

Flexural Performance of Steel Girders Retrofitted Using CFRP Materials

K. Galal, M.ASCE¹; H. M. Seif EIDin²; and L. Tirca³

Abstract: There are many existing deteriorated steel bridges that need to be retrofitted. This paper investigates the effectiveness of using carbon fiber-reinforced (CFRP) composite systems in retrofitting deteriorated steel beams. A total of 13 medium-scale steel I-beams with a span of 1.6 m was tested in a four-point bending setup. The tested beams were divided into four groups according to their studied parameter. Group 1 consists of four unretrofitted beams with different percentages of artificial deterioration to simulate corrosion in the bottom flange with the aim to investigate their behavior and to determine the residual flexural capacity. The other three groups have deteriorated steel beams that were retrofitted with different CFRP systems with the aim to evaluate the effectiveness of the proposed retrofit schemes. Four deteriorated beams were retrofitted with CFRP sheets bonded to the tension flange and were tested in Group 2. Group 3 consists of two deteriorated steel beams that were retrofitted with CFRP plates externally bonded to the bottom flange of the tested beams. Group 4 consists of three beams retrofitted using an unbonded CFRP sheet attached to two ductile anchorage systems at the beams' ends. The study shows that steel beams retrofitted with external bonded CFRP systems experienced limited ductility upon the failure of the CFRP either by debonding or rupture at higher load capacities than that of the unretrofitted beams. The proposed anchorage system could increase the strength of the deteriorated beam, behave in a ductile manner, and eliminate the early peel off of the CFRP sheet. DOI: [10.1061/\(ASCE\)CC.1943-5614.0000264](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000264). © 2012 American Society of Civil Engineers.

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Introduction

A large number of existing steel bridges are in need for replacement or strengthening. The replacement alternative would involve high costs and closure of traffic for long periods or, at least, substantial interference with the normal traffic, whereas the strengthening alternative appears to be more attractive. Corrosion is a significant cause of deterioration in steel bridge structures. An experimental and analytical study of simulated deteriorated steel beams was conducted by Lindt et al. (2005). The purpose was to develop a model that can provide structural engineers with a simplified approach for computing the reduced capacity of different steel beam sections. Rahgozar (2009) carried out an analytical study on two different deteriorated I-beam section models to obtain the minimum curves for reliable estimation of the remaining shear and moment capacities. The analytical results show that for uniform thickness losses equal to 50%, the remaining ultimate load capacity is only 30%, and attributable to corrosion, the member can pass easily from one class of section to another.

A traditional retrofit method that aims to increase the flexural and shear capacities of corroded or structurally deficient steel

girders would involve attaching steel cover plates to these members by means of bolts or weld. Several disadvantages are associated with this method, such as (1) the procedure is labor intensive and time consuming; (2) installation of externally bonded millimeters wide CFRP plate may interrupt traffic for a period of time; (3) it requires drilling and extensive lap splice detailing; (4) there is a potential for weld fatigue cracking at the cover plate ends and regions of high stress concentrations that may develop near the bolts; and (5) there will be an increase in the weight of the retrofitted members.

Consequently, in recent years, extensive research has focused on strengthening of steel bridge members by using fiber-reinforced polymer (FRP) composite materials, which are applied either by a bonded or an unbonded technique. The advantages of using these composites are the following: high strength-to-weight ratio, high fatigue resistance, ease and speed in strengthening work intervention.

One of the shortcomings of using carbon fiber-reinforced polymer (CFRP) composites for retrofitting structural steel elements is galvanic corrosion. Galvanic corrosion is the corrosion that results when two metals with different electrochemical potential are in contact with an electrolyte (e.g., water). Such galvanic-induced corrosion would occur in the case of applying CFRP composites on steel members. On the basis of research conducted by West (2001), it was concluded that isolating the CFRP from the steel surface by using an adhesive layer is sufficient to prevent the formation of galvanic corrosion. Dawood (2005) found that mixing a small amount of glass beads into 1 mm adhesive thickness between the steel and the CFRP material slows the process of galvanic corrosion significantly.

Previous research work on retrofitting steel structures focused on applications of adhesive-bonded FRP sheets or plates. One of the major disadvantages of such technique is its susceptibility to sudden failure attributable to FRP debonding. The bond between FRP and steel members was identified as one of the main variables

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that would significantly affect the behavior of steel members retrofitted with bonded FRP systems. As such, several researchers have conducted experimental and analytical studies that focused on the bond behavior between the FRP and steel plates or steel beams.

There are several factors that characterize the bond performance between the FRP and the steel, such as adhesive type, adhesive thickness, surface preparation, bond length, and the FRP type. Buyukozturk et al. (2004) reported a review of the progress achieved in the area of strengthening steel members using FRP composites, and Stratford and Cadei (2006) described a method for modeling adhesion between an FRP plate and a steel beam. An experimental study conducted to emphasize the behavior of steel plates strengthened with CFRP plates under axial loads was carried out by Fawzia et al. (2006). In addition to the experimental program, a nonlinear finite element analysis was conducted to predict the failure modes and the load capacity of the strengthened steel members. Nozaka et al. (2001) defined the development length of the FRP externally bonded to steel member as the shortest bond length engaging the largest possible strength of the FRP composites. A simplified equation for the development length was suggested by Frauenberger et al. (2003):

$$L_d = \frac{f_{frp} t_{frp}}{\alpha \mu_{max}} \quad (1)$$

where L_d = the effective development length; f_{frp} is the limit stress of the FRP; t_{frp} = the FRP thickness; μ_{max} = the debonding stress; and α = a factor of safety.

Full scale experimental tests on aged highway bridge girders were conducted by Gillespie et al. (1996). Four deteriorated steel girders were taken from a 55-years-old demolished highway bridge. After the four girders were tested under three-point-loading test to measure the stiffness loss, two of the four deteriorated steel girders were rehabilitated by using externally bonded pultruded carbon fiber applied to the tension flange and subsequently tested until failure. The results showed an average of 25% increase in stiffness and 100% increase in the ultimate load-carrying capacity. Liu et al. (2001) investigated the behavior of steel bridge girders strengthened with FRP composite laminated strips. Four $W12 \times 14$ girders with clear span of 96 in. (2,400 mm) were tested under three-point bending configuration. The first beam was tested as a control beam whereas the second beam had a 4-in. (100 mm)-wide notch in the tension flange to simulate the severe loss of section attributable to corrosion. The third and fourth beams were upgraded with 3.94-in. (100 mm)-wide CFRP laminates that covered the full length of the beam and one quarter of the beam's length, respectively. On the basis of the experimental and analytical results, it was found that an increase in stiffness and plastic load capacity of deteriorated steel girders can be achieved when CFRP laminates are bonded to the tension flange. Patnaik et al. (2008) conducted an experimental study to examine the behavior of steel beams strengthened with CFRP laminate strips to increase their flexural capacity. A four-point-loading steel beam with a clear span of 3.15 m was tested. It was reported that by adding CFRP laminate strips, the ultimate flexural capacity was considerably improved by 30%. Furthermore, Hollaway et al. (2006) discussed the advantages and limitations of bonding FRP composites to steel structural members. Ultra high and high stiffness CFRP composite materials were used in that study to reinforce artificially degraded structural steel members, and two test techniques, namely, double shear and flexural test, were performed. Meanwhile, a similar experimental study was conducted by Photiou et al. (2006), who strengthened simulated degraded steel beams by using CFRP plates.

From the above, it could be seen that there is a need to evaluate the effectiveness of FRP systems in enhancing the flexural

performance of deteriorated steel beams. The current study will evaluate three retrofit schemes, namely (1) adhesive-bonded CFRP sheets, (2) adhesive-bonded CFRP plates, and (3) unbonded CFRP sheets with two end ductile anchorage systems.

Experimental Program

Test Matrix

A series of 13 medium-scale steel beams was tested to evaluate the behavior of the deteriorated and FRP-retrofitted steel girders. The deteriorated beams simulate corroded beams with area loss in its bottom flange. The steel beams had a section type $W150 \times 30$ and were tested in four-point bending with a span of 1.60 m between supports. The beams were loaded in a displacement-control mode to track the full load-deflection response of the tested beams. The tested beams were divided into four groups according to their studied parameter. Group 1 consists of four unretrofitted beams with different percentages of artificial deterioration to simulate corrosion in the bottom flange with the aim to investigate their behavior and to determine the residual flexural capacity. The other three groups have deteriorated steel beams that were retrofitted with different CFRP systems with the aim to evaluate the effectiveness of the proposed retrofit schemes. Four deteriorated beams were retrofitted with CFRP sheets bonded to the tension flange and were tested in Group 2. Group 3 consists of two deteriorated steel beams that were retrofitted with CFRP plates externally bonded to the bottom flange of the tested beams. Group 4 consists of three beams retrofitted using unbonded CFRP sheets attached to two ductile anchorage systems at the beams' ends. Table 1 shows the test matrix of the tested beams. The details of the 13 beams are described in the following subsections according to their group.

As shown in Table 1, all the tested beams except the control one (BFO) have induced area losses in their bottom flange to simulate deterioration in the flexural capacity of steel beams attributable to corrosion. Two different damage levels of 33 and 50% area reduction of the tension flange were considered in this study. To avoid local web yielding and web crippling during the tests, the beams were designed to have transverse stiffeners of 100 mm depth on each side of the web at the location of the applied loads and supports reactions, as specified by the CSA S16-09 (2009). Fig. 1 shows the test setup.

Materials

Properties of the Studied Steel Beams

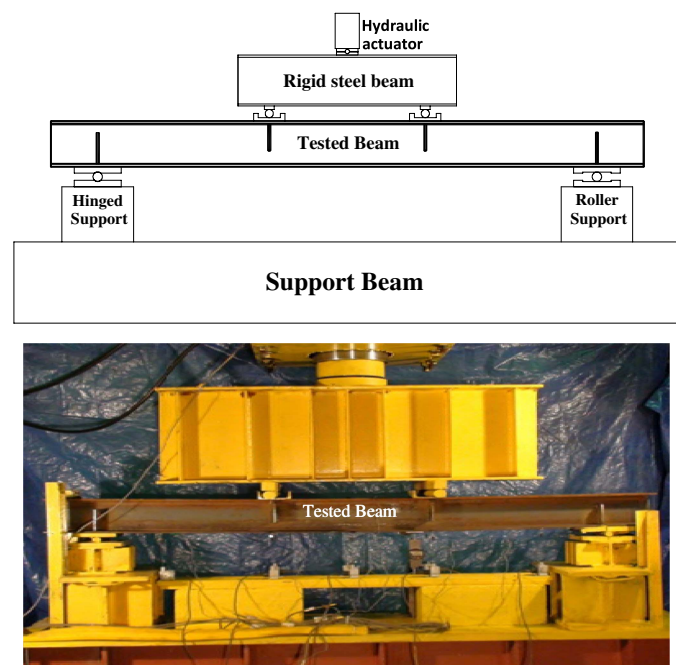
A uniaxial tension test was performed on three dog-bone specimens according to ASTM E8M-1 (2007). The specimens were taken from the same patch of the tested steel I-beams. Among the three tested specimens, one specimen was cut from the web, whereas the other two were cut from the top and bottom flange, respectively. The average yield strength F_y of 310 MPa and a modulus of elasticity E of 198.7 GPa were obtained experimentally.

CFRP Composite Materials

Two types of CFRP composite materials were used in this study, namely Tyfo SCH-11UP and Tyfo UC (Fyfe 2009). The composite material Tyfo SCH-11UP is a unidirectional carbon fiber sheet with a typical CFRP laminate thickness of 0.27 mm and a tensile modulus of 102 GPa. The Tyfo UC composite is a unidirectional carbon fiber plate with a higher tensile modulus equal to 155 GPa and a thickness of 1.4 mm. To prevent any galvanic-induced reaction between the steel surface and the CFRP material, one layer of GFRP sheet, Tyfo EP-DB (Fyfe 2009), was used between the CFRP and the bottom flange of the tested steel beams. Two types of adhesives were used in this experimental program: the Tyfo S and Tyfo MB-3

Table 1. Test Matrix

Group	Beams	ID	Area losses mm ²	Retrofit material CFRP	Epoxy material	Retrofit method
G1	B1	BFO	None	—	—	—
	B2	BF-H0.33	8 – 6.35 mm dia. holes	—	—	—
	B3	BF-N0.33	2 – (25.4 × 450) mm ² Notch	—	—	—
	B4	BF-N0.50	2 – (38.1 × 450) mm ² Notch	—	—	—
G2	B5	BF-H0.33-F1(5)-B1	8 – 6.35 mm dia. holes	SCH-11UP (5 layers)	Tyfo S	Fully bonded
	B6	BF-N0.50-F1(5)-B1	2 – (38.1 × 450) mm ² Notch	SCH-11UP (5 layers)	Tyfo S	Fully bonded
	B7	BF-H0.33-F1(5)-B2	8 – 6.35 mm dia. holes	SCH-11UP (5 layers)	MB-3	Fully bonded
	B8	BF-N0.50-F1(5)-B2	2 – (38.1 × 450) mm ² Notch	SCH-11UP (5 layers)	MB-3	Fully bonded
G3	B9	BF-H0.33-F2(1)-B2	8 – 6.35 mm dia. holes	UC strip (1strip 1.4 mm)	MB-3	Fully bonded
	B10	BF-H0.33-F2(1)-B2w	8 – 6.35 mm dia. holes	UC strip (1strip 1.4 mm)	MB-3	Fully bonded
G4	B11	BF-H0.33-F1(1)-A1D1	8 – 6.35 mm dia. holes	SCH-11UP (1 layers)	—	Ductile anchorage
	B12	BF-H0.33-F1(1)-A2D1	8 – 6.35 mm dia. holes	SCH-11UP (1 layers)	—	Ductile anchorage
	B13	BF-H0.33-F1(5)-A1D2	8 – 6.35 mm dia. holes	SCH-11UP (5 layers)	—	Ductile anchorage

**Fig. 1.** Test setup

(Fyfe 2009). The Tyfo S adhesive is a two-component saturant epoxy that can be combined with the Tyfo SCH-11UP sheets to provide a wet-layup composite system, whereas the Tyfo MB-3 is a two-component viscous epoxy adhesive that is designed to bond the CFRP laminates to the steel surface. Tables 2 and 3 provide the mechanical properties of the used CFRP composites and epoxies, respectively.

To establish the development length of the CFRP materials (sheets and plates) using Eq. (1), a series of uniaxial tension tests was conducted. These tests were conducted on CFRP sheets and plates that were bonded using two types of adhesives on steel plates that were cut from the flanges of the tested beams. On the basis of test results, the debonding stress between the CFRP sheets and the deteriorated steel beams is 1.4 and 3.2 MPa when using saturant and viscous epoxy, respectively. Whereas the debonding stress between the CFRP plate and the deteriorated steel beams is 4.8 MPa when using viscous epoxy. The debonding load is a function of both the debonding stress and the contact area between the retrofitted beams and the CFRP composite material.

Table 2. Properties of Used CFRP Materials (Fyfe 2009)

	CFRP sheets	CFRP plates
Width	150 mm	50.8 mm
Thickness / layer	0.27 mm	1.4 mm
Tensile strength	1062 MPa	2790 MPa
Tensile modulus	102 GPa	155 GPa
Ultimate elongation	1.05%	1.8%

Table 3. Properties of Epoxies Used (Fyfe 2009)

	Saturant epoxy	Viscous epoxy
Tensile strength	72.4 MPa	16.7 MPa
Tensile modulus	3.18 GPa	—
Ultimate elongation	5.0%	2.7%
Compressive strength	—	28.2 MPa
Thickness / layer	—	0.25 mm

Description of Group 1 Beams

The four beams of Group 1 were prepared to investigate the behavior of steel beams that could have deterioration either localized at the midspan section or spread along the constant moment zone (approximately along the middle one-third of the span). Among the four beams, the BFO beam was considered to be the control beam, whereas the other three steel beams BF-H0.33, BF-N0.33, and BF-N0.50 represent beams with different levels and spread of deterioration. Beams BF-N0.33 and BF-N0.50 have a flange area reduction of 33 and 50%, respectively, in the form of longitudinal notches with a length of 450 mm located in the midspan zone. Its purpose is to simulate beams with uniformly distributed corrosion developed at the tension flange of the beam (bottom flange in this case). The BF-H0.33 beam has a 33% area reduction in the tension flange in form of holes that are localized at the midspan section. It was prepared to simulate a local deterioration attributable to corrosion at midspan. The designation N0.33 and N0.5 represents beams with notches that have a reduction of 33 and 50%, respectively, in the bottom flange area in the middle one-third of the span, whereas the designation H0.33 represents beams with 33% bottom flange area reduction in form of holes at the midspan section. Fig. 2 shows perspective that illustrates the types and levels of induced deterioration in the bottom flange.

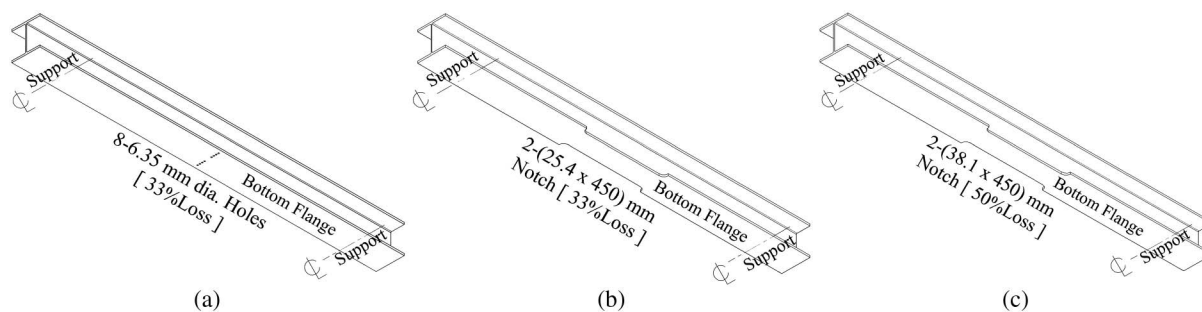


Fig. 2. Two types of induced deterioration in the middle flange: (a) holes at the midspan section with 33% loss in bottom flange area; (b) notches in the middle one-third of the beam 33% in the bottom flange area; (c) notches in the middle one-third of the beam 50% in the bottom flange area

Description of Group 2 Beams

In this group, four deteriorated steel beams were tested when retrofitted with CFRP sheets externally bonded below the bottom flange. The variables among the beams of this group were the level and type of deterioration and the type of adhesive bonding the CFRP to steel beam. The two beams BF-H0.33-F1(5)-B1 and BF-N0.50-F1(5)-B1 were retrofitted with five layers of 150-mm-wide CFRP sheets that were applied underneath the bottom flange using a saturating epoxy as both a resin and adhesive in a conventional wet-layup system. The first beam had 33% bottom flange area loss as holes localized in the midspan section whereas the second beam had 50% bottom flange area loss spread in the middle one-third of the beam's span. In the other two beams, BF-H0.33-F1(5)-B2 and BF-N0.50-F1(5)-B2, the same number and width of CFRP sheets of the first two beams were used, i.e., five layers of 150-mm-wide CFRP sheets, but the difference is that a viscous epoxy was used as a resin for the CFRP sheets and as an adhesive between the CFRP sheets and the steel beam. It should be noted that a layer of GFRP sheet was used at the interface between the CFRP layers and the bottom flange for the four tested beams to avoid galvanic corrosion.

Description of Group 3 Beams

The aim of this group is to study the behavior of deteriorated steel beams that are retrofitted using CFRP plates. One layer of 50-mm-wide CFRP plate was externally bonded to the bottom side of the tension flange of two beams, BF-H0.33-F2(1)-B2 and BF-H0.33-F2(1)-B2w, using viscous epoxy. One layer of GFRP system was used as dielectric barrier between the CFRP plate and the bottom of the tension flange to prevent the galvanic corrosion reaction. Beam BF-H0.33-F2(1)-B2w is different than beam BF-H0.33-F2(1)-B2 in that it has one layer of 200-mm-wide CFRP sheet that is wrapped around the longitudinal CFRP plate and the bottom flange at both ends of the beam. The aim is to evaluate the efficiency of this wrapping system on delaying and/or eliminating the debonding of the CFRP plates and thus enhancing the flexural performance of the retrofitted steel beams.

Description of Group 4 Beams

As mentioned earlier, previous research on the bond between FRP and steel identified several parameters that influence the FRP-steel bond behavior. Premature debonding of the CFRP materials is one of the most common failure modes, and it could occur before the stress in the CFRP composite material reaches its tensile strength.

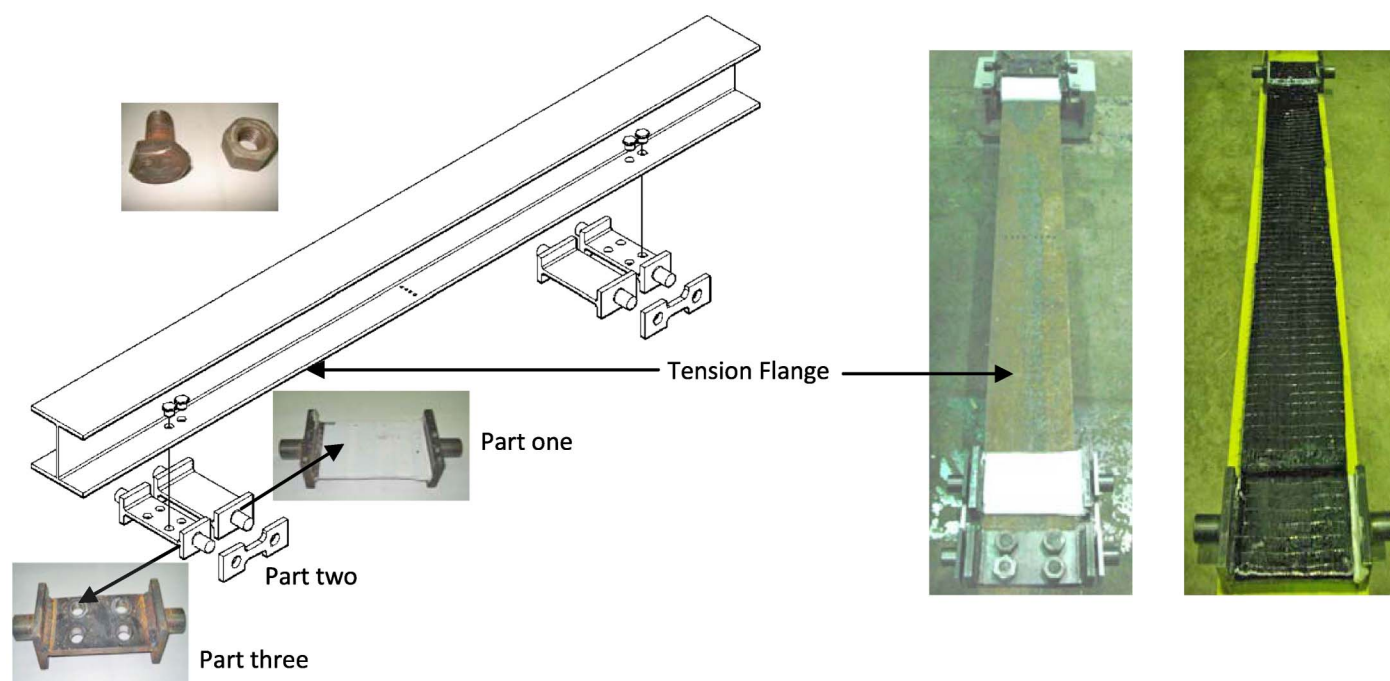


Fig. 3. Ductile anchorage system used in Group 4 beams

The proposed retrofitting scheme used in this group is shown in Fig. 3. It consists of an unbonded CFRP sheet that is attached to two ductile anchorage systems, one at each end of the beam. As can be seen in the figure, the ductile anchor system consists of three parts. The CFRP sheets that are wrapped around Part 1 are connected to Part 3, which is rigidly attached to the steel beam by means of ductile link members designated as Part 2. In such a system, the steel link members that are designed as a conventional tensile dog-bone specimen will always be in axial tension without permitting any moments to be developed in them. To reduce the stress concentration and to avoid the rupture of the FRP sheets at its connection with the anchorage system, Part 1 of the anchorage system was designed with rounded corners. Also, a duct tape was wrapped around Part 1 in order to separate the contact between Part 1 and the CFRP sheets such that this part could be reused in following tests. The main objective of this retrofit scheme is to utilize a higher level of strain of the CFRP sheets by avoiding premature debonding through transferring the tensile stresses developed upon deformation of the loaded beam to the two ductile anchorage systems, located at the beam ends. A similar strengthening technique using ductile anchorage systems and FRP sheets was used for enhancing the flexural performance of reinforced concrete beams (Galal and Mofidi 2009). In such a hybrid system, two design approaches could be adopted: (1) the ductile link members are designed to yield before the rupture of the CFRP sheets occurs, or (2) the CFRP sheets are designed to reach their full capacity,

i.e., rupture, before yielding of the ductile link members. The Appendix provides more information about the details and design of ductile anchorage system used in for beams tested in Group 4. Several detailing aspects had to be considered in the construction of the ductile anchorage system.

The three beams of this group, BF-H0.33-F1(1)-A1D1, BF-H0.33-F1(1)-A2D1, and BF-H0.33-F1(5)-A2D2, were designed to evaluate the efficiency of the hybrid CFRP/ductile anchorage system in enhancing the flexural capacity of deteriorated steel beams. The three beams had a 33% loss in the bottom flange area that is localized as holes at the midspan section of the beam. In this group, two details of the connection between the CFRP and Part 1 of the ductile anchorage system and two design approaches of the ductile anchorage system were examined. Beams BF-H0.33-F1(1)-A1D1 and BF-H0.33-F1(1)-A2D1 were retrofitted using one layer of CFRP sheet that is wrapped around Part 1 of the anchorage system with an overlap of 350 mm (end Detail 1), as can be seen in Fig. 4. The length of the overlap was determined based on several tension tests conducted on the CFRP-Part 1 attachment with several overlapping lengths, where the criteria was to ensure that the system develops the full capacity of the CFRP sheet, i.e., rupture, without debonding. Another detail, Detail 2 as shown in Fig. 5, of the connection between the CFRP sheet and Part 1 of the anchorage system was examined in beam BF-H0.33-F1(5)-A2D2. In that detail, five layers of the CFRP sheets are wrapped around Part 1 of the anchorage system in an attempt to reduce the overlap length of the

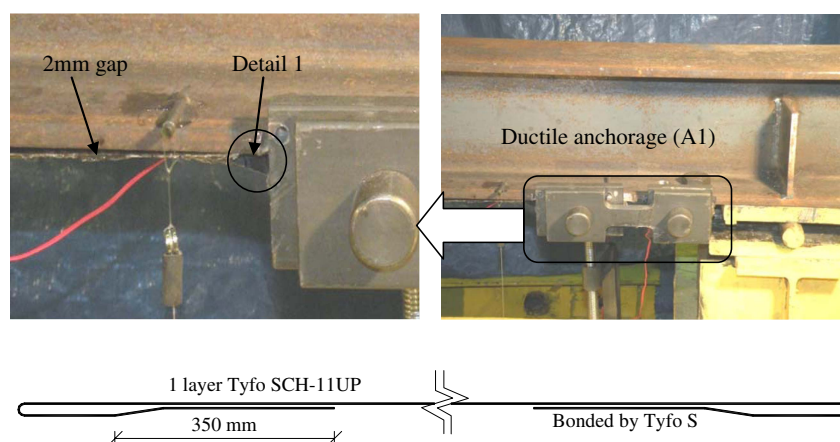


Fig. 4. Ductile anchorage system with Detail 1 that was used in BF-H0.33-F1(1)-A1D1 and BF-H0.33-F1(1)-A2D1

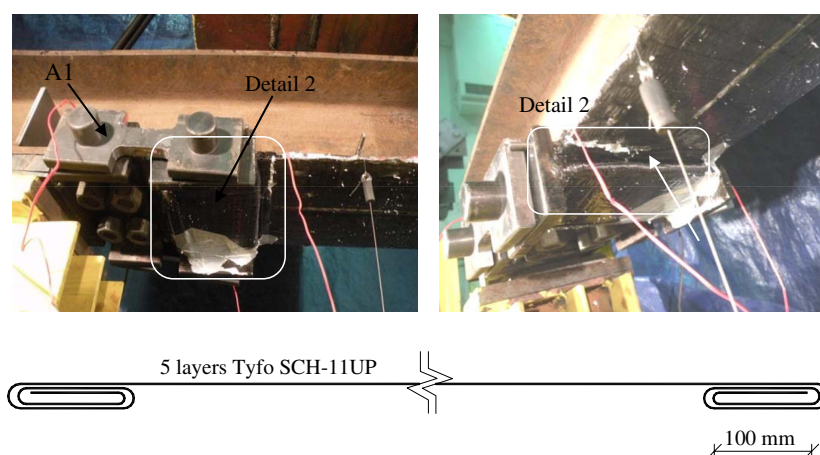


Fig. 5. Ductile anchorage system with Detail 2 that was used in BF-H0.33-F1(5)-A1D2

CFRP sheet of Detail 1. As for the capacity of the strengthening system, beam BF-H0.33-F1(1)-A1D1 was designed such that the steel link member, designated as A1, has higher capacity than the CFRP sheet to have rupture in the CFRP sheet before yielding of the steel link. Beams BF-H0.33-F1(1)-A2D1 and BF-H0.33-F1(5)-A2D2 were designed such that the yield stress of the steel link is less than the tensile strength of the CFRP sheet(s) used (the design is designated as A2).

Analysis of Experimental Results

Fig. 6 shows a schematic of load-deflection relationship of simply supported beams that has either (a) no deterioration; (b) local deterioration in the midspan section; or (c) spread deterioration in the midspan zone. Beam types (b) and (c) are to represent tested beams that have holes and notches in their bottom flanges, respectively. The load-deformation relationships are based on a bilinear stress-strain relationship of the steel beams. Compared with the load-deflection curve of an undeteriorated beam [beam type (a) shown in Fig. 6], a beam with spread deterioration would have less stiffness than that with a local deterioration, yet both will have the same value of the load that causes the first yield in the deteriorated section (depicted with a circle in the figure) if the level of deterioration in the flexure capacity is the same in the two beams. After the point of first yield, beams with different levels of deterioration [i.e., beam types (b) and (c) in Fig. 6] will undergo a nonlinear load-deflection relationship with reducing stiffness until the deteriorated section (or zone) is fully plastic as depicted with a triangle in the figure. After the point of full plasticity of the deteriorated zone, the beam will continue to deform following a reduced stiffness value.

From the above it can be seen that for a nondeteriorated beam, the yielding load is defined as the load at the first yielding. Whereas, unlike a nondeteriorated beam, the beams with local or spread deterioration will experience a nonlinear behavior after the deteriorated section (or zone) reaches its yield strain value. This nonlinear transition relationship is steeper in locally deteriorated beams and flatter in beams with spread deterioration. In this research and to represent the difference between the load-deformation responses of locally versus spread deterioration in steel beams, the point of yield is defined as being an arbitrary point resulting from a proof displacement, Δ_{proof} , and follows the initial

stiffness of the beam as shown in Fig. 6. This assumption is analogous to that established in determining the yield point for ductile materials with stress-strain relationship that do not have a clear yield plateau. Fig. 6 shows the points of yield for different beam types as solid circles. The point identifies the load at yield, P_y , and the corresponding displacement, Δ_y , for the two beam types (b) and (c). The value of Δ_{proof} for each specimen is taken as 0.002 of the displacement at the first measured yield strain in the original section of the beam. Table 4 summarizes the analyses of test results showing the load and deflection values in the midspan section at yield and at failure and the modes of failure.

The failure of the tested beams was one of five modes of failure as shown in Fig. 7. The first mode of failure, which occurred in all unretrofitted beams, was identified as being an in-plane plastic buckling of the compression flange (Gioncu and Petcu 1997a, 1997b). The second and third modes of failure were observed to be peeling off (at the CFRP ends) and rupture of the CFRP composite material, respectively. The occurrence of these two failure mechanisms was observed in all bonded CFRP-retrofitted beams based on the type of the CFRP composites and/or the type of adhesive. The last two failure modes occurred in the unbonded CFRP-retrofitted steel beams either as a yield in the ductile link followed by CFRP rupture or as a CFRP premature rupture, respectively.

Discussion of Test Results

The performance of each group is discussed individually. Table 5 shows a summary of performance indicators for all tested beams. The performance indicators considered in this study are:

- Ultimate-to-yield load ratio = P_u/P_y
- Displacement ductility ratio, $\mu_\Delta = \Delta_u/\Delta_y$
- Effective elastic stiffness, $K_e = P_y/\Delta_y$
- Retrofit effect on moment at yield = $M_y/M_{y,orig.}$
- Retrofit effect on ultimate moment = $M_u/M_{u,orig.}$

where Δ_y and P_y = the displacement and load at yield, respectively; Δ_u and P_u = the displacement and load at ultimate, respectively; M_y and M_u = the yield and ultimate flexure capacities of the retrofitted beam, respectively; and $M_{y,orig.}$ and $M_{u,orig.}$ = the yield and ultimate flexure capacities of the deteriorated unretrofitted beam, respectively.

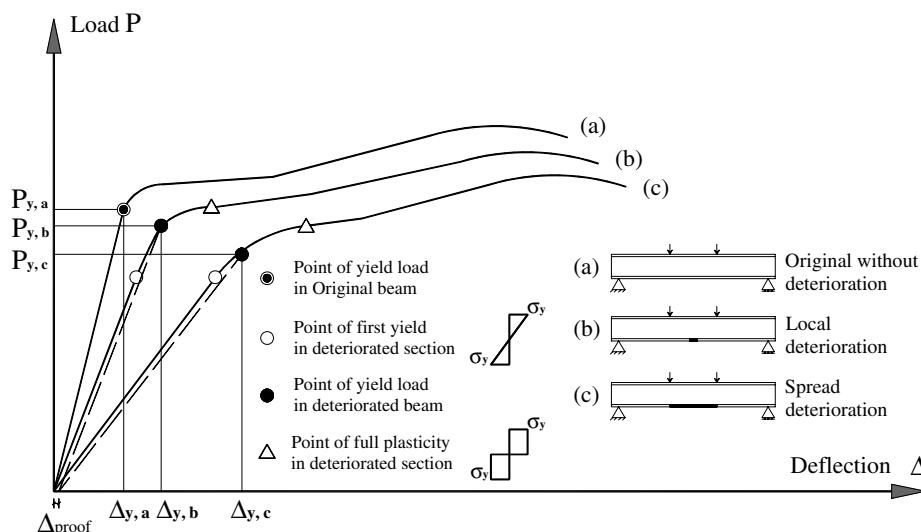
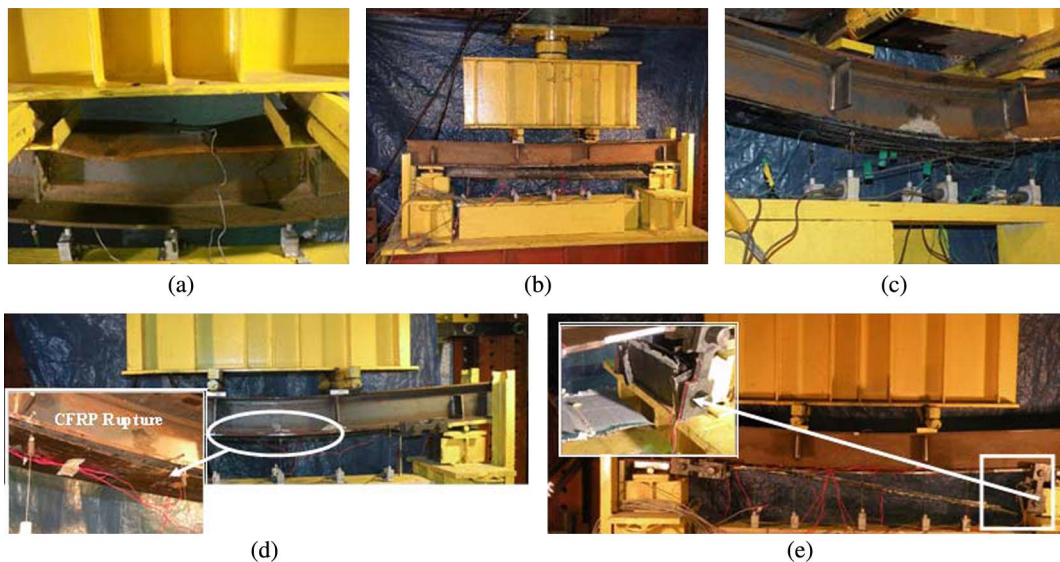


Fig. 6. Load-deflection relationship of simply supported steel beam that has: (a) no deterioration; (b) local deterioration in the midspan section; (c) spread deterioration in the midspan zone

Table 4. Summary of Test Results

Group	Beam ID	At yield		At failure		Mode of failure
		Load P_y (kN)	Defl. Δ_y (mm)	Load P_u (kN)	Defl. Δ_u (mm)	
G1	BFO	245	7.7	331	73.6	Steel-buckling
	BF-H0.33	242	8.0	330	69.8	Steel-buckling
	BF-N0.33	190	6.4	311	74.3	Steel-buckling
	BF-N0.50	155	5.8	290	95.4	Steel-buckling
G2	BF-H0.33-F1(5)-B1	245	7.9	307	18.1	CFRP peeling
	BF-N0.50-F1(5)-B1	181	5.7	262	12.1	CFRP peeling
	BF-H0.33-F1(5)-B2	253	8.2	375	51.0	CFRP rupture
	BF-N0.50-F1(5)-B2	195	6.5	307	24.0	CFRP rupture
G3	BF-H0.33-F2(1)-B2	250	8.4	344	46.1	CFRP peeling
	BF-H0.33-F2(1)-B2w	260	8.6	347	48.7	CFRP peeling
G4	BF-H0.33-F1(1)-A1D1	240	7.3	323	43.9	CFRP rupture
	BF-H0.33-F1(1)-A2D1	246	7.8	320	44	Yield in ductile link followed by CFRP rupture
	BF-H0.33-F1(5)-A2D2	250	7.7	304	32	

**Fig. 7.** Modes of failure: (a) in-plan plastic buckling; (b) CFRP peeling off; (c) CFRP rupture; (d) yield in the ductile link followed by CFRP rupture; (e) CFRP premature rupture**Table 5.** Summary of Performance Indicators for the Tested Beams

Group	Beam ID	P_u/P_y	$\mu_\Delta = \Delta_u/\Delta_y$	$K_e = P_y/\Delta_y$ (kN/mm)	$M_y/M_{y,orig.}$ (%)	$M_u/M_{u,orig.}$ (%)
G1	BFO	1.35	9.6	31.8	100	100
	BF-H0.33	1.36	8.7	30.3	99	100
	BF-N0.33	1.64	11.6	29.7	78	94
	BF-N0.50	1.87	16.5	26.7	63	88
G2	BF-H0.33-F1(5)-B1	1.25	2.3	31.0	101.2	93.0
	BF-N0.50-F1(5)-B1	1.45	2.1	31.8	116.7	90.3
	BF-H0.33-F1(5)-B2	1.48	6.2	30.9	104.5	113.6
	BF-N0.50-F1(5)-B2	1.57	3.7	30.0	125.8	105.9
G3	BF-H0.33-F2(1)-B2	1.38	5.5	29.8	103.3	104.2
	BF-H0.33-F2(1)-B2w	1.33	5.7	30.2	107.4	105.1
G4	BF-H0.33-F1(1)-A1D1	1.35	6.0	32.9	99.2	97.9
	BF-H0.33-F1(1)-A2D1	1.3	5.6	31.5	101.7	97.0
	BF-H0.33-F1(5)-A2D2	1.2	4.2	32.5	103.3	92.1

Effect of the Area Loss in Tension Flange on the Flexure Behavior of Deteriorated Steel Beam

The results of Group 1 beams shown in Table 5 illustrate that the remaining yielding load capacity of a deteriorated beam can be approximately 63% of the beam's original yield capacity if there is a uniform area loss of 50% of the bottom flange spread in the midspan zone. The same area loss would result in approximately 12% reduction in the beam's ultimate load capacity. Increasing the level of corrosion in the bottom flange (i.e., from local deterioration of 33% area loss in beam BF-H0.33 to spread deterioration of 33 and 50% area loss in beams BF-N0.33 and BF-N0.50, respectively) resulted in widening the gap between the loads (and deflections) at yield and at ultimate. Increasing the corrosion level increased the ratio of the ultimate-to-yield load, P_u/P_y , from 1.35 for beam BFO to 1.87 for beam BF-N0.50 and increased the displacement ductility μ_Δ from 9.56 to 16.53 for the same respective beams.

Fig. 8 shows the load-deflection and load-strain relationships at midspan section for Group 1 beams. As illustrated in Fig. 8(a), the difference between the load-deformation performance of the locally deteriorated beam BF-H0.33 and the control beam is insignificant. For both beams, the recorded displacement and load values at yield and ultimate levels were close. Although the load-deflection of locally deteriorated beams was close to that of the original beam, there is a significant difference in their load-strain performance as shown in Fig. 8(b). The difference between the load-deflection and load-strain performance of beams BF-H0.33 and BFO indicates

that beams with local corrosion (simulated by deterioration in mid-span section only) will experience a local increase in strains in the corroded section, yet this local deterioration will have negligible effect on the load-deformation performance of the beam mainly attributable to the insignificant reduction in the stiffness of the beam. Also, the strain readings shown in the load-strain diagram(s) terminates when the strain capacity of the gauge is reached.

Fig. 8 shows that increasing the level of area loss in the bottom flange for beams with spread deterioration from 33 to 50% in beams BF-N0.33 and BF-N0.50, respectively, results in a respective reduction in both load-deflection and load-strain performance of the beams compared with the original beam BFO.

Effect of Adhesive Types on the Effectiveness of the Retrofitting Scheme

Fig. 9 shows the load-deflection and load-strain performance of Group 2 beams at their midspan. From Figs. 9(a) and 9(b) it can be seen that the load-deflection and load-strain relationships for the four retrofitted beams was linear until yielding of the steel occurred, after which the beams experienced a nonlinear behavior until the failure of the CFRP sheets occurred by either rupture or debonding. The degree of nonlinearity of the performance beyond the first yield strain is more emphasized in beams with spread deterioration (notches in the midspan zone with a 50% area loss in bottom flange) compared with beams with local deterioration (holes in the midspan section with a 30% area loss in the bottom flange). As expected, after the failure of the CFRP, the retrofitted beams followed a similar

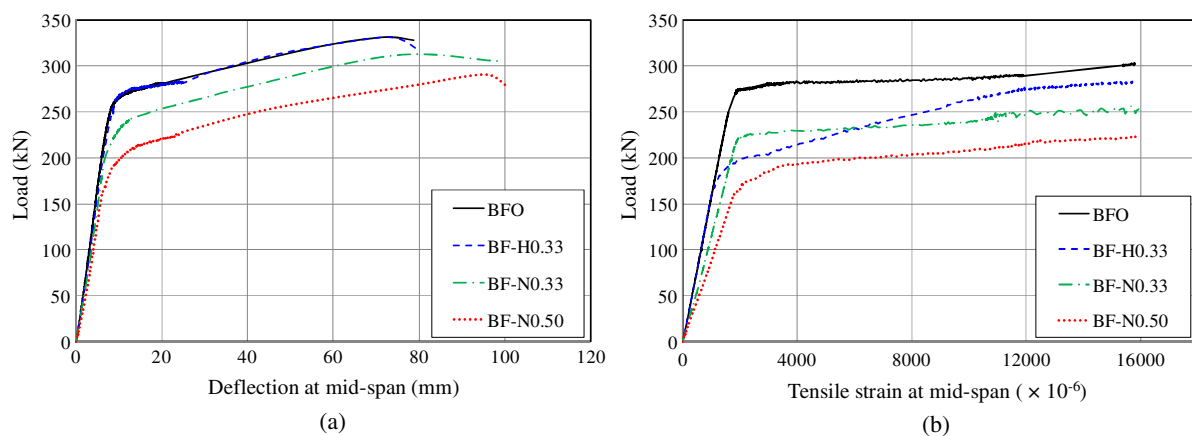


Fig. 8. Performance of Group 1 beams: (a) load-deflection; (b) load-strain relationships at midspan

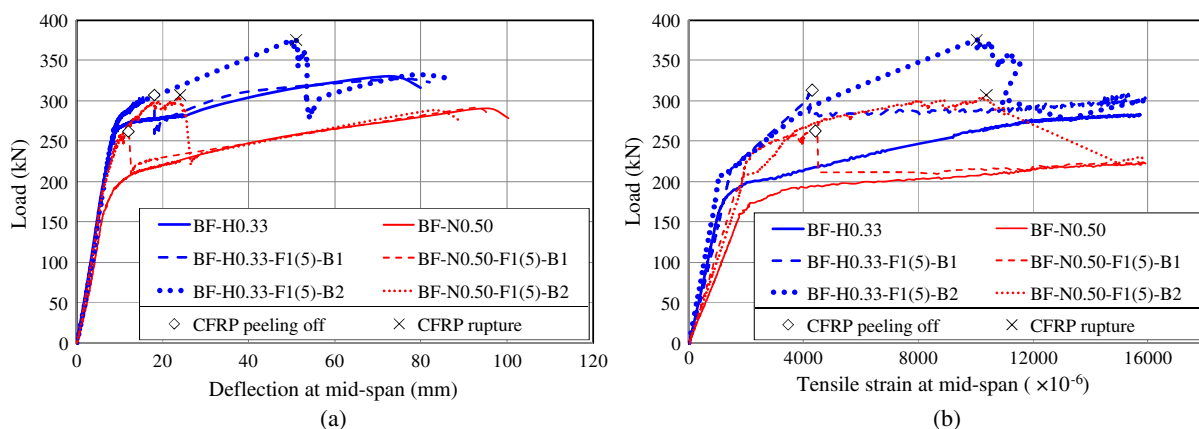


Fig. 9. Performance of Group 2 beams: (a) load-deflection; (b) load-strain relationships at midspan

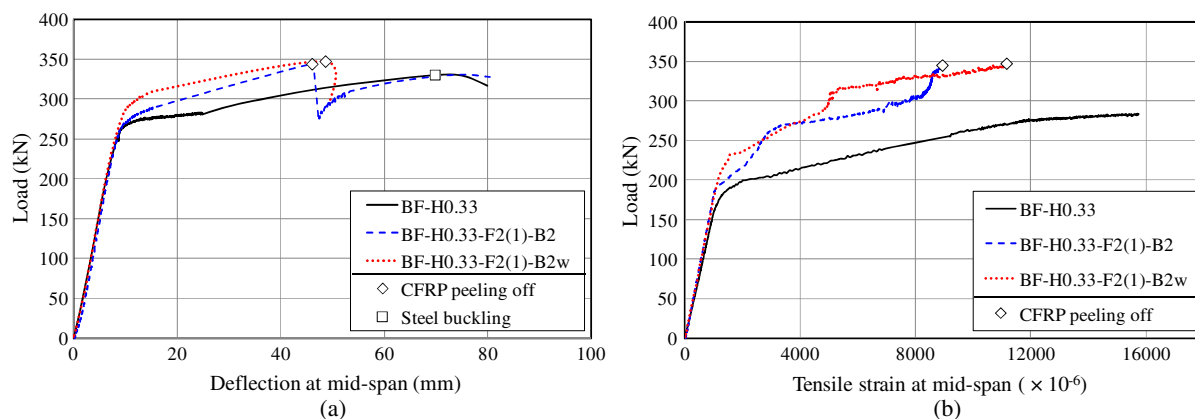


Fig. 10. Performance of Group 3 beams: (a) load-deflection; (b) load-strain relationships at midspan

behavior to that of their respective unretrofitted ones until an in-plane plastic buckling of the compression flange occurred.

As shown in Fig. 9(a) and Table 4, the type of adhesive has slight influence on the load at yield, whereas it has a significant effect on the ultimate flexural capacity and the mode of failure of the CFRP-retrofitted beams. Regardless whether the beam had a local or spread deterioration, beams BF-H0.33-F1(5)-B2 and BF-N0.50-F1(5)-B2 retrofitted using viscous epoxy showed higher ultimate flexural capacity compared with beams BF-H0.33-F1(5)-B1 and BF-N0.50-F1(5)-B1 that used saturating epoxy to bond the CFRP sheets to the steel beam. This resulted in higher

ultimate-to-yield load, P_u/P_y , and higher displacement ductility, μ_Δ , as shown in Table 5.

The performance of Group 2 beams could be considered to represent the general performance of deteriorated steel beams retrofitted with externally bonded FRP composites. The performance is characterized by an increase in the yield moment that reached approximately 126% that of the unretrofitted deteriorated beam as shown in Table 5. This is attributed to the postyield stiffening effect of CFRP sheets on the load-deflection performance of the retrofitted beams. This increase in the yield moment capacity is associated with a reduction in the displacement ductility, μ_Δ , and the ultimate

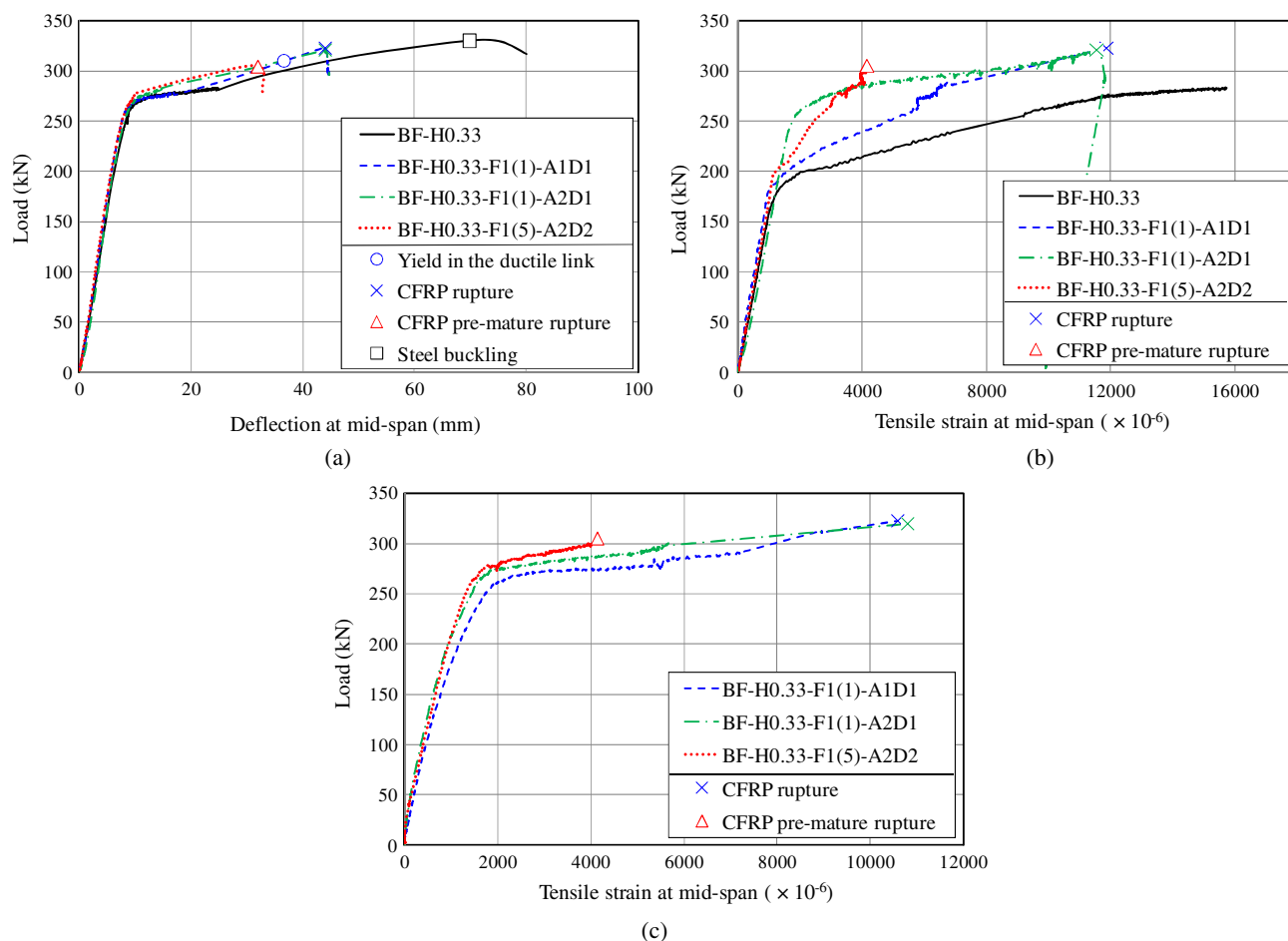


Fig. 11. Performance of Group 4 beams: (a) load-deflection; (b) load-strain; (c) load-CFRP strain relationships at midspan

moment ratio, $M_u/M_{u,orig}$, of the CFRP-retrofitted beams as can be seen in Table 5.

Effectiveness of Wrapping the CFRP Plates at the Beams' Ends

Fig. 10 shows the load-deflection and load-strain relationships of Group 3 beams that are used to evaluate the effectiveness of transverse wrapping around the longitudinal CFRP plates at the beams' ends on the bond between the CFRP plate and the retrofitted beams. As can be seen in the figure, using transverse CFRP wraps around the bottom flange at the ends of bonded CFRP retrofit system did not have significant influence on delaying the debonding of the CFRP plates. Table 4 shows a slight increase in the yielding load from 250 to 260 kN and an insignificant increase in the ultimate load from 344 to 347 kN. The failure was observed to occur due to peeling of the CFRP plate at the plate ends.

Efficiency of the Proposed Ductile Anchorage System

Fig. 11 shows the load deflection and load strain in steel and in CFRP sheets relationships of Group 4 beams compared with that of the deteriorated unretrofitted beam BF-H0.33. The two beams BF-H0.33F1(1)-A1D1 and BF-H0.33F1(1)-A2D1 that were retrofitted using one sheet of CFRP and had an anchor with Detail 1 (shown in Fig. 4) both failed according to their respective design criteria. Beam BF-H0.33F1(1)-A1D1 failed by rupture of the CFRP sheet, whereas the maximum recorded strain in the steel links was $380 \mu\epsilon$, which is considerably less than the yield strain. Whereas for beam BF-H0.33F1(1)-A2D1, the steel links yielded at 85% of the CFRP sheet capacity followed by rupture of the CFRP sheet at ultimate deflection of 44 mm as shown in Fig. 11(a). The method of transferring the stresses from the CFRP sheet to the anchor by means of Detail 1 used in these two beams did not experience a local failure, which implies that it is effective in this regard. Increasing the number of CFRP sheets would result in lower strain in them when the steel link yields, which would result in higher ultimate deflection. As can be seen from Fig. 11(c), the CFRP sheets started to strain and get engaged in increasing the flexure capacity of the retrofitted beams at an early load level. The rate of increase in CFRP strain in the three retrofitted beams increased in the post-yielding range (at approximately 270 kN). This implies that the CFRP strains in this retrofit technique follow a trend similar to that of the load-deflection relationship of the beams attributable to the fact that the strains in CFRP sheets are induced attributable to the vertical deformation of the beam, thus stretching the CFRP retrofit system. As the two beams BF-H0.33F1(1)-A1D1 and BF-H0.33F1(1)-A2D1 were retrofitted using one CFRP sheet only, the beams did not show significant increase in their retrofit-to-original ratios of yield moment ($M_u/M_{y,orig}$) and ultimate moment ($M_u/M_{u,orig}$). Rather, the retrofit scheme showed an excellent potential for securing a ductile performance (by having high displacement ductility ratio μ_Δ) and control over the mode of failure compared with Group 2 beams that used five externally bonded CFRP sheets.

Beam BF-H0.33F1(5)-A2D2 that was originally designed to have a ductile failure by yielding of the steel link members and to reach an ultimate deflection more than 51.0 mm [the ultimate deflection of the specimen BF-H0.33-F1(5)-B2] experienced a premature failure in the connection between the CFRP sheets and anchorage system (Detail 2 in Fig. 5). This highlights the importance of the detailing of the anchorage system and the means of transferring stresses in its components.

Conclusions

This paper presents the results of an experimental investigation carried out on the behavior of deteriorated steel beams before and after retrofitting using different CFRP systems. The control beam and the unretrofitted beams failed because of in-plane plastic buckling of the compression flange. On the basis of the test results, the following conclusions are drawn:

1. Local deterioration in the bottom flange of unretrofitted steel beams has negligible effect on the load-deformation behavior of the beam. This is mainly attributable to the insignificant reduction in the stiffness of the beam. However, there is a local increase in the strains of the deteriorated section.
2. Retrofitting deteriorated steel beams with externally bonded CFRP composites would increase its postyield carrying capacity yet will result in reduced displacement ductility.
3. Failure of steel beams retrofitted using externally bonded CFRP composites is sudden attributable to either CFRP peeling off or CFRP rupture, which implies the need for exploring means of enhancing the FRP retrofit systems for steel beams to maintain their ductile performance characteristic.
4. The type of adhesive has a slight influence on the load at yield, whereas it has a significant effect on the ultimate flexural capacity and the mode of failure of the CFRP-retrofitted beams. Regardless whether the beam had local or spread deterioration, steel beams retrofitted using viscous epoxy showed higher ultimate flexural capacity compared with beams that used saturating epoxy to bond the CFRP sheets to the steel beam. This resulted in higher ultimate-to-yield load and higher displacement ductility.
5. Transverse wrapping of the CFRP longitudinal retrofit plates (that are externally bonded to the bottom of the tension flange of steel beams) using CFRP sheets wrapped around the bottom flange at both ends did not have significant influence on delaying/eliminating the debonding of the CFRP plates of the retrofitted steel beam.
6. Retrofitting steel beams using the proposed anchorage system could prevent the early peel off of the CFRP sheet, which would enhance the flexural performance of the CFRP-retrofitted steel beams. The proposed anchorage systems used in CFRP-retrofitted steel beams could be designed to have a ductile performance by controlling the failure to be through yielding of its link members.
7. The end detail of the unbonded CFRP sheets in a ductile anchorage system has a significant influence on the efficiency of retrofitting scheme.

The writers emphasize that the conclusions drawn are based on a limited number of tests and that more tests and analyses need to be conducted to generalize the aforementioned conclusions.

Appendix. Design Approaches of the Ductile Anchorage System

This appendix provides the criteria used in the design of the ductile anchorage system shown in Fig. 3. The design limits for the anchorage system components followed the CSA S16 (2009) standard. The system is composed of three parts (shown in Fig. 12): Part 1 is a rigid plate with rounded edges that is attached to the CFRP sheets, Part 2 is a link member that is made of a ductile material, and Part 3 is a rigid plate that is attached to the bottom flange of the beam. Parts 1 and 3 have two cylinders (one on each side of the plate, see Fig. 12). These two parts are connected using Part 2 (that would act like a fuse) as can be seen in Fig. 12. The aim

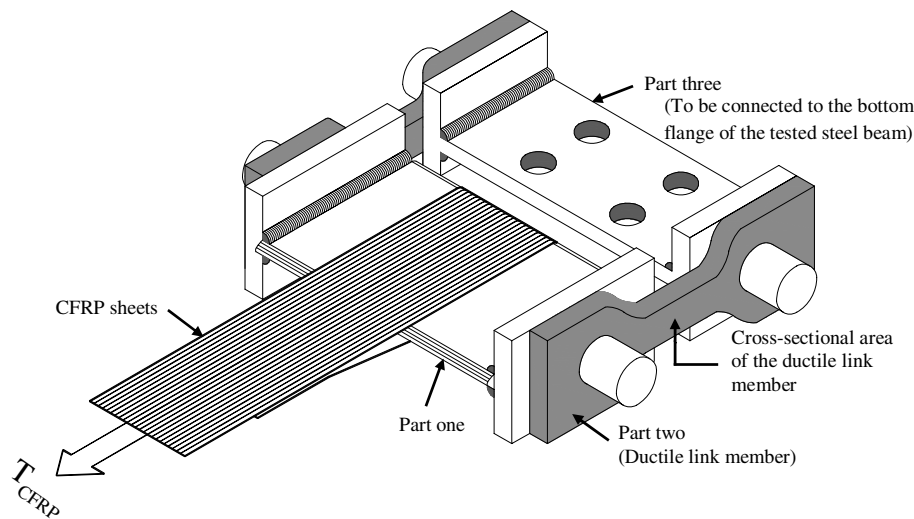


Fig. 12. Details of the ductile anchorage system

of the ductile anchorage system is to eliminate debonding of CFRP sheets and to provide the possibility of ductile elongation of Part 2 before rupture of CFRP sheets. There are two design approaches that are used in this experimental work, namely (1) to control the failure to be by rupture of the CFRP sheets before yielding of the ductile link member, and (2) to design the system such that the ductile link member will yield first before the CFRP reaches its ultimate strength.

Part 1 is designed to transfer the load from the CFRP sheets to the anchorage system. The dimensions of the rigid plate of Part 1 are designed to have a flexural capacity higher than the required moment needed to transfer the CFRP sheets tension load using an arbitrary factor of safety equal to 1.25. Part 3 is designed to transfer this load (tension load in the CFRP sheets) from the anchorage system to the tension flange. The same arbitrary factor of safety, i.e., 1.25, is used to design the number and the shear strength of the shear bolts to connect Part 3 to the bottom flange as shown in Fig. 3. Parts 1 and 3 are designed to be capable of transferring a load of five layers of CFRP sheets with the same width of the bottom flange of the tested beams without having any permanent deformations in these two parts.

Beam BF-H0.33-F1(1)-A1D1 is designed to evaluate the capability of the anchorage system in transferring the load from the CFRP sheets to the bottom flange of the retrofitted beam near the supports. The first design approach is used for this beam such that the yield load capacity of the link member, Part 2, has a higher capacity than the ultimate capacity of the CFRP sheet. The governing design equations are as follows:

$$T_{CFRP,u} < T_{link,y} \quad (2)$$

$$T_{link,y} = A_{link} \times f_{link,y} \quad (3)$$

$$T_{CFRP,u} = A_{CFRP} \times f_{CFRP,u} \quad (4)$$

where $T_{CFRP,u}$ = rupture load of the CFRP sheets; $T_{link,y}$ = yielding load of the link member (Part 2); A_{link} = total cross-sectional area of the link members (see Fig. 12); $f_{link,y}$ = yield strength of the link member; A_{CFRP} = total cross-sectional area of the CFRP sheets; and $f_{CFRP,u}$ = ultimate tensile strength of the CFRP sheets (see Table 2).

The second design approach is followed in the retrofit scheme of beam BF-H0.33-F1(1)-A2D1. To enhance the ductility of this retrofitted beam, the link member is designed to yield before the CFRP sheet reaches its ultimate capacity using the following equation

$$T_{CFRP,u} > T_{link,y} \quad (5)$$

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