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Strengthening of steel-concrete composite beams with composite slab

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Abstract. Steel-concrete composite beam with profiled steel sheet has gained its popularity in the last two decades. Due to the ageing of these structures, retrofitting in terms of flexural strength is necessary to ensure that the aged structures can carry the increased traffic load throughout their design life. The steel ribs, which presented in the profiled steel deck, limit the use of shear connectors. This leads to a poor degree of composite action between the concrete slab and steel beam compared to the solid slab situation. As a result, the shear connectors that connects the slab and beam will be subjected to higher shear stress which may also require strengthening to increase the load carrying capacity of an existing composite structure. While most of the available studies focus on the strengthening of longitudinal shear and flexural strength separately, the present work investigates the effect of both flexural and longitudinal shear strengthening of steel-concrete composite beam with composite slab in terms of failure modes, ultimate load carrying capacity, ductility, end-slip, strain profile and interface differential strain. The flexural strengthening was conducted using carbon fibre reinforced polymer (CFRP) or steel plate on the soffit of the steel I-beam, while longitudinal shear capacity was enhanced using post-installed high strength bolts. Moreover, a combination of both the longitudinal shear and flexural strengthening techniques was also implemented (hybrid strengthening). It is concluded that hybrid strengthening improved the ultimate load carrying capacity and reduce slip and interface differential strain that lead to improved composite action. However, hybrid strengthening resulted in brittle failure mode that decreased ductility of the beam.

Keywords: strengthening; steel-concrete composite; FRP; welded plate; longitudinal shear; profiled steel sheeting; steel deck

1. Introduction

Steel-concrete composite structures are widely adopted in the construction industry for the construction of building, bridges, etc. In recent years, steel-concrete composite beam with composite slab (concrete slab with profiled steel sheet) has gained attention in terms of research and construction. Due to the ageing of existing structures and to meet the increasing demand to carry additional load, a number of steel-concrete composite structures require external strengthening. However, research on retrofitting of steel-concrete composite beams with profiled sheet is still limited in the literature.

Most of the available works that focus on the retrofitting of steel-concrete composite beams are related to the solid concrete slab (without profiled sheet). Retrofitting of these composite beams are achieved by strengthening the steel I-beam using steel plate or carbon fibre reinforced polymer (CFRP). To repair a composite beam member, the effect of CFRP layer/thickness on the flexural strength of the composite beams are investigated in (Miller *et al.* 2001, Sen *et al.* 2001, Tavakkolizadeh and Saadatmanesh 2003, Afefy *et al.* 2016, El-Zohairy *et al.* 2017, Yousefi *et al.* 2017, Subhani *et al.* 2018). The effect of the width of CFRP strips were investigated in (Ellobody 2011, El-Zohairy *et al.* 2017).

Tavakkholizadeh and Saadatmanesh (2003) explored the suitability of using pultruded standard modulus and high modulus CFRP on the girder with varying layers to observe its effect on the strength gain. Fam *et al.* (2009) examined the effect of CFRP thickness and type (standard, high and ultra-high modulus) on the flexural stiffness and strength of a composite beam. Daouadji *et al.* (2016) investigated the effect of adhesive layer on the CFRP retrofitted concrete beams. The stiffness and moment-rotation behaviour of CFRP plate strengthened steel-concrete composite beams were also reported in (Mosavi and Nik 2015). Pre-stressed CFRP plates were also used for increasing the flexural performance of steel-concrete composite beams (Deng *et al*. 2011, El-Hacha and Aly 2012). The effect of anchorage for the CFRP used for retrofitting composite beams were explored by (El-Hacha and Aly 2012, Karam *et al*. 2017)

While the above studies strengthened the steel flange only to improve the structural properties of the composite beam, strengthening other components of the composite girders are also reported in some literature. Al-Saidy *et al*. (2004) strengthened both the steel web and flange separately and combined to increase the flexural strength of a composite beam. Sallam *et al*. (2005) suggested the application of steel plate welded and/or bonded to the compression flange of the steel girder as well. In addition, another study of the same authors (Sallam *et al*. 2006)

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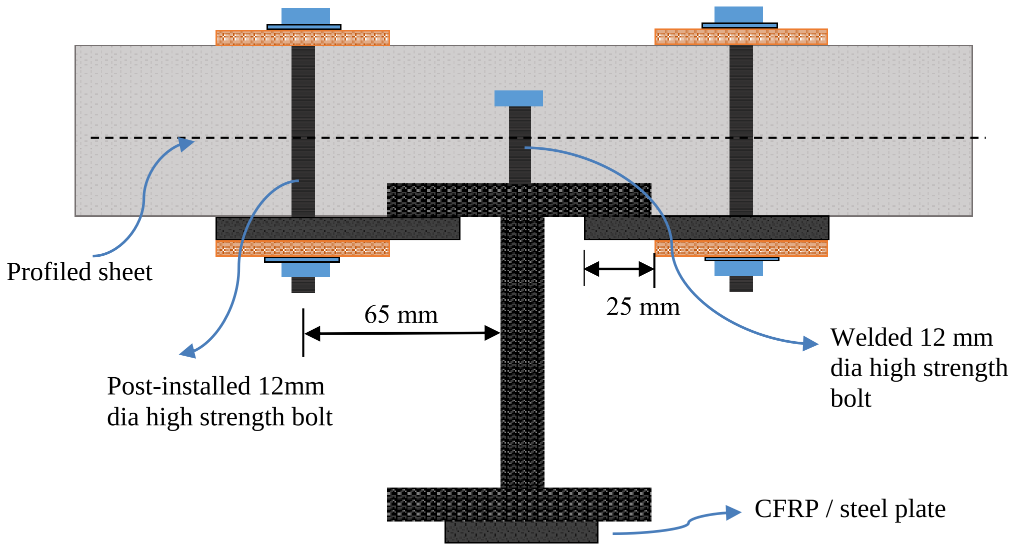


Fig. 1 Flexural and longitudinal shear strengthening of steel-concrete composite beam with profiled sheet

compared three different strengthening technique that included the application of CFRP plate on the tension flange only, CFRP plates on tension flange and steel plate on the compression flange and steel plates on both flanges. The conclusions of this study include, CFRP sheet is more effective than CFRP plates (one layer) in terms of improvement in ultimate load and bonded or welded steel plates performed better for load transfer. Similar conclusion was reported using pre-stressed CFRP sheets in (Bansal *et al.* 2016). Al-Saidy *et al.* (2007) studied the effect of the compressive strength of concrete, the cross-sectional area of the bottom flange of the steel beam and the stiffness, thickness and ultimate strain of the CFRP on the flexural strength of composite beams.

It can be noted that the aforementioned studies on the retrofitting of steel-concrete composite beam with solid slabs focused primarily on the flexural strengthening of the beam. Even though slip exist at the interface of a steel-concrete composite beam with a solid slab at a very small load as well (Johnson 2008, Saravanan *et al.* 2012), this effect is more prominent in steel-concrete composite beams with profiled sheet due to the limited number of shear connectors which can be placed within the rib of the profiled sheet. The effect of slip due to the limited number of shear connectors in a profiled sheet contained composite slabs were also reported in (Nie *et al.* 2005, Nie *et al.* 2008). Moreover, the shear capacity of the connectors in the profiled steel sheet is lower compared to the solid slab because of the local failure of the concrete rib (EN 1994; Construction 2000). Accordingly, enhancement in longitudinal shear capacity of a composite beam with profiled steel sheeting require attention while rehabilitation of this type of structural system is the point of interest. Friedrich *et al.* (2017) reported the use of a novel steel sheet which enhance both the longitudinal shear and flexural capacity of the composite slab. The slip behaviour of blind bolt and welded stud connectors in grout in rehabilitated steel-concrete composite beams were explore in (Pathirana *et al*. 2016, Pathirana *et al.* 2016, Henderson *et al*. 2017).

In addition to the longitudinal shear failure, flexural and vertical shear failure are also common in composite beam with composite slab (Gholamhoseini *et al.* 2014). Therefore, retrofitting of composite beam with composite slab perhaps require the strengthening of more than the flexural enhancement only. Consequently, in order to strengthen the longitudinal shear between steel beam and concrete slab in composite system with profiled sheeting, Pathirana *et al.* (2015) proposed the use of two special types of post-installable bolted connections and welded headed stud connector. Kwon *et al*. (2010) also suggested the implementation of post-installable bolts to rehabilitate non-composite bridges. Demir *et al.* (2018) also proposed the implementation of external steel member to enhance both flexural and shear capacities of conventional reinforced concrete members.

The present study focuses on the retrofitting of steel-concrete composite beam with profiled sheet in terms of both the longitudinal shear and flexural strength. The flexural strength is enhanced by using CFRP and steel plate on the soffit of the steel I-beam, while additional shear studs are installed in the concrete slab to increase the degree of shear connection. Since space is limited in the rib of the profiled sheet and above the steel top flange, the post-installed studs are placed further away from the top flange of the I-beam, but connected to the top flange using a steel plate, as shown in Fig. 1. The CFRP is attached to the steel beam using epoxy, whereas steel plate is welded on the bottom of the steel I-beam.

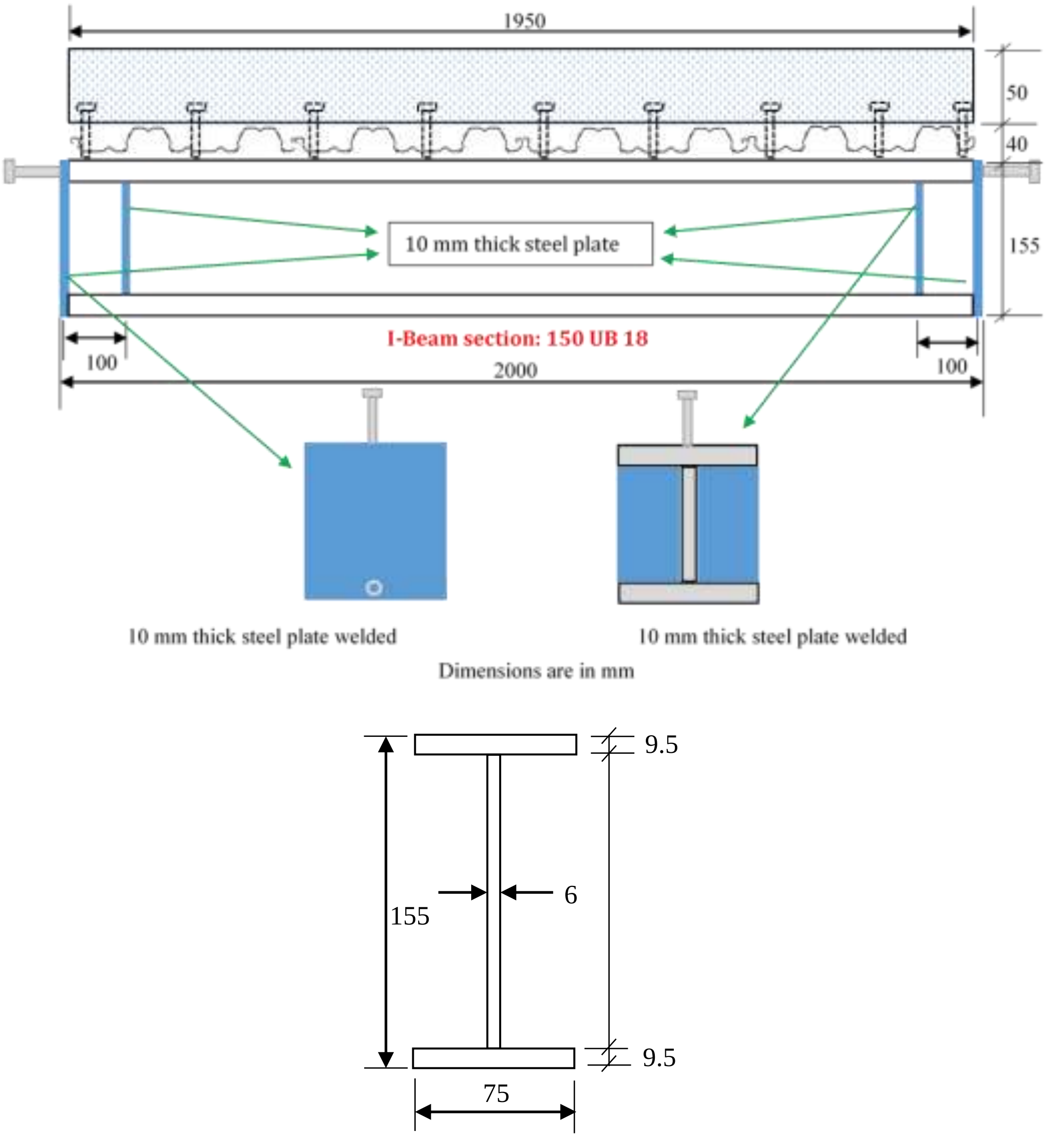


Fig. 2 Schematic of the control composite beam (not to scale)

2. Experimental program

In this study, steel-concrete composite beams were fabricated on a profiled steel sheeting/decking, and three-point bending test was conducted to study the efficacy of various strengthened beams in comparison with the unstrengthen beam. High strength bolts were used to provide shear connection between steel beam and concrete slab. Two types of strengthening schemes were implemented. The first type focused on the flexural strengthening of the beams using CFRP or steel plate on the soffit of the steel beam, whereas the second scheme strengthened the composite beam in terms of both flexure and longitudinal stress. For the longitudinal strengthening, high strength bolts were installed by drilling holes in the concrete slab. A total of six beams including one control were cast in the experiments where five beams were strengthened with five different strengthening schemes.

2.1 Fabrication of the specimen

All the six composite beams were 2000 mm long with an overall depth of 245 mm. The height of the 150UB18 steel beam was 155 mm while the thickness of the concrete slab was 50 mm and the height of the rib of the profiled sheet was 40 mm. The detailed dimension of the control beam is shown in Fig. 2. The effective width of the concrete slab was 500 mm.

To strengthen the supporting region, 10 mm thick steel plates were welded to both ends of the steel beam covering the full cross-section (155 mm height and 75 mm width). Two more steel plates with the same thickness (136 mm height and 34.5 mm width) were welded on both sides of the steel web at 95 mm clear distance from each end of the steel beam in order to stiffen the support regions. Accordingly, the clear span of the beam was 1.8 m.

Concrete slabs were fabricated from six batches with same mix design. Three concrete cylinders (200 mm long and 100 mm diameter) were cast for each batch to obtain the compressive strength on the day of testing of composite beams. A maximum coarse aggregate size of 7 mm was used due to the small slab thickness. All the concrete cylinders and slabs were cured in humid air (covered with wet hessian) for 28 days.

Two shear connectors were welded at the end of the beam for lifting purpose.

Spacing of the shear connectors was controlled by the centre-centre distance of the ribs of the steel deck
Therefore, a total of 9 M12 grade 8.8 bolts were welded to the top flange of the steel beam in one row to provide shear connection between the steel beam and concrete slab. The threaded part of the blot was cut and hence, the shank region of the bolt was used to provide connection between concrete slab and steel beam. The height of the shear connectors was 65 mm with a clear cover of 25 mm from the top of the concrete slab. SL81 wire mesh (8 mm bars with 100 mm spacing in both direction) was used as longitudinal and transverse reinforcement for the concrete slab, as depicted in Fig. 3.

2.2 Materials

The material properties of various components of the composite beams are as follows-

Steel beam: Steel beams used in the study are 150UB18.0 which has an overall depth of 155 mm, flange width of 75 mm, flange thickness of 9.5 mm and web thickness of 6.5 mm. The yield stress and tensile strength of the UB are 375 and 480 MPa, respectively, as per manufacturer's specification.

Concrete: The compressive strength of concrete was measured for all the six batches used to cast the aforementioned six composite beams. Three cylinders for each batch were prepared with a height of 200 mm and a diameter of 100 mm. The concrete cylinders were cured for 28 days in the same manner as for the concrete slabs of the composite beams.

The average compressive strength of the concrete related to each beam is presented in Table 1.

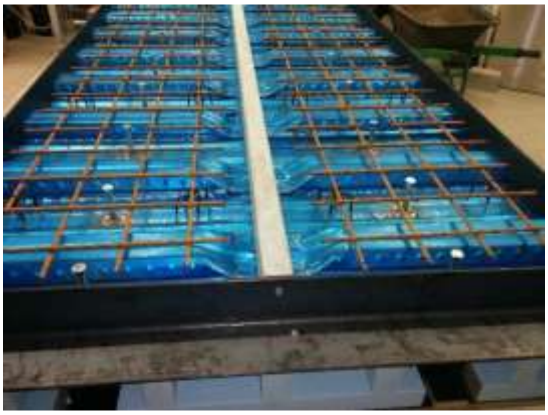


Fig. 3 Rebar arrangement of composite beams (2 beams were cast together)

Table 1 Compressive strength of concrete related to each beam in MPa

Beam1	Beam2	Beam3	Beam4	Beam5	Beam6
28.96	26.48	30.55	28.94	37.1	33.66

Table 2 Properties of steel deck

Yield strength	550 MPa
Thickness	0.6 mm
Mass per unit area	7.04 kg/m^2
Span length	487 mm
Width	500 mm
Rib height	40 mm

It is evident from the table that the compressive strength varies among six beams with the lowest being 26.48 MPa (Beam 2), in contrast to the maximum value of 37.1 MPa (Beam 5).

Steel reinforcement: SL81 steel mesh was used for the reinforced concrete to provide longitudinal and transverse reinforcement in the composite beams. The longitudinal steel reinforcement was placed at a clear distance of 10 mm from top of the ribs. The diameter of the SL81 mesh is 8 mm, and the centre to centre distance of the bars were 100 mm. The characteristic yield stress of the steel reinforcement is 600 MPa, while tensile strength is reported to be 700 MPa, as per manufacturer's specification.

Shear connector: M12 high strength (grade 8.8) bolts were used to provide connection between the steel beam and concrete slab. The bolts were placed at the centre of the concrete slab. The yield stress and tensile strength of the shear connector are 660 and 830 MPa, respectively, according to the manufacturer specification.

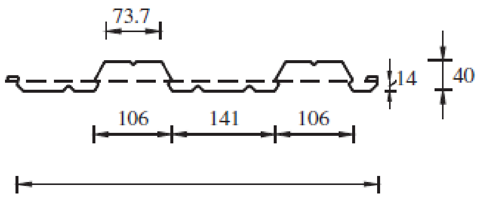
Steel plate: The yield stress and ultimate strength of the steel plate used for strengthening of composite beam was 250 and 410 MPa, respectively and thickness was 10 mm.

CFRP: MBrace CF 230/4900 was used in this investigation to strengthen the composite beams. The thickness of one layer CF 230/4900 is 0.17 mm, and the modulus of elasticity, ultimate tensile strength and rupture strain are 230 GPa, 4900 MPa and 2.1%, respectively (as per the manufacturer's specification).

Adhesive (CFRP-to-steel beam): MasterBrace 4500 was used to attach CFRP on steel I-beam and steel deck surface. The modulus of elasticity, ultimate strength and Poisson's ratio are 3034 MPa, 55.2 MPa and 0.4 (as per the technical data sheet from the manufacturer).

Primer (CFRP-to-steel): MasterBrace P 3500 primer was used before applying CFRP to steel I-beam. The modulus of elasticity and tensile strength are 2,097 MPa and 35 MPa, respectively (as per the technical data sheet from the manufacturer).

Steel Deck: The KF40 steel decking manufactured by BLUESCOPE Steel was used as a permanent formwork of the concrete slab. The properties of steel deck, obtained from manufacturer's data sheet, are provided in Table 1 and a schematic of steel deck's cross-section is provided in Fig. 4.



(All dimensions are in mm)

Fig. 4 Cross-section of the KF40 profiled sheet

(a) attachment of CFRP on steel deck

(b) steel plate welded on the deck to attach CFRP

Fig. 5 Strengthening of Beam 2 (beam upside down)

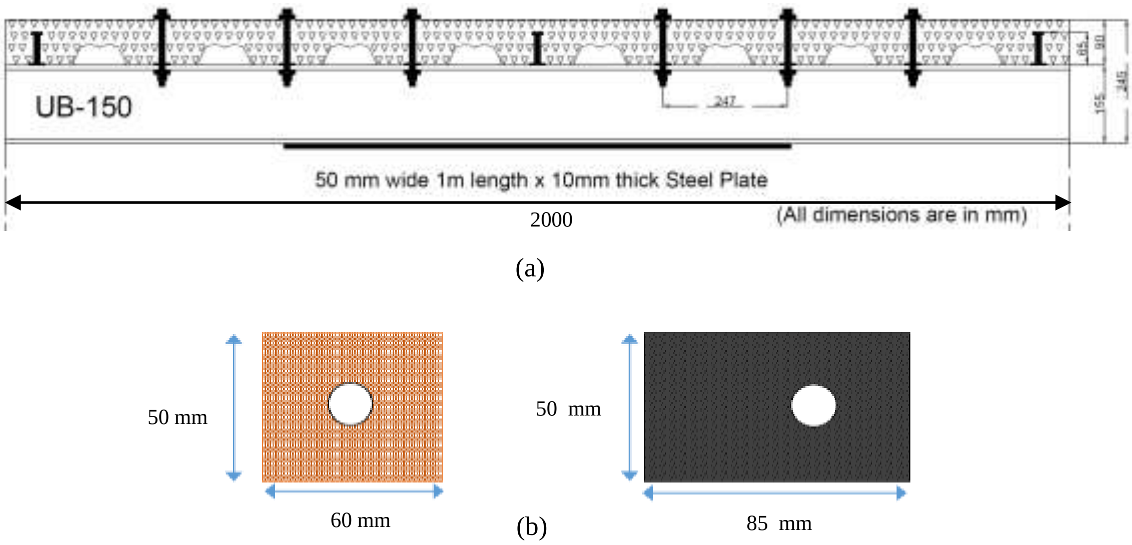


Fig. 6 (a) Strengthening of Beam 5 and (b) Steel plates used for anchoring of steel bolts in Beam 5 and 6

2.3 Strengthening schemes

One beam was used as control beam (***Beam 1***) where no external strengthening scheme was implemented. The degree of shear connection for the control beam was 0.42. The other five beams were strengthened with five different strengthening schemes. The description of all beams are as follows:

Beam 2: This beam was strengthened using one layer of 1.2 m long x 50 mm wide CFRP sheet, and applied on the soffit of the steel deck (both overhanging sides of the steel deck). One layer of 200 mm long and 100 mm wide CFRP strip was used as transverse anchor on each end of longitudinal CFRP sheet resulting in 1 m of clear distance between two anchors (Fig. 5(a)). No primer layer was used on steel deck before CFRP was bonded. Thin steel plates were stitch welded to the steel deck to fill the gaps between

(a) (b)

Fig. 7 (a) Strengthening of Beam 6 (upside down) and (b) top view of longitudinal shear strengthening

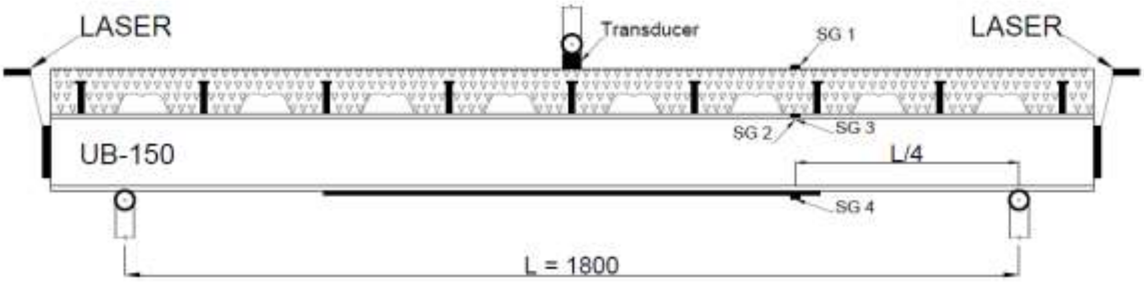


Fig. 8 Test set-up, location of transducer, strain gauges (SG) and LASER

ribs (Fig. 5(b)) and to create a platform for CFRP attachment.

Beam 3: Beam 3 was strengthened by welding a 1 m long and 50 mm wide steel plate to the steel I-beam. The steel plate thickness was 10 mm.

Beam 4: This beam was strengthened with one layer of 1.2 m long CFRP sheet with a width of 50 mm bonded to the steel beam's tension (or bottom) flange using the primer and epoxy. After curing of prime layer for one day, CFRP was adhered to steel beam using epoxy resin and left for curing for seven days.

Beam 5 (Hybrid 1): Beam 5 was strengthened with hybrid scheme which included strengthening of steel I-beam by welding steel plate with the same dimension as used for Beam 3, and strengthening of concrete slab and steel deck by 12 additional 150 mm long M12 grade 8.8 steel bolts. These bolts were drilled through the steel deck and concrete slab (6 on each side of steel I-beam) to contribute to the additional longitudinal shear capacity (Fig. 1 and Fig. 6). Each steel bolt was anchored with two steel plates (top and bottom of slab) of 60 mm × 50 mm × 10 mm dimension by nut and washer assembly (Fig. 1 and Fig. 6b). Additional steel plates with 85 mm×50 mm × 8 mm dimension were used to clamp the steel I-beam bottom flange with the composite slab. The overlap of the steel plate and the I-beam flange was 25 mm. These additional post-installed bolts were also installed within the rib of the steel deck with a distance of 125 mm on both sides of the initially welded shear connectors. The rib at mid-span was not strengthened in terms of longitudinal shear, since theoretically, the slip at this location should be zero.

Beam 6 (Hybrid 2): Beam 6 was strengthened with the same scheme as Beam 5 except the steel I-beam was strengthened with a 1.2 m long CFRP instead of steel plate welding, as shown in Fig. 7.

2.4 Test set-up

Three-point bending tests were conducted using a universal testing machine with a capacity of 500 kN. The test was performed on all beams under displacement control at a speed of 0.6 mm/min until failure, and mid-span deflection was recorded. The test set-up of the beam is shown in Fig. 8. The end slip was measured by using a 50 mm capacity LASER to observe the relative slip between steel beam and concrete slab. In order to obtain the strain values, a total of four strain gauges were attached on each beam along the cross-section at a distance of L/4 from the support. These four gauges were placed on top of the concrete slab (denoted as SG1), bottom of the profiled steel deck (SG2), bottom of the top flange of the steel I-beam (SG3) and on the soffit of the composite beam (SG4), as shown in Fig. 8. Therefore, the SG4 was attached on the soffit of the steel I-beam for Beam 1 and 2, whereas SG4 was attached on the soffit of the CFRP (Beam 4 and 6) or welded steel plate (Beam 3 and 5) for the other four beams. For Beam 2, one additional strain gauges was placed on the bottom of the attached CFRP on the profiled steel deck.

3 Experimental results

3.1 Load-deflection curve and failure modes

Fig. 9 represents the load-deflection curves of all the beams. Most common failure modes were the crushing of concrete, failure of shear connectors and debonding of CFRP. Fig. 10 illustrates the failure modes of the tested beams. The detail comparison and failure modes are explained below.

Beam 1: The steel beam of the control beam yielded at 176 kN. The ultimate load was recorded to be 214.14 kN where flexural failure of the composite slab was observed. At mid-span, tearing of profiled steel deck was also noticed due to bending at the deflection of 30 mm. As shown in Fig. 10(a), flexural crack initiated from bottom of the concrete slab (at the upper rib of steel deck), and the crack propagated towards the point load which ultimately led to concrete crushing at the deflection of 37 mm.

Beam 2: The load deflection behaviour of Beam 2 is similar to Beam 1. There was no improvement observed in the load-carrying capacity which reached the maximum load of 214.96 kN. However, the failure mode of Beam 2 was different than the control beam. The main mode of failure of Beam 2 was governed by a local shear failure, rapidly followed by flexural failure, occurred at the deflection of 38 mm, as illustrated in Fig. 10(b). Due to the presence of shear failure, debonding of steel deck from concrete was more prominent. In addition, the steel deck was stiffer due to the presence of CFRP on the deck. Accordingly, the different curvature between the concrete slab and the steel deck initiated cracks at the interface that led to the debonding of this interface. Debonding of CFRP was only noticed in the mid-span region and no complete separation of CFRP from steel deck occurred due to the fact that the debonding of steel deck from concrete slab prevented the stress transfer to CFRP.

Beam 3: Beam 3 showed slightly higher stiffness in the elastic stage (steeper slope) and higher maximum load (237.46 kN) compared to Beam 1 and 2. The load - deflection behaviour showed a sudden load drop after reaching the ultimate load (at 11 mm deflection) followed by a pseudo-ductile behaviour. This sudden drop of load was attributed to the shear connector failure. The welded steel plate on the soffit of the steel beam made the steel beam stiffer that resulted in higher load carrying capacity of the steel beam. However, due to the same number of shear connectors, the shear connectors became the weakest component. As a result, the failure of the furthest shear connectors from mid-span were observed. After the load drop due to shear connector failure, the composite action reduced and induced more deflection of the beam with pseudo-ductile failure. At the end, the concrete slab failed due to crushing.

Beam 4: Beam 4, retrofitted with CFRP attached to the bottom flange of steel beam, showed the lowest ultimate load (206.26 kN), slightly (3.68%) smaller than the control beam. After yielding of steel (at 176 kN), the beam reached ultimate load followed by debonding of CFRP from the bottom flange of the steel I-beam which can be seen by progressive small load drops starting from 15 mm deflection as shown in Fig. 9. Finally, the compression failure of the concrete slab occurred.

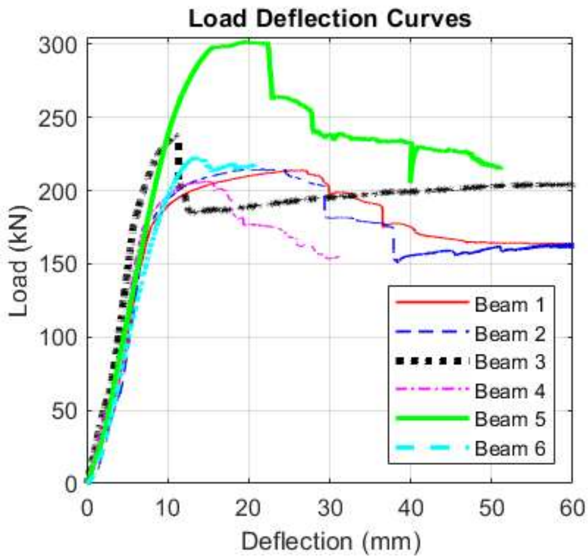


Fig. 9 Load-deflection curves of all the beams

Beam 5: Beam 5 failed after reaching the maximum load of 302.4 kN which is the highest among all the six beams. This beam reached its ultimate load in a more ductile manner compared to Beam 3 and 4. The longitudinal shear strengthened by additional post-installed shear connectors improved the performance of the beam in longitudinal shear significantly. Unlike Beam 3, there was no shear connector failure observed in this beam which can be expected. In addition, the debonding of steel deck from concrete slab was also prevented due to the anchorage provided by the additional bolts. However, the beam showed much less ductile behaviour compared to control beam and failed by concrete crushing at the deflection of 23 mm. This is due to the fact that the concrete became the weakest component in this strengthening scheme, as both the steel beam and shear connection were strengthened, which cased sudden failure.

Beam 6: Beam 6 reached its ultimate load at 222.46 kN, which is more than the control beam. However, this improvement is marginal while comparing against Beam 5. Debonding of CFRP occurred after the beam reached its ultimate load at a deflection of 15 mm. Finally, the beam failed due to concrete crushing. Nevertheless, better composite action was achieved in this beam which will be explained later.

3.2 Ultimate load

The maximum load carrying capacity and percentage difference of maximum load for all the strengthened beams compared to control beam are presented in the column 2 and 3 of Table 3. It is evident that the retrofitting of steel deck with CFRP (Beam 2) contributed to negligible increase in load carrying capacity. In addition, Beam 4, the steel I-beam retrofitted with CFRP alone, exhibited reduced ultimate load capacity. In contrast, steel beam retrofitted

Table 3 Comparison among all the beams in terms of ultimate load, stiffness and ductility

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
Beam ID	Ult. Load (kN)	% increase	Ductility (Δu/Δy)	% increase
Beam 1	214.14	-	3.04	–
Beam 2	214.96	+0.38	2.75	- 9.57
Beam 3	237.46	+10.89	1.58	- 48.06
Beam 4	206.26	-3.68	1.90	-37.64
Beam 5	302.4	+41.22	1.81	- 40.53
Beam 6	222.6	+ 3.95	1.33	- 56.20

with steel plate (Beam 3) performed notably well (10.89% increase in peak load from control) due to the strong welded connection between the I-beam and steel plate. The positive effect of steel plate welded scheme on the load capacity can also be proven by the performance of the hybrid strengthening scheme of Beam 5 which showed the highest increase by 41.22% compared to the control beam. Beam 6 which had hybrid strengthening scheme like Beam 5 with the exception of CFRP adhered to the I-beam's bottom flange had only 3.95% capacity improvement which is even lower than the scheme of steel plate welding to I-beam alone (Beam 3). However, the positive effect of hybrid strengthening can be observed in Beam 6 while comparing against Beam 4.

3.3 Ductility

The ductility of the composite beams are measured as the ratio of ultimate and yield deflection (Carlin 1998, Bank and Arora 2007)

$$\mu_d = \frac{\Delta_u}{\Delta_y} \tag{1}$$

where, Δ_u = deflection at ultimate load and Δ_y = deflection at the end of elastic load.

The Column 4 and 5 of Table 3 present the ductility and % difference in ductility of the strengthened beams with respect to the control one. The results show that all the strengthened beams (Beams 2-6) have reduced ductility compared to the control beam. The least reduction by 9.57% was observed in Beam 2. Beams 3-6 showed significant reduction in ductility, primarily due to the brittle failure modes (concrete crushing and failure of shear connectors) related to these beams. In case of Beam 3 and 5, strengthening of steel I-beam made the steel section much stronger than the concrete slab and/or shear connections.

Accordingly, steel section could withstand higher load with less deflection in the plastic region that shortened the plastic region, as illustrated in Fig. 9. As a result, reduction in ductility was observed.

The Beam 5 performed better compared to Beam 3 in terms of ductility due to the presence of more shear connectors. Beam 6 had the highest reduction of ductility with an amount of 56.2% compared to the control beam. The second highest reduction of ductility (by 48.1%) was observed in Beam 3, and the reason is the failure of shear connector immediately after the steel beam started to yield. However, this beam showed a pseudo-ductile behaviour with a slight increasing slope in load-deflection diagram until 60 mm of deflection. Beam 4 also showed 37% reduction in ductility which was much less than Beam 3; the beam only reached up to 15 mm deflection before it started to show continuous decreasing trend due to debonding of CFRP from steel beam.

In short, the reduction in ductility of the strengthened beams can be related to the failure modes. The failure mode of the control beam was the flexural failure, while the strengthened beams (except Beam 2) failed due to either concrete crushing (Beam 4, 5 and 6) or shear connector failure (Beam 3) which usually shows less ductile behaviour. The best ductility among the five strengthened beams was observed in Beam 2 where flexural failure was observed. However, sudden local shear failure in Beam 2 reduced ductility.

3.4 End slips

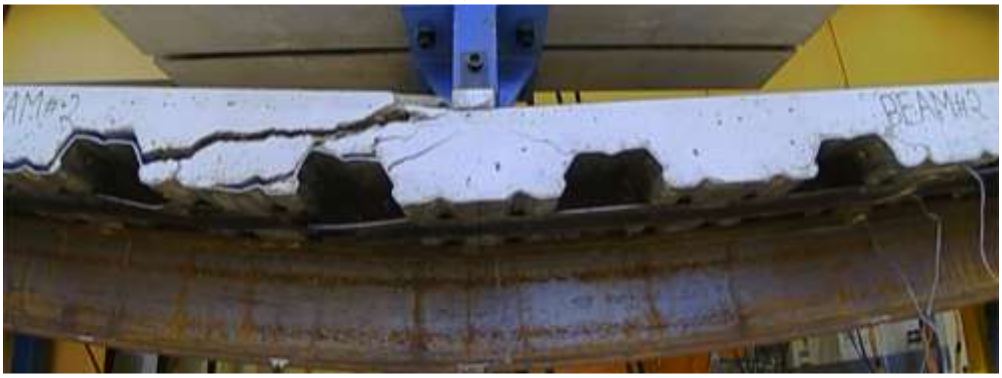
Fig. 11 demonstrates the load vs. end-slip curves of strengthened and control beams. It can be observed that with the exception of Beam 4, all the strengthened beams showed lesser end slip than the same in Beam 1 in the elastic region. Beam 4 did not show considerable non-linearity like other beams because it failed at a very low slip value of less than 1.5 mm. Similarly, Beam 2 also exhibited less slip (slightly more than 1 mm) due to the separation of steel deck from concrete slab and cracking of concrete initiated from longitudinal shear.

In comparison, Beam 3, 5 and 6 can carry same amount of slip at higher load compared to Beam 1, 2 and 4 without the failure of shear connector. The longitudinal shear strengthening scheme (of Beam 5 and 6) improved the shear transfer between the concrete slab and steel beam that resulted in higher load carrying capacity at the same slip value before the beam failed by concrete crushing.

3.5 Strain and interface differential strain

The load vs. strain curve of all beams are presented in Fig. 12. The strain profiles of all the beams along the cross-section is depicted in Fig. 13. For the strain profile, four load values are considered out of which three values are within the elastic stage and the last value is associated with the ultimate load of the corresponding beam. The presence of dual neutral axes due to partial interaction is clearly visible in case of Beam 1, 2, 4 and 5 where the concrete slab and top of steel flange show compression, while the bottom of the steel profiled sheet and bottom flange of the I-beam are subjected to tension.

(a) Beam 1



(b) Beam 2

(c) Beam 3 (d) Beam 4

(e) Beam 5

(f) Beam 6

Fig. 10 Failure modes of all the beams

In case of Beam 3, only the bottom of the steel I-beam is under tension, while all other components are in compression. For Beam 6, it is observed that up to 70 kN, the concrete slab and steel deck are in compression, while the whole steel I-beam is under tension. As the load increases, the slab, deck and top of the steel flange go into compression. This reflects that the Beam 6 shows the best composite action.

The jump at the interface is defined as interfacial differential strain and is indicative of composite action at the slab-to-beam interface (Lorenz and Stockwell 1984, Chen and Yossef 2015). As described in Fig. 14, the better composite action will have less jump in strain at the interface. This jump in strain is calculated at four different load levels and are compared in Fig. 15.

It is clear from Fig. 15 that all the strengthening techniques enhances the interfacial differential strain, while Beam 5 and 6 exhibited the lowest differential strain at the interface which was expected due to the installation of additional external bolts. And Beam 6 outperformed all other beams in terms of composite action. Beam 2 and 3 also showed significant improvement in enhancing composite action at the steel-concrete interface.

This improvement in composite action can be explained based on the works conducted by Hawileh *et al.* (2015) and Nawaz *et al*. (2016). In these works, it was concluded that flexural strengthening using CFRP increase concrete shear capacity. Accordingly, the longitudinal shear resistance of concrete can also be expected to increase which contribute to the increased composite action between concrete slab and steel beam.

It was also pointed out that CFRP is more effective in terms of increasing concrete shear strength in the beams with low steel reinforcement ratio (Hawileh et al. 2015; Nawaz et al. 2016). This explains the better composite action in Beam 6 while comparing against Beam 3 and 5 (Fig. 15), since Beam 3 and 5 has more steel in the cross-section due to the strengthening using steel plates.

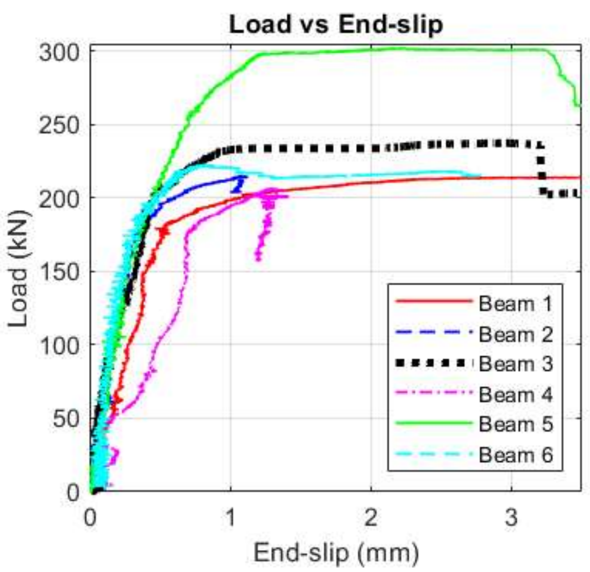


Fig. 11 Load vs end-slip all the beams

4. Conclusions

This article reports the flexural and longitudinal strengthening of five steel-concrete composite beams with steel profiled sheet / deck and compares against one control beam. Five strengthening schemes investigated in the study are strengthening of steel deck by CFRP sheet and end anchors, steel plate welded to steel I-beam, CFRP adhered to steel I-beam, combination of steel plate welding to steel I-beam and longitudinal shear strengthening (Hybrid 1), and combination of CFRP attached to steel I-beam and longitudinal shear strengthening (Hybrid 2). The effect of these strengthening schemes on the failure mode, ultimate load carrying capacity, end-slips and strain profile is investigated.

It is found that the combination of flexural strengthening of steel I-beam using welded steel plate and longitudinal shear strengthening of the interface using post-installed bolt attained the maximum load or moment carrying capacity. Use of steel plate alone can also enhance the maximum load carrying capacity of the composite beam. However, the failure of shear connectors may occur if the latter method is considered.

Both the hybrid strengthening schemes increases the composite action at the steel-concrete interface significantly, since differential strain at the interface was reduced by 4 to 13 times while comparing against the control beam. However, satisfactory improvement can also be observed for the beam strengthened using steel plate only.

The ductility of all the five strengthened beams are reduced compared to control beam. This is due to the fact that the aforementioned strengthening schemes lead to undesirable failure modes. The failure mode of the control beam is the flexural failure, while the strengthened beams (except one) are failed due to either concrete crushing (three beams) or shear connector failure (one beam) which usually show less ductile behaviour. The best ductility among the five strengthened beams is observed in the scheme where the profiled steel deck is strengthened using CFRP (but the ductility value is less than control). In this beam, the failure mode is found to be flexural-shear and lead to better ductility than other strengthened beams. More works will be conducted in future to address the issue of the undesirable failure modes.

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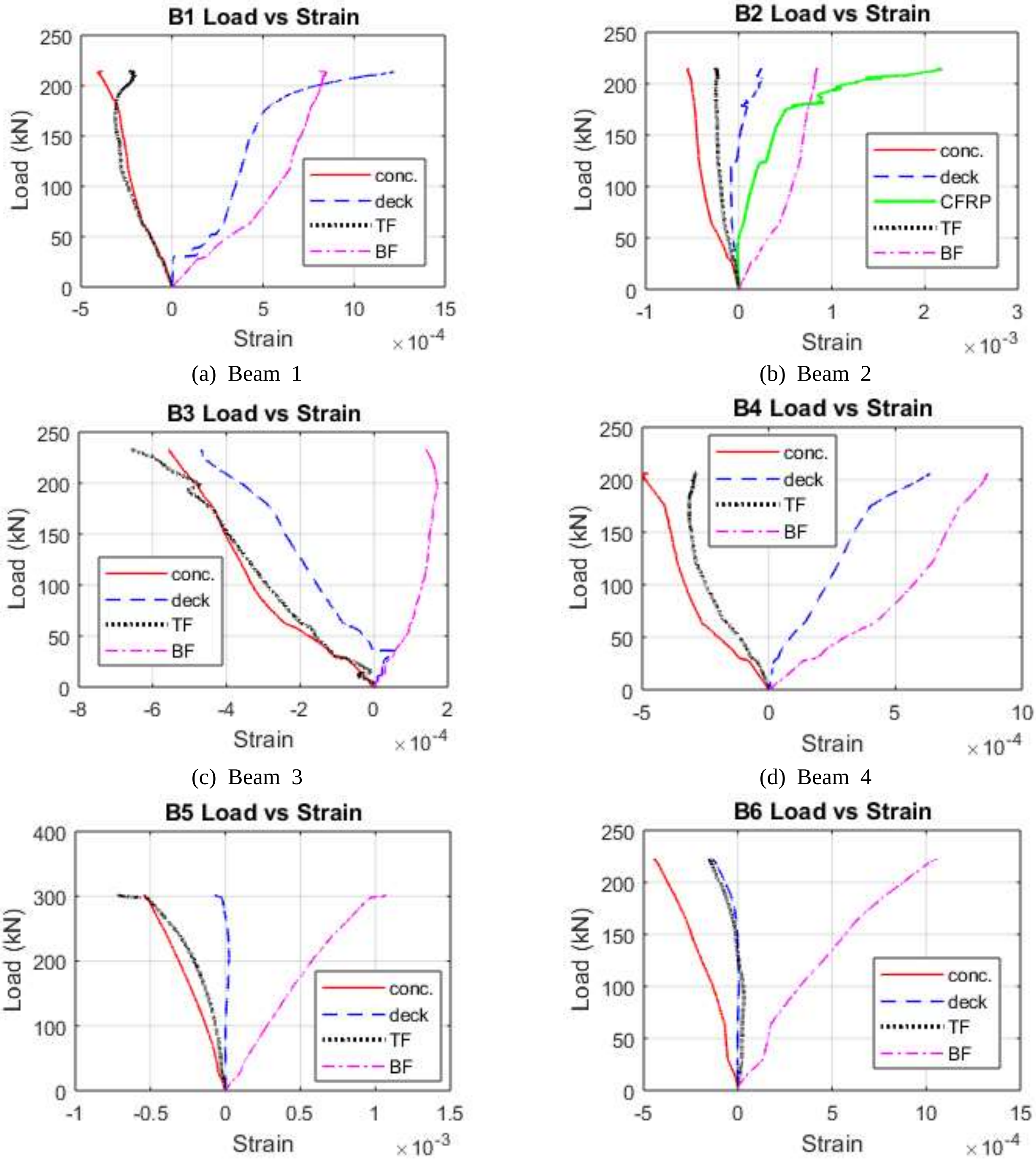


(a) Beam 1

(b) Beam 2

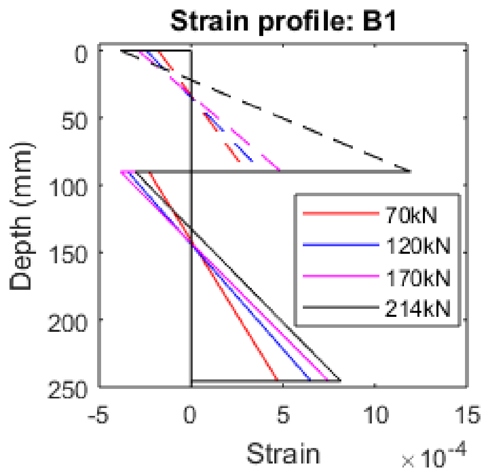
(c) Beam 3

(d) Beam 4

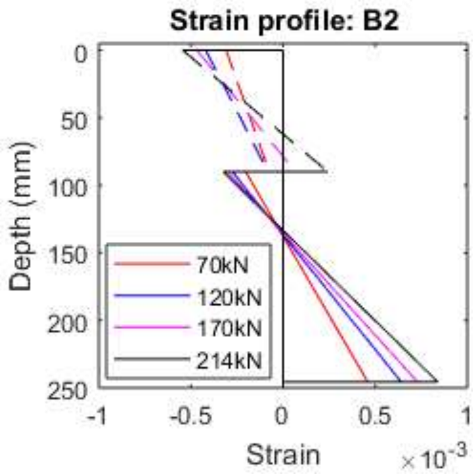
(e) Beam 5

(f) Beam 6

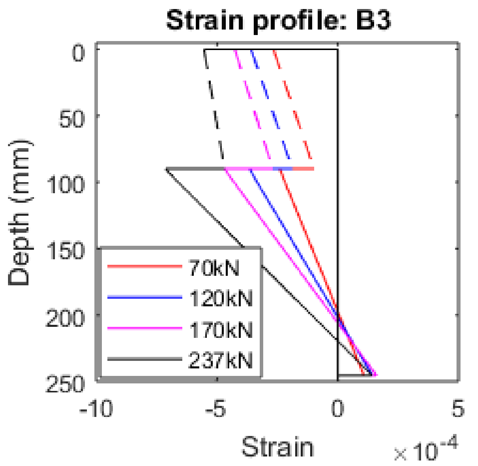
Fig. 12 Load vs strain curves of all beams
(+ = tension, - = compression, TF = top flange, BF = bottom flange)



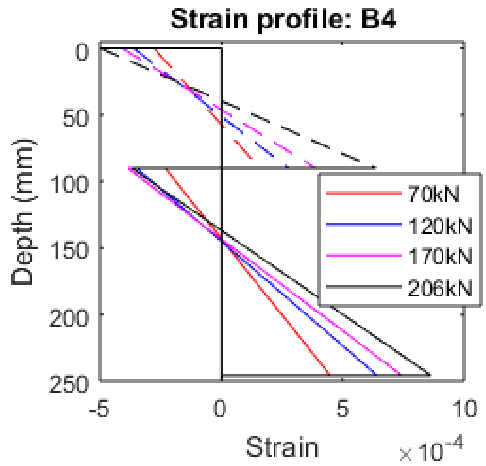
(a) Beam 1



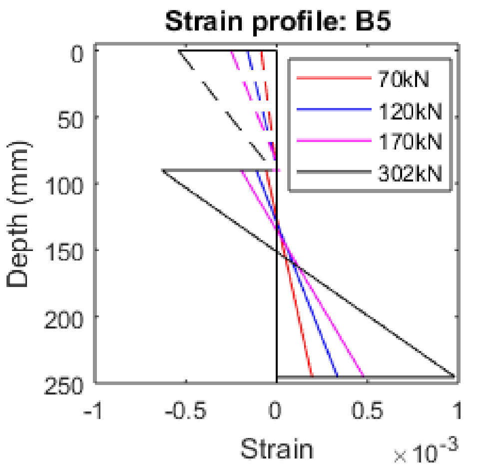
(b) Beam 2



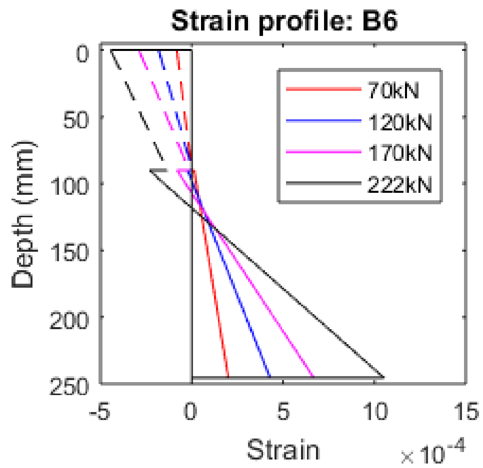
(c) Beam 3



(d) Beam 4



(e) Beam 5



(f) Beam 6

Fig. 13 Strain profile of all beams at different load level

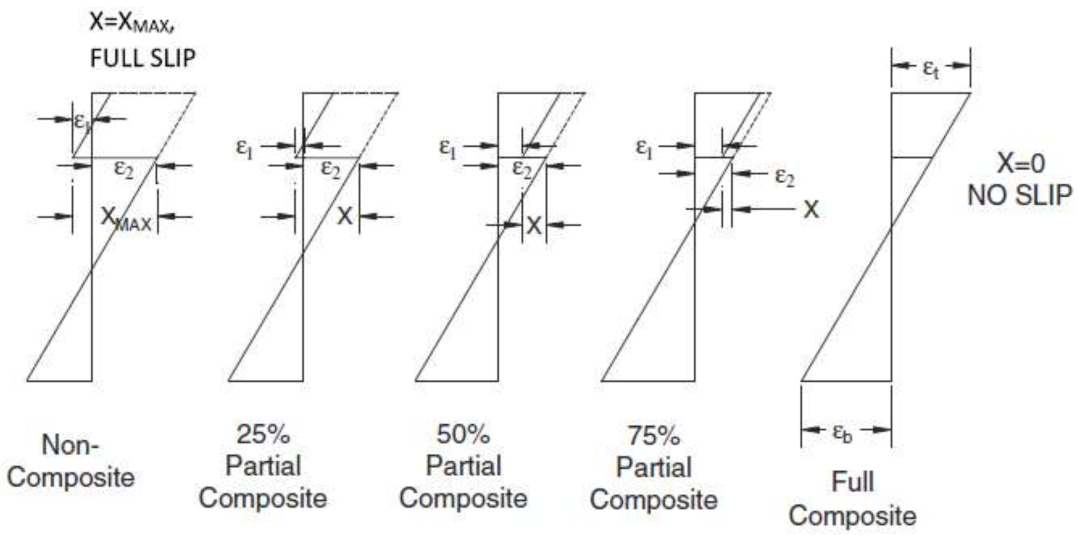


Fig. 14 Strain profile considering different degree of composite action (Chen and Yossef 2015)

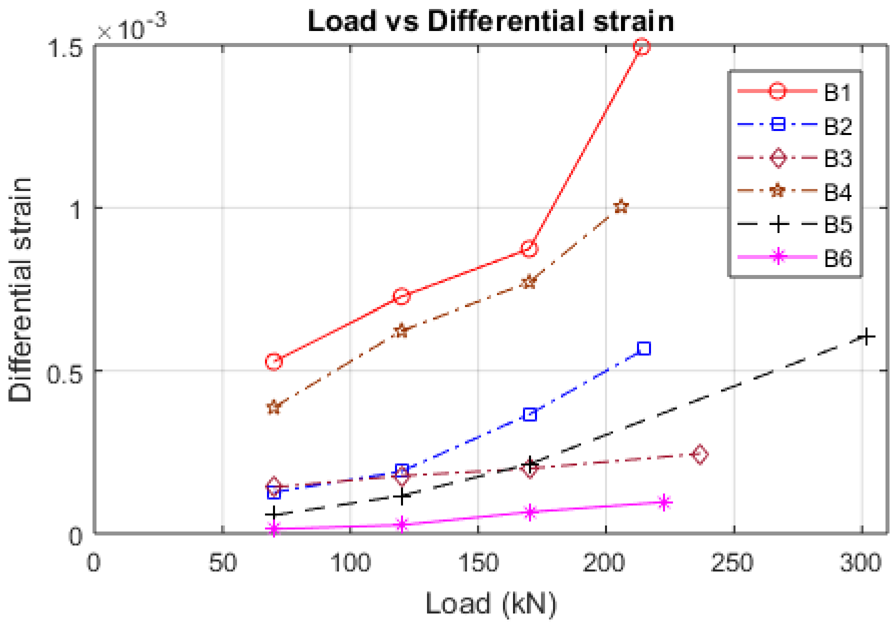


Fig. 15 Differential strain at various load level for all the beams

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