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Viability of Utilizing CFRP Composites for Improving the Structural Behavior of Steel Beams

Khairedin M. Abdalla*, Ghazi Abu-Farsakh, and Montaha Al-Shdiefat

Department of Civil Engineering, Jordan University of Science & Technology, Jordan

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Abstract

This paper presents an experimental investigation to evaluate the viability of utilizing carbon fiber reinforced polymer (CFRP) composites for strengthening of indeterminate steel beams. Ten steel I-beams were strengthened using unidirectional CFRP composites in a form of plates attached on the flanges and/or web. The beams were tested as fixed-ends under a one center load distributed over the piston area. The indeterminate steel beams having two fixed ends required high load to cause failure. Although, the maximum load carrying capacities were not significantly improved due to debonding of the CFRP plates, but the strengthened-beams demonstrated reasonable improvement in the flexural-stiffness and slight increase in the torsional-stiffness. In most cases, the governing buckling-mode for the strengthened and un-strengthened beams was inelastic lateral-torsional buckling combined with local flange- buckling. The findings of this study show encouraging enhancement in the structural behavior of intermediate steel beams after strengthening with CFRP composites. This study provides an important guidance for future research toward development of means to realize the full potential use of CFRP composites for strengthening of steel beams.

Keywords: Strengthening, CFRP composites, indeterminate steel beams, flexural stiffness

1. Introduction

Structural steel and reinforced concrete structures are employed in construction daily all over the world, and the infrastructure and construction are ever proliferating and developing. Large number of structures becomes unsafe to use or deteriorates on a daily basis owing to changes in design configurations, loading, the use of low-quality building materials, and/or due to natural events like earthquakes. It is more economical to repair and retrofit the deteriorating components than replacing the entire structure. Moreover, in certain cases as in bridges, the process of strengthening and rehabilitation take less time and reduce the possibility of service interruption. The rehabilitation and strengthening processes were traditionally performed through attachment of steel plates, but this process is recently achieved by the use of CFRP composites if a form of sheets, strips, or plates [1- 7].

Over the past few years, the material strength and stiffness characteristics of the CFRP sheets improved greatly [8]. Recently, some types of CFRP composites have almost double elastic modulus of structural mild-steel (Figure 1). Also, the advantages of corrosion resistance and light weight of CFRP composites over steel in highly corrosive environments make them more effective such as in off-shore structures [9]. In-service characteristics of the CFRP composites made them a good choice for rehabilitation of damaged bridge box girders, because of their excellent fatigue and strength properties. Their high strength-to-density ratio made them excellent choice for retrofitting/strengthening of steel beams and structures [10]. Variations of the mechanical characteristics of CFRP composites and their effect on the strengthened systems under various environmental and loading conditions were studied [11]. The authors studied retrofitting and strengthening of double-strap joints of corroded steel plates under tension and also, they investigated the flexural performance of deteriorated steel I-beams using externally bonded CFRP composite plates [11]. The strengthened beams with CFRP plates experienced limited ductility upon failure; either by debonding or rupture, at higher load capacities than those of the unstrengthened beams [12].

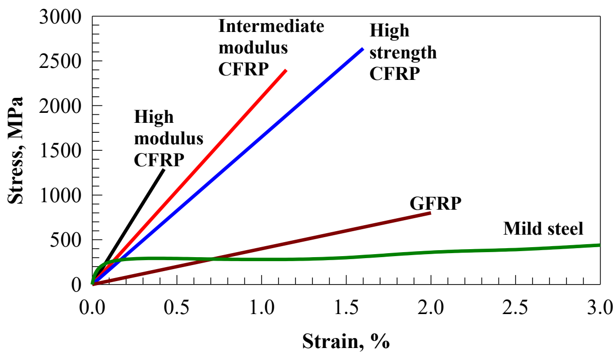

Fig. 1 Comparison of stress–strain relations of mild steel, CFRP, and GFRP composites [9]

Narmashiri et al. [12, 13] evaluated the load-carrying capacities of strengthened steel I-beams, which increased with increased length and thickness of the CFRP plates. The use of short CFRP plates led to premature end-debonding,

*E-mail address: abdalla@just.edu.jo
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while using long plates raised the resistance against the end-debonding. Also, they carried out an experimental testing on flexural-strengthening of steel I-beams using CFRP strips, which showed an improved flexural behavior. Galal et al. [14] investigated the effectiveness of using CFRP composites in retrofitting of deteriorated steel beams, and proposed an anchorage system to enhance the ductile strength of the deteriorated beams to remove early peel-off from the CFRP sheets.

Dawood et al. [15] found that the use of reverse-tapered joint configuration may improve the joint-capacity. Such findings indicate that steel beams can be strengthened using CFRP laminates, both in shear and in flexure. They concluded that shear strength improvement of nearly 25 to 39% can be readily achieved by attaching CFRP strips to webs of steel beams [16]. Peiris [17] conducted experimental and analytical investigations on bond characteristics and flexural-behavior of steel members strengthened with ultra-high and normal modulus CFRP laminates. In both cases, failure occurred at the interface of laminate edges followed by progressive debonding initiated at laminate-edge towards its center. Few studies investigated the use of pre-stressed CFRP laminates to strengthen steel structures. It was noticed that the use of CFRP composites for retrofitting steel structures is not very popular [18, 19].

Strengthening of notched steel beams using CFRP plates was found to double its strength, while the brittle fracture caused by intermediate debonding initiated from the notch location limits the ductility enhancement of the retrofitted beams [20]. Alternatively, notched steel beams strengthened with pre-stressed CFRP plates with end-anchorage systems showed a delayed debonding propagation of the strengthened beams, preventing premature failure [21]. Mohammed et al. [22] studied the effectiveness of CFRP composites to recover strength and stiffness of steel beams having web-openings. The load carrying capacity of beams achieved an increase over the un-altered beams from 5 to 20%. A parametric study carried out by Omar et al. [23] revealed that CFRP sheets were very efficient in reinforcing compact mono-symmetric sections, whereas the effect on non-compact sections was very small. The bonded CFRP sheets allowed for reaching the ultimate strength of steel beams provided that enough bond-length was ensured [23].

2. Experimental Setup

a) Details of the tested beams

Ten indeterminate steel I-beams were tested under one-point center load (Figure 2) that was distributed over the piston area (about 50 mm x 300 mm). The span length was 1530 mm and the ends were both fixed. Table 1 provides description of the ten steel beams. Two specimens were tested as reference without strengthening, one without stiffeners and one with stiffeners; designated as B0RN and B1RNS, respectively. Two specimens strengthened with CFRP plates attached to the flanges in the middle without and with stiffeners; designated as B2NCF and B7SCF, respectively. Two specimens strengthened with CFRP plates in the web in the middle of the beam without and with stiffeners; designated as B3NCW and B8SCW, respectively. Two specimens strengthened with CFRP plates in the web and flanges in the middle of the beam without and with stiffeners; designated as B4NCFW and B9SCFW, respectively. Lastly, two specimens strengthened with CFRP plates in the flanges and web at a distance of 38 cm from the support on both sides without and with stiffeners; designated as B5NCFW38 and B10SCFW38, respectively.

The used CFRP composite is factory-pultruded plate consists of unidirectional, stretched carbon fibers in epoxy resin matrix. Strain gages were used for measurements of the CFRP and steel strains. All CFRP plates have a length of 300 mm and two widths, 1) Four CFRP plates of 50 mm-width attached at the center of the web on both sides, and 2) Four CFRP plates of 10 mm-width attached at the inner-sides of the top and bottom flanges.

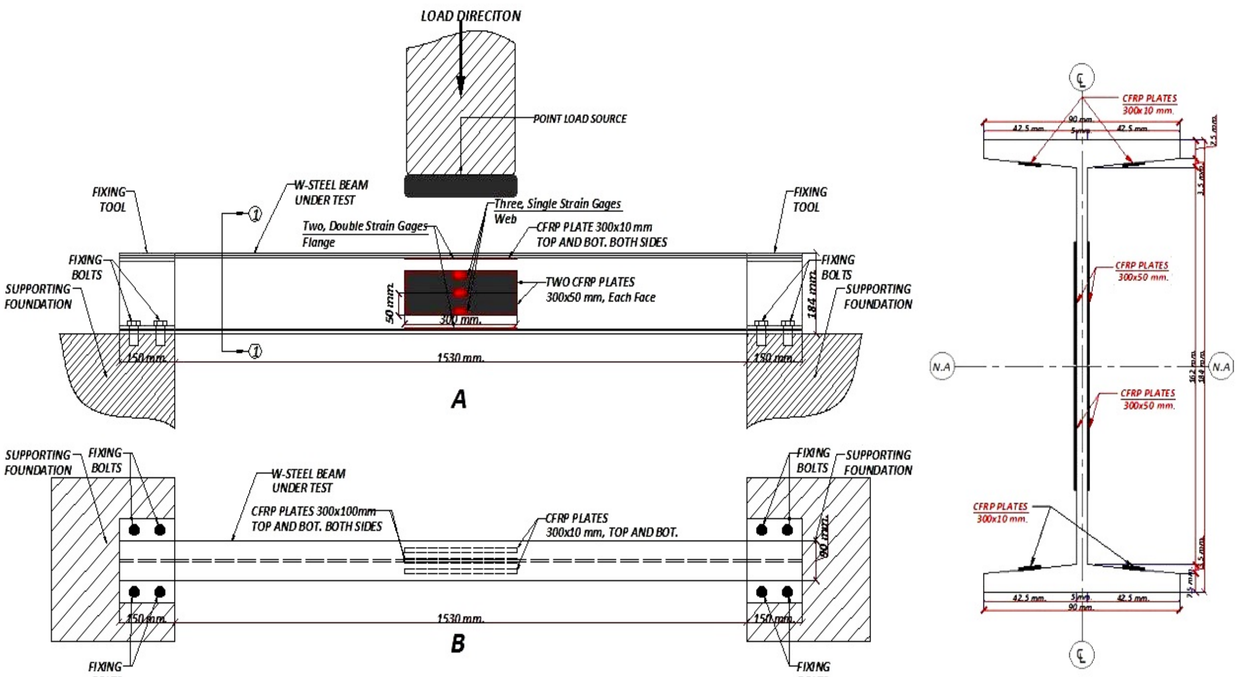


Fig. 2 Geometric details of beam cross-section showing the dimensions of the CFRP-plates and distribution of strain gages and LVDT's.

Table 1. Designation of the Tested Beams Based on the Parameters

Sample	Description of steel beam
B0RN	Beam 0: Reference beam with no stiffeners and without CFRP.
B1RS	Beam 1: Reference beam with stiffeners and without CFRP.
B2NCF	Beam 2: Steel beam with no stiffeners and with CFRP in flange in the middle of the beam.
B3NCW	Beam 3: Steel beam with no stiffeners and with CFRP in web in the middle of the beam.
B4NCFW	Beam 4: Steel beam with no stiffeners and with CFRP in web and flange in the middle of the beam.
B5NCFW38	Beam 5: Steel beam with no stiffeners and with CFRP in flange and web at a distance of 38 cm from the face of support on the two web-faces at both ends.
B7SCF	Beam 7: Steel beam with stiffeners and CFRP in flange in the middle of the beam.
B8SCW	Beam 8: Steel beam with stiffeners and CFRP in web in the middle of the beam.
B9SCFW	Beam 9: Steel beam with stiffeners and CFRP in flange and web at the middle of the beam.
B10SCFW38	Beam 10: Steel beam with stiffeners and with CFRP in flange and web at a distance of 38 cm from the face of support on the two web-faces at both ends.

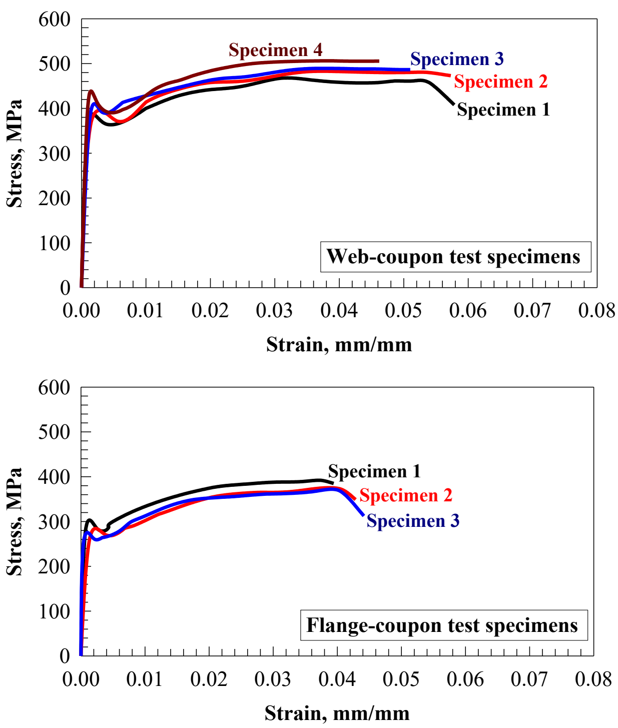


Fig. 3 Stress-strain curves for the coupon test specimens

b) Steel stress-strain relations

In order to determine the tensile mechanical properties of the steel beams, four coupon web-specimens and three coupon flange-specimens were tested in axial tension tests. The stress-strain curves for flange and web specimens of the beams are shown in Figure 3. All tests were carried out using 1200 kN Dartec testing machine at the Structural Laboratory of Civil Engineering Department at Jordan University of Science and Technology.

3. Experimental Results and Discussion

a) Mid Span Deflections

A comparison of the central mid-span deflections between the different beams without stiffeners is shown in Figure 4. The results showed strength-variation ranging between 165 kN to 180 kN. The highest strength was recorded for the reference beam (B0RN), which indicates that there is no improvement in the beam strength after using CFRP plates. This may be attributed to the pre-mature debonding of the CFRP plates for the strengthened specimens at loads lower than the reference beam (B0RN). Consecutive debonding of the CFRP plates was observed; the first incidence of debonding occurred at a load of 40 kN. As shown in Figure 4, B5NCFW38 exhibited the maximum deflection of 35 mm, which indicates that the CFRP plates improve the beam stiffness significantly. The yielding loads were approximately comparable. The recorded yielding loads for B4NCFW and B5NCFW38 were 143.5 kN and 144.5 kN, respectively. The lowest and highest yielding loads of 130.2 kN and 160.2 kN occurred in B3NCW and B0RN, respectively. The maximum deflections arranged in an ascending order along with the corresponding maximum loads were: B5NCFW38 (25 mm at 173 kN), B4NCFW (27 mm at 174 kN), B3NCW (31 mm at 165 kN), B2NCF (44 mm at 174 kN) and B0RN (55 mm at 180 kN). The reference beam without stiffeners (B0RN) exhibited more ductile behavior than the rest of the strengthen beams.

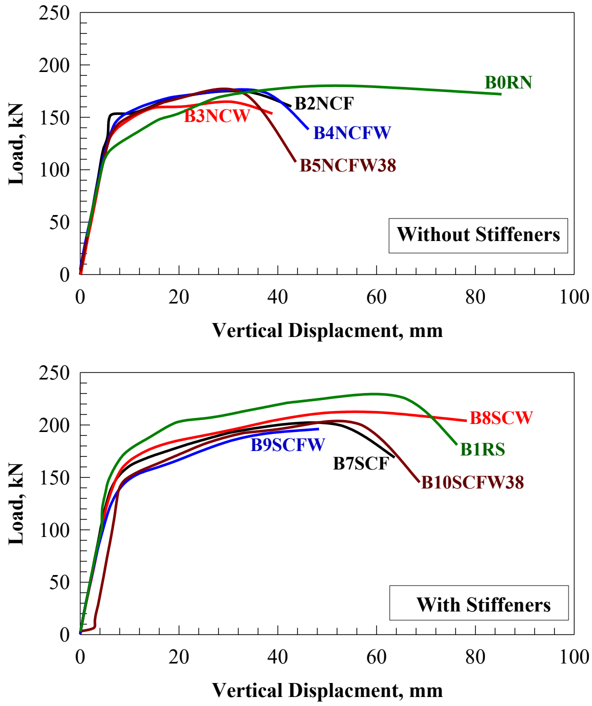


Fig. 4 Load versus mid span deflection of all beams

The results of this group also showed no strength improvement after applying the CFRP plates; the reference beam (B1RS) showed the highest strength. This might be attributed to the distortion of beams resulting from the unsymmetrical behavior of the debonded CFRP plates placed on either side of the beams; the first incidence of debonding occurred at a load of 134.35 kN. This indicates that CFRP-plates are disadvantageous to indeterminate beams of high strength. The beams unsupported lateral-length (L_b) was 1.53 m, which is between the Lp and Lr (Table 2) and this drives the steel beam to fail inelastically at smaller moments. Some of the beams were affected and their vertical displacement decreased. Beam B9SFW had the maximum vertical displacement of 50 mm (Figure 4) indicating that the use of CFRP plates resulted in an improvement in the beam stiffness. The yielding points for some of the beams with stiffeners were close to each other. For instance, the yielding loads for beam B9SFW, B10SCFW38, and B7SCF were 144.7 kN, 147.7 kN, and 150.2 kN, respectively. The yielding load was slightly higher for beam B8SCW (159.0 kN). The highest yielding load of 170.7 kN was recorded for beam B1RS. As shown in Figure 4, the mid span deflections increased in an ascending order as: B7SCF (40 mm at 200 kN), B9SFW (47 mm at 196 kN), B1RS (53 mm at 228 kN), B10SCFW38 (58 mm at 200 kN), and B8SCW (62 mm at 212 kN). The maximum load carrying capacity of 228 kN was for the reference beam (B1RS), while the maximum deflection of 62 mm was in beam B8SCW at a load of 212 kN.

b) Beam horizontal-displacement

Figure 5 and 6 show the horizontal displacement of the tested beams without and with stiffeners, respectively. In most of the test beams, with and without stiffener, the horizontal upper displacement was higher than the horizontal lower displacement. In few cases, the horizontal lower displacement at the maximum load was not observed. Moreover, the horizontal and vertical displacements were higher for the beams with stiffener than for the companion beams without stiffeners.

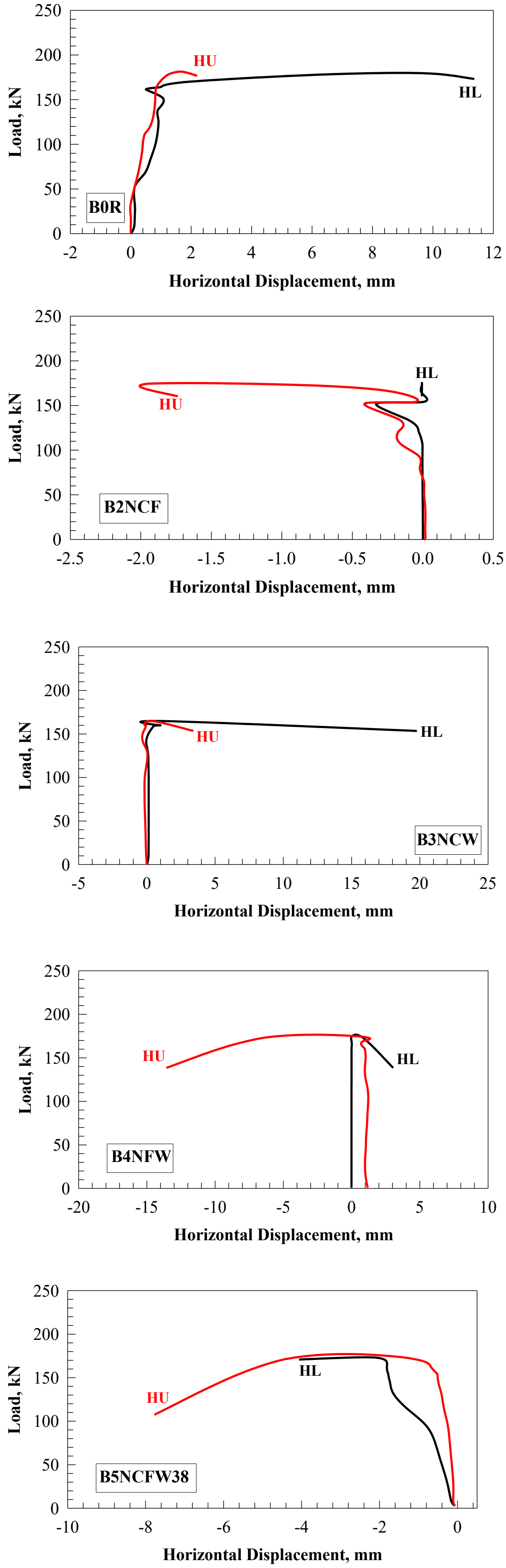


Fig. 5 The load–horizontal displacement curves for the beams without stiffeners.

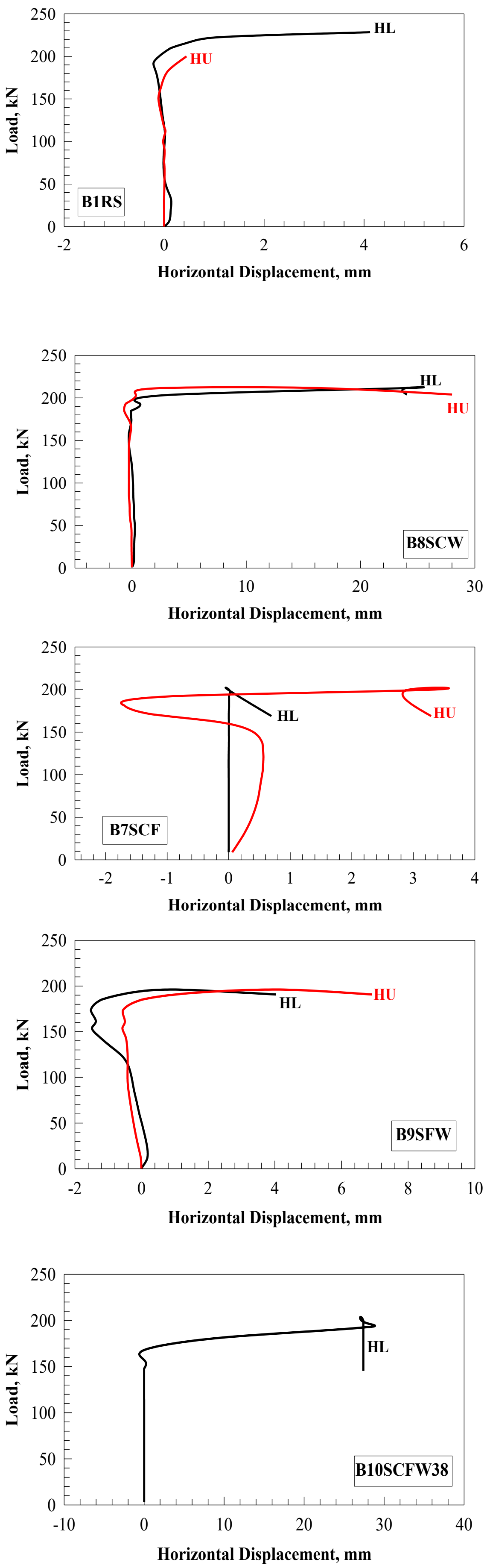


Fig. 6 The load–horizontal displacement curves for the beams with stiffener

Table 2. Beams' Section Properties

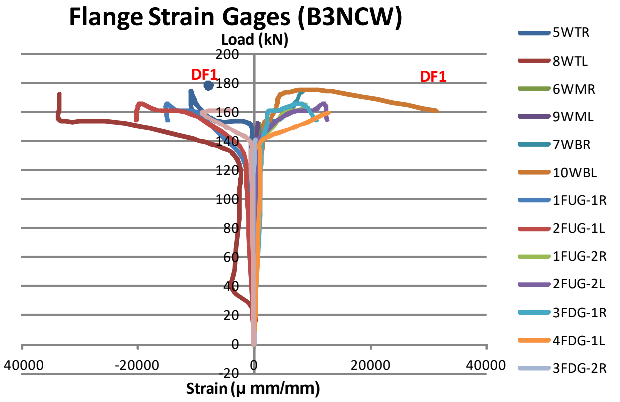
Properties	Value
A	2492.5 mm^2
$\overline{Y}$	92 mm
$\overline{X}$	45 mm
I_x	1.462×10^7 mm^4
I_y	1.0255×10^6 mm^4
S	1.589×10^5 mm^3
Z	179629.425 mm^3
M_y	60.68 kN.m
M_n	68.605 kN.m
L_p	0.8586 m
L_b	1.53 m
L_r	3.51 m
Local Flange Bending	192.881 kN
Local Web Yielding	581.644 kN
Web Crippling	461.433 kN

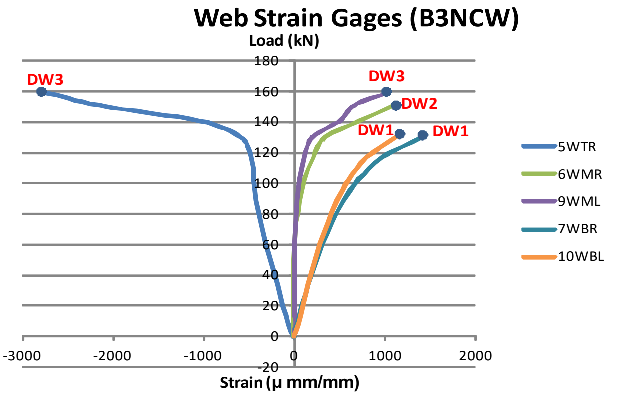
c) Strain-gages readings

Figure 7 compares the strains of the beams without stiffener. A maximum flange strain of $35,000x0^{-6}$ was observed in beam B3NCW, which reflects the significant impact of the use of CFRP plates on the web. The maximum web strain of $30,000x0^{-6}$ was observed in beam B2NCF, which reflects the considerable impact of the use of CFRP plates on the flanges. The use of CFRP plates a distance of 38 cm from the face of support on the two web-faces had a moderate effect on the web strain of $20,000x0^{-6}$ and on the flange strain of $15,000x0^{-6}$. Finally, the use of CFRP plates on the web and flanges in the middle of the beam had notable impact on the flange strain of $30,000x0^{-6}$ and little impact on the web strain of $3,500x0^{-6}$ Figure 8 shows that the beams with stiffeners showed higher strains in the flange and web than the companion beams without stiffener. A maximum flange strain of $30,000x0^{-6}$ was reported in beam B8SCW. The maximum web strain of $35,000x0^{-6}$ was observed in beam (B9SFW). The use of stiffeners and CFRP plates at a distance of 38 cm from the face of support on the two web-faces had a moderate effect on the web strain of $30,000x0^{-6}$ and on the flange strain of $10,000x0^{-6}$. Finally, the use of stiffeners and CFRP plates on the web and flanges had notable impact on the flange and web strains of $30,000x0^{-6}$.

d) Beam twist-angle

Figure 9 compares the angle of twist between the beams. It was noticed that the angle of twist for most of the tested beams was less than that of the control beam (0.0447 at a load of 179.8 kN). Only in the case of B4NFW was the angle of twist (0.0403 at a load of 173.3 kN) very close to that observed in the control beam.





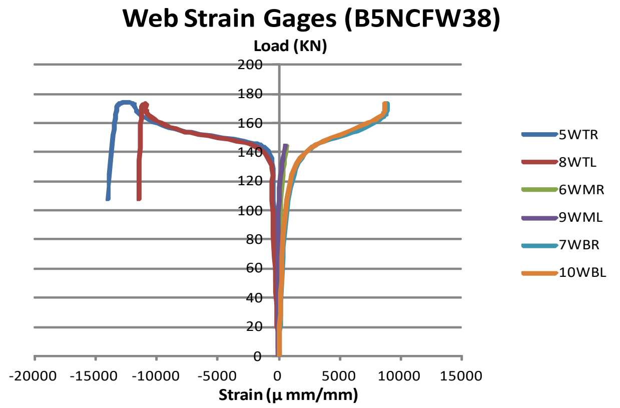
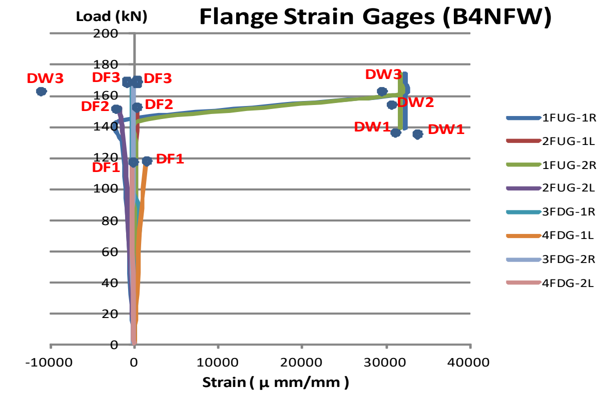
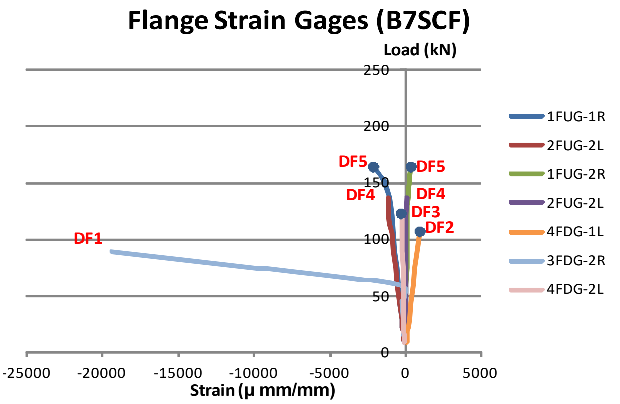
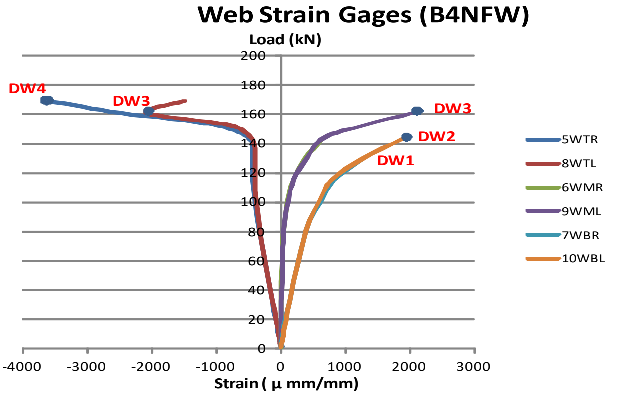
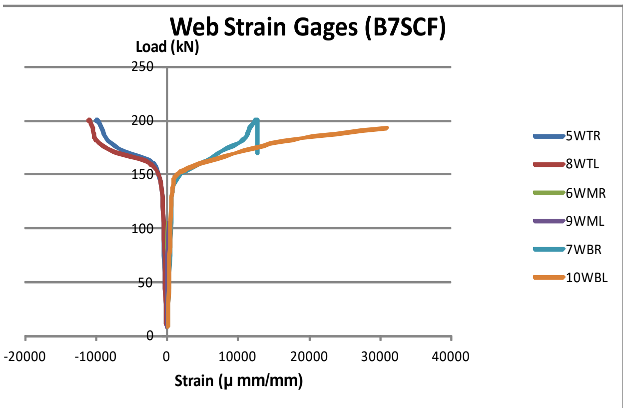


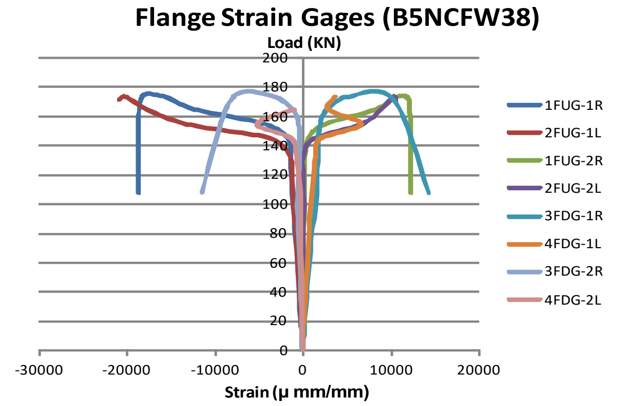
Fig. 7 The load-strains for the beams without stiffener

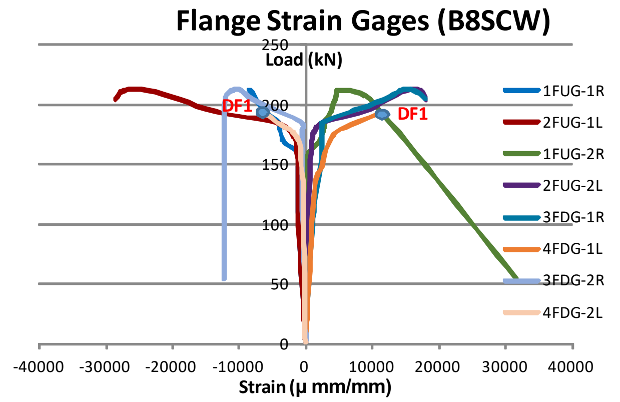












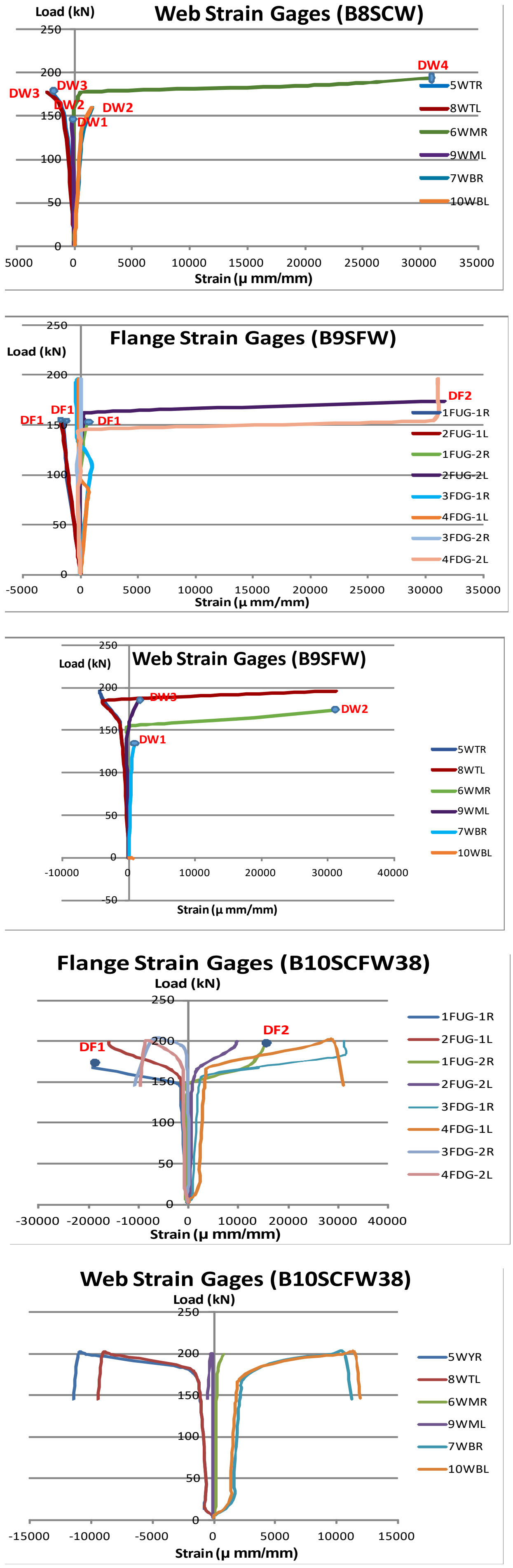


Fig. 8 The load displacement for strain gages of beams with stiffener

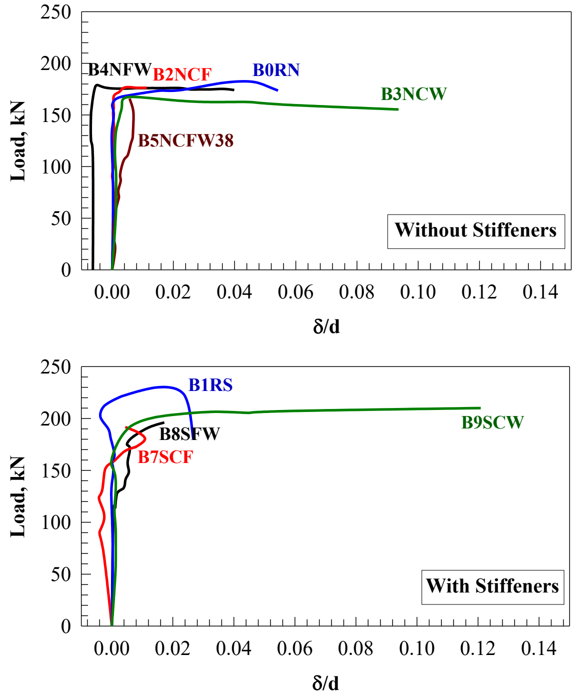


Fig. 9 Comparison in angle of twist between beams with and without stiffeners

The angles of twist of the other beams followed the descending order of B5NCFW38 (- 0.0062), B3NCW (0.0041), and B2NCF (- 0.0110). The loads affecting the latter twisting were 164.4 kN, 164.8 kN, and 174.2 kN, respectively. On the other hand, the twisting induced by the applied loads was in general limited and twisting of the beams without stiffeners took place at slightly lower loads than that of beams with stiffeners. Also, Figure 9 shows that the angle of twist was the lowest for the control beam (~0.0086 at a load of 227.9 kN). The second lowest angle of twist characterized the beam B7SCF (~ 0.0047 at the load of 192.5 kN). The beam B9SFW had a much higher angle of twist (0.0177 at a load of 196.1 kN) whereas beam B8SCW had the highest angle of twist (0.1198 at a load of 211.2 kN).

e) CFRP-debonding

As summarized in Table 3, debonding of the CFRP plates occurred in many places in the beams, which passively affected the steel beam strengthening with CFRP plates. For example, in beam B2NCF all CFRP plates debonded from all areas on which the plates were placed (1FFUH, 2FBUH, 3FFUV, 4FBUV, 5FFDH, 6FBDH, 7FFDV, and 8FBDV). As such, these beams interacted to the applied load much as if they were separate, non-strengthened steel beams at higher loads than 153.74 kN. In another example, the CFRP plates debonded in web from all places (9WFU, 11WFM, 12WBM, 13WFD, and 14WBD), except 10WBU, on beam B3NCW at loads higher than 160.55 kN. Regarding beam B4NFW, Table 3 points out that the CFRP plates debonded in flange from 6 of the 8 locations on which they were placed (2FBUH, 4FBUV, 5FFDH, 6FBDH, 7FFDV, and 8FBDV). No CFRP plate debonding in flange was observed in the beams 1FFUH and 3FFUV. On the other hand, the CFRP plates debonded from web of beam B4NFW at five points: 9WFU, 10WBU, 12WBM, 13WFD, 14WB (Table 3). No CFRP plate debonding was reported by 11WFM.

The results summarized in Table 4 disclose that: for B7SCF, debonding was only observed in flange, for B8SCW, debonding was only noticed in web, and for B9SFW, debonding took place both in web and flange. The results (Table 4) underline that the CFRP plates debonded in flange at seven points on beam B7SCF: 1FFUH, 2FBUH, 3FFUV, 4FBUV, 6FBDH, 7FFDV, and 8FBDV. The loads bringing these debonding incidences ranged from 89.4 kN to 136.7 kN (Table 4). Regarding B8SCW, debonding took place in the web only. The results (Table 4) spotlight that the CFRP plates debonded in web from six areas on this beam: 9WFU, 10WBU, 11WFM, 12WBM, 13WFD, and 14WBD. The loads leading to these events of debonding were in the range 158.9-193.3 kN (Table 4). For B9SFW, debonding occurred both in flange and in web. The study results (Table 4) show that the CFRP plates debonded in flange from four areas on this beam: 1FFUH, 2FBUH, 4FBUV, and 5FFDH. The loads resulting in these debonding incidences fell in the range 153.9-173.2 kN. Debonding of the CFRP plates was recorded at three points: 11WFM, 12WBM, and 13WFD. The loads affecting these debonding events ranged from 134.3 kN to 184.8 kN (Table 4).

This study found that in the cases of beams with and without stiffeners, the applied loads affected vertical displacement, horizontal lower displacement, horizontal upper displacement, twisting, and CFRP debonding to varying degrees. The applied loads resulted in debonding of CFRP plates from the web and flanges. Consequently, the beams suffering from CFRP plate debonding behaved much like non-strengthened beams. However, to an extent, the beams with stiffeners needed somewhat higher load to bring about CFRP plate debonding than the beams without stiffeners. The governing buckling mode for the tested beams was in-elastic lateral-torsional buckling combined with local-flange buckling in most cases. Fig. 10 shows photos of the some of the tested beams after failure.

4. Conclusions

The indeterminate steel beams having two fixed ends required high load to cause failure. The obtained maximum loads for the strengthened beams were sufficiently higher than the loads which caused CFRP-plate debonding. Therefore, no improvement was noticed in the strength of the strengthened-beams at flanges and/or webs. Stiffness of the strengthened-beams showed reasonable improvement concerning their flexural-stiffness as obtained from the load-vertical displacement behavior. The resulted total vertical-deflection of such beams was decreased and hence, showed relatively stiffer behavior than the corresponding un-strengthened beams. The torsional-stiffness was relatively increased for the strengthened-beams, especially, steel beams with strengthened flange and/or web. The governing buckling mode for the tested beams was in-elastic lateral-torsional buckling combined with local-flange buckling in most cases.

Table 3. Load at which Deboning of the CFRP Plates Occurred in the Unstiffened Beams

	B2NCF (Load)	B3NCW (Load)	B4NFW (Load)
1FFUH	DF2 (153.7 kN)		
2FBUH	DF2 (153.7 kN)		DF2 (152.3 kN)
3FFUV	DF2 (153.7 kN)		
4FBUV	DF2 (153.7 kN)		DF2 (152.3 kN)
5FFDH	DF1 (93.1 kN)		DF3 (169 kN)
6FBDH	DF1 (93.1 kN)		DF1 (115.1 kN)
7FFDV	DF1 (93.1 kN)		DF3 (169 kN)
8FBDV	DF1 (93.1 kN)		DF1 (115.1 kN)
9WFU		DW3 (160.6 kN)	DW4 (169 kN)
10WBU			DW3 (160.8 kN)
11WFM		DW2 (150.7 kN)	
12WBM		DW3 (160.6 kN)	DW3 (160.8 kN)
13WFD		DW1 (130.2 kN)	DW1 (129.8 kN)
14WBD		DW1 (130.2 kN)	DW2 (143.5 kN)

Table 4. Load at which Deboning of the CFRP Plates Occurred in the Stiffened Beams

	B7SCF (Load)	B8SCW (Load)	B9SFW (Load)
1FFUH	DF5 (161.8 kN)		DF1 (153.9 kN)
2FBUH	DF4 (136.7 kN)		DF1 (153.9 kN)
3FFUV	DF5 (161.8 kN)		
4FBUV	DF4 (136.7 kN)		DF2 (173.2)
5FFDH			DF1 (153.9)
6FBDH	DF2 (105 kN)		
7FFDV	DF1 (89.4 kN)		
8FBDV	DF3 (120.8 kN)		
9WFU		DW3 (176.8 kN)	
10WBU		DW3 (176.8 kN)	
11WFM		DW4 (193.3 kN)	DW2 (173.2 kN)
12WBM		DW1 (147.8 kN)	DW3 (184.8 kN)
13WFD		DW2 (158.9 kN)	DW1 (134.4 kN)
14WBD		DW2 (158.9 kN)	

Fig. 10 Typical failure mode for observed for most specimens

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