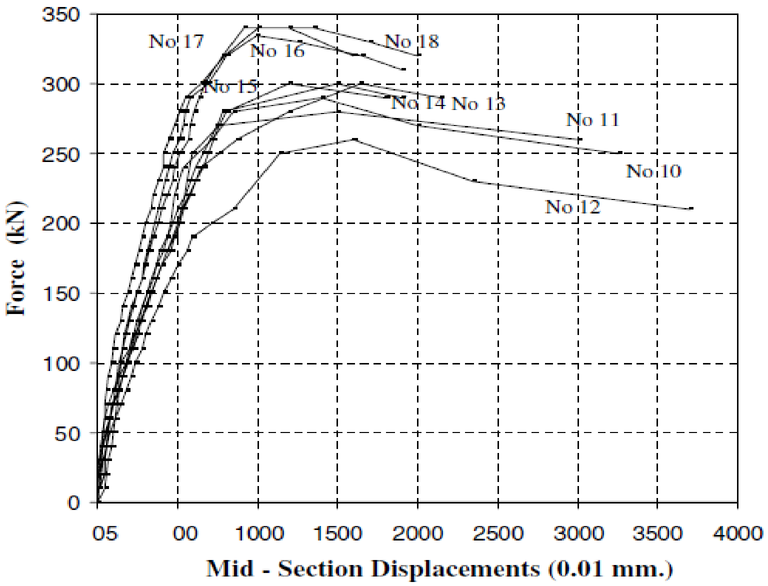
Table 2. 6-The properties of specimens and test results

Specimen	Depth (mm)	Width (mm)	Amount of Tensile Steel	Experiment Ultimate Load (kN)	Disp at Yielding δy (mm)	Disp at Ultimate δu (mm)	Ductility Ratio
1	150	150	2φ8	19.85	8.9	18.5	2.08
2	150	150	2φ8	23.25	8	13.65	1.71
3	150	150	2φ8	23	9.9	15.8	1.60
4	200	150	3φ8	31.5	9.3	13.1	1.41
5	200	150	3φ8	28.05	12.8	19	1.48
6	200	150	3φ8	28.45	13	23.5	1.81
7	200	200	3φ8	39.95	19	28.5	1.50

8	200	200	3φ8	40.25	18	30	1.67
9	200	200	3φ8	40.8	15	24.5	1.63
10	350	350	4φ2	262	7.1	15	2.11
11	350	350	4φ2	247	7.65	16.5	2.16
12	350	350	4φ2	246	8.54	17.2	2.01
13	400	350	5φ2	283	8.7	21.5	2.47
14	400	350	5φ2	296	7.3	20.75	2.84
15	400	350	5φ2	295	7.8	20.2	2.59
16	400	400	5φ2	337	6.7	12.5	1.87
17	400	400	5φ2	343	6.07	12	1.98
18	400	400	5φ2	339	6	11.8	1.97



(a)



(b)

Figure 2. 20- Load versus displacement at midspan of beams tested by Altun et al. (2004).

Cheong et al. (2000) conducted the investigation for the behavior of reinforced concrete beams strengthened by jacketing under static and dynamic load until failure. The simply supported beams were tested statically or dynamically and the continuous beams (see Figure 2.21) were tested statically as the details are shown in Table 2.7. Figure 2.22 presents the test results of control specimen 2-1, jacketed beams 2-2, 2-3 and 2-9 that strengthened with various bond conditions as listed in Table 2.7. Figure 2.23 gives the test results of jacketed beams 2-4, 2-5 and 2-6 with deliberately reduced host jacket bond. Beam 2-8 has additional stirrups at 2*d* only (*d*= effective depth), 2-10 has additional stirrups grouted into underside of flange, jacketed beam 4-1 without additional links enclosing longitudinal reinforcement (see Figure 2.24). Figure 2.25 plots the relation of static load and displacement of continuous monolithic beams 6-1 and 6-2. The details of dynamic tests on simply supported beams 8-1, 8-2, 8-3, 9-1 and 9-2 are listed near the bottom of Table 2.7 (PA means preplaced aggregate concrete and PC means plain concrete). The experimental results revealed that the reinforcement should be adequately anchored past the point of contraflexure and the support of simple beams. Also adequate anchorage is necessary for additional stirrups near the underside of flange. The fully anchored stirrups contribute fully to the strength of the jacketed beam. Width of the upgraded beam should be similar to support width. The effect of roughening in interface does not influence the behavior of jacketed beams very much.

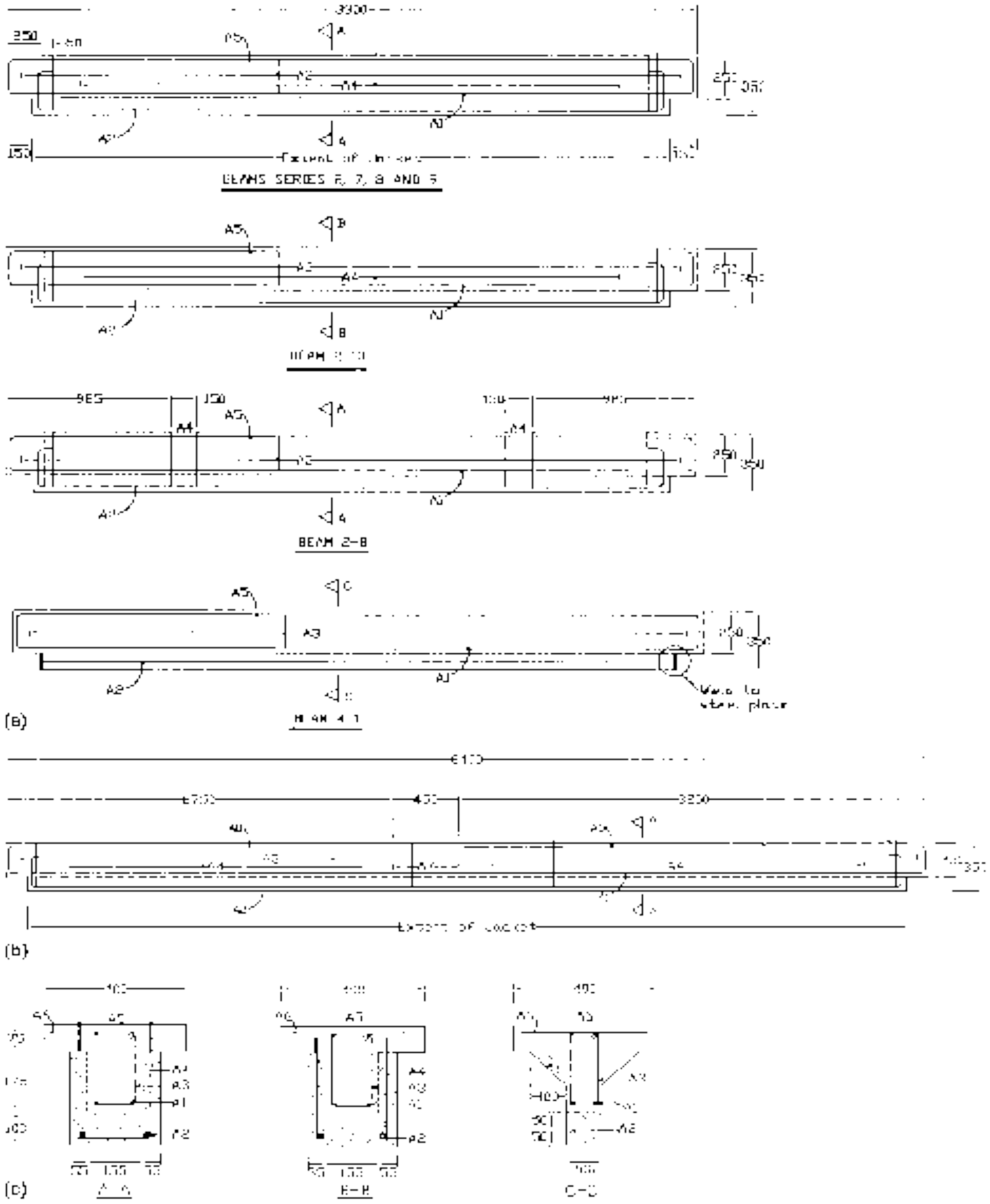
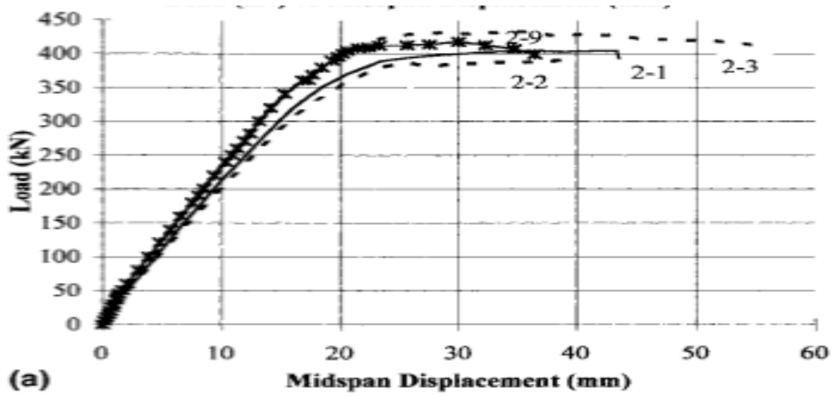


Figure 2. 21- Simply supported and continuous beams (a) 1; (b) 2; (c) 3

Table 2. 7- Details of beams tested under static loads.

Beam	Construction	Interface preparation	*fcu* of PA (N/mm²)	*fcu* of PC (N/mm²)	Failure load (kN)	Failure mode

1	2	3	4	5	6	7
2−1	Monolithic	−	−	45	404	Flexure
2−2	Composite	Partially roughen	55	45	393	Flexure
2−3	Composite	Fully roughen	50	45	433	Flexure
2−4	Composite	Fully roughen I1,I2,PaintI3	70	30	430	Flexure
2−5	Composite	Fully roughen I2,I3,PaintI1	70	50	422	Flexure
2−6	Composite	Fully roughen I2, Paint I1,I 3	70	30	410	Shear
2−8	Composite	Partially roughen	55	50	351	Anchorage
2−9	Composite	Partially roughen	55	50	418	Flexure
2−10	Composite	Partially roughen	55	50	376	Anchorage
4−1	Composite	Partially roughen	60	50	135	Interface
6−1a	Monolithic	−	−	30	309	Bearing
6−2a	Composite	Partially roughen	60	30	374	Bearing
1-Aug	Monolithic	−	−	50	411	Shear
2-Aug	Composite	Partially roughen	55	45	407	Flexure
3-Aug	Composite	Fully roughen	55	45	393	Flexure
1-Sep	Monolithic	−	−	50	Cycle 608,738	Fatigue
2-Sep	Composite	Partially roughen	60	30	Cycle 436139	Fatigue



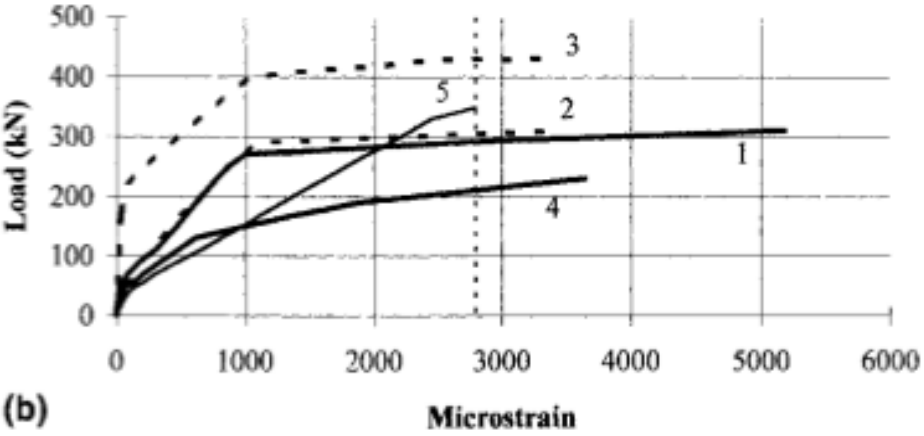


Figure 2. 22- Beams 2-1, 2-2, 2-3 and 2-9: (a) load versus mid-span displacement: (b) load versus Longitudinal steel strain. (Cheong et al. 2000).

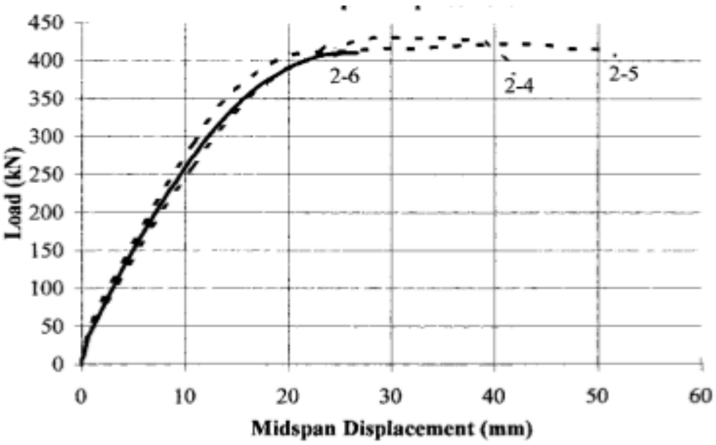


Figure 2. 23- Load versus. mid-span displacement- beams 2-4, 2-5 and 2-6 (Cheong et al. 2000).

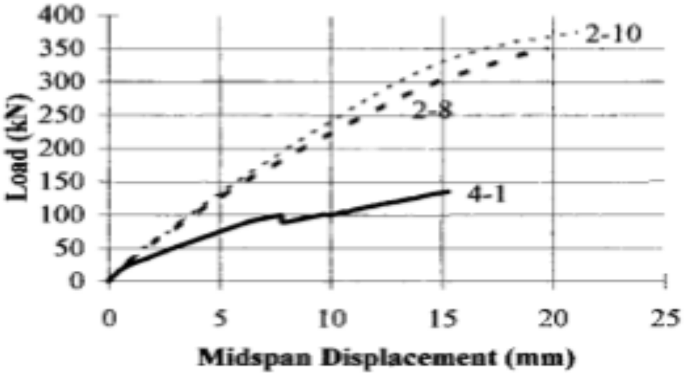


Figure 2. 24- Load versus displacement- beams 2-8, 2-10 and 4.1 (Cheong et al. 2000).

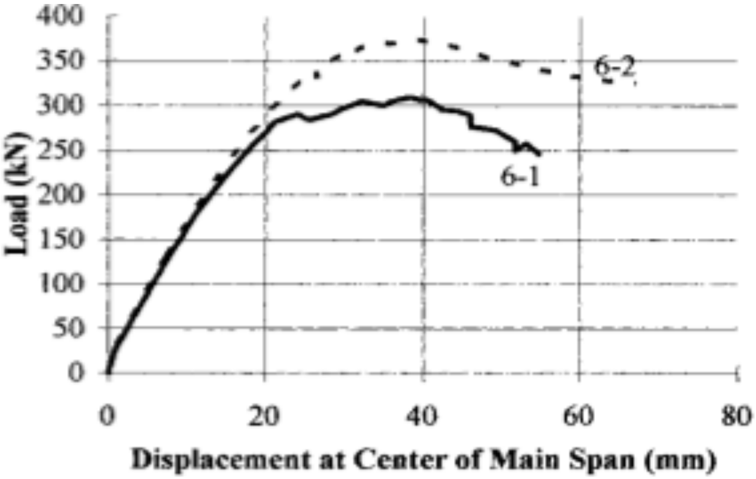
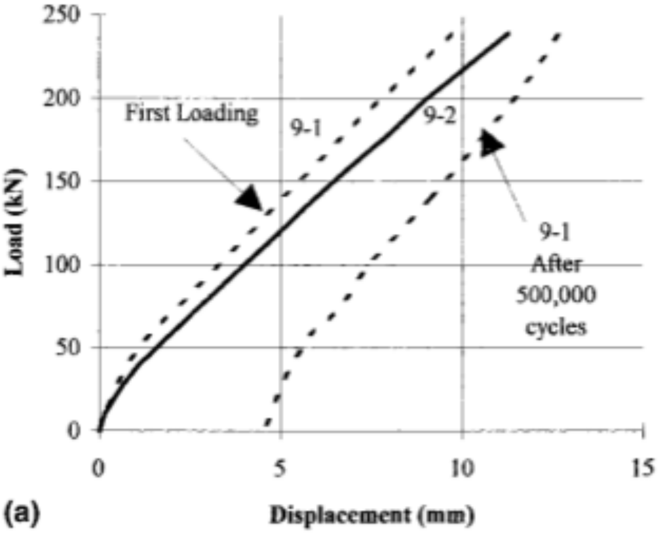


Figure 2. 25- Load versus mid-span displacement-beams 6-1 and 6-2 (Cheong et al. 2000).



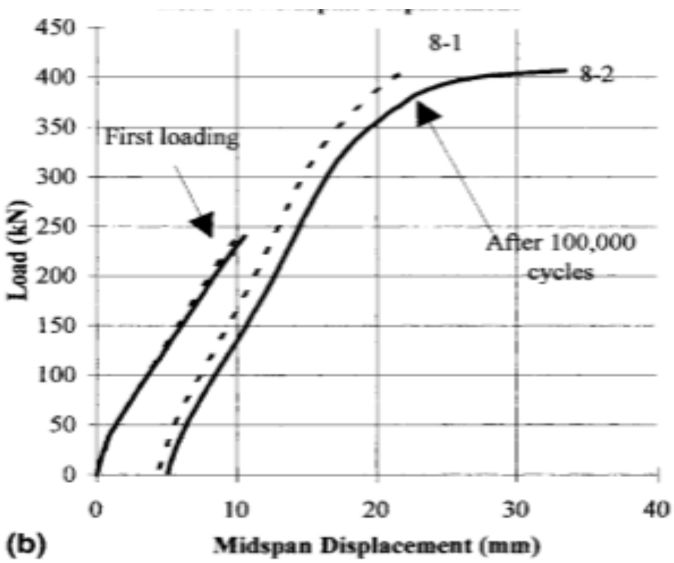


Figure 2. 26- Load versus mid-span displacement: a) beams 8-1 and 8-2; b) beams 9-1 and 9-2 (Cheong et al. 2000).

2. 4 Summary and Conclusion

Jacketing by reinforced concrete and section enlargement may be the relatively easy and economic strengthening method compared to attachment of an external steel, external post-tensioning or externally bonded composite system. It effectively increases the load carrying capacity or stiffness. However, the addition of concrete and steel to repair beams increases the weight of beams. So, the lightweight concrete may be considered as better applied when strengthening the beams. Strengthening with concrete and steel rebar might lead to corrosion in beams. Hence, section enlargement and concrete jacketing are limited to use in harsh environment and the protecting corrosion is important work.

CHAPTER 3 External Reinforcement

3. 1 Introduction

Retrofit of RC beams can be achieved by adding external longitudinal reinforcement to the RC beams in order to increase their load carrying capacities. This method can overcome many drawbacks of other methods. It is inexpensive and easy to execute.

3. 2 Description of Previous Research

Kothandaraman et al. (2010) devised a new technique that retrofit RC beams with external reinforcement at soffit level. This retrofitting method is cost effective, simple, and easy to achieve. As shown in Figure 3.1, Kothandaraman et al. used special chemical adhesives anchored to the bars into two pieces and welded them together in the soffit level of beams ERB1 and ERB2. All the specimens were tested under two-point loading till failure. The deflections at mid-span and one-fourth are shown in Figure 3.2 and test results are presented in Table 3.1. The test results showed that the retrofitting the external bars on the soffit level in this way significantly decreased the width of cracks, deflections, and the moment carrying capacity was increased compared to un-retrofitted beams.

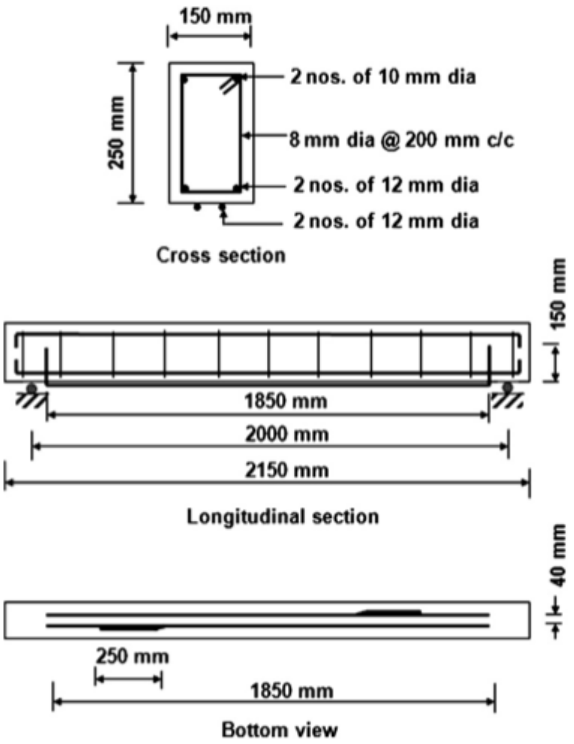


Fig. 4. Externally reinforced beams – ERB1 and ERB2.

Figure 3. 1-The externally reinforced beams-ERB1 and ERB2 (Kothandaraman et al. 2010)

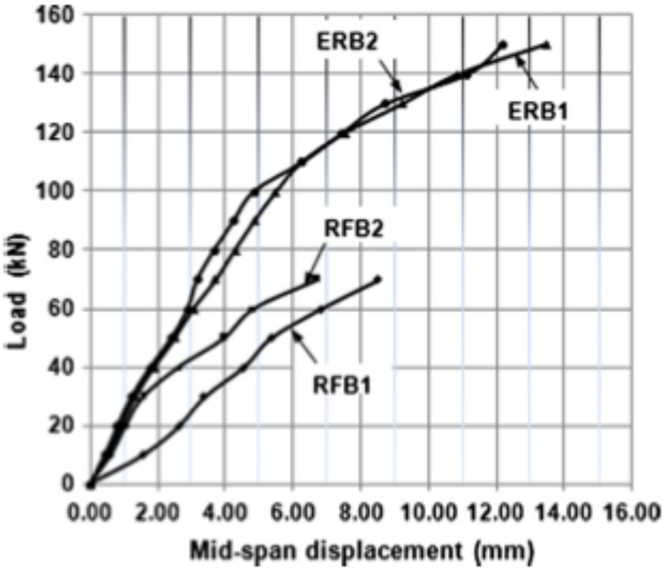


Figure 3. 2- Load-deflection diagram at mid-span (Kothandaraman et al. 2010).

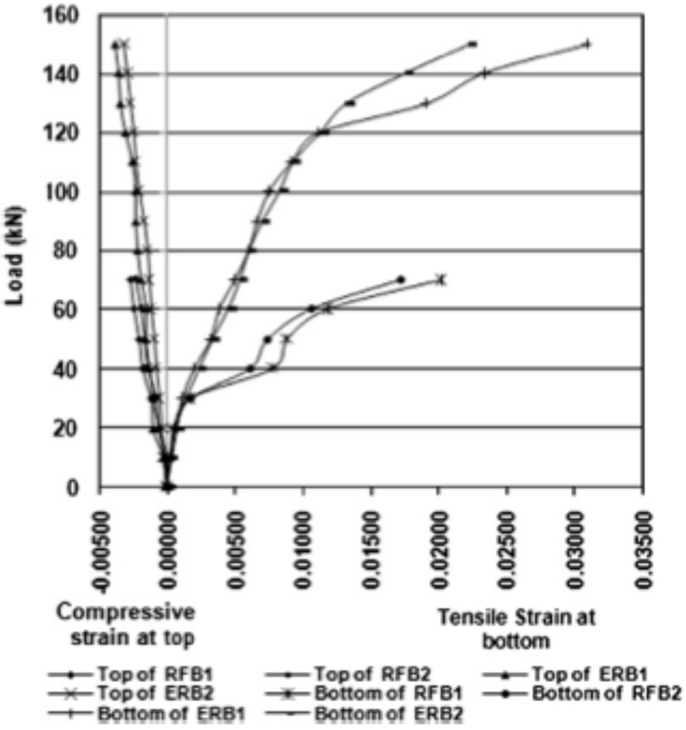


Figure 3. 3- Load-strain curves (Kothandaraman et al. 2010).

Table 3. 1-Test results using external bar in two pieces and tied by welding.

Beam	Concrete Strength (N/mm)	Mid-span deflection at ultimate load (mm)	Maxium crack width (mm)	Tested ultimate moment $M_{u,test}$ (kNm)	Calculated ultimate Moment $M_{u,cal}$ (kNm)	Comparative ultimate moment $M_{u,comp}$ (kNm)	$M_{u,test}$ $/M_{u,cal}$	$M_{u,test}/$ $M_{u,comp}$	Mode of failure
RFB1	46.71	8.78	4.00	31.2	26.29	-	1.19	-	Yielding of bars
RFB2	8.74	8.74	5.00	30.88	26.2	-	1.18	-	Yielding of bars
ERB1	15.19	15.19	2.00	53.63	52.25	43.39	1.03	1.24	Crushing of concrete
ERB2	14.5	14.5	2.00	53.63	53.25	44.38	1.01	1.21	Crushing of concrete

Cairns et al. (1997) conducted 21 tests with six beams cast in three sets which each set contained two beams. Every first specimen in each group was set as reference specimen and the other one was strengthened with external reinforcement anchored at the end of the beam. All specimens had 3500 mm overall length and 3000 mm span were tested under four-point bending (Table 3.2 and Figure 3.4). The reference beams were first loaded up to 65% of the ultimate capacity. The beams were then removed from the load, and finally they were reloaded to failure. The second sets of beams were also subjected to a pre-loading cycle to develop service crack patterns. The applied load and mid-span deflection are reported in Table 3.3. In order to identify beams during test, each test is described by a five characters identifier which as shown in Table 3.3. The first letter means the test series and the second letter indicates the specimen number within that series. The third number represents the shear span/effective depth ratio (a_v/d) for the test and the forth represents the effective depth of the external bars, d_{ext}. The last number represents the load cycle. Orb indicates ordinary reinforced beam, EUBRF represents beam with additional external unbonded reinforcement. The test results revealed that the loading arrangement, effective depth of external reinforcement, and geometric ratio of bounded reinforcement increased ultimate flexural strength of RC beams especially for the lightly reinforced beams.

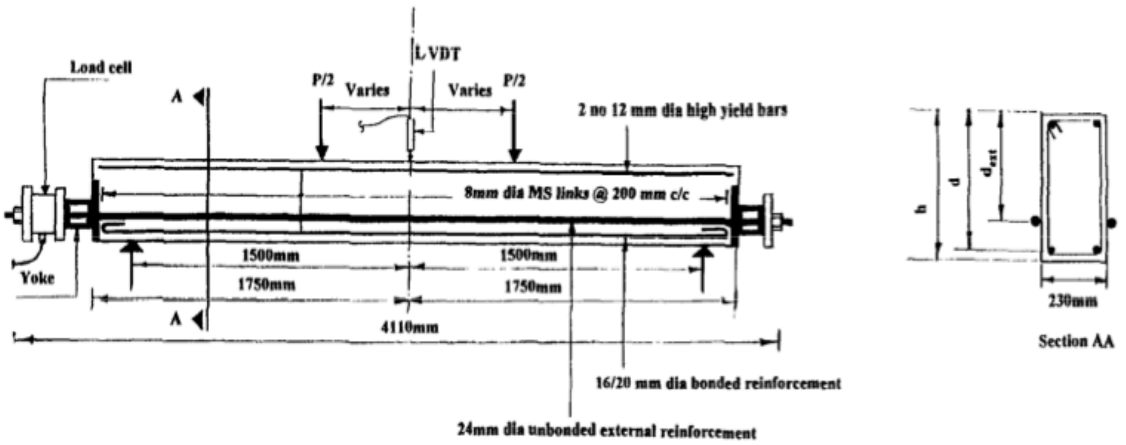


Figure 3. 4- Details of test specimens (Cairns et al. 1997).

Table 3. 2- Details of test specimens as cast.

Specimen Ref	Beam Depth *h* (mm)	Effective depth of bonded bars *d* (mm)	Bonded Reinforcement			Concrete compr. Strength *fcu* (N/mm^2)
			No.	Dia. (mm)	Yield Strength *fy* (N/mm)	
A1	400	355	2	20	493	42
A2	400	355	2	20	493	45
B1	400	355	2	16	510	35.4
B2	400	355	2	16	510	39.5
C1	300	255	2	16	497	39
C2	300	255	2	16	497	36.5

Table 3. 3- Applied loads and mid-span deflections.

Specimen Ref.	Shear Span av (mm)	Effective depth of ext-bar dext (mm)	P65 (kN)	Neutral axis depth factor x/d	Max comp strain in conc $\times 10^{-6}$	Surface strain at the level of bonded bars $\times 10^{-6}$	Force in external bars Fext mm	Mid-span deflection Δc mm	Comments
A1/20/2	1250	–	105	0.377	709	1172	–	5.31	Cracked ORB: to failure
A2/20/2	1250	–	105	0.36	868	1543	–	5.97	Cracked ORB: to failure

A2/21/3	1250	355	105	0.63	1270	746	36.1	5.59	EUBRF: to P65
A2/23/4	1250	255	105	0.45	1161	1420	28	5.7	EUBRF: to P65
A2/32/5	1100	305	105	0.595	1212	825	23	4.25	EUBRF: to P65
A2/12/6	1400	305	105	0.377	990	1637	31.5	5.96	EUBRF: to P65
A2/22/7	1250	305	105	0.58	1308	948	30	4.93	EUBRF: to failure
B1/20/2	1250	–	72	0.376	762	1265	–	4.25	Cracked ORB: to failure
B2/20/2	1250	–	72	0.37	776	1322	–	4.62	Cracked ORB: to P65
B2/21/3	1250	355	72	0.382	508	823	32	4.23	EUBRF: to P65
B2/23/4	1250	255	72	0.343	549	1052	26.5	4.15	EUBRF: to P65
B2/32/5	1100	305	72	0.368	498	855	25.5	3.18	EUBRF: to P65
B2/12/6	1400	305	72	0.33	492	999	40.5	4.1	EUBRF: to P65
B2/22/7	1250	305	72	0.351	491	908	29.4	3.85	EUBRF: to failure
C1/20/2	1100	–	56	0.369	888	1518	–	7.73	Cracked ORB: to failure
C2/20/2	1100	–	56	0.403	819	1214	–	7.54	Cracked ORB: to P65
C2/21/3	1100	355	56	0.453	646	782	32	4.24	EUBRF: to P65
C2/23/4	1100	275	56	0.401	713	1066	26	5.68	EUBRF: to P65
C2/32/5	800	215	56	0.43	637	845	22.5	5.84	EUBRF: to P65
C2/12/6	1400	215	56	0.393	1032	1594	38.5	7.82	EUBRF: to P65
C2/22/7	1100	215	56	0.41	721	1039	29.5	5.81	EUBRF: to P65

Tan et al. (2003) investigated the shear deficiency of reinforced concrete continuous beams strengthened with different external tendons. The four two-span externally prestressed T-beams are shown in Figure 3.5. The top of the beams had four or six deformed 10 mm diameter steel bars. The transverse reinforcements consisted of mild steel closed stirrups with diameters of 6 and 8 mm. Strengthening the beams with seven-wire prestressing steel strands that each one has a diameter of 9.5 mm and a cross-area of 66 mm^2. The strengthening details are provided in Table 3.4. Four strengthened beams and one unstrengthened reference beam S0 were tested monotonically under four-point loads until failure. Ultimate load of beams, deflections and stress of internal stirrups are recorded and exhibited in Table 3.5, Figures 3.6 and 3.7. In Table 3.5, strengthening ratio indicates the ratio of observed ultimate load of the test beam to the predicted ultimate load of the unstrengthened Beam S0. The experimental results proved the effectiveness of using external tendons strengthening continuous beams. However, the flexural capacity of strengthened beams was limited by lower shear capacity. Furthermore, strengthening concrete beams with draped or parabolic profile tendon minimized the shear failure risk because they increased the shear strength.

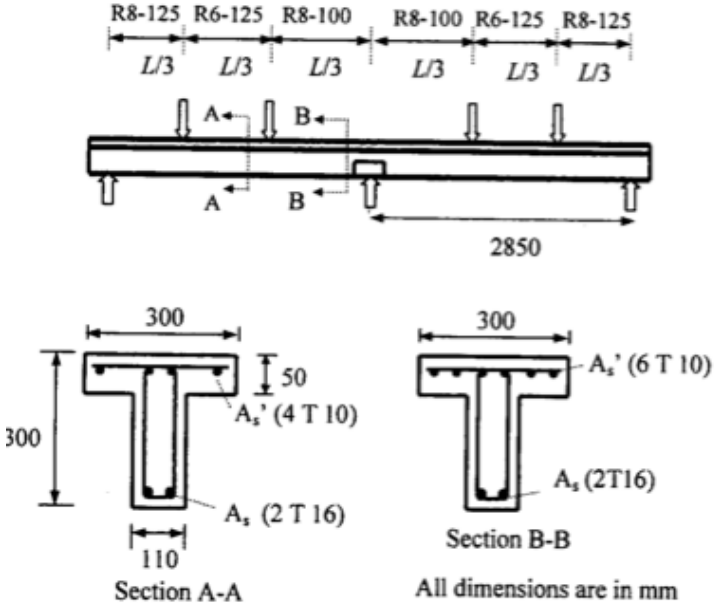
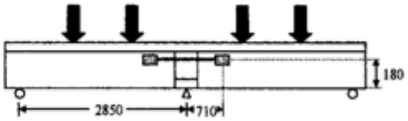
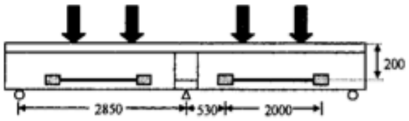
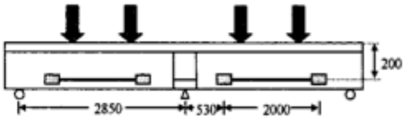
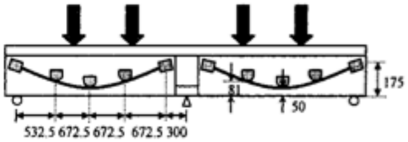


Figure 3. 5- Cross-section of unstrengthened beam (Tan et al. 2003).

Table 3. 4- Details of test beams.

Beam	Profile	Effective prestressing force F_{pe}, kN	Concrete strength f_c', MPa
S1		56.4	27.6
S2		104.5	36.3
S3		33.4	37.8
S4		33.4	33.3

Note: All dimensions in mm.

Table 3. 5- Ultimate loads of test beams.

Beam	Ultimate load *Pu*, kN			Test/Predicted	Strengthening radio
	Test	Predicted			
		Flexure	Shear		
S0	-	322	398	-	1
S1	382.2 (FT)	364	410	1.05	1.18
S2	410.2 (SC)	470	414.4	0.99	1.27
S3	421.3 (FT)	418.4	419.2	1.01	1.3
S4	397.8 (SC)	413.2	409.6	0.97	1.23

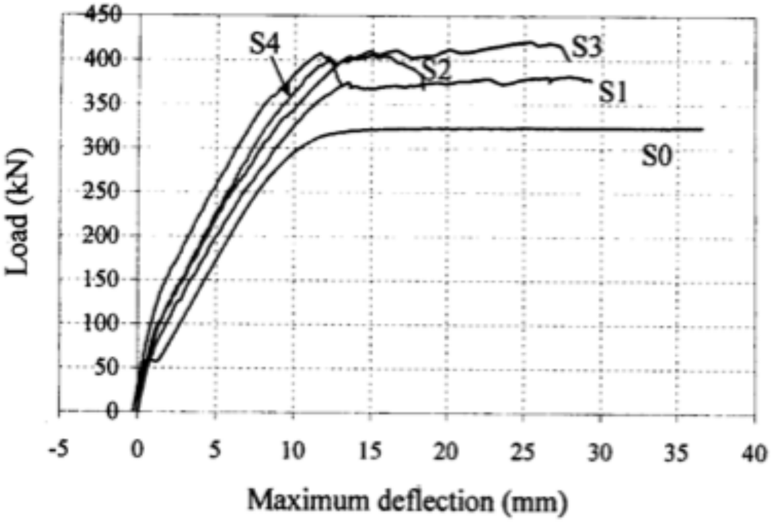


Figure 3. 6- Load-deflection response (Tan et al. 2003).

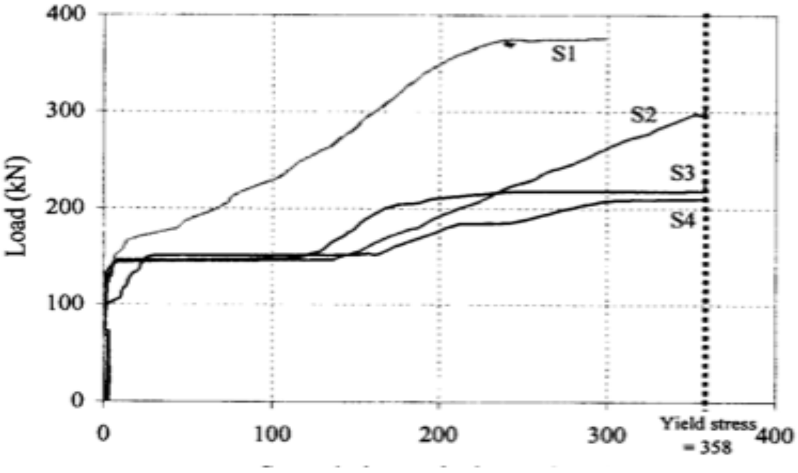


Figure 3. 7- Load-stress relations in internal stirrups (Tan et al. 2003)

3. 3 Summary and Conclusion

Strengthening and retrofitting of RC beams by attaching external reinforcement is an effective, easy, and economic method due to it being easy to install, speedy execution, and cost less compared to other methods. However, the extra reinforcement may also increase the weight of beams. Protection from corrosion and fire need to be considered.

CHAPTER 4 Strengthening Beams Using of Steel Plates

4. 1 Introduction

Attaching steel plates to certain external surface of the beams is another popular strengthening technique. Anchoring or bonding steel plates to reinforced concrete beams can increase flexural and shear capacity. Furthermore, it can control deflections and cracking of beams. The efficiency of steel plates is influenced by some factors such as the dimension of the steel plate, the arrangement of bolts, and bonding method. This chapter discusses how to optimally repair and strengthen the beams by considering these factors.

4. 2 Description of previous research

Hussain et al. (1995) explored the steel plate bonding repair technique including the effects of plate thickness and end anchorage on ductility, ultimate load and mode of failure. They tested eight beams named from FRB1 to FRB8. FRB1 is kept as a control beam since it is not strengthened. Figure 4.1 shows the dimensions of beams that the length is 1200 mm, breadth is 150 mm and the width is 150 mm. As Table 4.1 shows, the different thickness plates are bonded by two-part epoxy glue in some beams and end anchorage was used for bonded steel plates by anchor bolts. Yield strength of main steel and stirrups of tested beams are 414 MPa and 275 MPa. Concrete strength of beams is 31 MPa. All the beams are preloaded to 85% of ultimate load. Figure 4.2 shows the strengthening details of two typical beams with and without end anchorage. The retrofits beams have 1100 mm long and 100 mm wide. The yield strength of steel plate is 269 MPa and shear modulus of glue is 120 MPa. The thickness of steel plates and bolt dimension details are given in Table 4.1. The repaired beams were tested to failure under four-point loading and the results properties are listed in Table 4.2, Figures 4.3 through 4.8 shows the experimental results that indicate the ductility of the repaired beams decreases as plate thickness increases. Although end anchorages to the bonded plate can improve ductility, the improvement in ductility due to end anchorages decreases as plate thickness increases. Instead of pure flexure failure, the failure could due to increased thickness of bonded steel due to the tearing of concrete in the shear span. Even end anchorages to the bonded plates could not help to prevent the premature failure.

Table 4. 1- The retrofits properties

Specimen	Thickness (mm)	Bolt dimension (mm)	
		Diameter	Length
FRB1	-	-	-
FRB2	1	No end anchorage	
FRB3	1.5	No end anchorage	
FRB4	1.5	15	75
FRB5	2	No end anchorage	
FRB6	2	15	75
FRB7	3	No end anchorage	
FRB8	3	15	75

Table 4. 2- Summary of Test Results.

Specimen	Experimental Maximum Load (kN)	Modulus of Toughness (kN-mm)	Interface Shear stress (N/mm^2)	Max Displacement (mm)
FRB1	54	954	-	55
FRB2	69.54	872	5.43	70
FRB3	75	325	6.15	75
FRB4	77.86	734	-	78
FRB5	60	178	4.79	60

FRB6	66	633	-	66
FRB7	58	102	4.55	58
FRB8	57.8	237	-	59

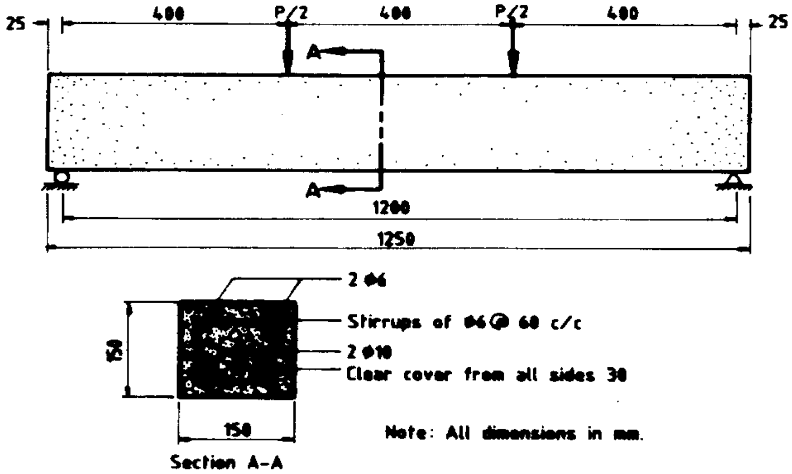
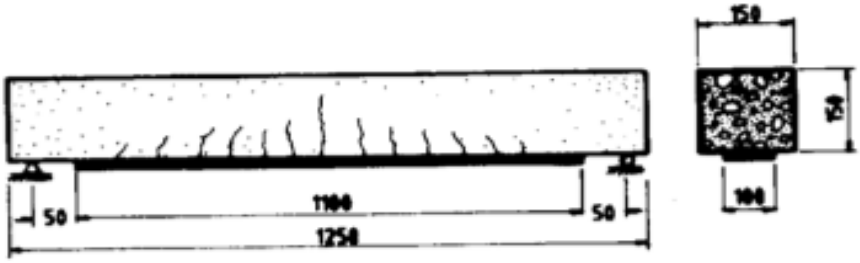
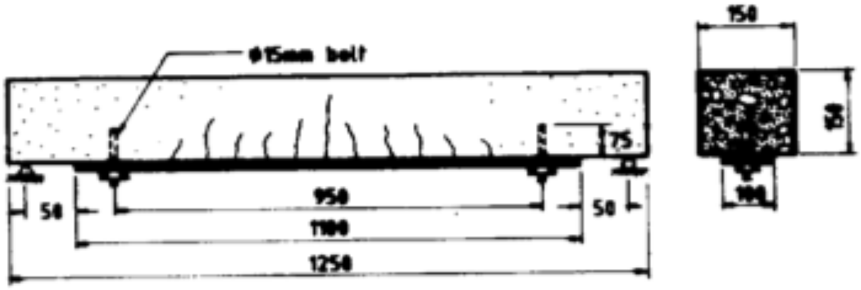


Figure 4. 1- Dimensions and reinforcement detail of Beams FRB2 through FRB8 (Hussain et al. 1995).



(a) Typical beam with unanchored plate.



(b) Typical beam with anchored plate

Figure 4. 2- **Strengthening detail of two typical beams, one with anchored plate and another with unanchored plate (Hussain et al. 1995).**

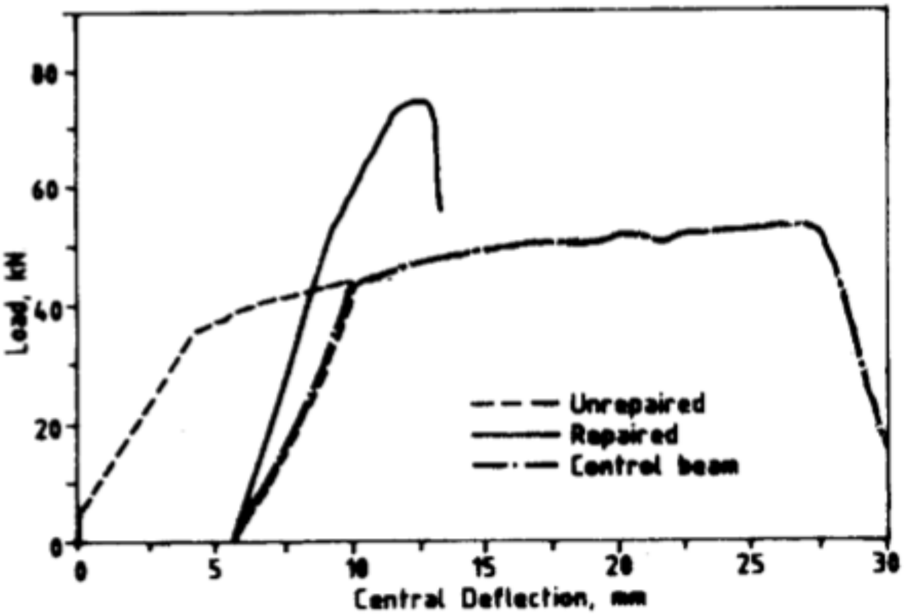


Figure 4. 3- Load-deflection curve of Beam FRB3, strengthened with 1.5-mm-thick steel plate without end anchorage (Hussain et al. 1995).

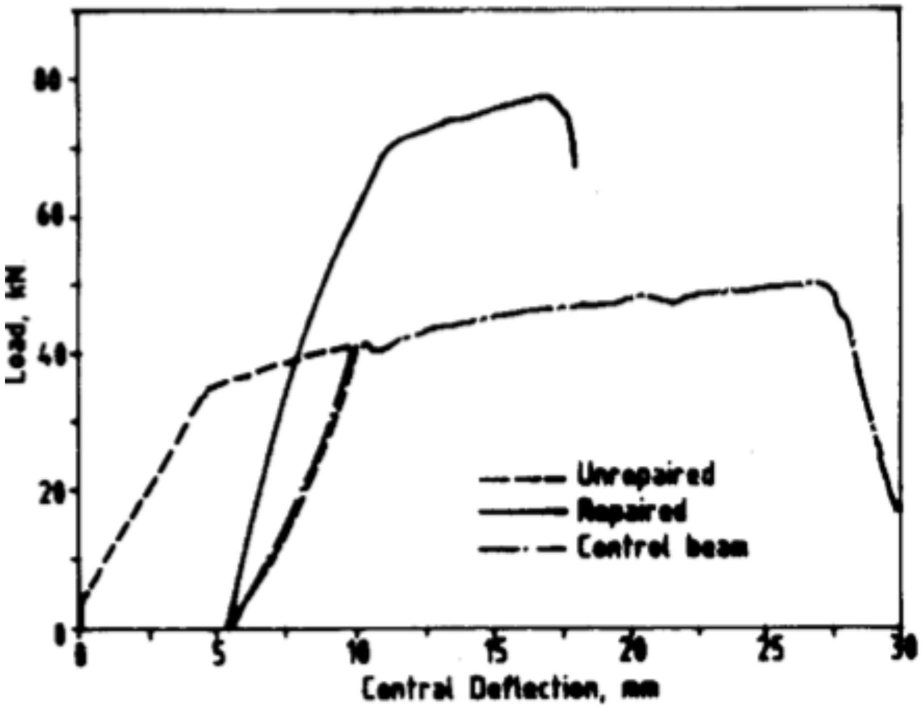


Figure 4. 4- Load-deflection curve of Beam FRB4, strengthened with 1.5-mm-thick steel plate with end anchorage (Hussain et al. 1995).

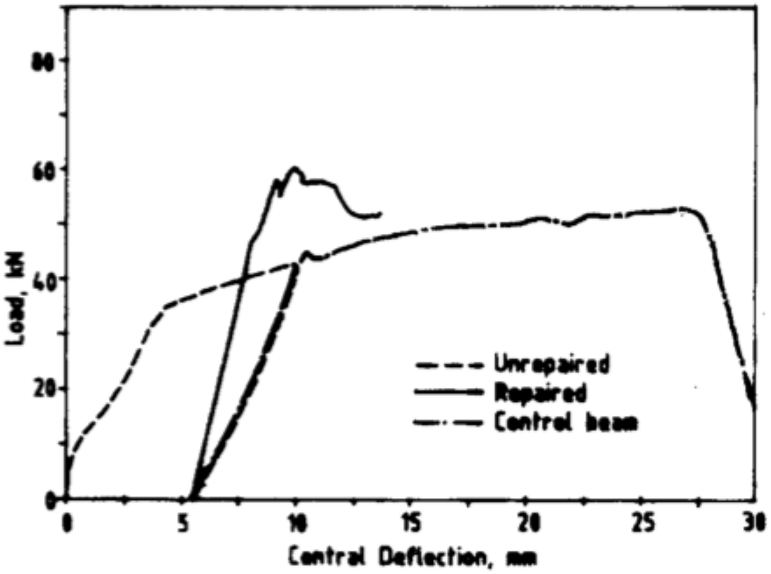


Figure 4. 5- Load-deflection curve of Beam FRB5, strengthened with 2.0-mm-thick steel plate without end anchorage (Hussain et al. 1995).

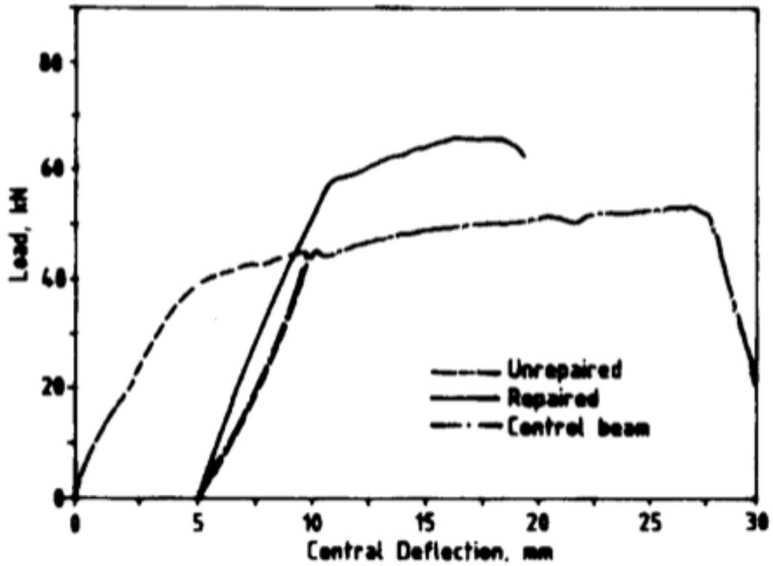


Figure 4. 6- Load-deflection curve of Beam FRB5, strengthened with 2.0-mm-thick steel plate with end anchorage (Hussain et al. 1995).

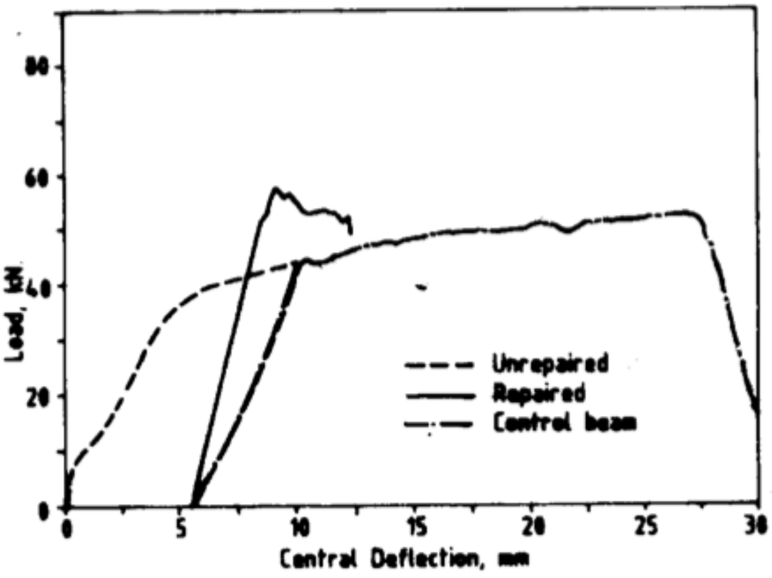


Figure 4. 7- Load-deflection curve of Beam FRB7, strengthened with 3.00-mm-thick steel plate without end anchorage (Hussain et al. 1995).

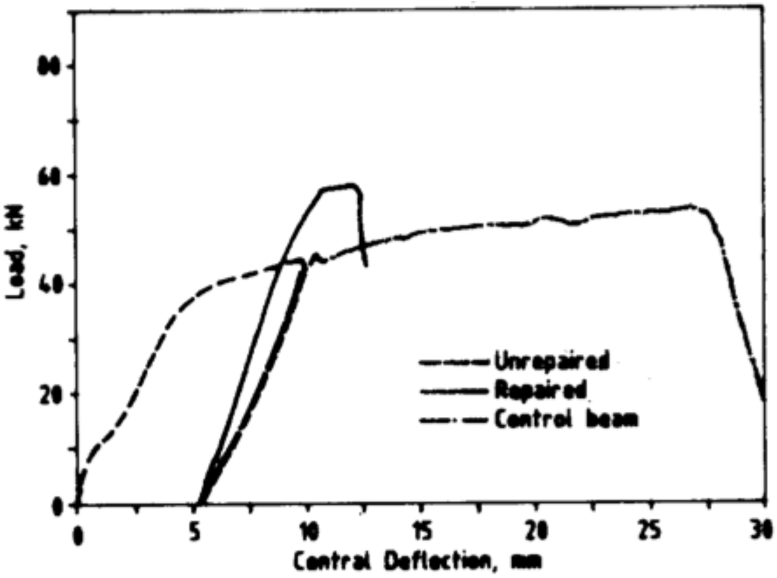


Figure 4. 8- Load-deflection curve of Beam FRB8, strengthened with 3.00-mm-thick steel plate with end anchorage (Hussain et al. 1995).

Barnes et al. (2001) compared the adhesive bonding and bolted external plate attachment techniques to increase shear capacity of beams. In both techniques steel plates are attached in the web areas of beams. Bolted plate attachment technique typically improves the connection between the steel plates and RC beam. Adhesive plate bonding is also a widely used strengthening technique that enhances the flexural capacity of beams by using advanced composites, such as GFRP and CFRP to bond the steel plates. Barnes et al. (2001) strengthened four beams with adhesively bonded steel plates, and three beams with bolted steel plates. Two beams were not strengthened and were treated as control specimens. Table 4.3 shows the properties, the strengthening details and the experimental results. The experiments show that when beams are carried high shear loading and anchorage is sufficient, the use of shear plates can improve serviceability and ultimate capacity of beams. The thin plates improve shear capacity very well, although heavy plates and additional strapping can significantly improve flexural capacity. The ultimate capacity of section depends on the bolting arrangement. The adhesively bonded plates highly control the surface crack but inadequate surface area can lead to sudden collapse and interface failure. The efficiency of bonded plates probably improves when the shear span and depth increase.

Table 4. 3- The properties of beam before and after test.

Beam Name	Shear span (*a*) Depth (*d*)	Plate Thick (mm)	Plate conn type	Conc strength N/mm^2	Thick ness (mm)	Yield strength (N/mm^2)	Ult capa (kN)	Yield Strength of steel (N/mm^2)
EP1.C	1.25	Control	-	60	2	248	765	340
EP1.2	1.25	2	Bolted	60	2	248	1412	340
EP1.4	1.25	4	Bolted	60	2	248	1884	340

EP1.6	1.25	6	Bolted	60	2	248	2001	340
EPG1.4	1.25	4	Bonded	67	4	239	1255	450
EPG1.4/2	1.25	4	Bonded	67	4	239	1373	450
EPG1.6	1.25	6	Bonded	67	4	239	1452	450
EPG1.C	0.78	Control	-	67	4	239	1422	450
EPG1.6/2	0.78	6	Bonded	67	4	239	1393	450

Adhikary and Mutusyoshi (2006) investigated shear strengthening of RC beams having internal shear reinforcement. In their experimental program, the five beams with dimensions 150mm(b)×200mm(h)× 2400mm(l) were tested until failure (Figure 4.9). The properties of reinforcement and epoxy adhesive details are provided in Table 4.4. Beam C-1 was kept as control beam while remaining four beams were strengthened with steel plates with different thicknesses. The steel plates were anchored to the beam sides with epoxy adhesive and anchor bolts. The test results (Table 4.5) indicate that increasing plate depth and thickness across the beam section can increase the ultimate shear strength. Figures.4.11 through Figure 4.13 show that, the displacement and strains in beam are reduced when steel plates are used. Use of thicker plates could not increase the strength proportionally. The best way to achieve the larger shear strength is to use deeper plates rather than thicker plates.

Table 4. 4- The properties of reinforcement and epoxy adhesive.

Epoxy Adhesive			Yield Strength L.S(MPa)	Yield Strength T.S(MPa)
T.S (MPa)	C.S (MPa)	S.S (MPa)		
49	72	15.6	391	346

Table 4. 5- Material properties and test results.

Specimen	Concrete strength (MPa)	Plate thickness (mm)	Plate depth (mm)	Yield strength (MPa)	Diagonal crack strength (kN)	Ultimate shear strength (kN)
C1	38.6	-	-	-	36.7	98.2
C2	42.5	2.3	100	378	63.7	116.4
C3	41.5	4.5	100	382	63.3	106.3
C4	37.6	6	100	398	63.7	126.4
C5	42	2.3	150	378	80.8	132.4

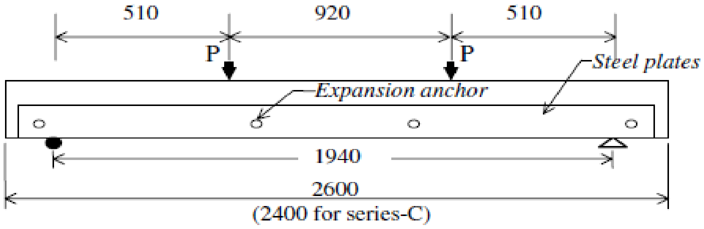


Figure 4. 9- Beams loading details

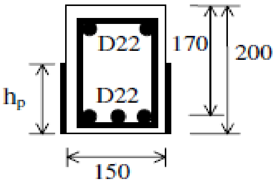


Figure 4. 10- Cross section of beams (Adhikary and Mutusyoshi 2006).

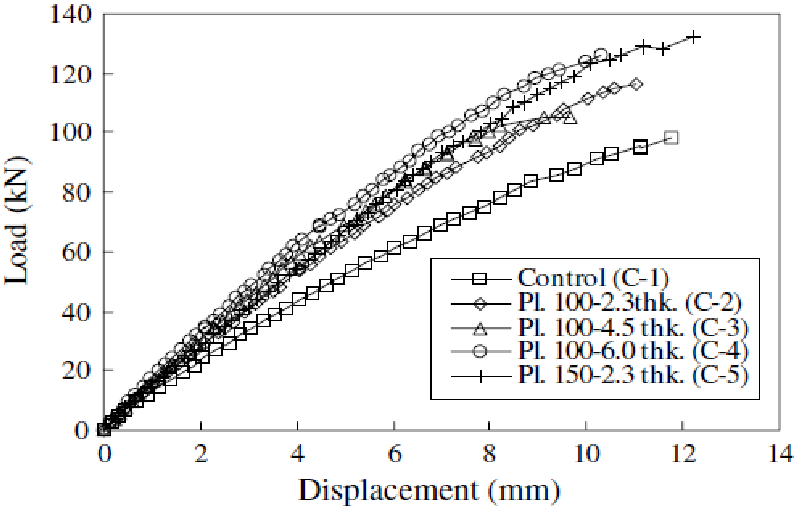


Figure 4. 11- Load versus mid-span displacement relationships for test beams (Adhikary and Mutusyoshi 2006).

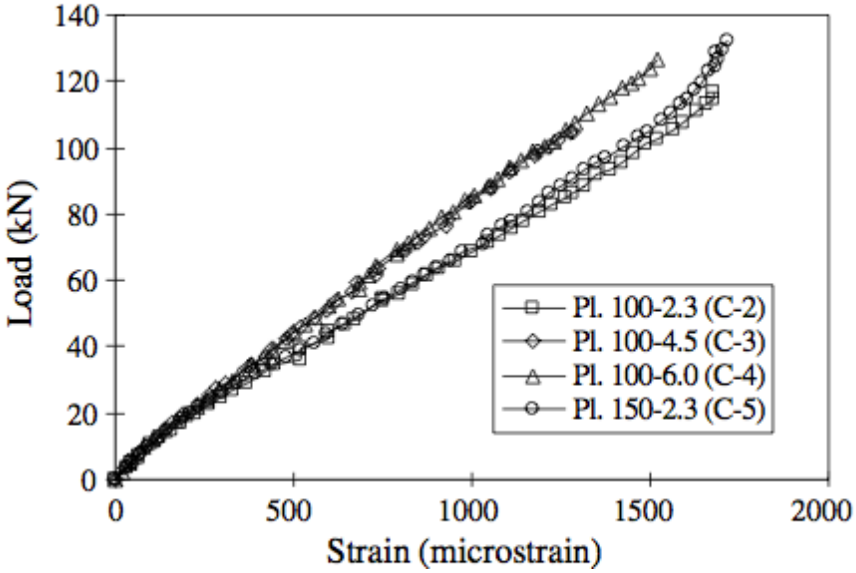


Figure 4. 12- Steel plate strain in mid-span of beam (horizontal) (Adhikary and Mutusyoshi 2006).

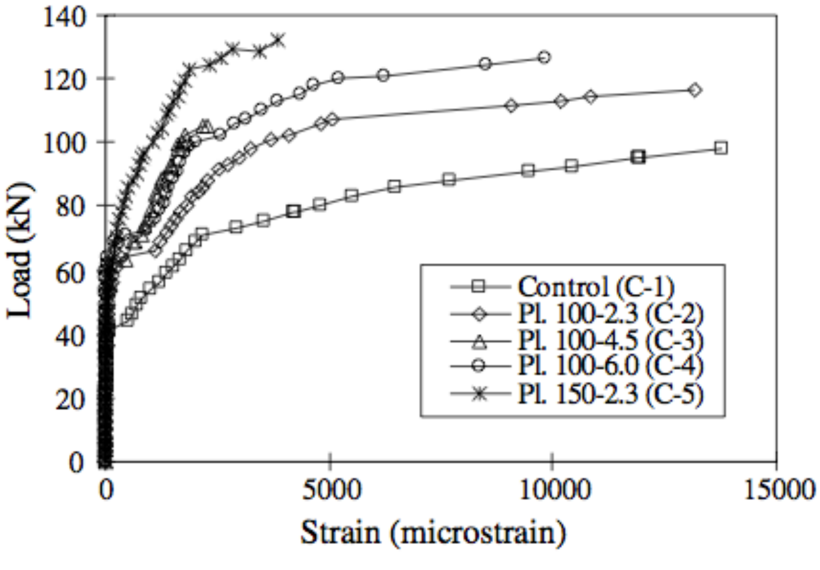


Figure 4. 13- Strains in internal shear reinforcement (Adhikary and Mutusyoshi 2006).

Su and Sui (2010) conducted four point bending tests for five simply supported RC and bolted side-plated (BSP) specimens. The beams have different bolt-plate arrangement to cover both under-reinforced and over-reinforced bolt side-plate conditions but same dimensions. Depth of strengthened strong plate specimens is 150 mm and the depth of strengthened weak plate is 75 mm (Figure 4.14). The strong and weak bolt arrangement is valued by degree of shear connection (*Pb/Fp,fi*) which is the ratio of the total strength of bolts on a shear span (*Pb)* and the plate force at ultimate state in full interaction analysis respectively (*Fp,fi*). The measured moment-deflection response of all specimens is shown in Figure 4.15. The test results imply that the strength of the bolts and plates greatly influences the two structural performance criteria: post-elastic strength enhancement and displacement ductility. The specimen strengthened by strong bolt arrangement and weak steel plate had sufficient strength enhancement and ductility. The beam strengthened by strong bolt arrangement and strong steel plate experienced brittle and undesirable failure. The amount of steel plates should be controlled, while sufficient bolts should be used to ensure the desirable ductile beam failure. Displacement ductility of 2.45 and post elastic enhancement of 1.17 gives impending warning prior to failure of beams for 'strong bolt weak plate'. The cost of strengthening arrangement for 'strong bolt weak plate' is also reduced since the depth of the plate is half the depth required for strong plate arrangement. However, lower depth of steel plate is not as effective as higher depth in enhancing the shear capacity of the beams.

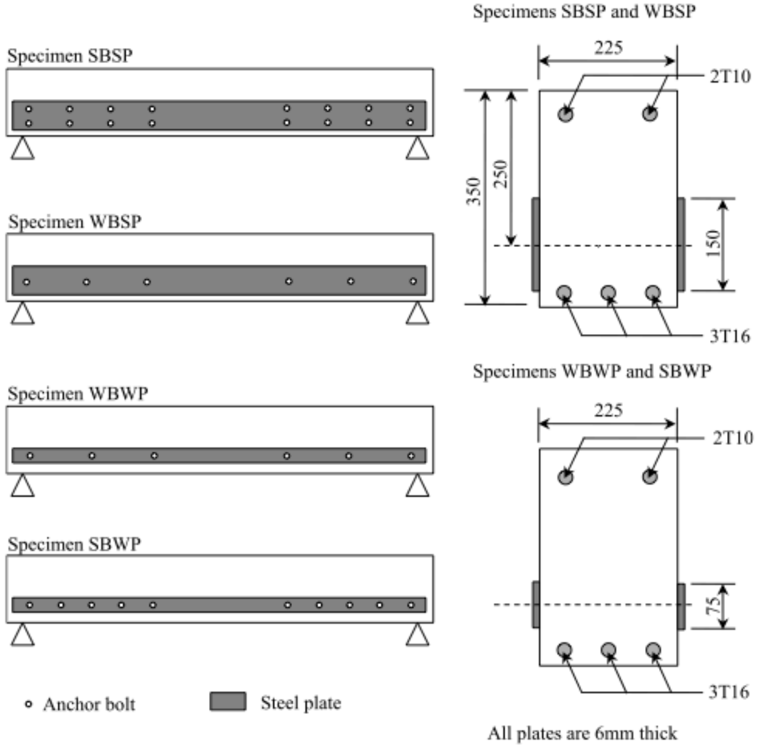


Figure 4. 14- RC and bolt-plate detail of BSP specimens (Su and Sui 2010).

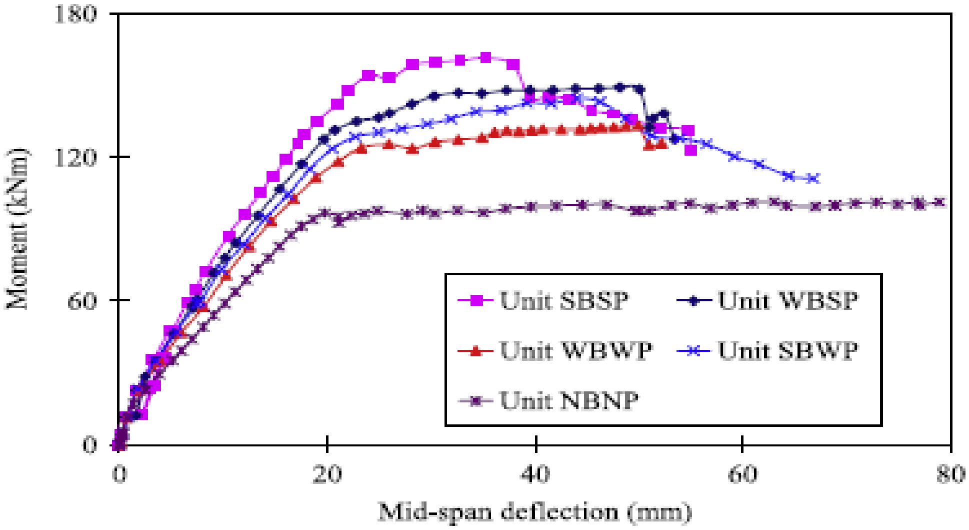
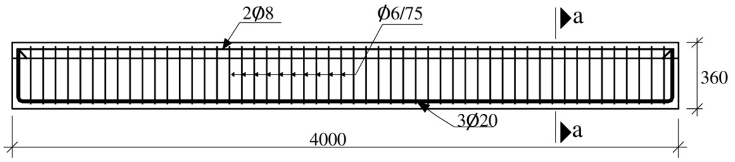
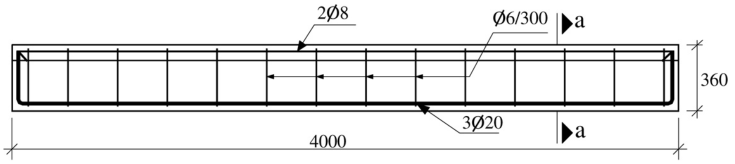


Figure 4. 15- Measured moment deflection response (Su and Sui 2010).

Altin et al. (2005) conducted an experiment on strengthening shear deficient beams with external web bonded steel plates. Altin et al. tested eleven beams. The dimensions, and reinforcement details of those beams are exhibited in Figure 4.16 and Table 4.6. Except for the control specimen Beam-1 which has greater shear strength than flexural strength, others did not have enough shear capacity. To strengthen shear deficient beams, steel straps and plates were bonded to the web of beams along the length of the shear span by using epoxy resin. All the steel plates had a thickness of 4 mm and the other geometric dimensions and arrangements of steel strap and plate are shown in Figures 4.17 and 4.18. Altin et al. tested all the beams under four-point loading. They loaded the beams to failure and then measured the mid-point deflection and shear cracks of the specimens (Table 4.7 and Figures 19 through 22). Experimental results show that the strength, stiffness and ductility of all the types of strengthened beams were improved. Strengthened beams had similar flexural capacity compared to control beam. Altin et al. (2009) found that the displacement ductility ratio has inverse proportionality to the spacing of the steel straps. Increasing the bonding area on the shear span led to a decrease in development and propagation of shear cracks. "L" type steel straps had the lowest ductility ratio among all of the specimens. Similarly to bonding steel plates, bonding the segmented steel plates to the shear span of beams also led to successful results in preventing shear cracks.



(a) Specimen Beam-1 (Control)



(b) Specimens Beam-2, 3, 4, 5, 6, 7, 8, 9, 10, Beam-11

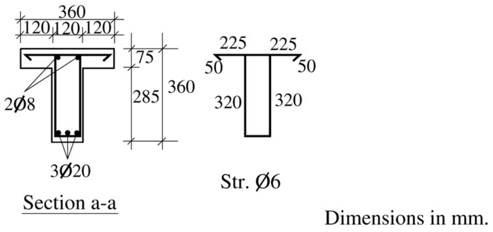


Figure 4. 16- Reinforcement details of specimens (Altin et al. 2005).

Table 4. 6-Specimen properties

Specimen #	*fc* (MPa)	Stirrups *ρw*	Ratio ρw/ρw Beam 1	Steel member used for strengthening		
				Dimensions	Type	Spacing (mm)
Beam-1	25.8	0.00224	1	-	-	-
Beam-2	27	0.00056	0.25	-	-	-
Beam-3	27.6	0.00056	0.25	40×285×40	Narrow steel strap	80
Beam-4	27.3	0.00056	0.25	40×405×40	Narrow L shape steel strap	80
Beam-5	26.5	0.00056	0.25	40×285×40	Narrow steel strap	40
Beam-6	26.5	0.00056	0.25	150×285×40	Wide steel strap	150
Beam-7	25.8	0.00056	0.25	150×405×40	Wide L shape steel strap	150
Beam-8	25.6	0.00056	0.25	150×285×40	Widee steel strap	75
Beam-9	26.7	0.00056	0.25	1550×285×40	Steel plate	-
Beam-10	26	0.00056	0.25	310×285×40	Wide steel strap	-
Beam-11	26.4	0.00056	0.25	1550×285×40	Steel plate with opening	-

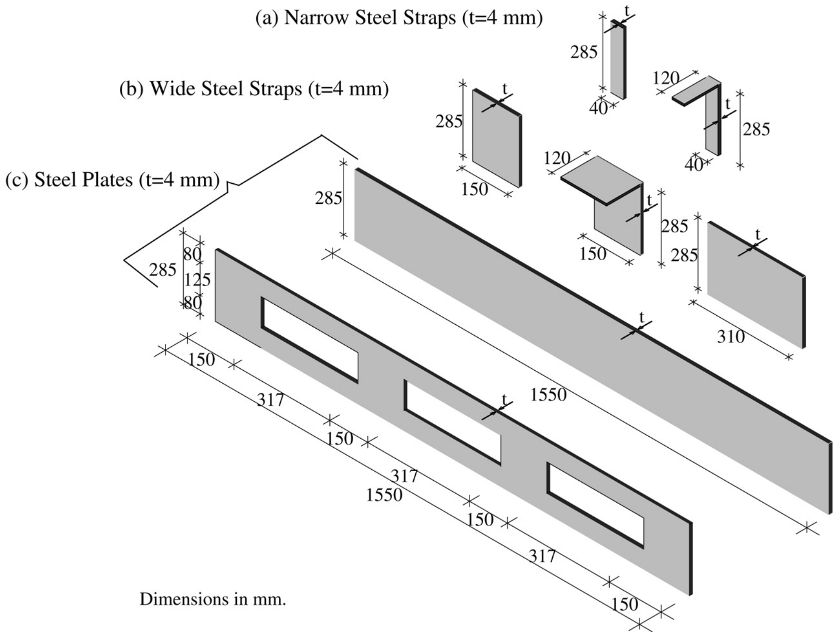
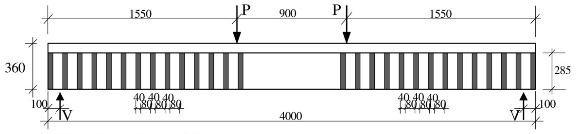
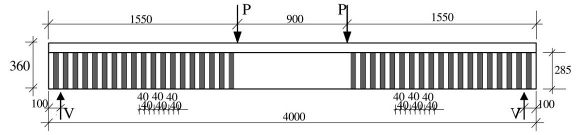


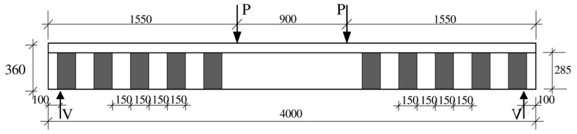
Figure 4. 17- Steel straps and plates used for strengthening (Altin et al. 2005).



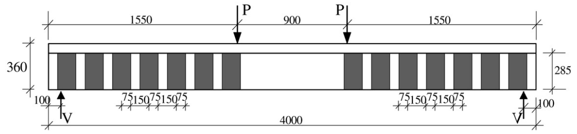
(a) Specimens Beam-3, Beam-4



(b) Specimen Beam-5

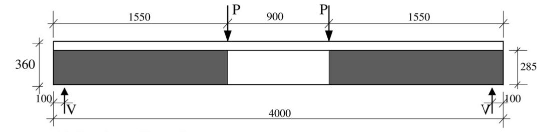


(c) Specimens Beam-6, Beam-7

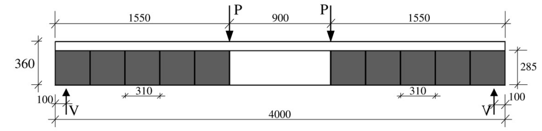


(d) Specimen Beam-8

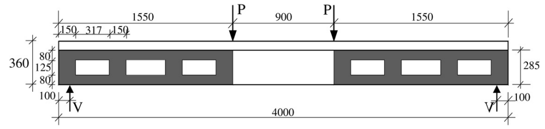
Dimensions in mm.



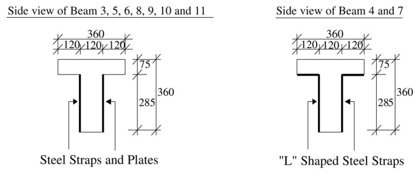
(e) Specimen Beam-9



(f) Specimen Beam-10



(g) Specimen Beam-11



Dimensions in mm.

Figure 4. 18- Steel strap and plate arrangement of strengthened specimens (Altin et al. 2005).

Table 4. 7-Test results.

Specimen #	Cracking Load (kN)		Yield load kN	Ultimate load kN	Yield disp. mm	Ultimate disp. mm	Stiffness at yield (kN/mm)	Ductility radio	Failure mode at ultimate
	Flexure	Shear							
Beam-1	13.4	36.0	81.0	90.4	23.5	84.9	3.45	3.61	Flexure
Beam-2	12.6	34.5	—	55.3	—	20.5	—	—	Shear
Beam-3	14.0	36.3	79.2	81.0	25.2	50.4	3.14	2.00	Shear
Beam-4	14.1	37.5	81.2	79.7	24.9	33.0	3.26	1.33	Shear
Beam-5	12.9	40.6	80.0	83.6	24.8	76.0	3.23	3.06	flexure
Beam-6	13.6	35.7	79.0	79.9	22.8	40.7	3.47	1.79	Shear
Beam-7	13.7	34.3	80.1	80.2	22.2	33.5	3.6	1.51	Shear
Beam-8	12.4	34.3	80.6	80.1	25.1	46.0	3.21	1.83	Shear
Beam-9	12.8	38.2	81.3	88.6	22.0	93.7	3.69	4.26	Flexure
Beam-10	12.2	38.0	81.0	87.5	20.7	88.0	3.91	4.25	Flexure
Beam-11	13.5	37.8	81.0	84.7	23.5	67.9	3.44	2.89	Shear

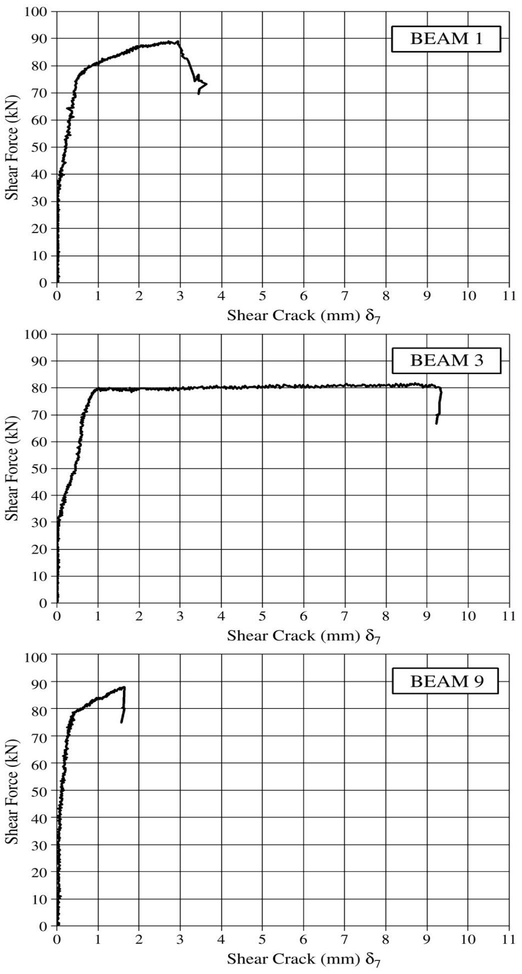


Figure 4. 19- Typical examples of crack width measurements (Altin et al. 2005).

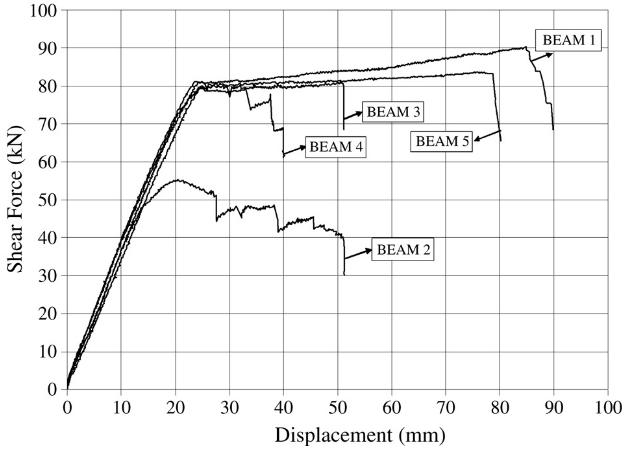


Figure 4. 20- Load-displacement curves of specimens strengthened with narrow steel straps (Altin et al. 2005).

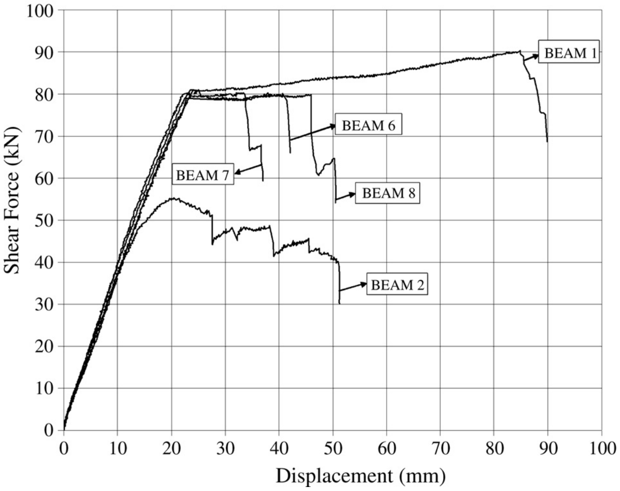


Figure 4. 21- Load-displacement curves of specimens strengthened with wide steel straps (Altin et al. 2005).

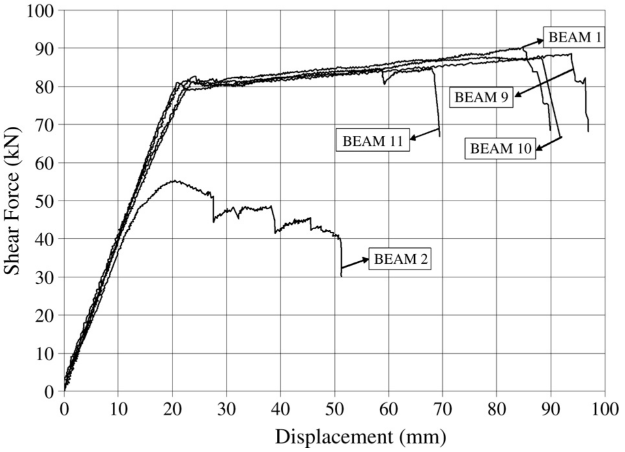


Figure 4. 22- Load-displacement curves of specimens strengthened with steel plates (Altin et al. 2005).

Oh et al. (2003) investigated the behavior of RC beams that were flexure-strengthened with steel plates under static and fatigue loads. A total 20 beams were tested. All but one was strengthened with steel plates before the test; while the other one, which was used as the control specimen, was not. The specimens had dimensions of 150 mm × 250 mm × 2400 mm as shown in Figure 4.23. After bonding steel plates to beams by injecting epoxy between steel plates and RC beams, 14 specimens were tested under static loads and the others were tested under fatigue loads. Oh et al. applied the static loads step-by-step up to 70 kN to the beams and then shifted to a displacement control method until they failed. The fatigue loads were 60%, 70%, and 80% of the static failure load of reference specimen S43 and the minimum load level was set to 10 kN. The main test of the experiments included thickness of plates, adhesive thickness and shear span to depth ratio (Table 4.8). Table 4.9 shows the test results including the separation loads, peak loads, displacements and strain at the peak loads, and the failure modes for tested beams. In Table 4.9, "PY" indicates plate yielding, "PS" means plate separation, "DT" represents diagonal tension failure and "SC" means shear compression failure. Test results indicate that the peak load is close to the separation load *Psep* for each beam, hence the separation of steel plates and beams are very risky for the strengthening method. Figure 4.24 exhibits the effects of plate thickness on the load-deflection curves and load-rebar strains. The figures indicate that increasing the thickness of plates effectively decreases the mid-span displacements, tensile rebar strains, and compressive rebar strains. Figure 4.25 shows the effects of adhesive thickness on the load-deflection curves and load-rebar strain. Compared to the unstrengthened control beam, strengthened beams have a much higher stiffness and peak load but slightly higher displacements at peak loads.

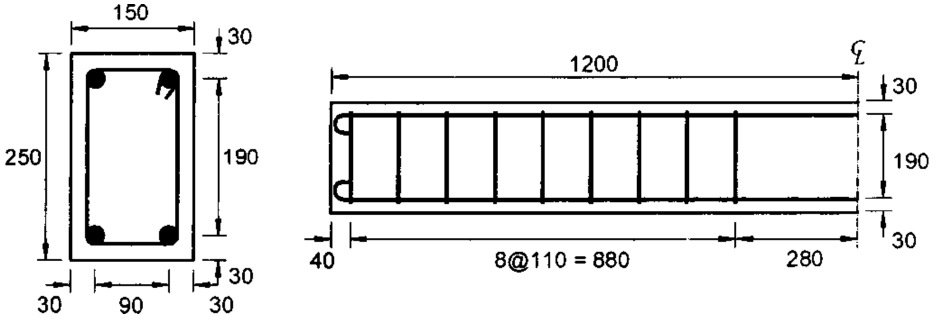


Figure 4. 23- Details of test specimen (unit: millimeter) (Oh et al. 2003).

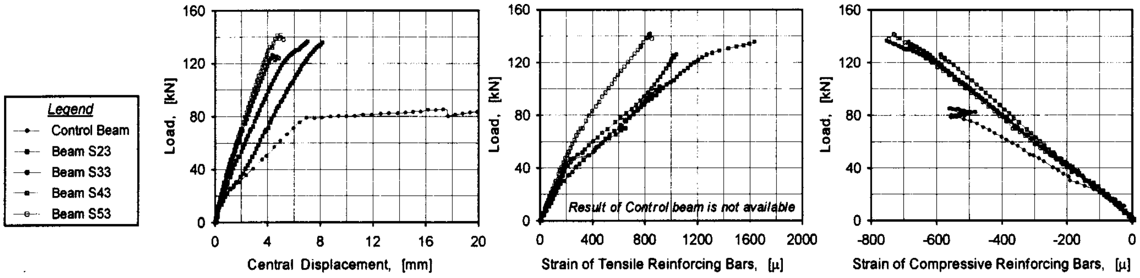
Table 4. 8- Test parameters and specimen identification

Beam identification	Test type	Plate thickness (mm)	Adhesive thickness (mm)	Shear-span-to-depth radio (*a/d)*
Control	Static	—	—	3.18
S23	Static	2	3	3.18
S33	Static	3	3	3.18
S43	Static	4	3	3.18
S53	Static	5	3	3.18
S41	Static	4	1	3.18
S43	Static	4	3	3.18
S45	Static	4	5	3.18
S47	Static	4	7	3.18
S43S1	Static	4	3	4.77
S4352	Static	4	3	4.09
S43	Static	4	3	3.18
S43S3	Static	4	3	2.27
S43S4	Static	4	3	1.36
Cont-F60	Fatigue	—	—	3.18
F60	Fatigue	4	3	3.18
F70	Fatigue	4	3	3.18
F80	Fatigue	4	3	3.18

Table 4. 9-Test Results for Steel Plate Beams

Beam identification	Spearation load *Psep* (kN)	Peak load *Pult* (kN)	Ratio to unstrengthened beam	*Pult*				
					Strain			
				displacement	steel plate	tensile rebar	stirrup	Failure mode

Control	-	89(79)	1	34.7 (7.2)	-	≥1800	691	Flexure
S23	131	136	1.53	8.15	≥1400	1633	1825	PY, PS, DT
S33	129	137	1.54	7.02	128	1484	1222	PY, PS, DT
S43	126	126	1.42	4.35	1079	1040	581	PS, DT
S53	132	142	1.6	5	1005	835	898	PS, DT
S41	120	125	1.4	4.68	1090	1220	1022	PS, DT
S43	126	126	1.42	4.35	1079	1040	581	PS, DT
S45	134	134	1.51	4.97	1172	1045	879	PS, DT
S47	140	150	1.69	5.35	1244	1207	1004	PS, DT
S43S1	129	132	1.48	5.94	1913	1642	594	PY, PS, DT
S4352	127	128	1.44	5.61	1427	1266	593	PS, DT
S43	126	126	1.42	4.35	1079	1040	581	PS, DT
S43S3	131	135	1.51	4.67	869	793	447	PS, DT
S43S4	214	221	2.48	5.13	691	689	1015	SC, PS

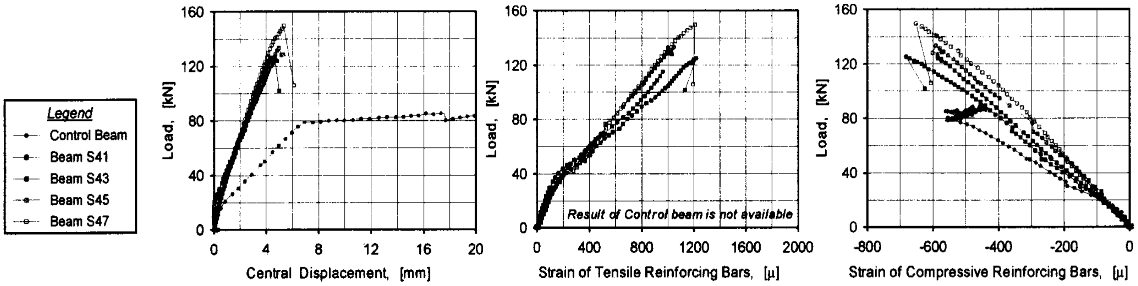


(a) Loads versus central displacement

(b) Loads versus tensile rebar strains

(c) Loads versus compressive rebar strains

Figure 4. 24- Effect of plate thickness on the load-deflection curves and load-rebar strain relations (Oh et al. 2003).



(a) Loads versus central displacements

(b) Loads versus tensile rebar strains

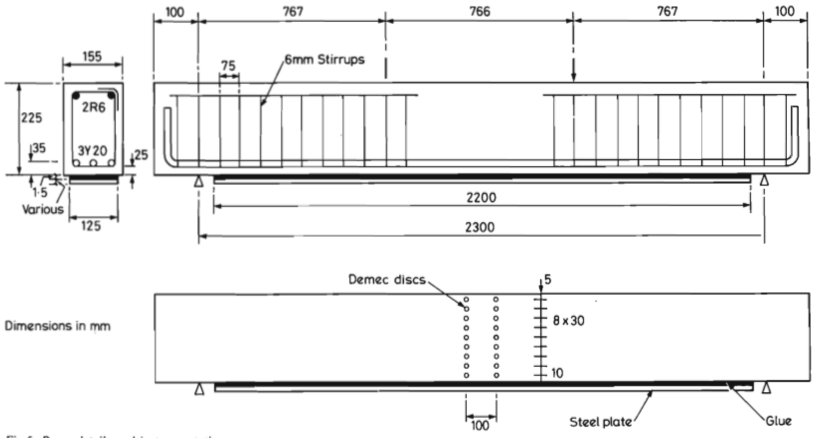
(c) Loads versus compressive rebar strains

Figure 4. 25- Effect of adhesive thickness on load-deflection curve and load-rebar strain relations (Oh et al. 2003).

Jones et al. (1988) focused on the problem of anchorage at the ends of steel plates glued to the tensile faces of reinforced concrete beams. To investigate the issue “sudden failure by steel separation”, the seven RC beams were strengthened by epoxy-bonded steel plates were tested with loading applied at the third points. All the beams had the same dimensions: 115 mm × 255 mm × 2500 mm. The strengthening details were provided in Figures 4.26 and 27. Details are summarized in Table 4.10. F31 was treated as base beam to compare with others since it failed suddenly by plate separation. Compared to beam F31, beam F32 were bonded with additional 3 mm thickness at the ends and failed at 14.3% higher load. The F33 was strengthened with tapered plate but the failure load only increased 4.9%. Beams F34 and F35 were similar to beams F31 and F32 but with anchored bolts. Their failure load increased 21.4% and 24.7% and the plate separations were prevented. Eventually the beams failed because the concrete crushed. Beams F36 and F37 strengthened using the most effective method with L-shaped anchor plates reached their theoretical ultimate loads. The load-deflection relationship is shown in Figure 4.28. This figure shows that the anchorage details won’t affect the deflection performance at service load. F31, F32, and F33 failed suddenly and did not show ductility. Beams F34 and F35 showed considerable ductility before failure. The response of beams F36 and F37 flatten more gradually but showed similar ductility compared to beams F34 and F35. The experimental results refer that anchorage arrangement effect on the ultimate strength and mode of failure. Using tapered and multiple systems increased the failure loads slightly. Using bolts did not prevent debonding, but prevented the separation and increased strength by 8%. After comparing the different anchor details, it can be concluded that the most effective method is using additional glued anchor plates.

Table 4. 10- Strengthening details and test results

Beam No.	Strengthening	Failure load (kN)	% over F31	% over unplated	Mode of failure
F31	1 no. 6mm plate	182	-	-13.3	Plate separation
F32	2 no. 3mm plates, curtailed	208	14.3	-1	Plate separation — inner plate
F33	1 no. 6mm plate tapered to 2mm	191	4.9	-9.1	Plate separation
F34	As F31 + bolts at end	221	21.4	5.2	Debonding followed by concrete crushing
F35	As F32 + bolts at end and curtailment	227	24.7	8.1	
F36	As F31 + one short and one long anchor plate	285	56.6	35.8	Plate yield and concrete crushing
F37	As F31 + short end and anchor plates	283	55.5	34.8	



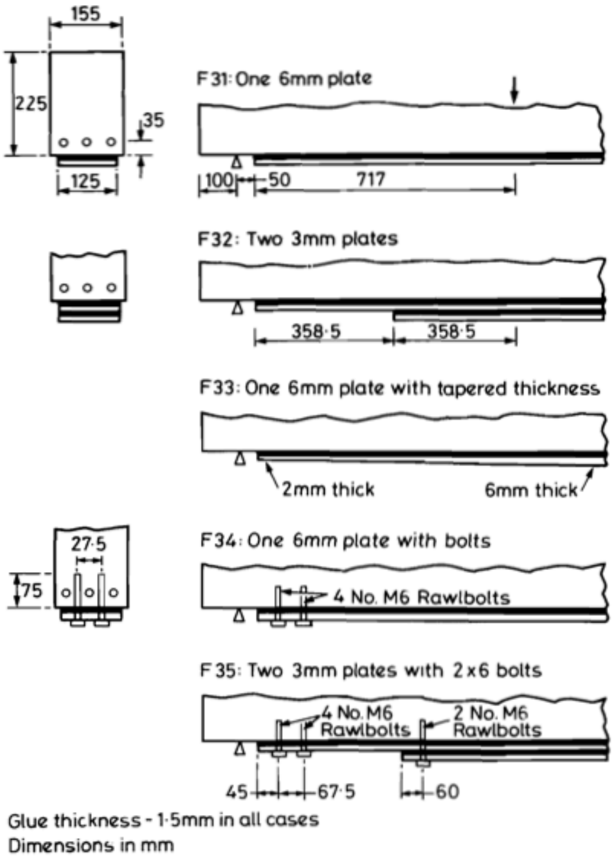


Figure 4. 26- Plating details: beams F31-F35 (Jones et al. 1988).

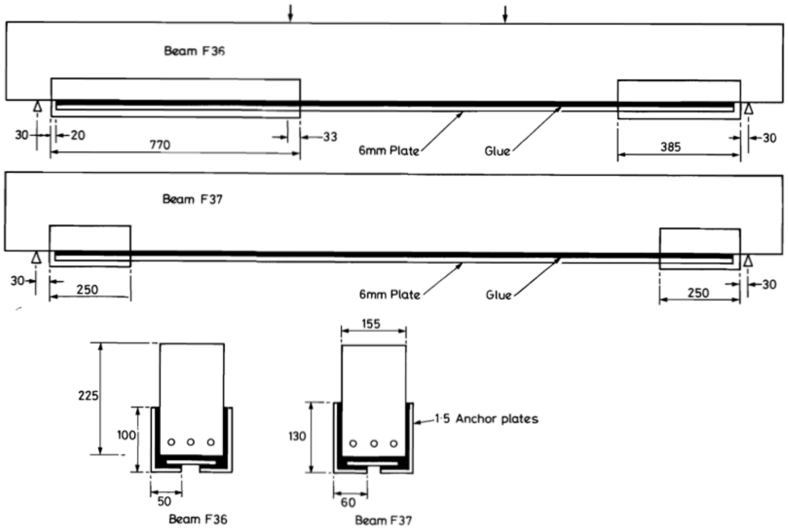


Figure 4. 27- Plating details: beams F31-35 (Jones et al. 1988).

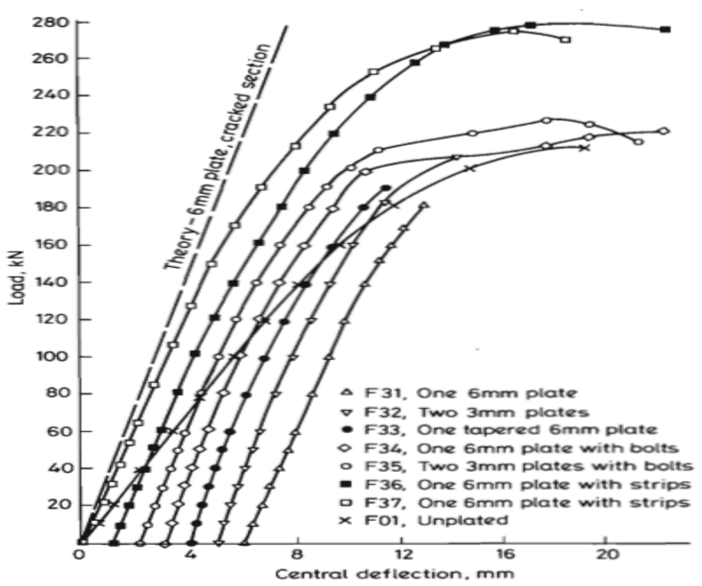


Figure 4. 28- Load-deflection curves (Jones et al. 1988).

Sirju and Sharma (2001) tested of reinforced concrete members that were strengthened with different methods under axial and bending compression. In the experiment, the control beams exhibited in previous chapter 2 Figure 2.7, and one beam was strengthened with steel plates fixed with adhesive and the other beam strengthened with steel plates fixed with bolts (Figure 4.29). The one concrete column was externally reinforced with 6mm×100mm×2300mm steel plates that were fixed by the adhesive under axial bending. Another beam strengthened with same dimension steel plates and were attached by 10 mm diameter × 60 mm long bolts. The beams were tested until failure. The test results and load deflection relationships, which are shown in Table 4.11 and Figure 2.9, reveals that compared to the unstrengthened control beam, strengthened beams have higher ultimate flexural strength and stiffness. The failure mode for strengthened beams was sudden and brittle.

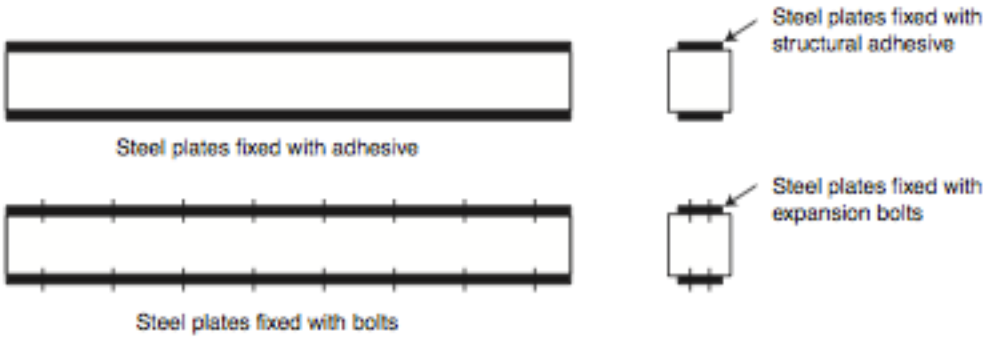


Figure 4. 29- Details of test beams (Sirju and Sharma 2001).

Table 4. 11- Test results and Comparison of flexural strength enhancement after strengthening

Strengthening technique	Moment at first cracks (kN m)	Increase (%)	Moment at failure (kN m)	Increase	Failure flexural load(kN)	Effective area of reinforcement based on BS 8110 Design Chart (%)
None (control)	12.87		18.02		48.05	1.13
Steel plates fixed with adhesive	36.78	185.8	40.46	124.5	107.9	5.1
Steel plates with expanding bolts	18.39	42.9	27.58	53.1	73.55	2.9

Adhikary [a] and Mutsuyoshi[b] (2006) compared effectiveness of various strengthening methods of RC beams in term of enhancing shear capacity. The two series of specimens were tested in flexural failure and shear failure (Figure 4.30). The experimental results indicated that although strengthening RC beam with epoxy bonding steel plates to the sides of beams can increase average 72% shear capacity compared to the control beam. The amount of increase is relatively low compared to the increase RC beams gained through other strengthening methods. However, the flexural strengthens of beams could not be increased (Figures 4.31 and 32).

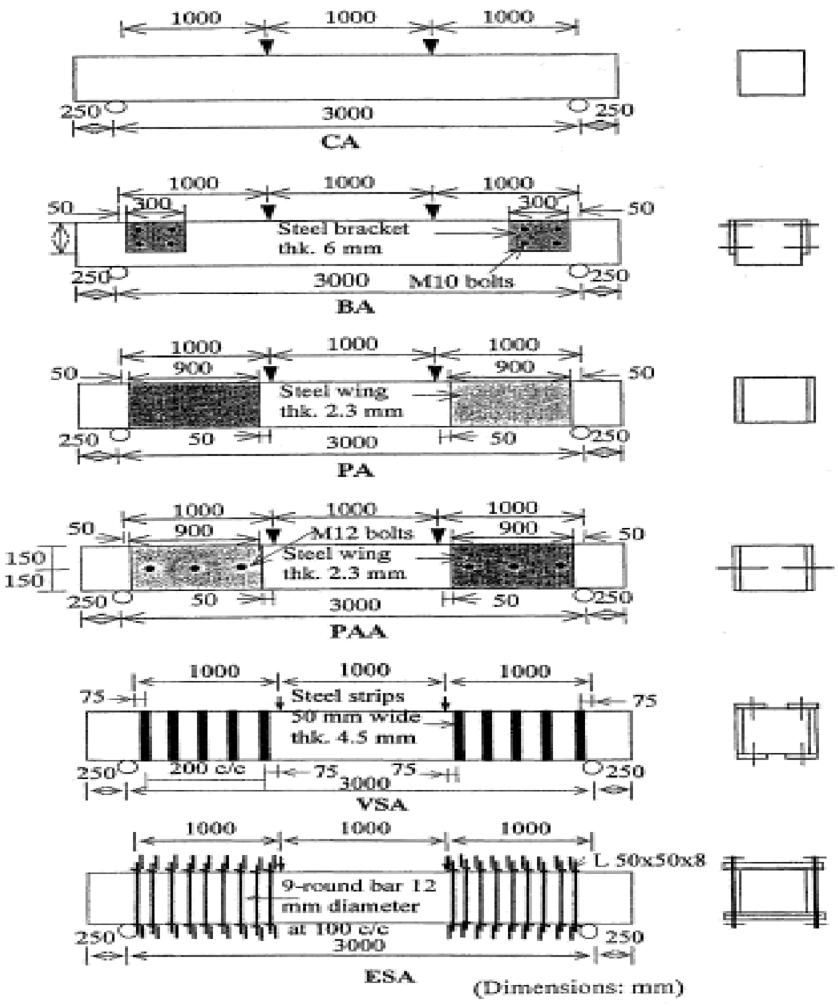


Figure 4. 30- (a) Different strengthening schemes for beams in series-A (Adhikary [a] and Mutsuyoshi[b] 2006).

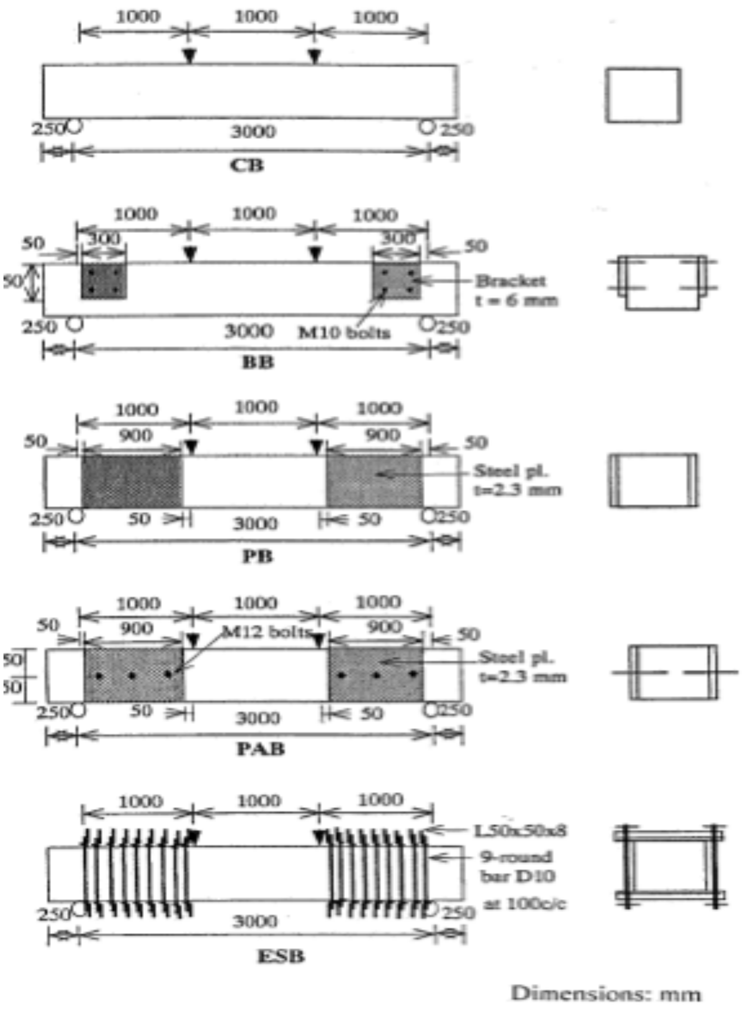


Figure 4. 31- (b) Different strengthening schemes for beams in series-B (Adhikary [a] and Mutsuyoshi[b] 2006).

Table 4. 12-Test results

	Specimen	Strengthing scheme	Yield strength (MPa)	Failure load (kN)	Mode
Series A	CA	Control Beam	-	187	shear
	PA	Steel Plates	378	279.3	flexure
	PAA	Steel Plates (anchors)	378	272.2	flexure
Series B	CB	Control Beam	-	233.6	shear
	PB	Steel Plates	320	405.5	shear

	PAB	Steel Plates (anchors)	320	400.6	shear

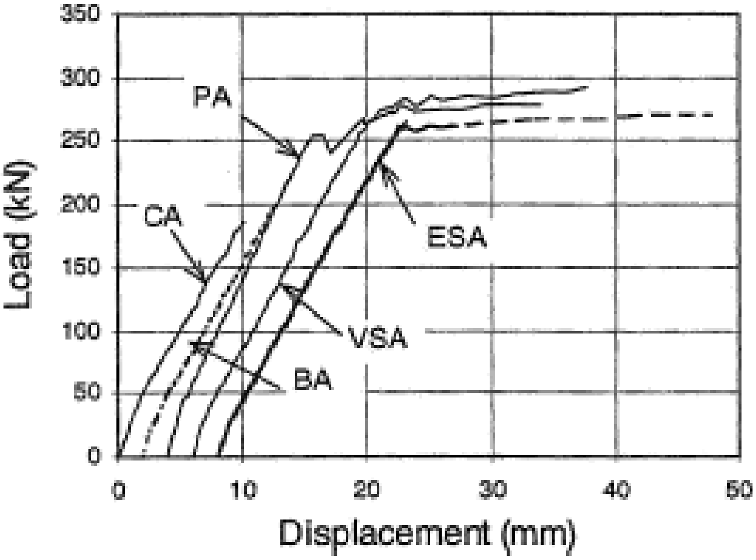


Figure 4. 32- Load versus mid-span displacement relationships for beams in series (Adhikary [a] and Mutsuyoshi[b] 2006)

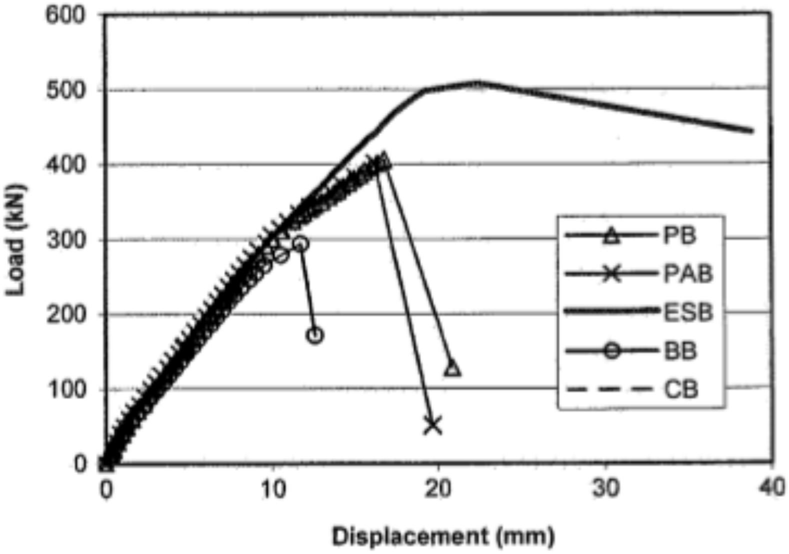


Figure 4. 33- Load versus mid-span displacement relationships for beams in series B (Adhikary [a] and Mutsuyoshi[b] 2006).

Aykac et al. investigated the influences to flexural behavior, ductility of externally plated reinforced concrete that are led by strengthened plate properties. For example, the thickness of the soffit plate, anchorage of the soffit plate to the beam, and the presence of perforations in the soffit plate and the height of side plates of collars. A total 13 full-scale rectangular reinforced concrete beams with different strengthening details were tested under two-point monotonic loading until failure. The beams strengthening details were exhibited in Figure 4.34 and Table 4.13 where “S” means that bottom plate is solid, “P” represents the bottom plate being perforated, “R” indicates that the beam was obtained by repairing a previously tested beam, “U” refers to when the soffit plate was anchored and “B” shows that the bottom plate is only epoxy-bonded to the beam. Figure 4.35 shows an example of perforated beams. The deflections of beams at mid-span points were listed in Figure 4.36. Table 4.14 shows the ultimate loads and failure modes of specimens. The test results indicated that strengthening beams with perforated steel plates instead of solid plates improved the ductility but decreased the ultimate strength of beams. The bolt anchorage in the thick steel plates is more effective compared to thin plates.

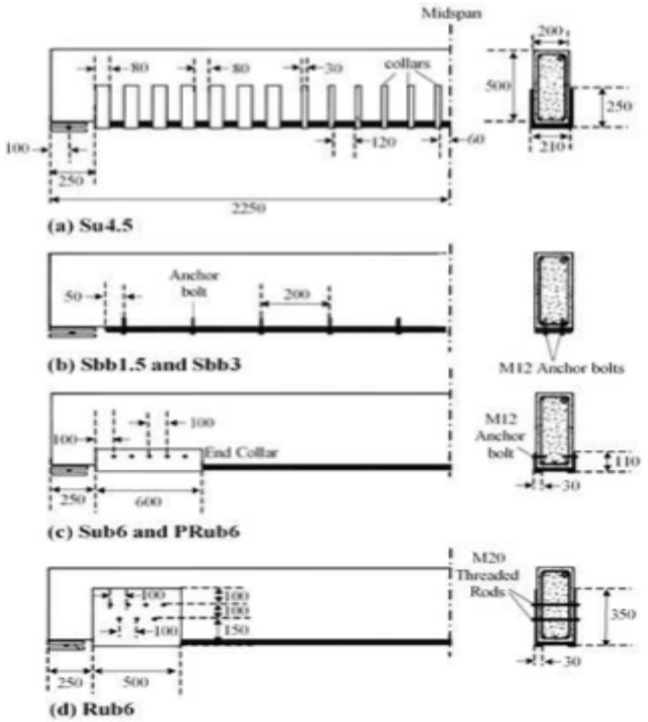


Figure 4. 34- Specimens details (Aykac et al. 2012).

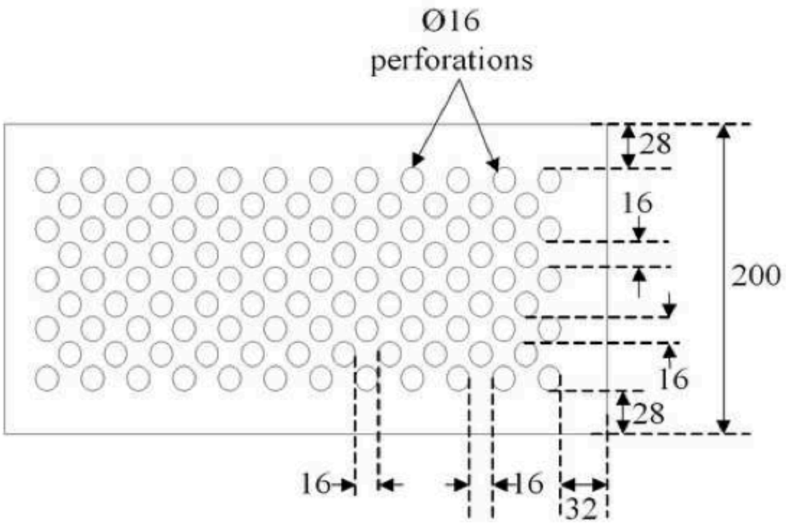


Figure 4. 35- Perforation pattern of the perforated plates (Aykac et al. 2012).

Table 4. 13- Properties of test specimens

Beam	Intervention	Soffit Plate& Thickness (mm)	Additional Spffit Plate Anchorage	Side Plate Depth of Collar
BS	Basic	-	-	-
Sb1.5	Strengthening	Solid-1.5	-	-
Sb3	Strengthening	Solid-3.0	-	-
Sb3.6	Strengthening	Solid-3.6	-	-
Sb4.5	Strengthening	Solid-4.5	-	-
Su4.5	Strengthening	Solid-4.5	Collars	250
Sb6	Strengthening	Solid-6.0	-	-
Sbb 1.5	Strengthening	Solid-1.5	Bolts	-
Sbb3	Strengthening	Solid-3.0	Bolts	-
Sub6	Strengthening	Solid-6.0	End Collars	110
Rub6	Repair	Solid-6.0	End Collars	350
PRb6	Repair	Perfor.-6.0	-	-
PRub6	Repair	Perfor.-6.0	End Collars	110

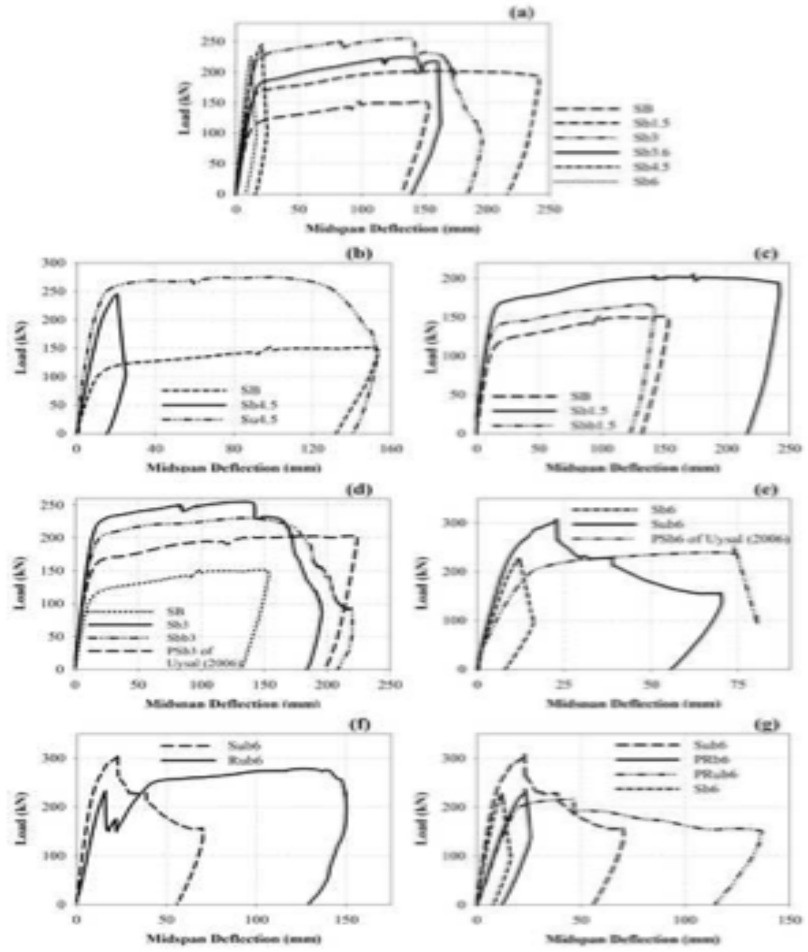


Fig. 8 – Load-Deflection Curves of the Specimens

Figure 4. 36- Load versus deflection curve of specimens (Aykac et al. 2012).

Table 4. 14 - Ultimate loads and failure modes of the specimens.

Beam	Ultimate load (kN)			Pex/Pan	Pex/P_{ACI}	Failure mode	Modulus of thougness		Deformation ductility index (*δu/δy*)	
	Test, *Pex*	Todeschini, *Pan*	ACI 318, *Paci*				Absolute	Relative	Absolute	Relative
BS	152	137	130	1.11	1.17	flexure	20994	1	16.8	1
Sb1.5	205	177	174	1.16	1.18	flexure	46602	2.22	18.6	1.1
Sb3	258	218	217	1.19	1.19	flexure	43728	2.08	9.4	0.56
Sb3.6	228	233	234	0.98	0.98	flexure	32240	1.54	9	0.54
Sb4.5	245	256	258	0.96	0.95	shear	3952	0.19	-	-

						peeling				
Su4.5	282	256	258	1.1	1.09	flexure	39200	1.87	10.4	0.62
Sb6	225	292	297	0.77	0.76	shear peeling	2290	0.11	-	-
Sbb 1.5	170	177	174	0.96	0.98	flexure	28248	1.35	13.5	0.8
Sbb3	225	218	217	1.04	1.04	flexure	44832	2.14	11.6	0.69
Sub6	304	292	297	1.04	1.02	shear peeling	15097	0.72	-	-
Rub6	275	292	297	0.94	0.93	flexure	36961	1.76	6.5	0.39
PRb6	229	233	234	0.98	0.98	shear peeling	3796	0.18	-	-
PRub6	216	233	234	0.93	0.92	flexure	23395	1.11	5.9	0.35

4. 3 Summary and Conclusions

Attaching external steel plates in different areas of reinforced concrete beams can certainly improve flexure and shear capacity of RC beams. Bolting or bonding plate to the certain external surface of the beams could effectively strengthen beams. The researchers focus on the different specific factors like bolt arrangement, thickness and depth of the steel plate, attachment method; which can influence the performance of steel plate. The obvious advantage of using this strengthening method is that it needs relatively short installation time and the steel plates do not disrupt operations compared to concrete jacketing. The disadvantages include debonding, expensive, temporary weakening, and corrosions.

CHAPTER 5 Unbounded External Strengthening

5. 1Introduction

Unbounded-type strengthening techniques include attachment of the externally unbounded steel units such as wire rope, steel clamping, post-tension units. It is an economical, environmentally friendly, and efficient strengthening method.

5. 2 Description of previous studies

Wire ropes are lightweight, high-strength and have high flexibility. They are used as external shear reinforcement in concrete beams. Kim et al. (2007) explored the significance and shortcomings of using wire rope techniques strengthening concrete beams. Kim tested 15 reinforced concrete beams to failure in shear then strengthened them with wire rope units and retested them. The specimens have various shear span-to depth ratio, pre-stressing force, orientation and spacing of wire rope units. The properties of specimens and properties of retrofit are shown in Figure 5.1 and Table 5.1. The results of experiment are shown in Figure 5.3s and 5.4. The investigation indicates that using wire units to strengthen beams that failed due to shear stress could control the crack distribution. The diagonal cracking loads increased after strengthening beams with wire units. Use of wire units strengthening beams will increase ultimate shear strength of beams by 20%-70% compared to un-strengthened damaged beams.

Table 5. 1- The properties of specimens tested by Kim et al. (2007).

Beam no.	f_c' (MPa)	*a/d*	*a* (mm)	*L* (mm)	config	space of wire rope s_w mm	Init ten force *Fi* (kN)	Init orque *T* (N m)	f_i/f_{pu}
1	24.5	1.5	540	1140	only repair	-	-	-	-
2	24.5				vertical	150	46.4	35	0.53
3	22.5				45° incline				

4	24.7	2.5	900	2160	only repair	-	-		-
5	24.7				vertical	150	33.2	35	0.38
6	24.5				45° incline				
7	24.1				vertical		46.4		0.53
8	22.5				45° incline			45	
9	22.5				vertical		60		0.69
10	20.6				45° incline				
11	24.7	3.25	1170	2700	only repair	-	-	-	-
12	24.5				vertical	150	46.4	35	0.53
13	22.5				45° incline				
14	24.8				vertical	100		26	0.4
15	20.6				vertical	200		52	0.8

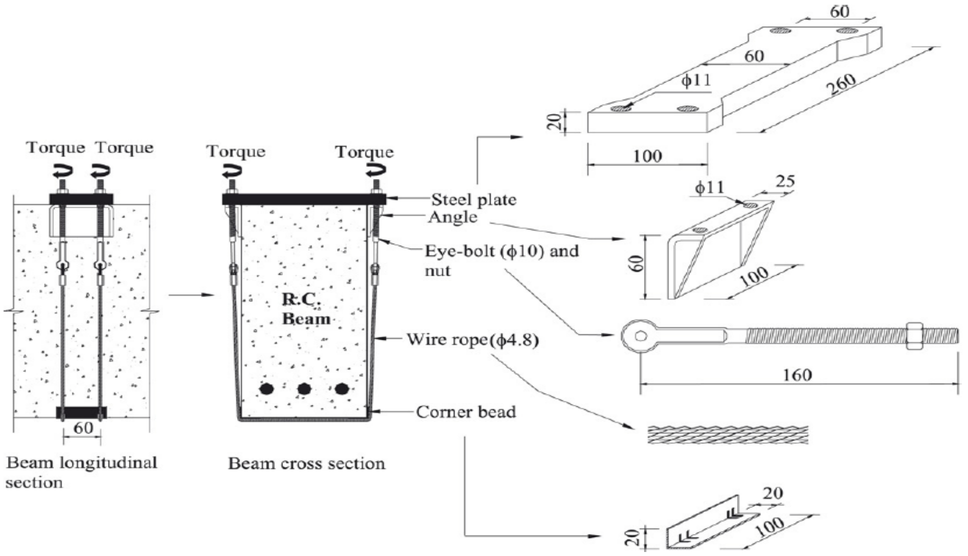
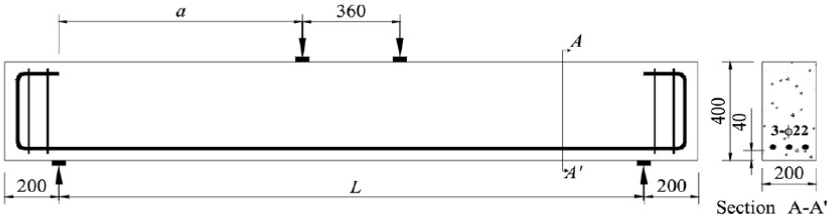
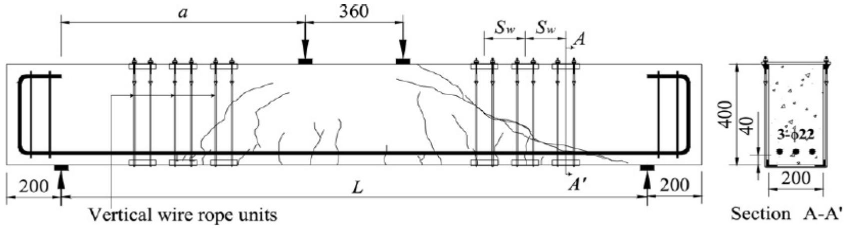


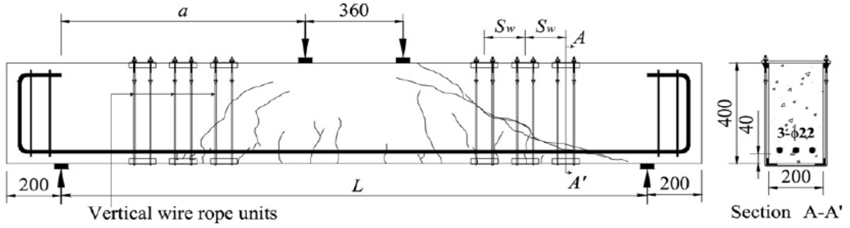
Figure 5. 1- Details of wire rope units and strengthening procedure (Kim et al. 2007).



(a) Original beams (for the first test).



(b) Vertically strengthened beams (for the second test).



(b) Vertically strengthened beams (for the second test).

Figure 5. 2- Specimen details and arrangement of wire rope units (Kim et al. 2007).

Table 5. 2-The mechanical properties

Type	f_y(MPa)	ε_y	f_{su} (MPa)	E_s (GPa)
Reinforcement (22mm)	445	0.00244	620	182
Steel plate	307	0.00157	448	195
Eye Bolt (10mm)	355	0.00187	465	190
Wire rope (4.8mm)	-	-	2145	120

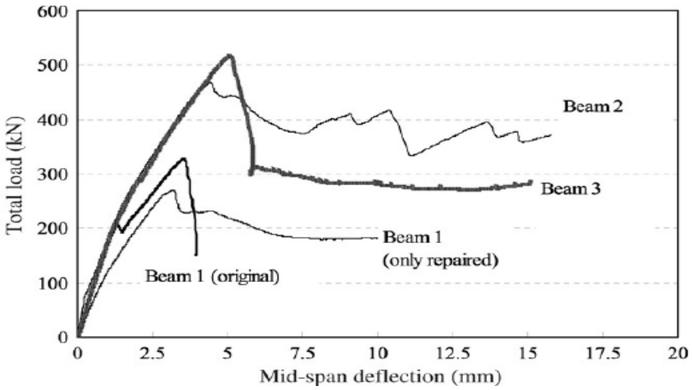


Figure 5. 3- Total Load versus mid-span deflection *a/h*=1.5 (Kim et al. 2007).

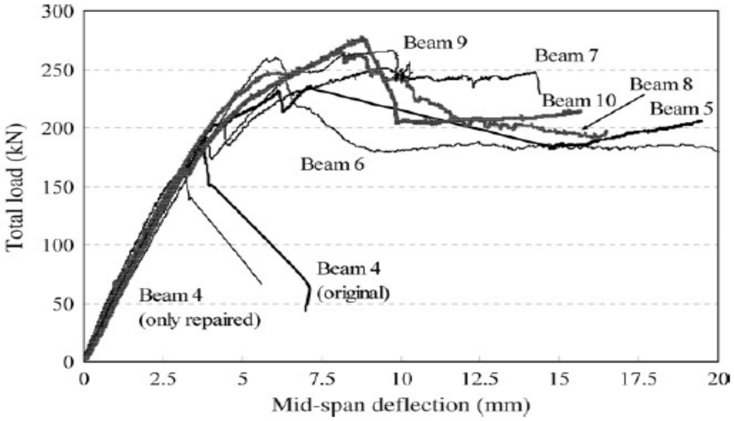


Figure 5. 4- Total load versus mid-span deflection *a/h*=2.5 (Kim et al. 2007).

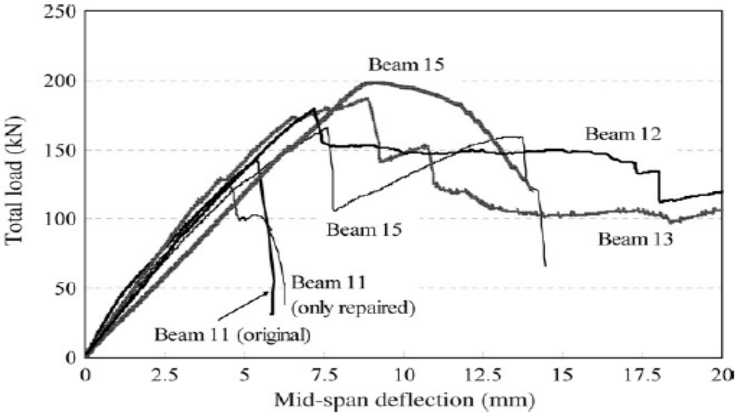
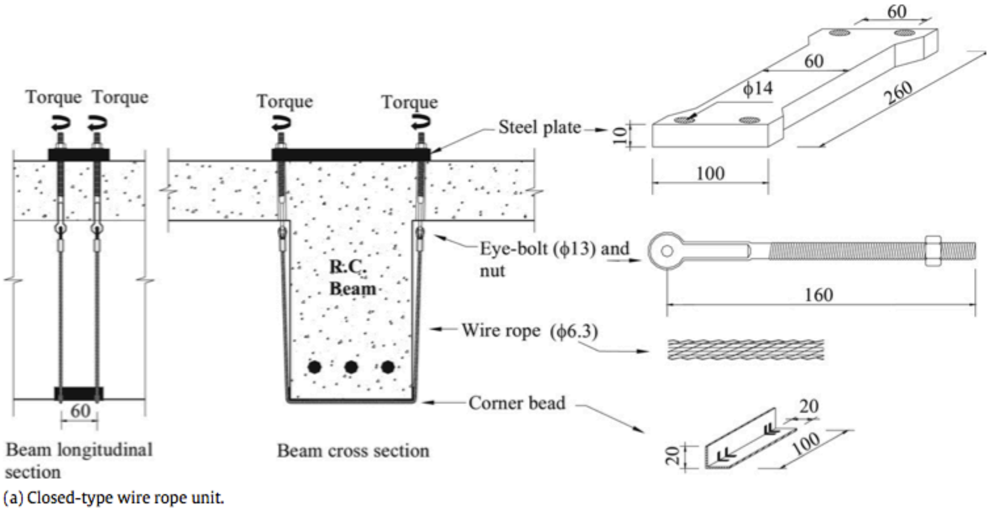
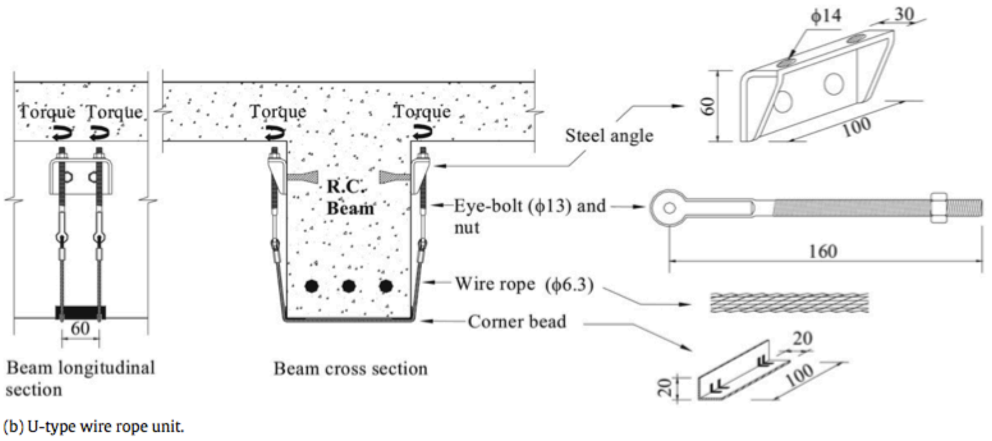


Figure 5. 5- Total load versus mid-span deflection *a/h*=3.25 (Kim et al. 2007).

Yang et al. (2009) also explored the significance and shortcomings of unbounded wire rope unit technique for continuous reinforced concrete (RC) T-beams. Ten two-span RC T-beams strengthened by U-type and closed-type wire rope units and one control unstrengthened specimen were tested until failure. Table 5.3 shows the properties of units, where s_w means spacing of wire rope units, *Ni* is the total initial prestressing force in a unit and *Ti* represents initial torque. Figures 5.6 and 5.7 and Table 5.3 give the details of these two types of developed wire rope units. The type of wire units rather than the amount of wire ropes influenced the cracks propagation of specimens. All the specimens had significant diagonal crack within the interior shear span except that the specimens with closed type wire rope units exhibited more ductile failure than others. The test results after strengthening are shown in Table 5.4 where M_{fl} indicates the nominal moment capacity of beam section obtained from ACI 318-05, subscripts *N* and *P* represent the hogging and sagging zone, subscripts *I* and *E* identify the interior and exterior shear spans respectively. The test results indicate the specimens with closed type rope units have higher load and shear capacities compared to specimens with U-type wire ropes when they have the same amount of wires. No matter what kind of wire rope units the beams are strengthened with, the diagonal shear cracks will decrease when the amount and initial prestressing force of wire ropes increase.

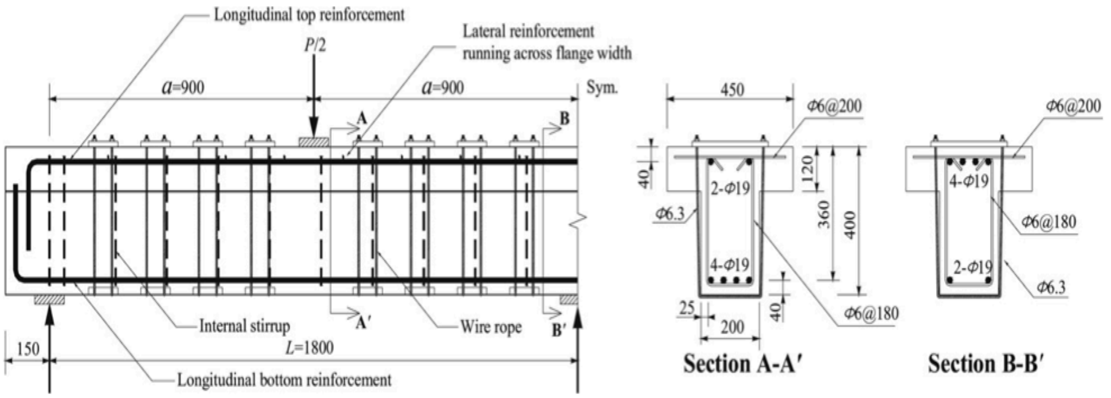


a) Closed-type wire rope unit.

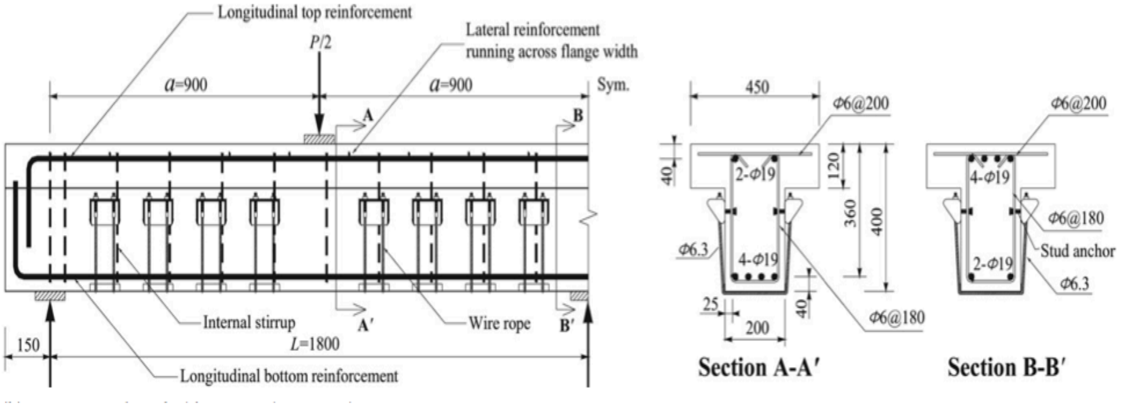


b) U-type wire rope unit.

Figure 5. 6- Details of wire rope units and strengthening procedure (Yang et al. 2009).



a) **Beams strengthened with closed-type wire rope units.**



f_i/f_{wu}

b) Beams strengthened with U-type wire rope unit

Figure 5. 7- Specimen details and arrangement of reinforcement and wire rope units (Yang et al. 2009)

Table 5. 3- Wire rope unit properties (Yang et al. 2009).

Specimen	fc' (MPa)	Details of wire rope unit						
		Type	ρ_w	ρ_w/ρ_{min}	s_w(mm)	f_i/f_{pu}	Ni (kN)	Ti (N*m)
N	26.8	N/A						
C2.0-0.6	25.9	Closed type	0.0017	2	223	0.6	78.8	76.8
C2.5-0.6	25.9		0.0021	2.5	178			
C3.5-0.6	26.4		0.0029	3	127			
C4.5-0.6	26.4		0.0038	4.5	100			
C2.5-0.45	25		0.0021	2.5	178	0.45	59.1	57.6
C2.5-0.75	26.4		0.0021	2.5	178	0.75	98.5	96
U2.5-0.6	26.7	U type	0.0021	2.5	178	0.6	78.8	76.8
U3.5-0.6	26.7		0.0029	3.5	127			
U4.5-0.6	26.7		0.0038	4.5	100			
U2.5-0.75	26.7		0.0021	2.5	178	0.75	98.5	96

Table 5. 4- Test results after strengthening (Yang et al. 2009).

Specimen	Initial Flexural Cracking Load Pft (kN)		Diagonal cracking load(pcr) and shear force(Vcr) (kN)				Load capacity (Pn) and Corresponding shear force (Vn) at failed span (kN)			Ultimate moment (Mn) (kNm)		$\frac{(M_n)_N}{(M_{fl})_N}$	$\frac{(M_n)_p}{(M_{fl})_p}$
			Interior		Exterior								
	$(P_{fl})_N$	$(P_{fl})_P$	$(P_{cr})_I$	$(V_{cr})_I$	$(P_{cr})_E$	$(V_{cr})_E$	P_n	$(V_n)_I$	$(V_n)_E$	$(M_n)_N$	$(M_n)_P$		
N	166.6	215	390.3	126.1	547.9	97.9	580.1	182	108	66.6	97.2	0.37	0.52
C2.0-0.6	178.2	197.2	411.5	131.6	555.9	103.3	707.2	214.9	138.5	68.9	124.7	0.39	0.67

C2.5-0.6	170.6	210.3	417.4	133.1	564.7	104.1	799.9	239.7	159.6	72.7	143.6	0.41	0.77
C3.5-0.6	176.9	218.9	431.6	137.8	565.8	103.7	966.5	308.2	174.9	120.1	157.4	0.67	0.84
C4.5-0.6	187.7	222.5	437.6	141.3	608.2	111.2	1063.7	341.6	190.5	135.8	171.5	0.76	0.92
C2.5-0.45	160.2	212.8	424.5	130.9	564.7	104.7	756.8	218.8	160.2	52.2	144.2	0.29	0.78
C2.5-0.75	181.6	220.2	418.6	135.7	575.9	103.4	868.5	269.3	165.8	92.4	149.2	0.52	0.80
U2.5-0.6	176.8	221.9	398.2	125.8	530.0	98.8	622.8	191	120.9	62.6	108.8	0.35	0.58
U3.5-0.6	172.5	215.5	394.8	124.5	546.6	101.1	617.6	191	118.1	65.3	106.3	0.37	0.57
U4.5-0.6	185.3	226.0	411.3	130.9	532.6	100.5	647.9	193	130.2	57.2	117.2	0.32	0.63
U2.5-0.75	184.0	217.5	410.2	131.4	589.3	109.4	689.5	208	136.4	64.8	122.8	0.36	0.66

Attaching external clamps to the beams is another unbonded-type strengthening method to solve the problem when reinforced concrete beams have sufficient moment capacities but insufficient shear strength. The external clamps are made by symmetrically bending the steel strap to improve ductility of the beams and by wring them together around the beams. Altin et al. (2003) investigated the effects of attaching clamps to beams under flexure. In total, 13 specimens were divided into two groups in term of their different ratio of the shear span to the effective height of specimens (*M/Vd* = *a/d*). One group had a shear span/effective depth ratio of 4.5 (*M/Vd* = *a/d*=4.5) and the other group had 3.3(*M/Vd* = 3.3). The specimens had 3900 mm or 3000 mm span and same sectional geometry and longitudinal reinforcement (Table 5.5 and Figure 5.8.). All the beams had three ϕ20 mm longitudinal tension reinforcement $f_{sy} = 517.3$ MPa, ϕ6 and ϕ8 mm reinforcement with $f_{sy} = 328.5$ MPa and $f_{sy} = 299.8$ MPa. The specimens were damaged by shear strength and then repaired by the clamps that are shown in Table 5.6 and Figure 5.10. The repairing details are given in Figure 5.9 and Table 5.6. All specimens were tested by four-point loading test until they failed. Load-deflection curves are presented in Figures 5.11 through 5.14. The results reveal that using clamp to strengthen the specimens can improve the ductility and rigidity. Clamps can control the size of the cracks and force the flexural behavior to govern in the section.

Table 5. 5-The properties and experiment results of specimens.

Specimen	Application Type	Concrete Strength *fc'* (MPa)	Failure Load (kN)	Yield Disp δ_y(mm)	Failure Disp δ_u(mm)	Ductility ratio $\delta u/\delta y$
S500	Control	24	104	22	69	3.14
S500A	-	21	61.5	-	19	-
S521	Strengthen	25.2	102.5	25	69	2.76
S531	Strengthen	20.1	93.2	25.2	29	1.15
S531-A	Strengthen	18.8	95	24	40	1.67
S531-B	Strengthen	22.4	95.9	22.2	51.1	2.3
S541	Strengthen	21.8	97	22	43	1.95
S521	Repair	21.7	93.6	22.9	55.9	2.44
S300	Control	26	147.5	14.4	44.2	3.07
S321	Strengthen	23.9	143.7	13.1	38.2	2.92
S331	Strengthen	25.6	146.5	13.5	36.5	2.7
S341	Strengthen	22.1	134.5	15.2	25.4	1.67
S320	Repair	24.6	138.5	15.8	39.1	2.47

Table 5. 6- The details of clamps.

Clamp			
Steel Bars	Steel box (mm)	Steel plates (mm)	Yield strength bar (MPa)
10 mm diameter	40x40x4	60x60x10	276.9
500 mm Length		40x40x10	

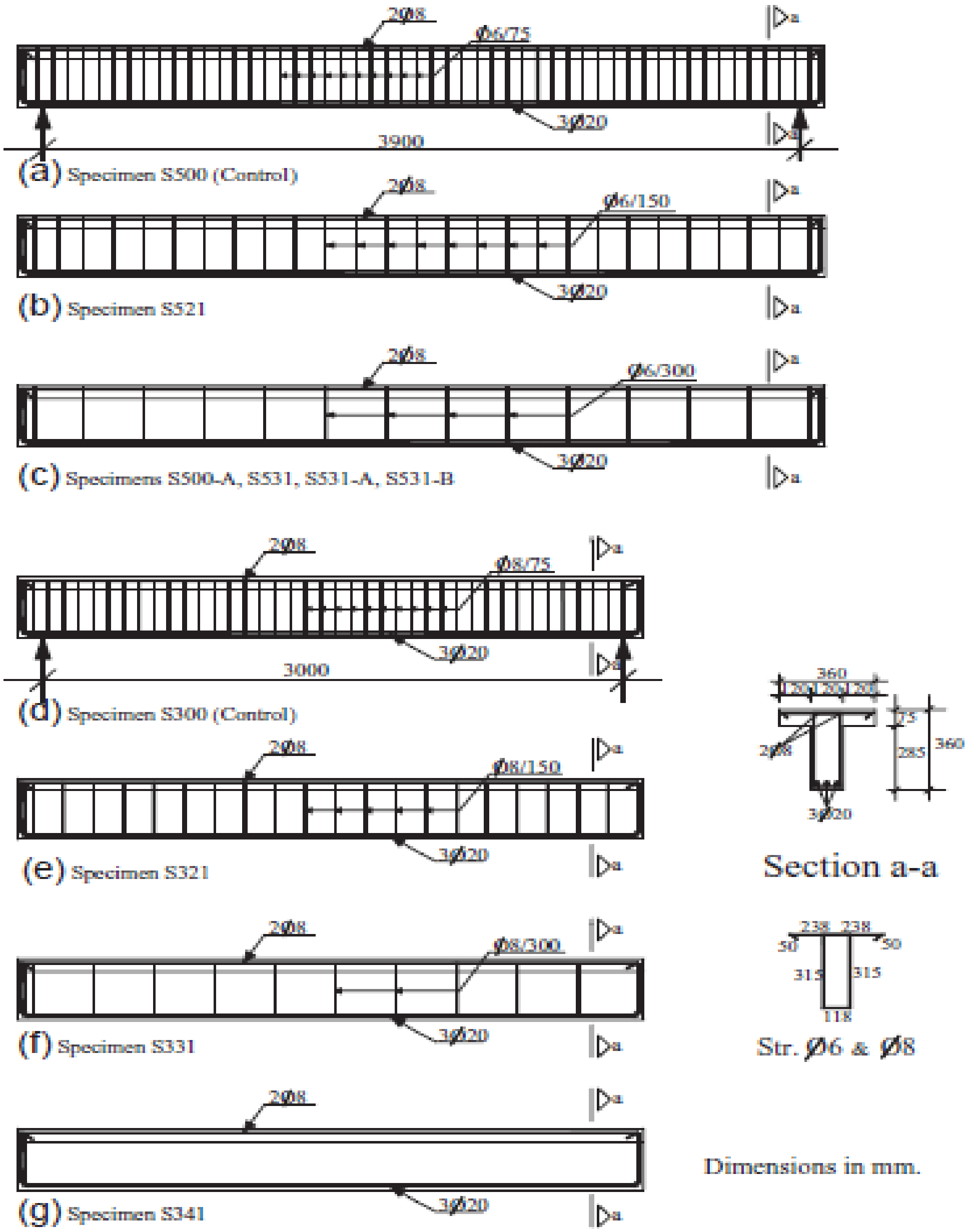


Figure 5. 8- Reinforcement layout of specimens (Altin et al. 2003).

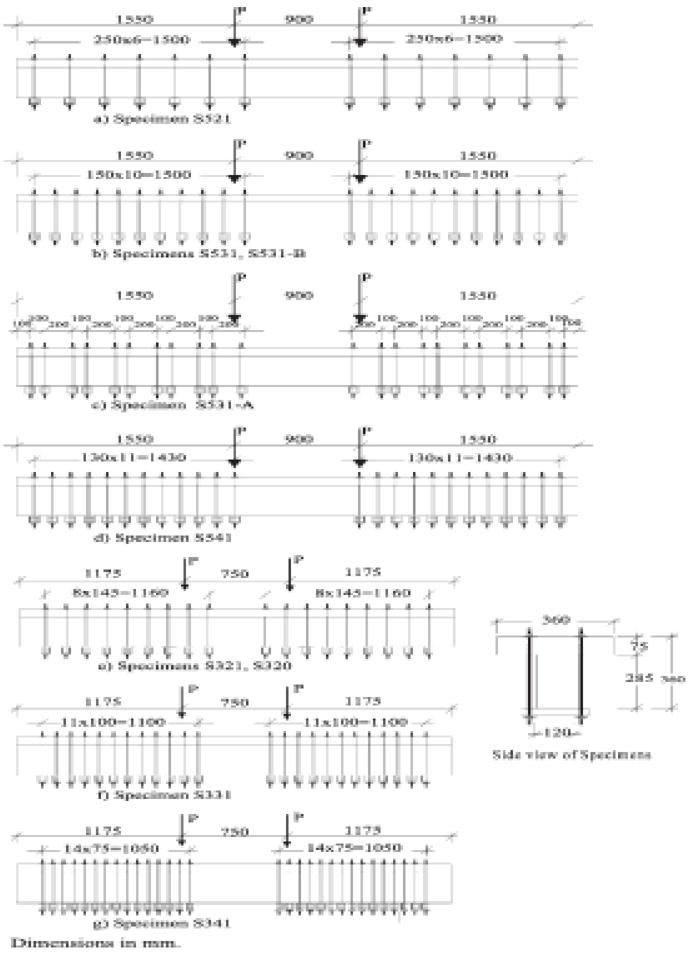


Figure 5. 9- Strengthened/repaired specimens (Altin et al. 2003).

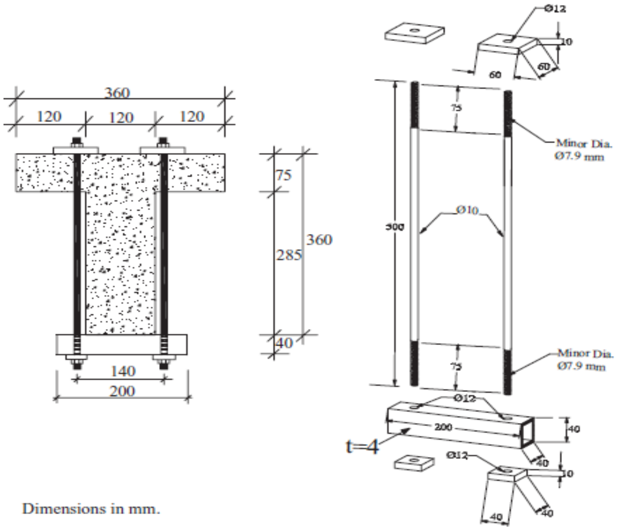


Figure 5. 10- Details of the clamps (Altin et al. 2003)

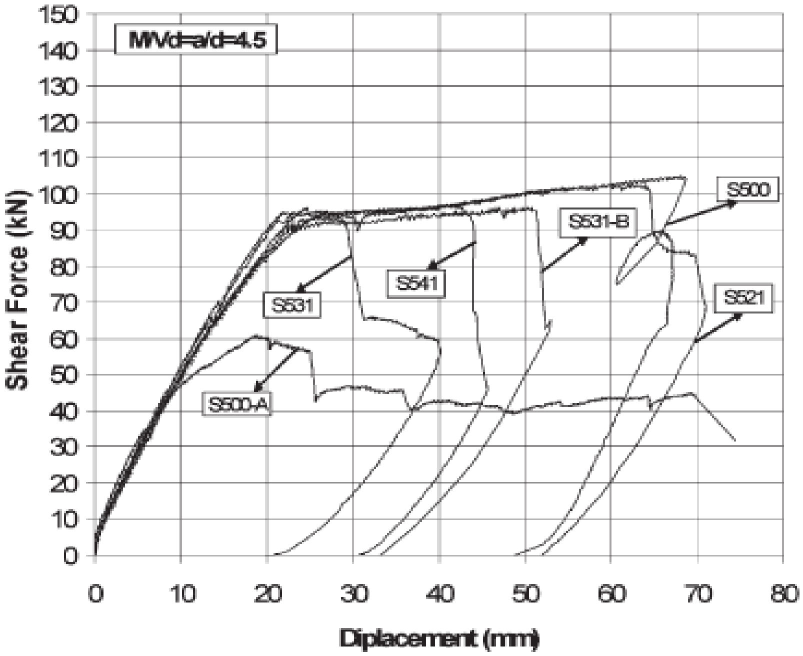


Figure 5. 11- Load-displacement relations; evaluation of number of clamps (*a/d*=4.5) (Altin et al. 2003).

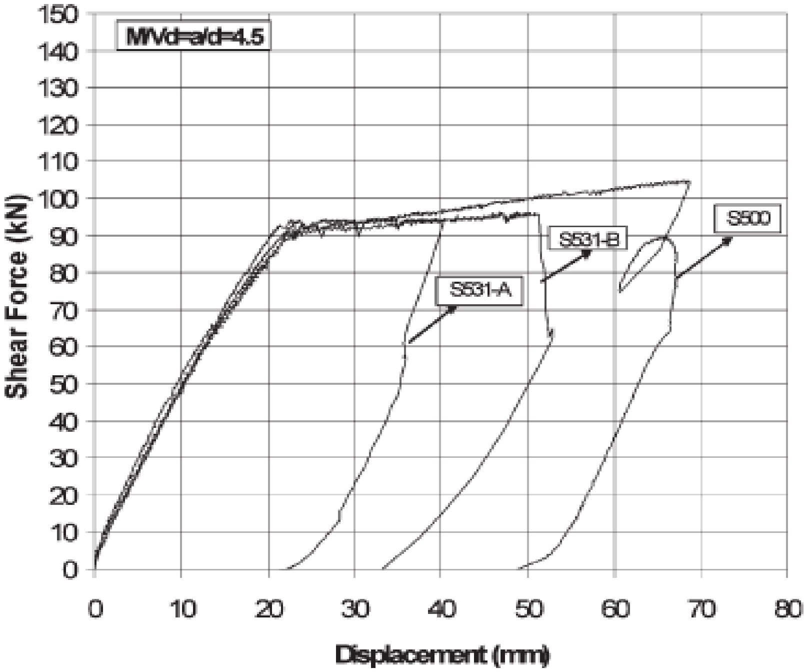


Figure 5. 12- Load-displacement relations; evaluation of clamp space (a/d=4.5) (Altin S. et al. 2003).

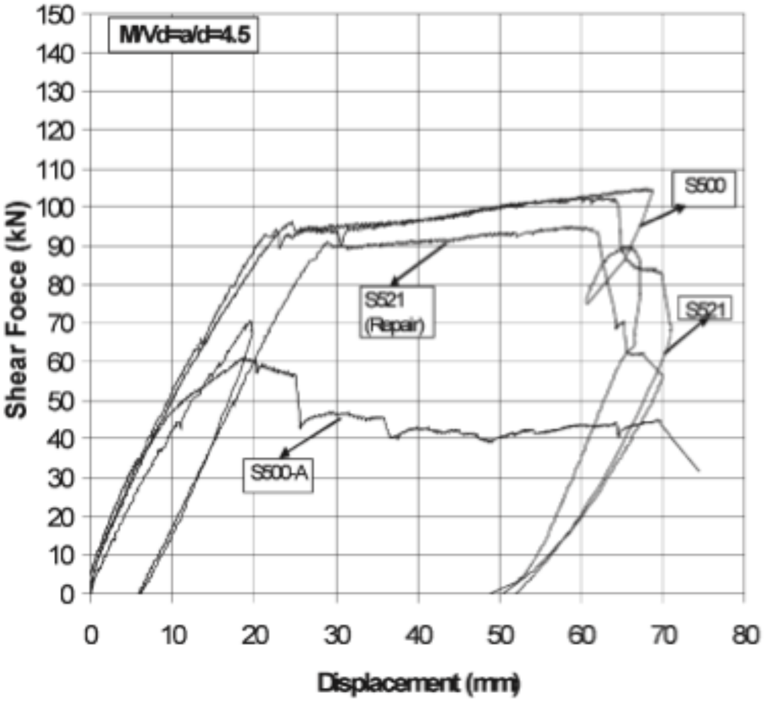


Figure 5. 13- Load-displacement relations; evaluation of application type (*a/d*=4.5) (Altin et al. 2003).

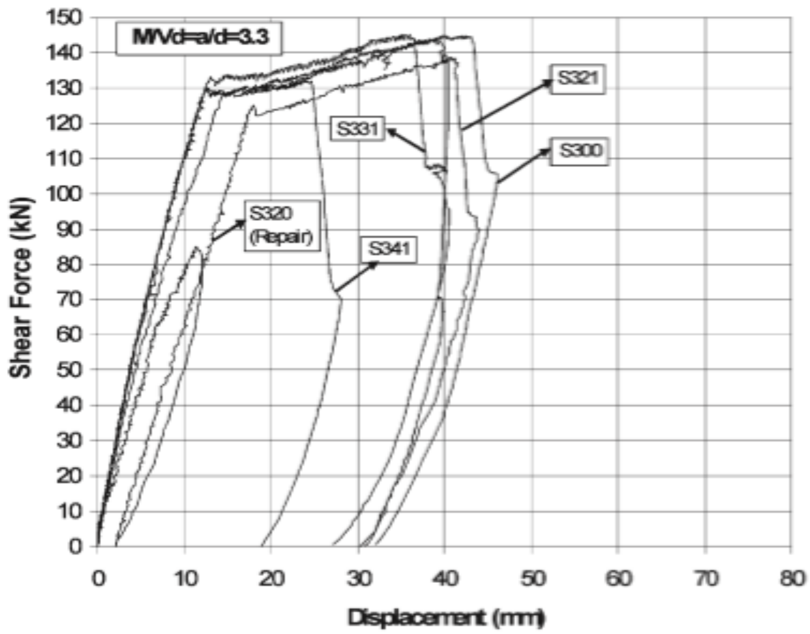


Figure 5. 14- Load-displacement relations (*a/d*=3.3) (Altin et al. 2003).

Shamsai et al. (2007) investigated another unbounded-type method that strengthened reinforced concrete beams with post-tensioning in the critical shear region. A total of 24 specimens are categorized into three groups: BB, SC and SB. Beams in BB are bare or unstrengthened beams. Beams in SC are specimens tested until large shear cracks developed and then were strengthened with post-tensioning in cracked shear region. SB beams are specimens post-tensioned before loading. The specimens' properties and strengthening details are listed in Table 5.7 and Figure 5.15 where a represents the shear span, ρ indicates longitudinal reinforcement ratio and ρ_{max} means the maximum allowed reinforcement ratio. Shamasi et al. tested 24 specimens until they experienced flexural failure near the middle of the beams. Figure 5.16 presents the experimental and theoretical load-deflection relations for these 19 specimens. It indicates that the factors including post-tensioning stress, existence of stirrups in the critical shear regions and an additional steel plate under the post-tensioning angles did not significantly impact the behavior of beams. The theoretical and experimental load-deflection relations for shear-critical beams are presented in Figure 5.17. The plots refer that the influence of the amount of longitudinal reinforcement on shear strength. Strengthening with post-tensioning is an effective method in terms of preventing shear failure and developing maximum flexural strength no matter strengthening is done before or after shear cracks occur.

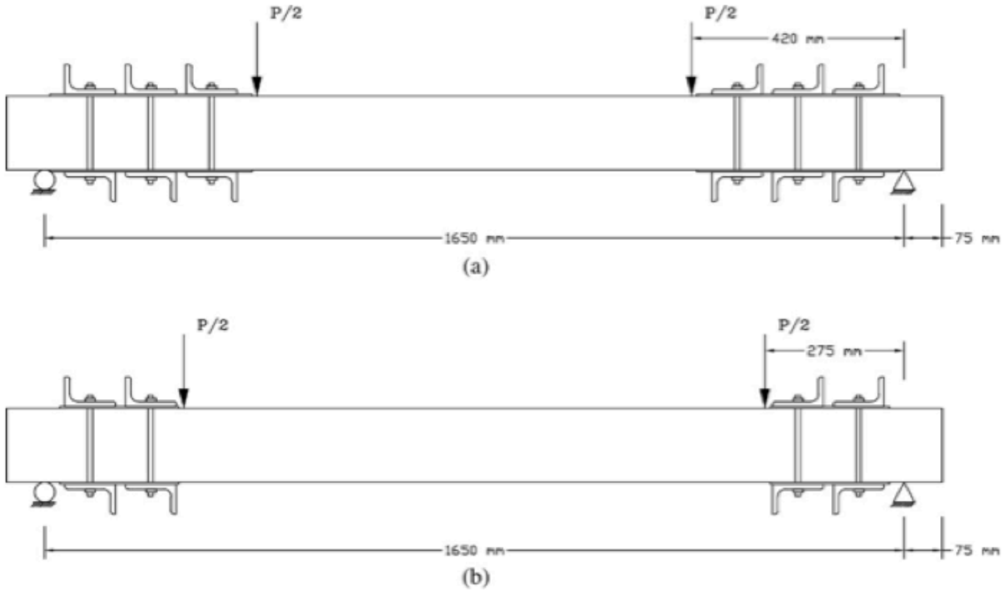


Figure 5. 15- Beam specimens with different shear span: a) a= L/4, b) a=L/6 (Shamsai et al. 2007).

Table 5. 7- Load and parameter specification for beam specimens.

#	Specimen name	f_c' (MPa)	a	f_p (MPa)	ρ/ρ_{max}	Stirrups at shear region

1	SB1	35	L/4	0.075 *fc'*	½	Yes
2	SC2	35	L/4	0.075 *fc'*	½	No
3	SB3	35	L/4	0.075 *fc'*	⅔	Yes
4	SC4	35	L/4	0.075 *fc'*	⅔	No
5	SB5	35	L/4	0.075 *fc'*	½	Yes
6	SB6	35	L/4	0.075 *fc'*	½	Yes
7	SC7	35	L/4	0.075 *fc'*	½	No
8	SB8	35	L/4	0.075 *fc'*	⅔	Yes
9	BB9	35	L/4	-	½	Yes
10	BB10	35	L/4	-	⅔	Yes
11	BB11	35	L/4	-	½	No
12	BB12	35	L/4	-	⅔	No
13	SB13	35	L/6	0.075 *fc'*	½	Yes
14	SC14	35	L/6	0.075 *fc'*	½	No
15	SB15	35	L/6	0.04 *fc'*	⅔	Yes
16	SC16	35	L/6	0.04 *fc'*	⅔	No
17	BB17	35	L/6	-	½	Yes
18	BB18	35	L/6	-	⅔	Yes
19	BB19	35	L/6	-	½	No
20	BB20	35	L/6	-	⅔	No
21	BC21	50	L/4	0.075 *fc'*	½	No
22	BC22	50	L/4	0.04 *fc'*	½	No
23	BB23	50	L/4	-	½	No
24	BB24	50	L/4	-	½	Yes

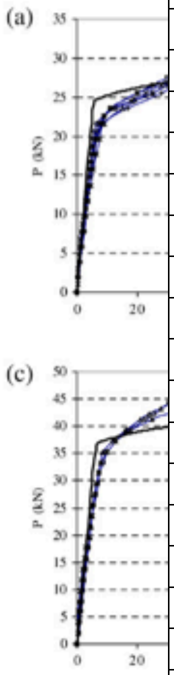


Figure 5. 16-Theoretical and experimental load-deflection relations for beams failing in flexure. (a) beams with fc'=35, ρ=1/2ρmax, a=L/4, (b) beams with fc'=35 MPa, ρ=2/3ρmax, a=L/4, (c) beams with fc'=35 MPa, ρ=1/2ρmax, a=L/6

(d) beams with fc'=35 MPa, ρmax, a=L/6, (e) beams with fc'=50 Mpa, ρ=1/2ρ_{max}, a=L/4 (Shamsai et al. 2007).

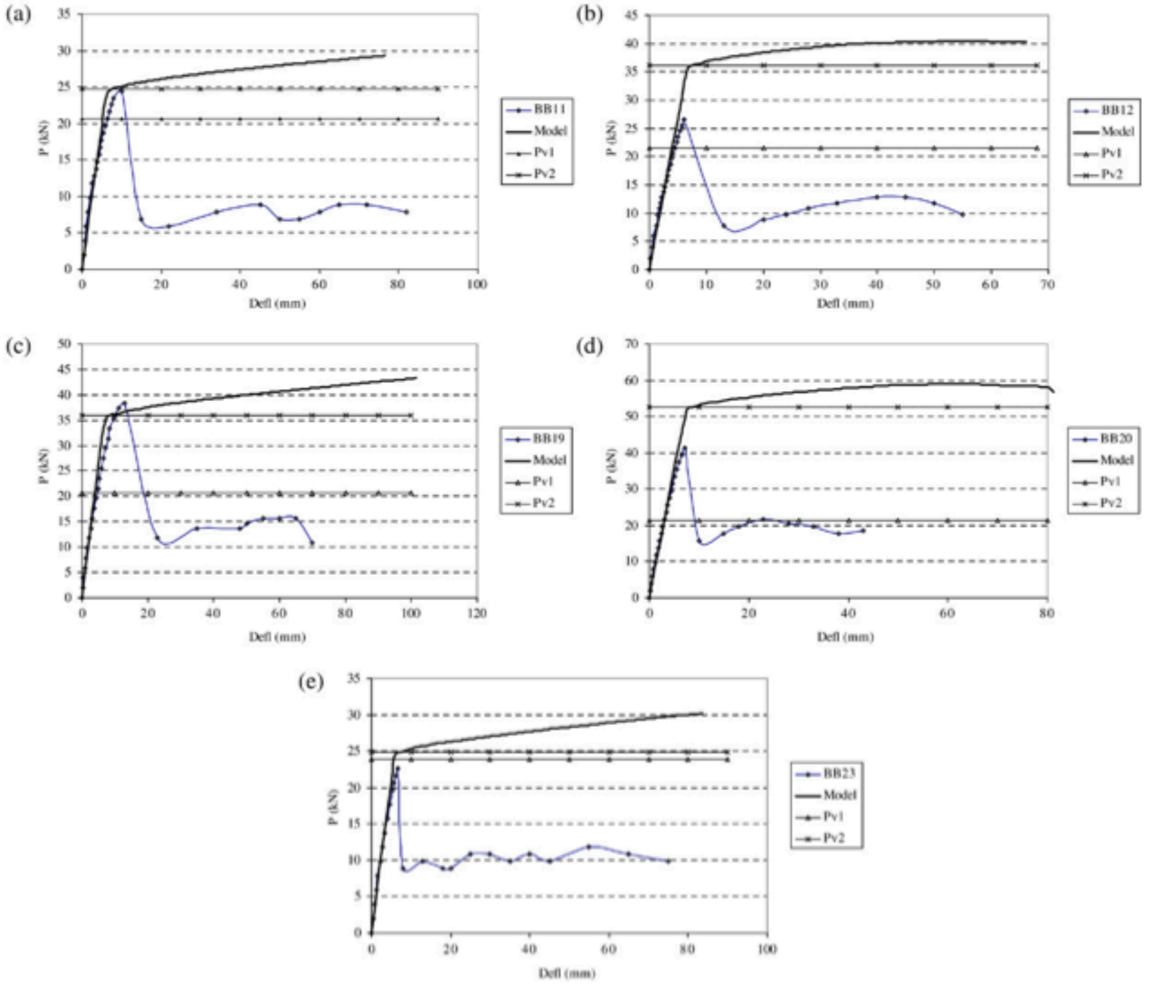


Figure 5. 17- Theoretical and experimental load-deflection relations for beams failing in shear. (a) beams with fc'=35 MPa, ρ=1/2ρmax, a=L/4 (b) beams with fc'=35 MPa, ρ=2/3ρmax a=L/4 (c) beams with fc'=35 MPa, ρ=1/2ρmax a=L/6 (d) beams with fc'=35 MPa, ρ=2/3ρmax, a=L/6 (e) beams with fc'=50 MPa, ρ=1/2ρmax, a=L/4 (Shamsai et al. 2007).

Ozturk et al. (2002) did research on behavior of beams strengthened with U connecting bars and V connecting stirrups. The average concrete compressive strength was 25 MPa, the tensile strength of steel was 565 MPa and tensile strength of stirrups was 516 MPa. The reinforcement details of specimens are presented in Figures 5.18 through 5.20. For each strengthening techniques contained a strengthened beam, a repaired strengthened beam and an unstrengthened beam are tested and compared. The comparison of beams was monolithic in construction. The span of test specimens was 1900 mm and other details of reinforcement and dimension of specimens are presented in Figures 5.18 through 20. All beams are firstly loaded by controlling load magnitude until tension steel yielded and then loaded by controlling their displacement until exceeded carrying capacity. The measured displacement and loading are reported in Table 5.8 and Figures 5.18 through 25. From the test results it can be inferred that the load carrying capacity of strengthened beam was slightly less than control beam, load carrying capacity of V connecting bars is higher than U connecting stirrups. However the energy dissipation capacity of U connecting stirrups is better than those of V connecting bars. The application of U connecting stirrups is more labor intensive than that of V connecting bars. From the ductility point of view, U connecting stirrups outweighs the advantage as compared to V connecting bars. Hence for a designer with seismic retrofit in mind, the U connecting stirrup will be preferred.

Table 5. 8- Results of the retrofit tests.

Specimen	Load at yielding (kN)	Load at failure (kN)	Displacement at yield (mm)	Displacement at failure (mm)	Displacement ductility	Energy dissipation (kNm)
BB	27	28.2	6.95	23.53	3.39	
SBV	69.55	72.5	6	28	4.67	1.629
SBU	60	69	9	38.82	3.15	1.543
RSBV	62	70	10.11	31.46	7.17	1.982
RSBU	58	62	7.59	35.52	4.31	1.995
CBV	59	75	4.6	33	4.69	1.758
CBU	63	72	6.3	38.9	6.17	2.264

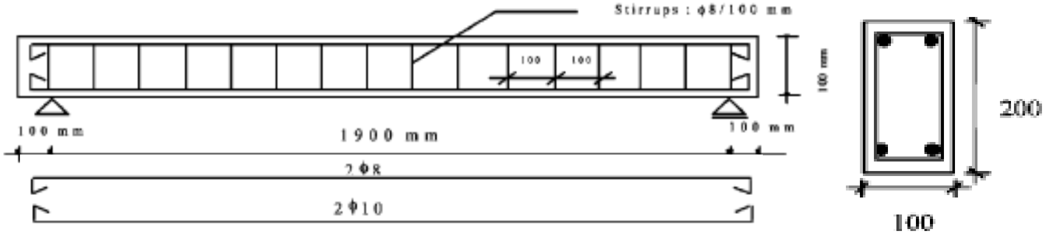


Figure 5. 18- Reinforcement Details of Beam BB (Ozturk et al. 2002).

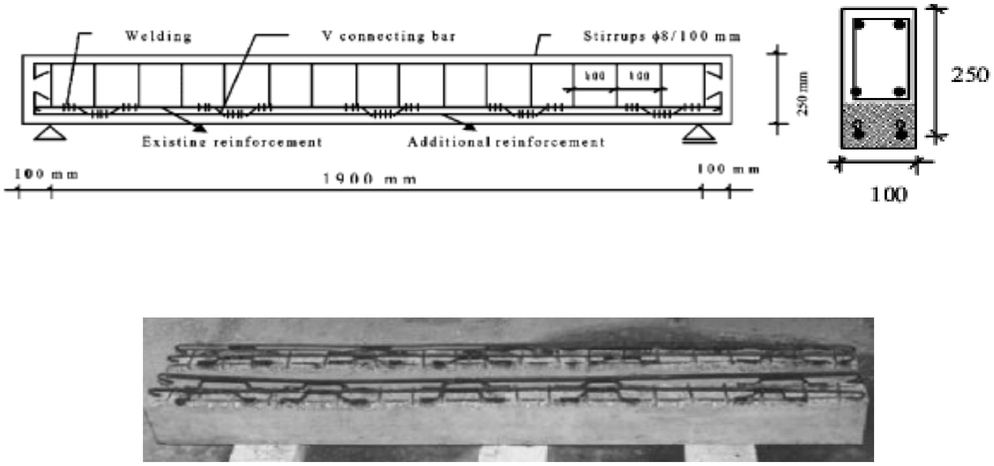
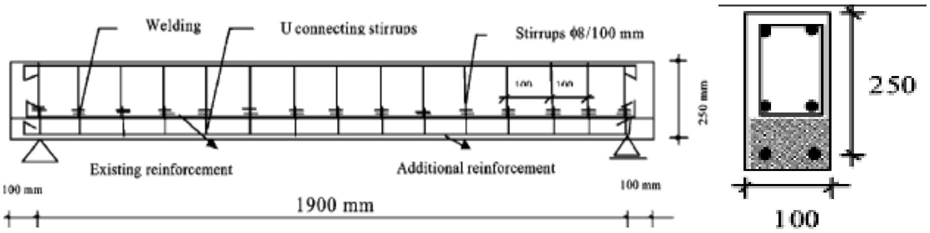


Figure 5. 19- Reinforcement details of SBV, RSBV and CBV (Ozturk et al. 2002).



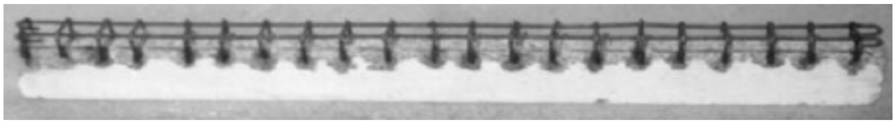


Figure 5. 20- Reinforcement details of SBU, RSBU and CBU (Ozturk et al. 2002).

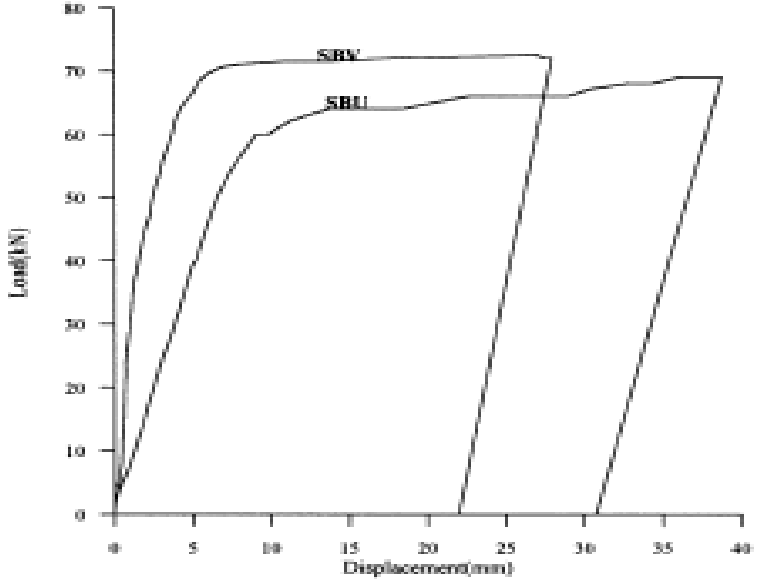


Figure 5. 21- Recorded load displacement curve of SBV and SBU (Ozturk et al. 2002).

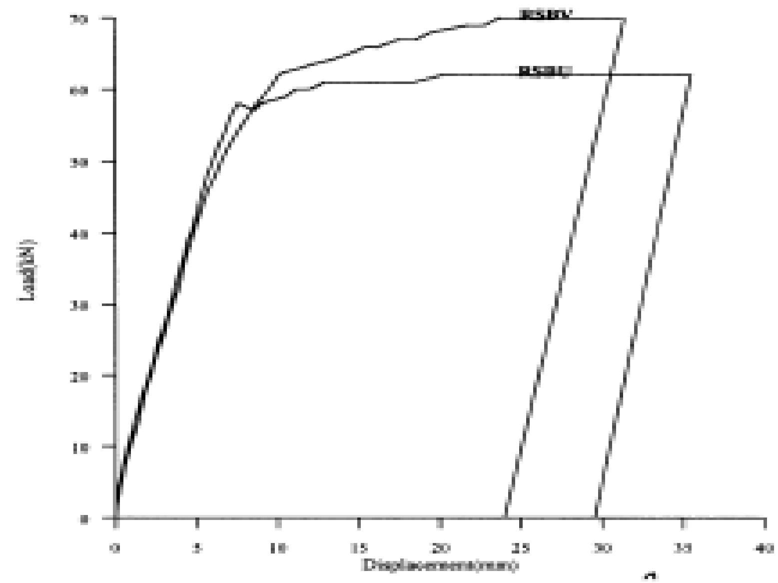


Figure 5. 22- Recorded load displacement curves of RSBV and RSBU (Ozturk et al. 2002).

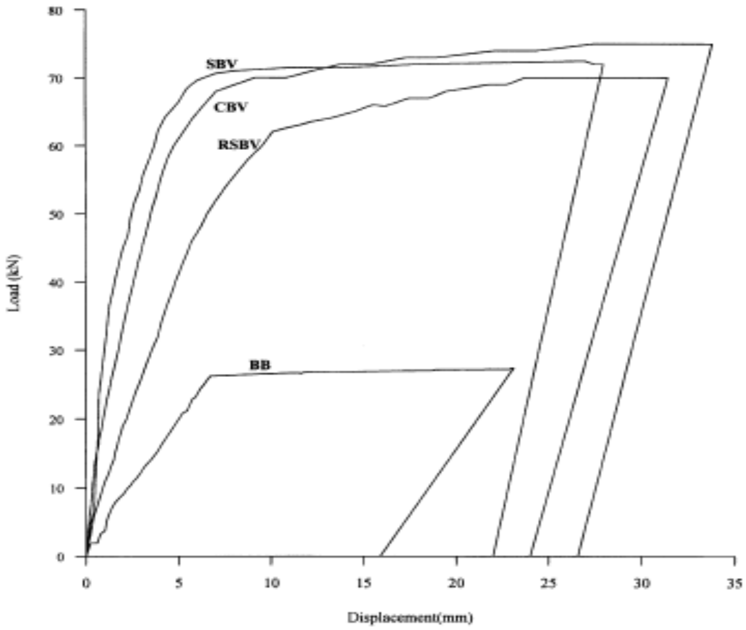


Figure 5. 23- Recorded load displacement curves of BB, SBV, RSBV and CBV (Ozturk et al. 2002).

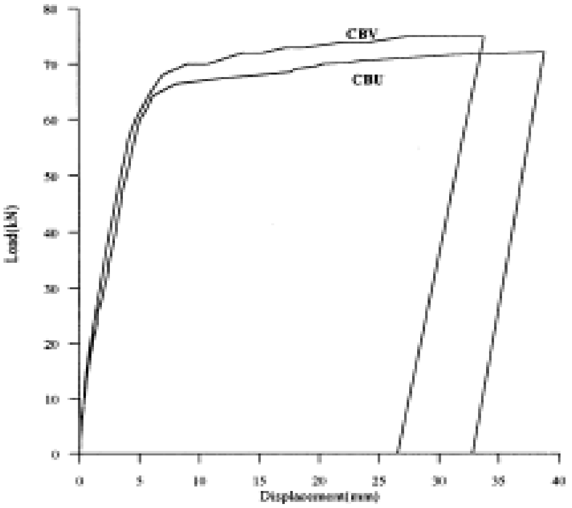


Figure 5. 24- Recorded load displacement curve of CBV, CBU (Ozturk et al. 2002).

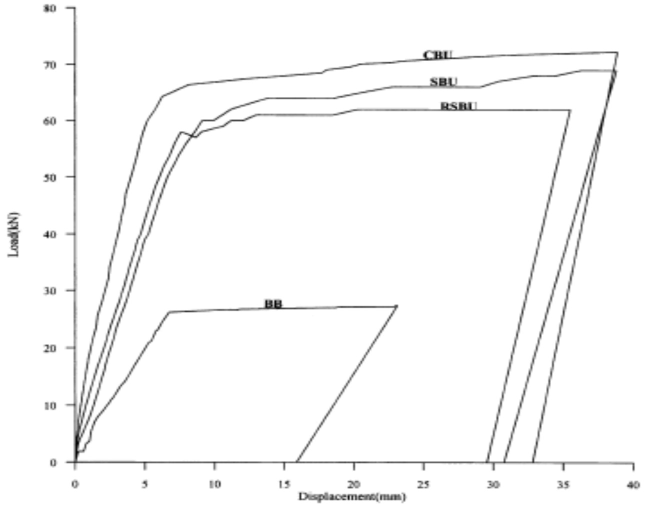


Figure 5. 25- Recorded load displacement curve BB, SBU, RSBU and CBU (Ozturk et al. 2002).

Adhikary and Mutsuyoshi (2006b) conducted experiments to compare the effectiveness of different strengthening methods. As the details mentioned in Chapter 4, all strengthening methods can effectively increase the shear capacity of RC beams. However, strengthening RC beams by using vertical strips has the most significant effect, increasing the shear capacity by 117%, which is much higher than other methods (Table 5.9).

Table 5. 9-Test results of Series A and B. (Adhikary and Mutsuyoshi, 2006b)

	Specimen	Strengthing Scheme	Yield (MPa)	Failure Load (kN)	Mode
Series A	CA	Control Beam	-	187	shear
	BA	Steel Brackets	398	201	shear
	PA	Steel Plates	378	279.3	flexure
	PAA	Steel Plates (anchors)	378	272.2	flexure
	VSA	Vertical Steel Strips	382	292	flexure
	ESA	Externally anchored Stirrups	480	272.5	flexure
Series B	CB	Control Beam	-	233.6	shear
	BB	Steel Brackets	347	220.5	shear
	PB	Steel Plates	320	405.5	shear
	PAB	Steel Plates (anchors)	320	400.6	shear
	ESB	Externally Anchored Stirrups	450	507.6	flexure

5. 3 Summary and Conclusions

Unbounded-type strengthening techniques not only increase the flexural and shear capacities but also can lower the cost and minimize environmental impact because they minimally increase the weight of beams, require short time to install, and produce no additional pollution during the strengthening process. However, they need sophisticated instruments and sufficient attention on protecting them from environmental effects such as corrosion and fire.

CHAPTER 6 Concrete Repair

6. 1 Introduction

Although most concrete structures perform satisfactorily, they may still have some durability or structural problems that need to be solved. For example, they are vulnerable in strict environments; they may carry higher loads compared to designed loads; and they maybe designed inadequately. In these scenarios, cracking and corrosion are two common factors that cause the deterioration of concrete structures. Hence, repairing of concrete structures is necessary.

6. 2 Description of Previous Study

The most common cause that leads to deterioration of concrete structures is cracking. Shash et al. (2005) investigated the causes of cracks on five concrete beams in the university campus. The five reinforced concrete roof beams were 16 m long, 1.5 m deep, and 0.3 m wide. The cracks of the beams were occurred in six months after construction. After cracks were cleaned, a liquid epoxy resin was injected to cracks. The properties, information of cracks and repair properties are exhibited in Figure 6.1 and Table 6.1. The repaired beams were tested by load test. The deflections of roof beams were recorded in Table 6.2. B1 is a roof beam located at the center and B2 is a roof beam located 8m from the center. The experimental results indicated that sealing the existing cracks by epoxy injection is an effective method to repair the cracked RC beams since it reduced the deflection under allowable value (6.4 mm). Epoxy injection does not increase the weight of beams very much and effectively reduce the deflections of beams, so it can be used to successfully repair the cracked RC beams.

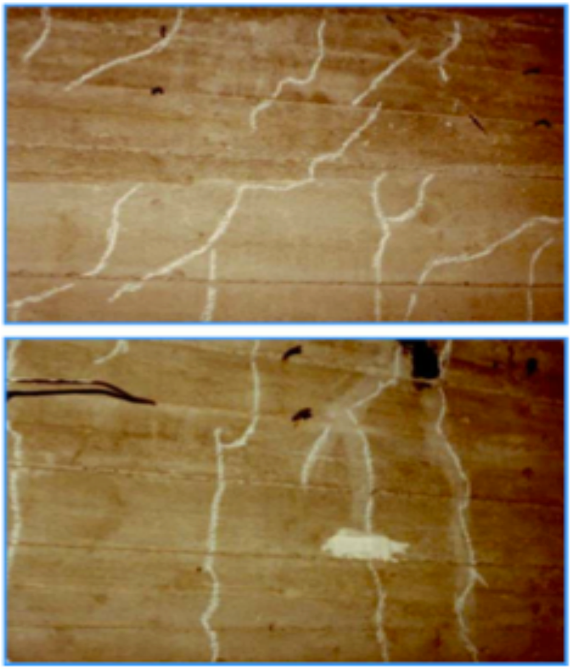


Figure 6. 1- Cracks noted on two beams (Shash et al. 2005).

Table 6. 1- Properties of cracks and injection

Crack #	Width (mm)	Length (m)	Volume of epoxy resin injected (ml)
1	0.3	0.4	125
2	0.2	1	245
3	0.3	0.2	100
4	0.4	0.5	245
5	0.2	0.9	125
6	0.4	0.85	368
7	0.4	0.9	368
8	0.4	0.5	245
9	0.3	0.85	125
10	0.2	0.65	368
11	0.4	0.87	125
12	0.2	0.9	245
13	0.4	0.5	125
14	0.2	0.95	100
15	0.2	0.5	125
16	0.2	0.5	180
17	0.2	0.5	125

18	0.2	0.9	125
19	0.2	0.65	60
20	0.2	0.65	125
21	0.2	0.4	125
22	0.2	0.6	125
23	0.2	0.6	125
24	0.2	0.6	125
25	0.2	0.65	125

Table 6. 2- Deflection of beams due to applied load

Measurement (mm)	Beam B1	Beam B2
Deflection immediately after loading	1.1	0.1
Deflection after 24 h of loading	2	0.3
Limiting deflection after 24 h of	6.4	3.8

Corrosion of reinforcement is another typical cause that leads to damage in concrete structures. Protecting reinforcement from harsh weather conditions is necessary and important. Nounu and Chaudhary (1999) conducted experiments with 18 large beams with same dimensions: 2.5×0.25×0.25m. All specimens were tested until they failed in bending. Then they were repaired in central area of their span, and they finally were tested in flexure under the third point loading (Figure 6.2). The beams were categorized to three groups: Batch A included six beams for preliminary corrosion, Batch B contained six control beams, and Batch C had six beams for the assessment of repairs in the aggressive climate conditions. The amount of chlorides varied between the sections and beams (Tables 6.3 and 6.4). Ordinary Portland cement (OPC) mortar and free flowing micro-concrete are two repairing materials that were applied and compared during the experiment (Tables 6.3 and Table 6.4). After curing the repairs, the beams of Batch C were placed in the weathering chamber. Table 6.5 provides the failure loads of all specimens. The comparison of corroded (D) beams and control (DC) beams indicated that the beam damage occurred much earlier than the loss of steel section and load bearing capacity. Furthermore, corrosion can slightly affect the bond. The comparison between repaired beams and control beams referred that, in short term, free flowing micro-concrete and OPC mortar show similarly effectiveness but for longer term durability, free flowing micro-concrete showed little cracking and restored only 40%~50% of the capacity, while OPC mortar showed extensive cracking and restore approximate 90% of the capacity. The Figures 6.3 through 6.6 provided the load-deflection curves. They indicate that the preliminary corrosion hardly influenced the stiffness, but the repaired beams had higher deflections compared to control beams. Overall, the free flowing micro-concrete performed better in resisting chloride ingress from and external source than OPC mortar under accelerated conditions.

Table 6. 3- Details of beams cast for load testing (un-weathered)

Batch	Beam code	Chloride level (% by weight of cement)				Corroded	Repaired	Repair material
		Section 1	Section 2	Section 3	Section 4			
A	D1	1	3	1	Nil	Yes	No	OPC mortar Free-flowing micro-concrete
	D2	1	3	1	Nil	Yes	No	
	DB1	1	3	1	Nil	Yes	No	
	DB2	1	3	1	Nil	Yes	No	
	R1	1	3	1	Nil	Yes	Yes	
	R2	1	3	1	Nil	Yes	Yes	
B	DC1	Nil	Nil	Nil	Nil	No	No	
	DC2	Nil	Nil	Nil	Nil	No	No	
	DBC1	Nil	Nil	Nil	Nil	No	No	
	DBC2	Nil	Nil	Nil	Nil	No	No	
	RC1	Nil	Nil	Nil	Nil	No	No	
	RC2	Nil	Nil	Nil	Nil	No	No	

Table 6. 4- Details of beams cast for accelerated weathering in climatic chamber (weathered)

Batch	Beam code	Chloride level (% by weight of cement)				Corroded	Repaired	Repair material
		Section 1	Section 2	Section 3	Section 4			
C	Beam 1	Nil	Nil	Nil	Nil	No	Yes	OPC mortar
	Beam 2	Nil	Nil	Nil	Nil	No	Yes	Free-flowing micro-concrete
	Beam 3	0.5	3	0.5	Nil	Yes	Yes	OPC mortar
	Beam 4	0.5	3	0.5	Nil	Yes	Yes	Free-flowing micro-concrete
	Beam 5	1	3	1	Nil	Yes	Yes	OPC mortar
	Beam 6	1	3	1	Nil	Yes	Yes	Free-flowing micro-concrete

Table 6. 5- Failure loads (kN)

Beam	Load (kN)	Beam	Load (kN)	Beam	Load (kN)
D1	260	DB1	22.5	R1	180
D2	248	DB2	see Beam 7	R2	160
DC1	250	DBC1	240	RC1	256
DC2	260	DBC2	241	RC2	266
Beam 1	120	Beam 3	110	Beam 5	136
Beam 2	230	Beam 4	230	Beam 6	245
Beam 7	270				

C=control beam; D=damaged beam; B=broken beam; R=repaired beam;

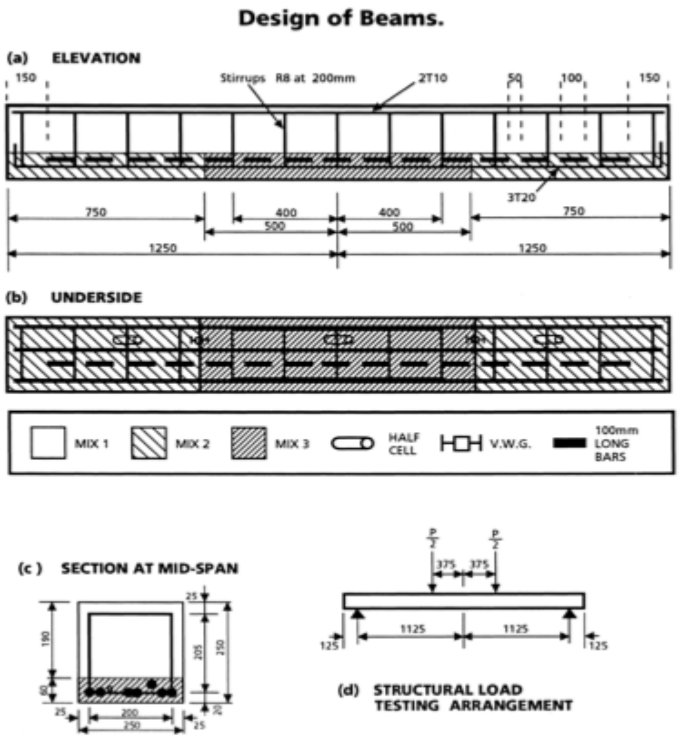


Figure 6. 2- Design of beams (Nounu and Chaudha 1999).

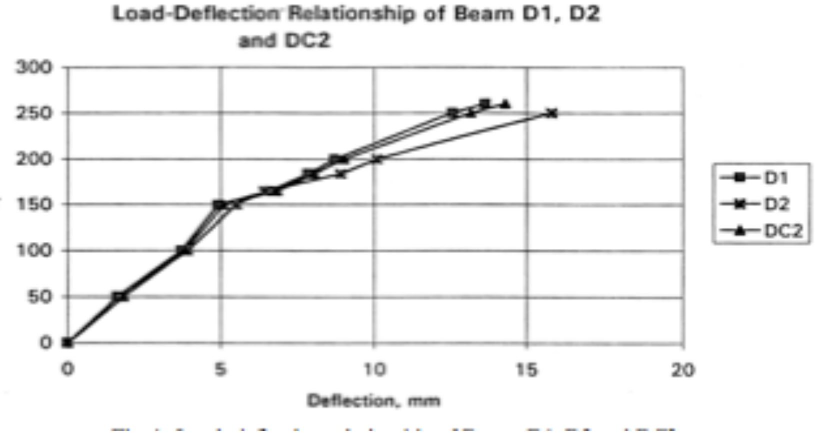


Figure 6. 3- Load-deflection relationship of Beams D1, D2 and DC2 (Nounu and Chaudha 1999).

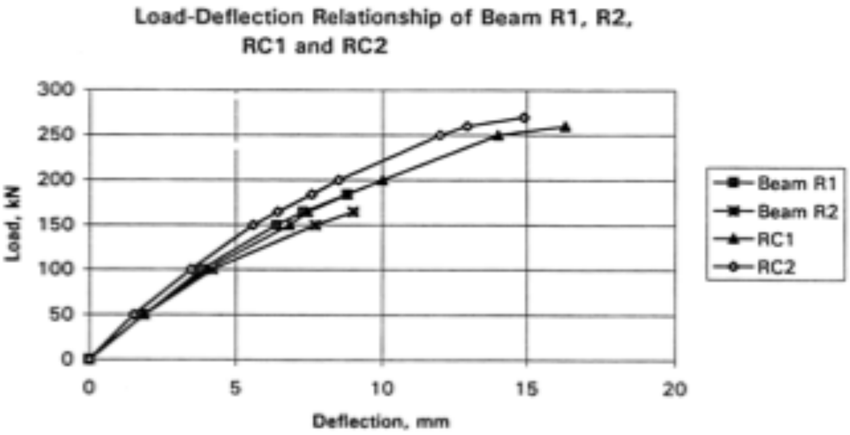


Figure 6. 4- Load-deflection relationship of Beams R1, R2, RC1 and RC2 (Nounu and Chaudha 1999).

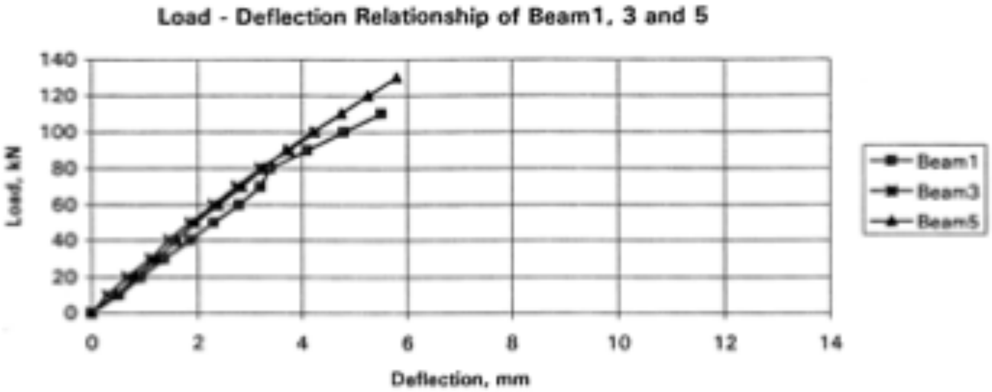


Fig. 6. Load−deflection relationship of Beams 1, 3 and 5.

Figure 6. 5- Load-deflection relationship of Beams 1, 3 and 5 (Nounu and Chaudha 1999).

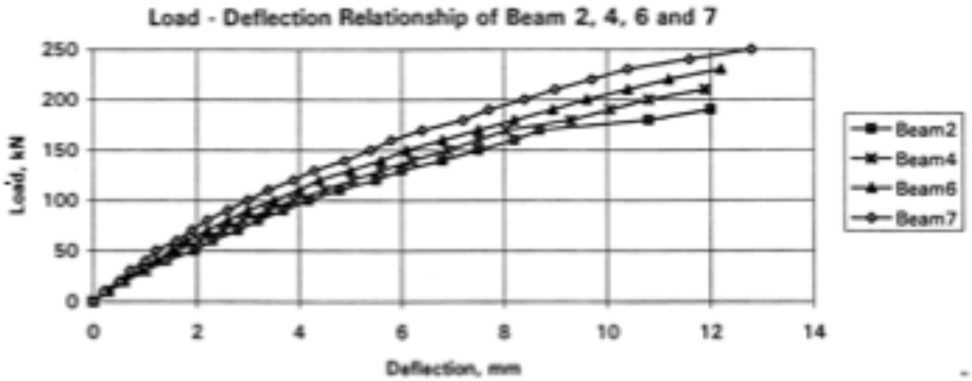


Fig. 7. Load–deflection relationship of Beams 2, 4, 6 and 7.

Figure 6. 6- Load-deflection relationship of Beams 2, 4, 6 and 7 (Nounu and Chaudha 1999).

6. 3 Summary and Conclusions

From the review of the previous studies, it can be concluded that epoxy injection is an easy and effective method to repair the cracked beams. It not only reduces the deflection and control the cracks but also slightly increases the weight of beams. Furthermore this method is inexpensive and unsophisticated. Using OPC mortar and a free flowing micro-concrete are two possible methods to repair the beams under corrosion conditions. However, for long term, a free flowing micro-concrete can restore higher structural capacity and resist chloride better under accelerated weather conditions compared to using OPC mortar.

CHAPTER 7 Summary and Conclusions

7. 1 Summary

This research analyzed the various reinforced concrete strengthening and repairing methods for reinforced concrete beams. The experimental results that are summarized from literature review were used to conclude the pros and cons of current strengthening and repair methods. The suggestions about retrofit and rehabilitation methods are provided in the thesis for future work and study. The problems and experiences reported in this research will be used to improve the future repair and strengthening research.

7. 2 Conclusions

Section enlargement and concrete jacketing can effectively increase the load carrying capacity and stiffness of reinforced concrete beams. Compared to other methods such as attaching external steel plates, they are relatively easy, cheaper and will add less weight to beams. However, using section enlargement and concrete jacketing can lead to beams gaining relatively more weight when compared them to using unbounded-type methods. So in order to minimize the extra weight, the light weight concrete can be used. Furthermore, the material properties used to determine the protection of concrete jackets and additional enlargement layers are important.

External reinforcement can increase flexural capacity of RC beams very well, but it will be limited by shear capacity sometimes. The external reinforcement can also increase the weight of beams, and they are vulnerable in harsh environment. Compared to other methods, this technique is inexpensive and easy to execution.

Attaching external steel plates can increase flexural and shear capacity of RC beams. However it may increase weight to beams and cost more than other methods. Attaching steel plates to beams also has the risk of peeling and corrosion. The construction process could be complicated and the cost of this method is higher compared to other methods. The efficiency of steel plates is influenced by some factors such as dimension of steel plate, the arrangement of bolts, and bonding method. So the strengthening should be designed based on the different situations.

Unbounded-type strengthening technique is adding externally steel units such as unbounded wire rope units, steel clamping or post-tension to the RC beams. These strengthening methods not only increase the carry capacity of beams but also add little weight to them. Compared to steel plates, this is a better option in term of increasing the shear strength of RC beams. The construction time of using this method is short, but it requires relatively more technical labor.

For damaged beams, injecting epoxy to seal the cracks is an effective method to repair the cracked beams. It is very easy to apply and slightly increases the weight of the beams. When it comes to corrosion, flowing micro-concrete and OPC mortar are two foundational materials to repair the damaged beams. They both work effectively in short term. However in long term, OPC mortar performs much better than flowing micro-concrete in terms of maintaining the load carrying capacity.

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