Scientific paper

Behavior of Beams Strengthened with Steel Fiber RC Overlays

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Abstract

Strengthening of reinforced concrete beams using steel fiber reinforced concrete (SFRC) overlays has been investigated in the test program described in this paper. Two methods have been used to connect the overlays to the original beams, i.e. chemical and mechanical bonding. In the chemical bonding, 2-component epoxy resin bonding agent has been used. The mechanical bonding was achieved by welding the stirrups in the overlays to the stirrups in the original beams only near each support. In general, the weld bonded strengthened beams have achieved a better structural behavior in terms of load carrying capacity and failure mode compared to the epoxy bonded beams. The epoxy bonded beams have reached same load and ductility levels obtained from an identical monolithically cast control beam which was included in the test program for comparison purposes. However, at failure separation cracks have occurred at the common interface between the overlays and the original beams. On the other hand, the weld bonded strengthened beams have behaved in a flexural ductile manner and achieved higher load carrying capacity compared to the control beam. The interlaminar shear failure did not occur in these beams which have acted as a single unit up to the failure.

1. Introduction

Sudden destruction or progressive deterioration of cities would result in damages in large number of structures that need rehabilitation or replacement. The adverse influence of environmental factors after long neglect, changes in building regulations and code requirements, the demand of increasing load levels have led to problems in the load carrying capacity and long-term durability of many structures. In addition to the need of recycling of old buildings, there are many structures that require either, rehabilitation or demolition because of inadequate design or detailing, poor construction practice, natural or manmade destruction, etc. The decision on whether to rehabilitate or to demolish a damaged structure is dependent on the anticipated functional life span requirements of the structure and the availability of cost-effective structurally upgrading solutions (Ziara 2007).

A variety of rehabilitation techniques are used in practice (ACI 546R 2004; REHABCON 2004). The sophistication of these techniques ranges from simple methods of enlargement of cross sections including the use of concrete overlays (Saiidi *et al*. 1990; Trikha *et al*. 1991) to advanced methods in which strengthening is achieved using externally bonded steel plates, external post-tensioning, carbon fibers, etc. Strengthening techniques of beams include external prestressing using steel or carbon fiber reinforced tendons (Federation 1991; Basler and Vaneck 2003; Ha Minh and Kyoji 2007), epoxy- or mechanical- bonded steel or plastic plates (Quantrill *et al*. 1996a; Quantrill *et. al*. 1996b; Lamanna 2004; Zhang and Hsu 2005; Adhikary and Mustsuyoshi 2006), ferrocement laminate with Skeletal bars (Jumaat and Alam 2006a), advanced polymer matrix composite materials (Meier 1992; Meier 1994; Adhikary *et al*. 2004) and the use of concrete overlays and underlays. A number of practical limitations have been identified with these techniques (Arduini *et al*. 1997; Arduini and Nanni 1997; Malek and Saadatmanesh 1998; ACI 546R 2004). For example, when external tendons are to be used consideration must be given to general access to the structural members, the feasibility of providing the necessary anchorage for tendons; and the practical limits placed on tendon layout and profile where straight tendons could be introduced more readily (Jumaat and Alam 2006b). Strengthened structures using fiber-reinforced plastics may result in brittle failure modes (Arockiasamy *et al*. 1995) in addition to their high cost (Meier 1994). In case of strengthening using steel plates special preparations are needed to install the externally mounted steel plates (Appleton and Silva 1995, Jumaat and Alam 2006b).

Strengthening of reinforced concrete beams using composite concrete overlays has been proved to be a cost effective solution (Ziara 2000; Haldane and Ziara 2000). This paper describes an investigation of using steel fiber reinforced concrete (SFRC) overlays for strengthening of reinforced concrete beams.

2. Strengthening techniques using SFRC concrete overlays

Concrete is considered a relatively brittle material with low tensile strength. In some applications, this deficiency can be overcome by combining concrete with small diameter fibers. The use of fiber reinforcement leads to improvement in strength and ductility of the

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concrete (ACI 544.1R 2002; Kwak *et al*. 2002; Choi, *et al*. 2007). The role of fibers is essentially to confine any advancing cracks by applying pinching forces at the crack tips, thus delaying their propagation across the cement matrix (El-Abboud 2003). Due to its superior characteristics fiber concrete can be used to strengthen conventional reinforced structures.

One of the most straightforward ways of increasing the capacity of existing beams is to add a concrete overlay. The effectiveness of this technique depends on the ability of the strengthened beam to act as one unit under ultimate load (Trikha *et al*. 1991; Federation 1991). This requires prevention of the inter-laminar shear failure to occur at the common interface between the original beam and the overlay. In spite of the existence of bond forces between the concrete overlay and the original beam the interface may crack as a result of the combined effects of the presence of externally applied and internal stresses. Initially the stresses in the bonded interface result from a combination of external loads and internal restraining forces resulting from shrinkage and temperature gradients in the new concrete overlay (Munger *et al*. 1995). The combined stresses may exceed the capacity of the initial bond leading to cracking in the interface. In the cracked interface the stresses may be considered to be due to only external forces since internal stresses are released.

In the undertaken research work, two methods for the development of full interaction between the original beams and the overlays have been investigated, i.e. chemical and mechanical bonding.

2.1 Chemical bonding

Previous tests using conventional concrete overlays have indicated that the effectiveness of chemical bonding across the interface has an influence on the strength and ductility which can be achieved through the addition of such overlays (Saiidi *et al*. 1990; Trikha *et al*. 1991). Premature failures have occurred in strengthened beams when the compression strain limit of the concrete either in the original beam or in the overlay was exceeded. In the test program reported in this paper, 2-component epoxy resin bonding agent that conforms to ASTM C881-78 specifications has been used to bond the old concrete of the original beam to the new SFRC of the overlay. The bonding agent was prepared, applied and cured in accordance with manufacture instructions.

2.2 Mechanical bonding

Strengthening of reinforced concrete beams using mechanically bonded traditional concrete overlays has been already validated for beams with rectangular, T and П shaped cross sections (Ziara 2000, Haldane and Ziara 2000). In the undertaken research work described in this paper, beams are strengthened using SFRC overlays. Strengthening of existing beams is achieved by casting the SFRC overlay on top of the existing beam to act as a concrete compression zone within the strengthened beam structure. The overlay is provided with closed stirrups. The interaction between the original beam and the overlay is achieved by locally welding the two sets of stirrups together in the overlay and in the beam. Only the stirrups in the part of the overlay near each support (2d from the support) are required to be welded to the corresponding stirrups in the original beam. In this method, the enhancement of the main parts of the beam can readily be achieved since the stirrups in the overlay are not required to extend down into the original beam hence avoiding the need to break into the lower regions of the original beam. The remaining regions of the top surface of the existing beam only require to be cleaned thoroughly before placement of the concrete in the overlay. The flexural strengthening and stiffness enhancements occur by utilizing the increased effective depth d of the strengthened beam. Welding together the stirrups in the existing beam and in the concrete overlay in the $2d$ distances near to the supports prevents an inter-laminar shear failure. The closed stirrups in the concrete overlay will prevent the propagation of any diagonal cracks into the concrete compression zone and prevent the occurrence of diagonal failures in the beams (Ziara *et al*. 1999; Ziara 2000). In addition, the compression steel bars which are normally placed in the top of the cross section of the original beam will be now located in the centre of the strengthened section. These central bars may contribute to the shear resistance of the beams (Desai 1995). The confining influence of the closed stirrups in the concrete overlay enhances the ductility of the beams and improves its resistance to seismic loading. The steel fibers contribute to shear resistance and enhance the ductility of the beams as well (ACI 544.1R 2002, Kwak *et al*. 2002, Choi, *et al*. 2007).

3. Test program

3.1 Test beams

The test program consisted of 9 beams shown in **Fig. 1** and detailed in **Table 1**. All of the beams had an overall length of 2000mm and width to depth dimensions of 150mm × 240mm (effective depth, d = 215 mm). To prevent anchorage failure, the beams were extended beyond the supports for a distance of 300mm. The design cylinder concrete compressive strength f'_c was 25 MPa. 2Φ-14 mm nominal diameter deformed steel bars were used for the longitudinal flexural reinforcement (A_s = 307.9 mm^2 and f_y = 420 MPa). The stirrups were fabricated from 8 mm nominal diameter plain round steel bars (A_s = 50.3 mm^2 and f_y = 280 MPa).

The steel cages were fabricated using 2Φ-8 mm bars as secondary reinforcement placed at the upper parts of the beams and the overlays. The beams were subjected to four-point loading with shear span a equals 562.5 mm as shown in **Fig. 1**. This loading system resulted in two shear spans subjected to constant shear with linearly variable bending moment and a middle span of 275mm subjected to pure bending moment of maximum con-

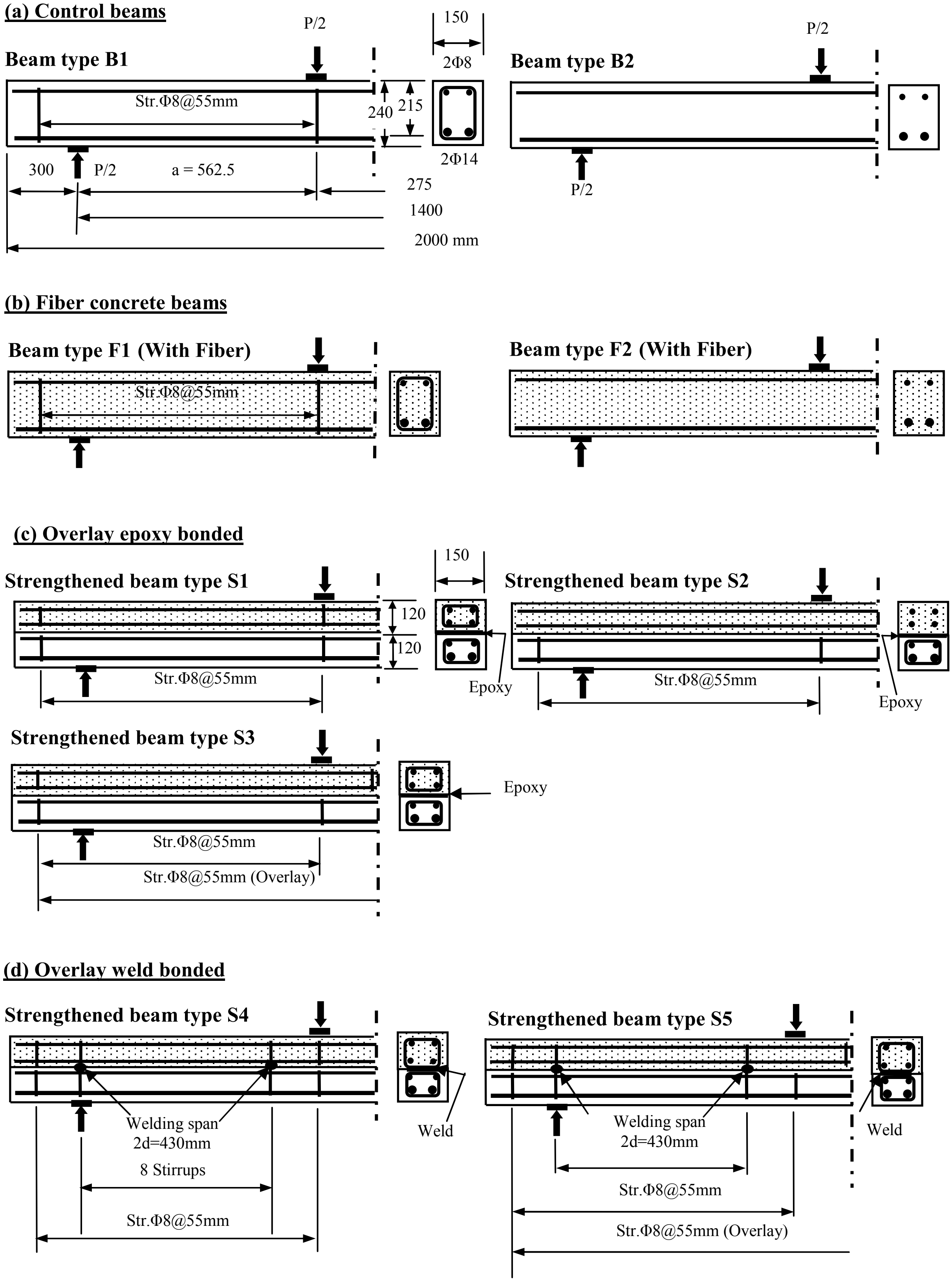

Fig. 1 Test beams.

stant value. All the beams had a shear span to depth ratio a/d = 2.6, i.e. the beams were shear-critical. Therefore, it was possible to study both the shear and flexural strengths of the test beams.

The identical monolithically cast control beam type B1 shown in **Fig. 1(a)** was included for comparison purposes. This beam was made with traditional concrete and has the final dimensions of the strengthened beams. This beam of tension-controlled section was designed to behave in a ductile manner and reach its full flexural capacity (ACI 2008). The shear failure was prevented in this beam by providing the shear span with Φ8 mm stirrups spaced at 55 mm. Beam type B2 was identical to beam type B1 except that it was not provided with shear stirrups, thus enabled the study of shear strength of the control beam without web reinforcement.

Beam types F1 and F2 shown in **Fig. 1(b)** were identical to beam types B1 and B2, respectively, except they were made from SFRC. The SFRC in the test program was made with the use of 1.5% by volume deformed steel fibers having an aspect ratio about 50, 30mm length and 0.28mm^2 cross section.

SFRC overlays were used in the test program to strengthen the beams. The inter-laminar shear failure was resisted, either chemically or mechanically as shown in **Fig. 1(c)** or **(d)**, respectively. The overlays were bonded to existing beams using either epoxy bonding agent as shown in **Fig. 1(c)**, or by welding together the stirrups of the existing beams to the stirrups of the overlays as shown in **Fig. 1(d)**. Strengthened beams were made of two parts that have final cross section (150 mm × 240 mm) equals to that used in the control beams and were provided with same flexural reinforcement (2Φ-14 mm). The lower parts represented existing beams in practice and had width to depth dimensions of 150 mm × 120 mm. The upper part was the SFRC overlays and has also width to depth dimensions of 150 mm × 120 mm.

The final dimensions and reinforcements of the strengthened beam types S1 and S2 were identical to beam types B1 and B2, respectively. The SFRC overlays were epoxy bonded to the existing beams in order to prevent inter-laminar shear failure. The shear spans in beam type S1 were provided with stirrups similar to beam type B1, however beam type S2 was without web reinforcement in order to examine the ability of SFRC overlay alone in resisting shear failure. Beam type S3 was similar to beam type S1, except the stirrups in this beam were extended also into the middle span which was not subjected to shear force. This detailing of stirrups envisaged to result in enhancement in the ductility of this beam. Strengthened beam types S4 and S5 were identical to beam types S1 and S3, respectively, except the inter-laminar shear failure was resisted mechanically in these beams. Stirrups were placed in the shear spans in beam type S4 and in the whole span for beam types S5. The stirrups of the overlays and of the existing beams were welded together in the support regions along the *2d* distance. It should be mentioned that in the mechanically bonded beams it was not possible to include a beam similar to beam type S2 because of the absence of stirrups in the overlay in such a beam!

Table 1 Details of test beams.

Beam Type	Cross Section b(mm)×h(mm)		Material		Reinforcement		Stirrups (Str.Φ8@55mm)		Bonding
	Existing	*Overlay*	*Existing*	*Overlay*	*Existing*	*Overlay*	*Existing*	*Overlay*	
B1	150×240	-	Concrete	-	2Φ14 (Bott.) & 2Φ8 (Top)	-	Shear span	-	-
B2							-		
F1			SFRC				Shear span		
F2							-		
S1	150×120	150×120	Concrete	SFRC		2Φ8 (Bott.) & 2Φ8 (Top)	Shear Span	Shear span	Epoxy
S2								-	
S3								Whole span	
S4								Shear span	Welding (8 Str.)
S5								Whole span	

3.2 Testing procedure and measurements

All of the test beams and the control cylinders were cast, compacted and were then stored under ambient conditions in the laboratory. The beams were whitewashed for easier identification of cracks during the loading sequences. Dial gauge of 0.05mm accuracy was used to measure the deflection at the mid-span. On completion of each increment of displacement the beams were inspected for cracks which were then measured using a crack micrometer of 0.01 mm accuracy.

4. Test results and discussion

4.1 Test results

Table 2 contains description of test results including maximum total load ($P_{meas.}$) and comparison between measured load for each beam type and measured load for the control beam type B1 ($P_{meas.}/P_{(B1)\ meas.}$).

Figure 2 shows the crack patterns at failure for all of the test beams. **Figure 3** shows the load deflection curves for groups of test beams with respect to control beam type B1. **Figure 3(a)** includes the curve for beam type B2 without stirrups. **Figure 3(b)** includes the curves for the SFRC beam types F1 and F2. **Figure 3(c)** includes the curves for the epoxy bonded strengthened beam types S1, S2 and S3. **Figure 3(d)** includes the curves for the weld bonded strengthened beam types S4 and S5. **Figure 4** shows the crack patterns at working load level for the control beam type B1, the epoxy bonded strengthened beam type S1 and the weld bonded strengthened beam type S4. The working load was taken to be approximately equal to 60% of the measured maximum load carrying capacity of the beams.

4.2 Discussion of results

Strengthening of beams using SFRC overlays is achieved due to the increase in the beam cross section. In the strengthened beams included in the test program, the cross section has been increased from 150mm×120mm in the original beams to 150mm×240mm in the strengthened beams. This increase in the cross section has resulted in an increase in the lever arm between the internal forces, i.e. the tension force "*T*" in the longitudinal tension reinforcement steel bars and the compression force "*C*" in the SFRC overlay. The strengthening techniques using overlays can be used in practice when architectural configurations allow casting overlays to the top of beams to increase their depths. In this case the strengthened beam acts as inverted beam. Also when rehabilitation of bridges is carried out, a new concrete layer is normally cast onto the deck top surface. This layer can be used to act as a concrete overlay. Off course it is not always necessary to double the beam depth to obtain the required strengthening.

Table 2 Test results.

Beam type	Description	Failure Type	f'_c (MPa)	$P_{meas.}$ (kN)	$P_{meas.}/P_{(B1)\ meas.}$
B1	Control beam	Flexure	25.2	113.5	1.00
B2	Without stirrups	Shear	26	75.7	0.67
F1	Fiber and stirrups	Flexure	26	126.8	1.11
F2	Fiber only	Flexure	25.5	120	1.06
S1	Epoxy bonded with shear span stirrups	Inter-laminar Shear	26.5 (Overlay 20.3)	115	1.01
S2	Epoxy bonded without stirrups	Inter-laminar Shear	25.3 (Overlay 19.8)	80	0.71
S3	Epoxy bonded with whole span stirrups	Inter-laminar Shear	25 (Overlay 19.8)	115	1.01
S4	Weld bonded with shear span stirrups	Flexure	25 (Overlay 21.5)	133	1.17
S5	Weld bonded with whole span stirrups	Flexure	25.7 (Overlay 21)	133.5	1.18

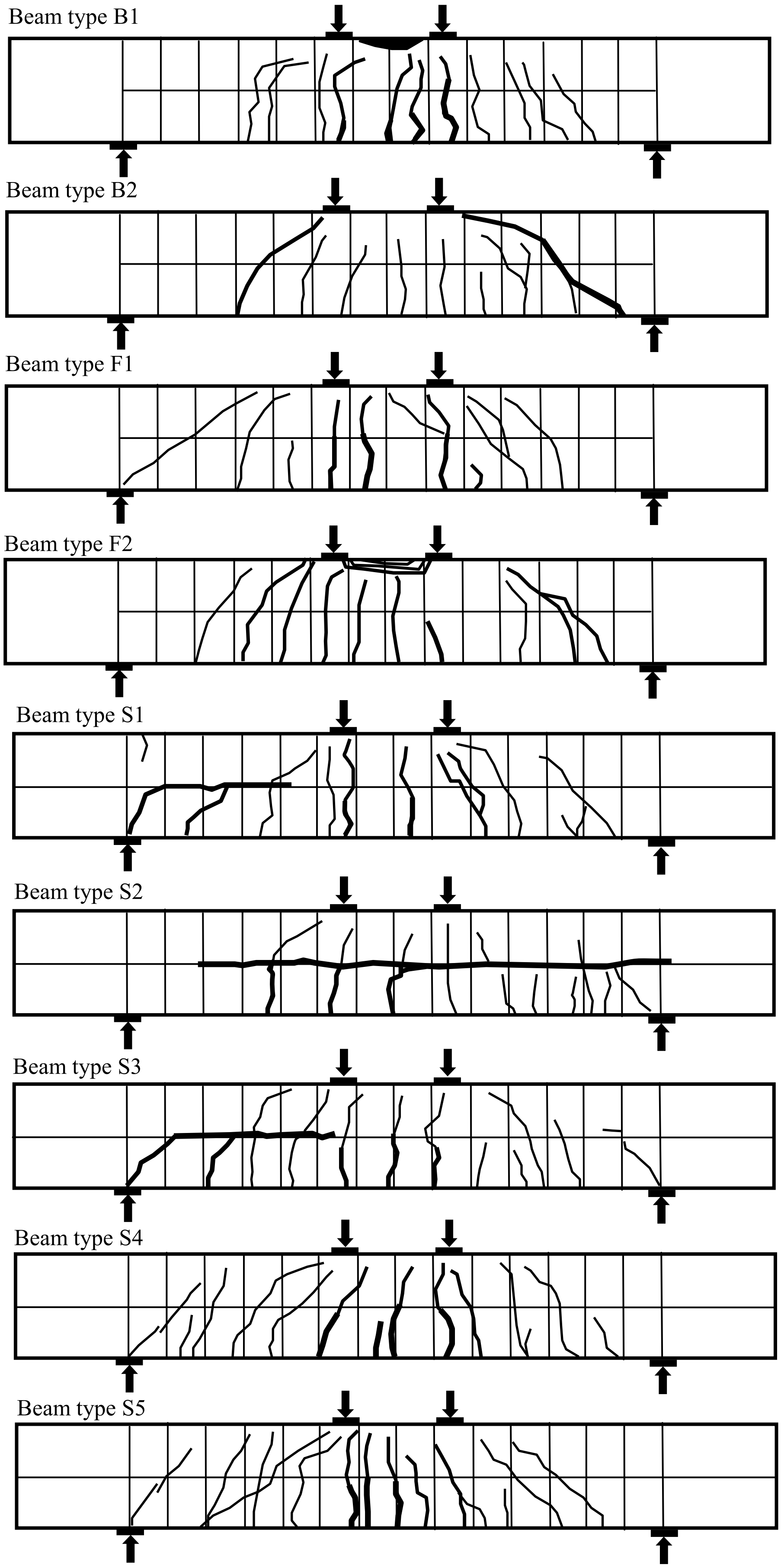

Fig. 2 Failure crack patterns for all test beams.

4.2.1 Failure mode

All of the test beams were tested to destruction until failure had occurred either prematurely or after reaching their full flexural capacities. Failure modes of the test beams can be best explained considering the failure crack patterns shown in **Fig. 2** as follows:

The traditionally designed and detailed control beam type B1 failed in a typical ductile flexure mode. Flexural cracks developed in the mid-span and in the shear spans close to the loading points. Diagonal cracks developed in the shear spans as an extension of the flexural cracks. Flexural cracks proliferated and widened under increasing loads. The shear reinforcement succeeded in preventing a premature shear failure. Failure of this beam eventually occurred by crushing of the compression concrete in the mid-span region where the maximum bending moment was experienced.

Beam type B2 failed in a brittle manner before reaching its full flexural capacity which is a characteristic of shear-critical beams without shear reinforcement. Flexural and diagonal cracks developed under increasing loads in the mid-span region and in the shear spans, respectively. Ultimate failure has occurred as one of the diagonal cracks widened and extended into the compression concrete just outside the loading plate.

The SFRC beam types F1 with shear reinforcement and F2 without shear reinforcement behaved in a ductile manner until failure had occurred after these beams reached their full flexural capacities. Similar to the control beam type B1, flexural and diagonal cracks occurred and proliferated in these two beams under increasing loads. The fibers in beam type F1 prevented the splitting of the compression concrete at failure which has occurred by widening of the flexural cracks in the mid-span region. However, in beam type F2 without shear reinforcement the compression concrete although remained intact had cracked at failure.

The epoxy bond used in the strengthened beam types S1, S2 and S3could not prevent the inter-laminar shear failure to occur in these beams. Flexural and diagonal cracks were developed first in the lower parts of the beams. The flexural cracks widened and extended into the SFRC overlay under increasing loading until they have reached the compression zone. At failure separation cracks occurred at the common interface surface between the original beam and the SFRC overlay.

Strengthened beam types S4 and S5 failed in a ductile manner after reaching their full flexural capacities. Flexural cracks developed in the mid-spans and in the shear spans close to the applied loading points. Diagonal cracks developed in the shear spans. Flexural and diagonal cracks proliferated, widened and extended from the original beams to the overlays under increasing loads. The stirrups in the SFRC overlay succeeded in preventing diagonal cracks from extending into the compression zone and thus prevented the diagonal failure. The fibers in concrete overlays prevented the splitting of the compression concrete at failure which has occurred by widening of the flexural cracks in the mid-span region. It is concluded that full interaction did develop between the overlays and the existing beams. Separation cracks at the common interface did not occur even after failure.

4.2.2 Load carrying capacity and ductility

The control beam type B1 which behaved in a typical flexural ductile manner reached its full load carrying capacity '$P_{meas.}$ = 113.5 kN' as shown in **Table 2**. The load carrying capacities of the other test beams were compared with the capacity of this beam in **Table 2**. **Figure 3** shows the load carrying capacities and ductilities of the test beams relative to beam type B1. For comparison purposes, the mid-span deflection was taken as a measurement of ductility.

As expected beam type B2 without shear reinforcement failed by diagonal tension cracking after reaching its ultimate load carrying capacity of 75.7 kN, i.e. 0.67 of type B1 capacity. The brittle behavior of this beam is shown in **Fig. 3(a)** where the deflection at failure was equal to 4.5mm compared to about 8mm for the ductile beam type B1.

SFRC beam types F1 and F2 reached their full carrying capacities which were equal to 126.8kN and 120 kN, i.e. 1.11 and 1.06 of type B1 capacity, respectively. The ductilities of these two beams were larger than the ductility obtained from beam type B1 as shown in **Fig. 3(b)**. It is known that fibers enhance both the tensile strength and ductility of concrete [ACI 544.1R 2002]. It is interesting to note that the shear failure did not occur in beam type F2 without shear reinforcement. The presence of the fibers contributed to preventing shear failure in this shear-critical beam (a/d = 2.6). The SFRC was able to resist alone the diagonal tension stresses due to shear. This confirms the conclusions of previous studies, especially for beams with high steel fiber content (Kwak *et al*. 2002; Choi, *et al*. 2007). The SFRC with enhanced structural behavioral characteristics has also resulted in an increase in the load carrying capacities compared to beam type B1.

The ultimate strengths of the epoxy bonded strengthened beam types S1, S2 and S3 were equal to 115 kN, 80 kN and 115 kN, i.e. 1.01, 0.71 and 1.01 of type B1 capacity, respectively. Thus, beam types S1 and S3 in which the SFRC overlays were provided with shear reinforcement were able to reach their full flexural capacities although ultimate failure occurred in these beams by inter-laminar shear at the common interface. However, beam type S2 in which the SFRC overlay was not provided with shear reinforcement failed prematurely by inter-laminar shear before reaching its full load carrying capacity. The ductilities obtained from all the strengthened beams, including type S2 were larger than the ductility of beam type B1 as shown in **Fig. 3(c)**. The fibers in the overlays have enhanced the ductile behavior of the concrete which has resulted in overall enhancement in the ductility of the strengthened beams. The increased deflection of beam type S2 can be attributed to the friction resistance developed at the common interface between the overlay and the original beam after the inter-laminar shear cracks have occurred at this interface. This resistance has allowed the beam to withstand additional deflection and even more loads as shown in **Fig. 3(c)**. It is concluded that strengthening of beams using SFRC overlays which were bonded to existing beams using epoxy bonding agent has succeeded in allowing the beams to reach same load and ductility levels obtained from the monolithically cast beam type B1. However, the overlays must be provided with shear reinforcement.

The ultimate strengths of the weld bonded strengthened beam types S4 and S5 were equal to 133 kN and 133.5 kN, i.e. 1.17 and 1.18 of type B1 capacity, respectively. Thus, these beams in which the stirrups in the SFRC overlays and the original beams were welded together in the support regions were able to reach their full flexural capacities. It should be mentioned that similar conclusion was reached previously for beams strengthened with traditional concrete overlays (Ziara 2000; Haldan and Ziara 2000). The obtained higher strengths can be attributed to the enhanced concrete strength due to the use of the steel fibers and the prevention of the inter-laminar shear failure using the mechanical bonding. The ductilities of these beams were in the same level as for beam type B1. Providing the whole overlay length with stirrups in beam type S5 has resulted in very little enhancement in the load carrying capacity and ductility of this beam compared to beam type S4 in which the shear spans only were provided with stirrups. It seems that the obtained ductilities in the two beams were controlled by the ductile behavior of the overlay concrete which was reinforced with steel fibers. It is concluded that strengthening of beams using SFRC overlays which are mechanically bonded to existing beams succeeds in allowing the beams to reach similar ductility and higher load levels compared to the identical monolithically cast beam type B1 and the epoxy bonded strengthened beams.

4.2.3 Serviceability

Serviceability limit state design methods require crack width and deflection not to exceed maximum limiting values at the working load levels (BS 1997, ACI 2008). In the case of the test beams the allowable deflection and the maximum crack width under working load conditions would be in the order of 5 mm and 0.3 mm, respectively. The maximum crack widths obtained from

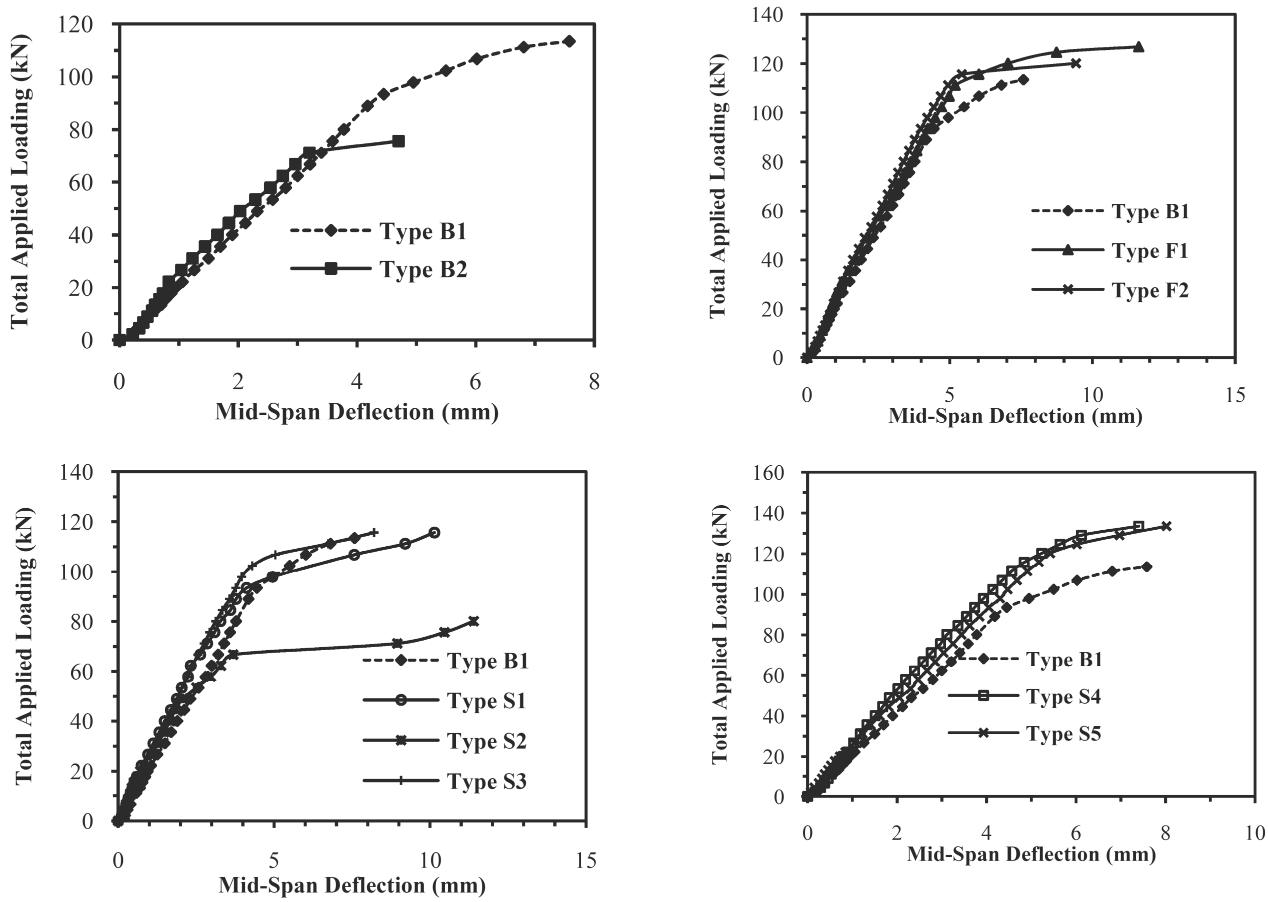

Fig. 3 Load deflection curves.

all of the strengthened beams at working load levels were significantly less than the 0.3 mm limit. The test results shown in **Fig. 3** indicate that all of the strengthened beams satisfied the deflection limit. The measured deflection at working load levels was in the order of 3 mm. It is worthwhile mentioning that the long-term deflections and maximum crack widths decrease with the use of steel fibers (Tan and Saha 2005).

It is also generally accepted that in traditional design approaches the deflections and widths of cracks satisfy the serviceability limit state requirements. The deflections and the maximum crack widths obtained from the strengthened beams at working load levels were in the same order compared to those obtained from the identical monolithically cast control beam type B1. **Figure 4** shows the crack patterns at working load level obtained from the control beam type B1. For comparison purposes **Fig. 4** also shows corresponding crack patterns obtained typically from epoxy bonded strengthened beam type S1 and the weld bonded strengthened beam type S4. The three beams have similar crack patterns since they have similar tension reinforcement and inter laminar shear separation in beam type S1 did not occur at working load level.

Therefore, it can be concluded that the strengthened beams have satisfied the serviceability limit state requirements with respect to maximum crack width and deflection.

5. Conclusions

(1) Existing beams can be strengthened or rehabilitated by casting steel fiber reinforced concrete (SFRC) overlays onto their upper compression surface. The overlays in the test program were bonded to the original beams using either epoxy bonding agent, or by welding the stirrups in the original beam to the stirrups in the overlay in only the support regions.

(2) The strengthened beams, regardless of the bonding method used, have satisfied the serviceability limit state requirements with respect to crack width and deflection. The maximum crack width and deflection values were at the same level obtained from the identical monolithically cast control beam.

(3) In general, weld bonded strengthened beams have achieved a better structural behavior in terms of load carrying capacity and failure mode compared to epoxy bonded beams.

(4) Weld bonded strengthened beams behaved in a typical flexural ductile manner and achieved similar ductility and higher load carrying levels compared to the monolithically cast control beam. This strengthening technique has prevented the occurrence of inter-laminar shear and diagonal tension failures in the strengthened beams which have acted as a single unit up to the flexural failure. Providing the entire overlay length with stirrups did not result in any significant enhancement in the structural behavior of the beam compared to the beam in which the stirrups were provided in only the shear spans. In this case the ductile behavior obtained from the concrete overlay reinforced with steel fibers has controlled the overall ductility of the weld bonded strengthened beams.

(5) Epoxy bonded strengthened beams have reached

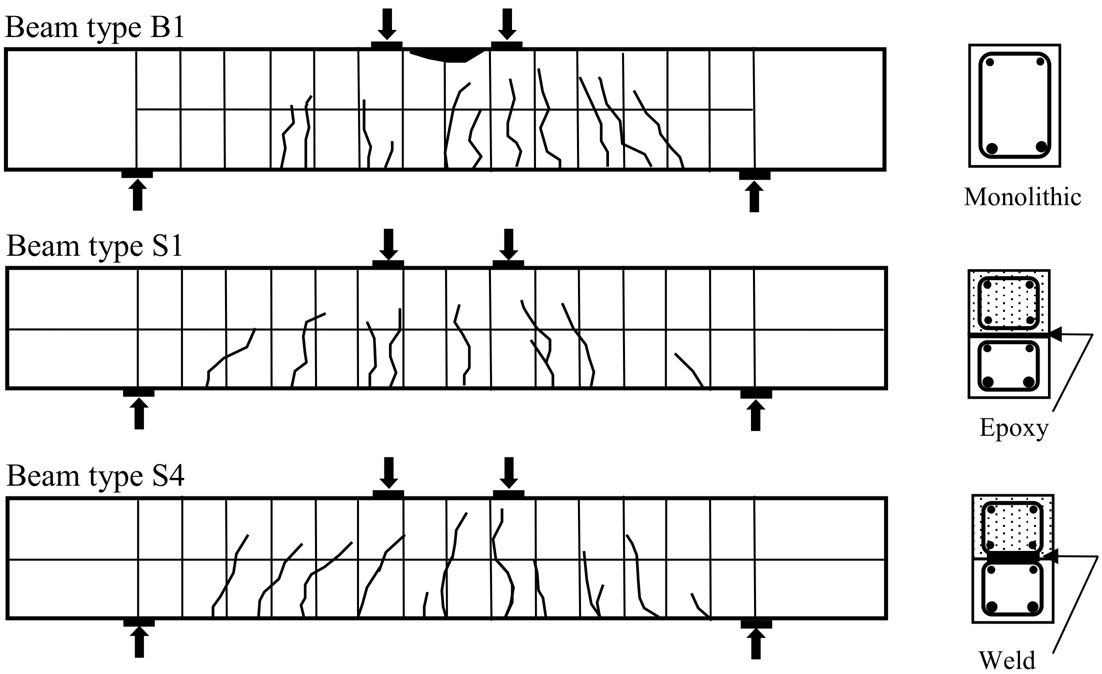

Fig. 4 Crack patterns at working load level for beam types B1, S1 and S4.

same load and ductility levels obtained from the identical monolithically cast control beam. However, the overlays must be provided with shear reinforcement to act as a stress transfer mechanism from the common interface to the overlays. The epoxy bonding could not prevent the inter-laminar shear failure to occur in these beams. At failure separation cracks occurred at the common interface between the overlays and the original beams.

(6) Weld bonding strengthening technique could have significant practical advantages over other comparable approaches to strengthening, since the stirrups over a significant part of the mid-span region of the original beam are not required to be welded to the stirrups in the overlay. In addition, the approach does not require any special treatment of the beam interface surface. In particular the adoption of this approach to the strengthening of beams e.g. bridge girders and beams which are subjected to additional loads in a renovation process, would appear to offer a practical solution. Improvements in strength can be obtained very quickly, if early strength cement was used in the concrete overlay. The use of SFRC overlays will provide excellent protection to the underlying and possibly damaged concrete. Another advantage of this strengthening technique is that the load carrying capacity of the strengthened beam is unrelated to the characteristics of the old concrete, which normally had been subjected to adverse loading and environmental conditions. In this case, the compression concrete will be located in the new SFRC overlay designed to have favorable characteristics.

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