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Effect of CFRP bond length on repair of composite steel girders with a cracked flange

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ABSTRACT: The results of an experimental study on the repair of artificially damaged steel-concrete composite beams repaired using adhesively bonded carbon fiber-reinforced polymer (CFRP) sheets are presented. Six beams composed of W150x22 steel sections with concrete slabs were tested in four-point bending to evaluate the bond length of CFRP sheets. Five beams had their tension flanges completely cut using a saw at mid span, to simulate a severe damage. CFRP sheets were then used to repair four damaged beams. The number of CFRP layers was kept constant, while the length of the CFRP repair varied from 8 to 97 percent of the span. The repair system was installed symmetrically around the centerline of the beam. Results showed that the induced damage has reduced flexural strength and stiffness by 60 and 54%, respectively. However, the strength of the CFRP-repaired beams ranged from 72 to 116% of the original undamaged strength, depending on bond length. Stiffness was generally recovered, but was not much affected by the bond length. Failure was initiated by debonding of CFRP layers.

1 INTRODUCTION

Carbon-FRP (CFRP) materials have a great potential in retrofitting steel structures, however, there are far less studies in this area, relative to studies and field applications on retrofitting concrete structures. Recent reviews of the available studies by Shaat et al. (2004) and Hollaway and Cadei (2002) have found that the focus of most studies has been on retrofit of naturally corroded girders or girders that are artificially damaged to simulate fatigue cracks or the effect of corrosion. Also, studies have looked into strengthening intact girders to increase their flexural strength and stiffness. Other studies looked into extending fatigue life of steel structures using CFRP (Jones and Civjan, 2003) as well as strengthening tubular steel columns, which are susceptible to buckling, using transverse and longitudinal CFRP layers (Shaat and Fam, 2006). Different techniques of artificial degradation of steel girders typically included notching the thickness of the tension flange (Tavakkolizadeh and Saadatmanesh, 2003), cutting part of the flange tips (Al-Saidy et al., 2004), complete removal of the tension flange at mid-span (Liu et al., 2001), or machining the tension flange to a reduced thickness throughout the entire span (Photiou et al., 2006).

In this paper, the tension flanges of steel beams connected to concrete slabs were completely saw-cut at mid-span. CFRP sheets of 91 GPa, adhesively bonded to both sides of the tension flange were used to restore the original strength and stiffness. The study investigated the effect of bond length on the repair effectiveness.

2 EXPERIMENTAL PROGRAM

Six steel-concrete composite beams were fabricated. The cross-section of the beams consists of W150x22 steel sections with a 75 mm thick and 465 mm wide concrete slabs cast on the upper steel flanges (Figure 1). The concrete slab was provided to simulate realistic composite girders.

The concrete slabs were designed such that failure would occur at the tension side prior to concrete crushing. The specimens included one control intact (undamaged) beam (B1) and five artificially damaged beams (B2 to B6) having their tension flanges completely cut at mid-span. Beams B1 and B2 were essentially control specimens. The remaining four specimens were repaired using CFRP sheets of different lengths. All four beams, however, had the same number of layers designed to provide a force equivalence index (ω) of 210%. This index was introduced to quantify the amount of FRP reinforcement on the basis of a relative strength of the flange, as follows:

$$\omega = \frac{\sum_{i=1}^{n}\left[A_{f_i} F_{fu_i}\right]}{A_{sf} F_y} \tag{1}$$

where F_{fui} and F_y are the ultimate tensile strength of FRP layer i, and the yield strength of steel, respectively. A_{f_i} and A_{sf} are the cross-sectional areas of FRP layer i and the steel flange, respectively.

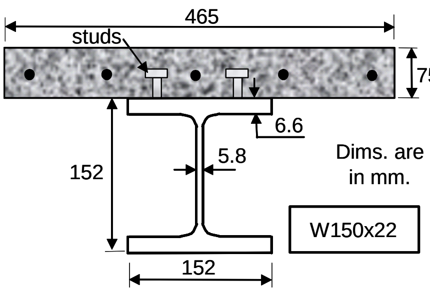

Figure 1. Cross-section of a typical test beam

The target index was higher than 100%, taking into account that failure was likely to occur due to debonding and not rupture of the FRP sheets. Table 1 summarizes all details of the beams, including total cross-sectional area of CFRP, force equivalence indices (ω), and the number of FRP layers and their dimensions, including the width and bonded length. The bonded length varied from 1900 mm to 150 mm for beams B3 to B6, respectively. In each beam, the layers were split almost equally between the upper and lower sides of the tension flanges. Also, a glass-FRP (GFRP) layer was placed between the CFRP and steel to prevent direct contact, which may lead to galvanic corrosion. The study simulated this system, even though this corrosion was unlikely in such a short term study, in order to account for any effect the lower modulus of GFRP may have on bond strength.

Table 1. Test Matrix

Beam	Area of CFRP (mm^2)	ω %age	Bonded sides of the tension flange	[number of layers x width x length] (mm) GFRP layer	CFRP layers
B1	Intact				
B2	Damaged				
B3	813	210	Lower	1 x 128 x 1900	1x128x1900 + 1x128x1850 + 1x128x1850
			Upper	2 x 64 x 1900	2x 64 x1900 + 2x 64 x1850 + 2x 46 x1850
B4	813	210	Lower	1 x 128 x 1900	1x128x1000 + 1x128x 950 + 1x128x 950
			Upper	2 x 64 x 1900	2x 64 x1000 + 2x 64 x 950 + 2x 46 x 950
B5	813	210	Lower	1 x 128 x 1900	1x128x 250 + 1x128x 200 + 1x128x 200
			Upper	2 x 64 x 1900	2x 64 x 250 + 2x 64 x 200 + 2x 46 x 200
B6	813	210	Lower	1 x 128 x 1900	1x128x 150 + 1x128x 145 + 1x128x 145
			Upper	2 x 64 x 1900	2x 64 x 150 + 2x 64 x 145 + 2x 46 x 145

2.1 *Materials*

W150x22 hot-rolled steel sections were used for the steel beams. The average yield strengths of the flange and the web were 386 and 406 MPa, respectively, based on coupon tests. Nelson headed steel studs, 41 mm long and 9.5 mm in diameter, with yield and ultimate strengths of 350 and 450 MPa, respectively, were welded to the steel sections to provide bond with the concrete slabs. The concrete mix for the slabs was designed for a 46 MPa compressive strength at 28 days. The average measured compressive strengths at the time of test was 50 MPa. Carbon and glass fibre fabrics were used in this study. The average tensile strengths and Young's modulli were 987 MPa and 90 GPa for the CFRP, and 336 MPa and 17.6 GPa for the GFRP, respectively, based on wet lay-up installation and coupons. A two-component epoxy resin was used and the mixing ratio was 2.9 to 1.0 by weight.

2.2 *Fabrication of Test Beams*

The steel beams were cut into 2030 mm long sections and Nelson studs were welded in pairs at a longitudinal spacing of 60 mm along the compression flange. After building the formwork for the concrete slab, a 150x150x5 mm welded wire mesh reinforcement was provided at mid-thickness and concrete was then pored, consolidated and cured. The steel tension flange of the w-section was completely cut at mid-span using a saw with a 1.4 mm thick blade, prior to casting the slab, in beams B3 to B6. In order to prepare the surface of the beams for FRP sheet installation, sandblasting was applied to the surface of the tension flange. The surface was cleaned with air pressure and wiped with acetone just before the FRP installation. FRP sheets were then applied within 24 hours.

2.3 *Test Setup and Instrumentation*

The specimens were tested as simple beams with a span of 1960 mm. The beams were monotonically loaded in four-point bending, as shown in Figure 2, with a distance of 400 mm between the applied loads. Short angle sections were bolted to the web from both sides and used as stiffeners under the loading and supporting points. Two linear potentiometers (LPs) were mounted on both sides of the beams, at mid-span, to measure the vertical deflection. Strains in the longitudinal direction were also measured. The load was applied to failure using stroke control at a rate of 0.75 mm/min.

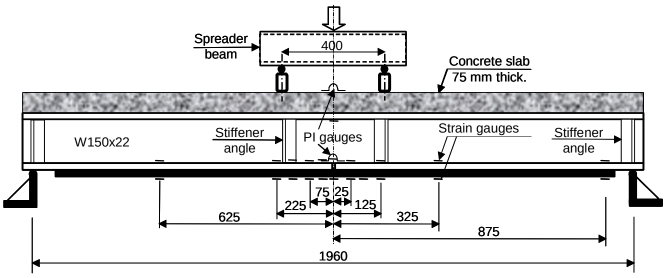

Figure 2. Test setup

3 **TEST RESULTS AND DISCUSSION**

A summary of test results of all beams is given in Table 2, in terms of the flexural stiffness and strength. Also given in Table 2, are the percentage differences between the stiffness and strength of all specimens, relative to the control intact specimen B1.

Figure 3 shows the load versus mid-span deflection of the control intact beam B1 and the control damaged beam B2. The figure shows that both the strength and stiffness have been severely degraded as a result of the complete cutting of the lower steel flange at mid-span. Table 2 reports 60 and 54 percent reductions in flexural strength and elastic stiffness, respectively. The failure mode of the intact control beam B1 was by yielding of the steel section, followed by concrete crushing. For the damaged beam B2, significant yielding, associated with crack propagation from the flange cut into the web, as shown in Figure 4(a), was observed. At the end of the test, the measured crack width and height in B2 were 14 and 67 mm, respectively. Because of the crack propagation, the neutral axis was significantly shifted upwards, inside the concrete slab. Therefore, tension cracks were observed at the bottom of the concrete slab. The concrete slab was eventually crushed at the compression side.

Table 2. Summary of test results

Specimen I.D.	Stiffness (kN/mm)	%age difference	Maximum load (kN)	%age difference
B1	33.8	---	357	---
B2	15.4	-54	144	-60
B3	33.3	-1	415	+16
B4	34.5	+2	353	0
B5	31.4	-7	319	-10
B6	31.0	-8	256	-28

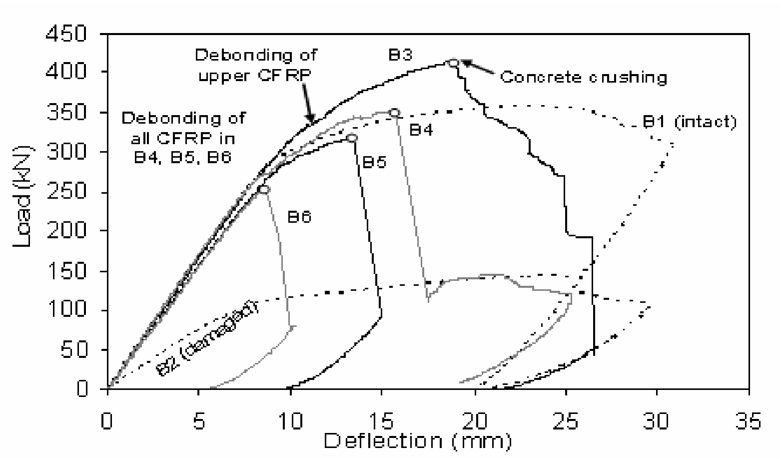

Figure 3. Load-deflection responses of test beams

Figure 3 shows that B3 with a 1900 mm bond length (97% of the span) has fully recovered the flexural strength and stiffness of the intact control beam and even exceeded them. The total bond length for B4 was 1000 mm, whereas for B5 and B6, the total bond lengths were 250 and 150 mm, respectively (i.e. completely within the 400 mm constant moment zone). Figure 5(a) shows the variation of the ultimate load, normalized to that of control beam B1, with the bond length, normalized to the span length (L_{sheet}/Span). A bi-linear trend is observed, with the steep part of the curve being within the constant moment zone, while the shallow part being within the shear span. Generally, the behaviour clearly shows a reduction in ultimate load with shortening the bond length of CFRP. As given in Table 2, B3 with CFRP sheets bonded along 97 percent of the span achieved a 16 percent higher strength than the intact beam B1. Beam B4, with a 1000 mm bond length (51 percent of span) achieved a load equals to that of the control beam B1. Beams B5 and B6 with bond lengths of 12.8 and 7.7 percent of the span, respectively, failed at lower loads, 10 and 28 percent lower than B1, respectively. Table 2 also shows that unlike the flexural strength recovery, the elastic stiffness of the intact beam B1 has generally been recovered in all beams (B3 to B6), regardless of the bond length, with little variations. Figure 3 also shows the effect of the bond length on the deflection of the repaired beams. The deflection at ultimate and the length of the nonlinear part of the curve reduced as the bond length was decreased. Figure 5(a) suggests that the optimum total bond length would be about 55 percent of the span for this particular beam configuration and force equivalence index.

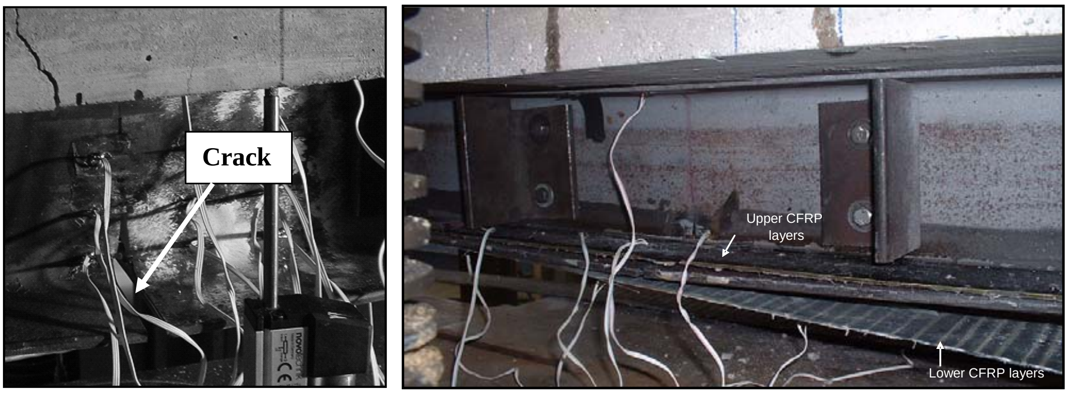

Figure 4. Failure modes (a), left: crack propagation in B2 and (b), right: FRP debonding

Figure 5(b) shows the variation of the maximum strain reached in the CFRP sheets at the lower side of the flange, at the location of the slit, with the bond length of CFRP. The figure shows a bilinear trend analogous to that shown in Figure 5(a). The first part, from 0 to 200 mm, represents the variation within the constant moment zone in absence of shear, whereas the second part, beyond 200 mm, represents the effect of the bond length on the maximum achieved strain within the shear span, in presence of both bending and shear. The first line when extended down, passes through the origin (point “a”), which represents a case without CFRP sheets, essentially beam B2. The line may also be extended up, to intercept a ceiling, which is the rupture strain of CFRP (point “b”). Point “b” represents the optimum bond length (i.e. achieving rupture and debonding simultaneously), required for repair under a constant bending, which is 180 mm in this case. The second part of the curve clearly has a shallower slope due to the presence of additional high shear stresses within the shear span of the beam. This necessitates a longer bond length to achieve a certain increment in strain, than within a constant moment region. By extending the second line to intercept the rupture strain ceiling at point “c”, the optimum bond length required to achieve rupture of CFRP can be obtained. In this case, it is about 1400 mm from mid span, which is clearly longer than the span of the beams tested in this study.

Failure of B3 started by debonding at the upper side of the flange, followed by concrete crushing at a higher load and then debonding at the lower side of the flange, at a reduced load. In beams B4 to B6, debonding occurred at both sides of the flange, followed by a very large drop in the load. A typical debonding failure is shown in Figure 4(b).

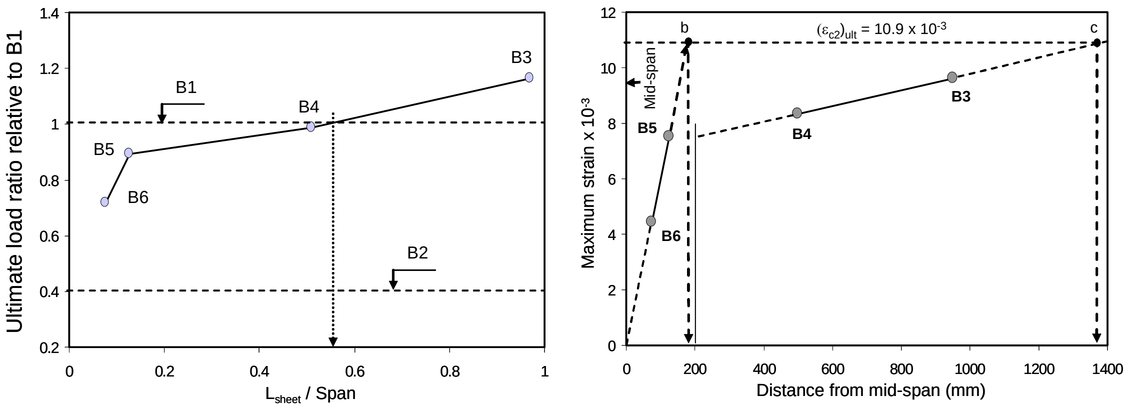

Figure 5. (a), left: Variation of ultimate strength with bond length and (b) right: maximum strain achieved with bond length

4 CONCLUSIONS

This study has shown that adhesively-bonded CFRP sheets can effectively be used to repair damaged steel-concrete composite beams. The following conclusions are drawn:

1. A complete cut in the tension flange at mid span has severely reduced the flexural strength and stiffness of the composite girder by 60 and 54 percent, respectively.
2. The CFRP-repaired girder with sheets covering 97% of the span recovered the original stiffness and the undamaged original strength was exceeded by 16 percent.
3. CFRP-repaired beams failed consistently by debonding of CFRP sheets
4. The bond length of CFRP sheets affected flexural strength significantly but not the elastic stiffness. The stiffness of the beams with CFRP sheets covering 8 to 97 percent of the span was 93 to 102 percent of the original stiffness, while their strengths ranged from 72 to 116 percent of the original strength.
5. Based on CFRP strain measurements at ultimate, it has been estimated that the minimum bond lengths, on one side of the crack, required to achieve rupture instead of debonding, for this particaluar repair system, would be 180 mm and 1400 mm, for the cases of pure bending and combined bending and shear, respectively.

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