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Flexural behavior of cold-formed steel concrete composite beams

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Abstract. Flexural behavior of thin walled steel-concrete composite sections as cross sections for beams is investigated by conducting an experimental study supported by applicable analytical predictions. The experimental study consists of testing up to failure, simply supported beams of effective span 1440 mm under two point loading. The test specimens consisted of composite box and channel (with lip placed on tension side and compression side) sections, the behavior of which was compared with companion empty sections. To understand the role of shear connectors in developing the composite action, some of the composite sections were provided with novel simple bar type and conventional bolt type shear connectors in the shear zone of beams. Two RCC beams having equivalent ultimate moment carrying capacities as that of composite channel and box sections were also considered in the study. The study showed that the strength to weight ratio of composite beams is much higher than RCC beams and ductility index is also more than RCC and empty beams. The analytical predictions were found to compare fairly well with the experimental results, thereby validating the applicability of rigid plastic theory to cold-formed steel concrete composite beams.

Keywords: composite beam; empty beam; shear connector; flexural strength; ductility

1. Introduction

In thin walled or cold formed steel sections, width to thickness ratio of plate elements is always large and the flexural failure occurs by buckling and not by yielding (Yu 2000), which limits its load carrying capacity. Instead, if the section is filled with concrete, not only is the premature buckling of thin plates prevented, but also steel section provides confinement to concrete thereby increasing the flexural capacity brought about by composite action.

One of the most important developments in steel -concrete composite construction is the use of composite slabs in which cold formed thin walled steel sheeting has been utilized successfully throughout the world. However thin walled composite sections as cross section for beams is a relatively new concept (Hossain 2003) and can serve as a suitable replacement for hot-rolled steel or reinforced concrete beam for small to medium, both spans and loading. Steel acts as formwork in the construction stage and later on it takes-up load in the service stage resulting in reduction in construction cost to some extent.

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Research work has been carried out on the behavior of steel concrete composite beams in several ways such as cold formed steel elements as soffits of the beam (Nguyen 1991), composite beam with profiled section (Oehlers 1993, Oehlers, Wright and Burnet 1994), composite beam with various interface connections (Hossain 2003, 2005), composite thin walled closed flexural members with in-filled concrete (Dubey, Nayak and Bhattacharyya 1999, Soundarajan and Shanmugasundaram 2008), modular composite profile beams (Ahn and Ryu 2007) and RCC beam provided with cold formed plain sheet, and unstiffened and stiffened channel in the tension zone (Kottiswaran, Sundararajan 2007). Recently studies on prefabricated cage reinforced steel concrete composite beam have been carried out (Chitra, Thenmozhi 2011, Chitra, Thenmozhi and Revathi 2011).

Earlier an experimental study was conducted on flexural behavior of empty cold formed steel channel sections with lip on compression side and lip on tension side (Sowmya and Valsa 2008). It was shown that lip on tension side has more flexural strength than that of lip on compression side. The study is now extended to composite section to quantify the composite behavior.

This paper discusses the flexural behavior of thin walled composite beams by means of an experimental study conducted mainly on two types of cross sections, namely, channel section with lip and box section. Behavior of composite section with bar type and bolt type shear connectors is included in this study. Beams of empty cross section (non composite) and equivalent rectangular reinforced concrete section were also tested for comparison. The test results are compared with analytical predictions based on Rigid plastic theory (Oehlers *et al.* 1994, Hossain 2003).

2. Analytical study

Rigid plastic analysis theory is used for developing the equations for moment carrying capacity of composite beams. Consider a simply supported composite beam with channel section having lip on compression side in-filled with concrete. The flexural strength of the composite beam can be determined by considering the distribution of forces in the concrete and steel sections. The two cases of interaction between steel and concrete, full and partial are considered.

Let $P_c =$ Force due to concrete, $P_B =$ Bond Force, $b_c =$ Width of concrete, N_c and N_s = Distances of neutral axis from the extreme compression face for concrete and steel respectively, M = Moment carrying capacity of composite beam, γ = Reduction Factor , $P_t =$ Force at top flange lip, $P_{wt} =$ Force at web portion above neutral axis, $P_{wb} =$ Force at web portion below neutral axis, $P_b =$ Force at bottom flange, b = Width of bottom flange, $d_b =$ Distance of bond force from compression face, d = Depth of web, l = Lip length, t = Thickness of steel, f_y = Yield stress of steel, $f_{ck} =$ Characteristics compressive strength of concrete.

The distribution of strains and forces individually in concrete and steel across the cross section is shown in Fig. 1. Cross section of composite beam, strain diagrams for full interaction and partial interaction are shown in Fig. 1(a) and corresponding forces in the individual concrete and steel section are shown in Figs. 1(b) and (c) respectively.

From Fig. 1(b) considering the equilibrium of forces in concrete

$$N_c = \frac{P_B}{0.85\gamma\, f_{ck} b_c} \tag{1}$$

where γ the reduction factor = 0.85γ- 0.007 (f_{ck} -28) (Oehlers, Wright and Burnet 1994)

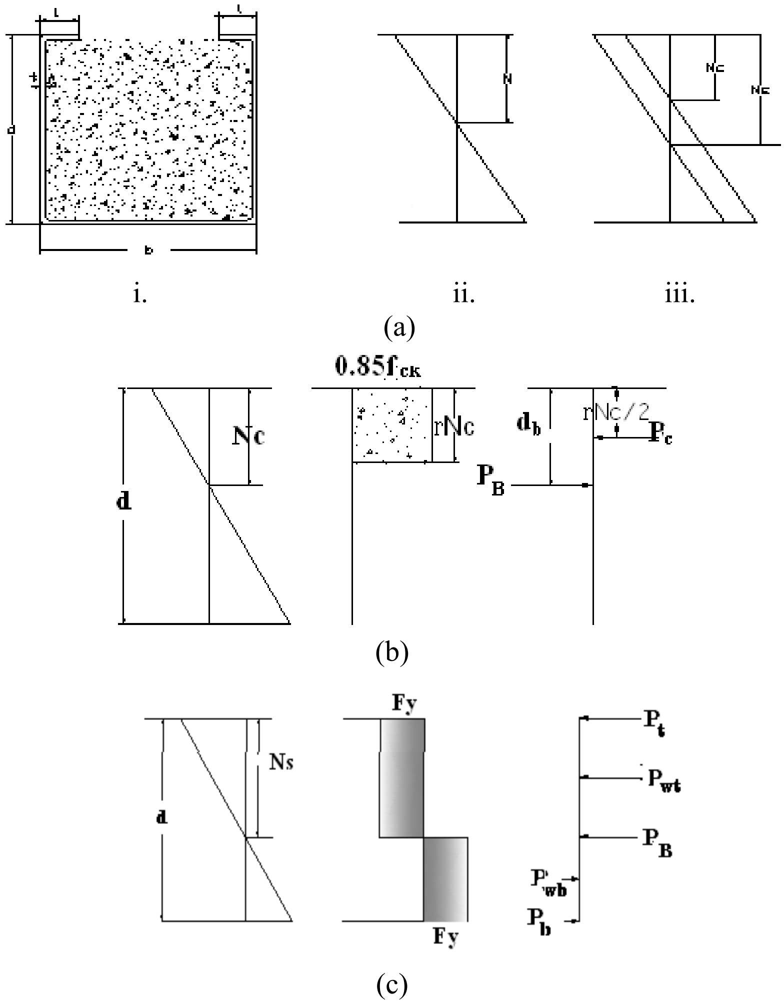


Fig. 1 (a) i. Cross section; ii. strain diagram for full interaction; iii. Strain diagram for partial interaction; (b) and (c) Distribution of forces in the individual concrete and steel section respectively

From Fig. 1(c) considering the equilibrium of forces in steel

$$N_s = \frac{tf_y(2d + b - 2l) - P_B}{4tf_y} \tag{2}$$

Taking moment of all the forces about the top fiber of the beam, the moment carrying capacity M of the channel section composite beam, can be determined from the expression.

$$M = tf_y(d^2 + db_c - 2N_s^2) - 0.425\gamma^2 N_c^2 f_{ck} b_c - t^2 f_y (l + 0.5b) \tag{3}$$

2.1 For full interaction between steel and concrete

$N_c = N_s = N$

$$P_B = \frac{0.85\gamma f_{ck} b_c t f_y (2d + b = 2l)}{0.85\gamma f_{ck} b_c + 4tf_y} \tag{4}$$

2.2 Partial interaction between steel and concrete

For partial interaction $N_c \neq N_s$, if the degree of interaction is 75% then replace P_B by 0.75(P_B).

2.3 No interaction between steel and concrete

$P_B = 0$ and hence $N_c = 0$.

In similar way for the box section, the corresponding forces and the moment carrying capacity can be written as

$$N_c = \frac{P_B}{0.85\gamma\, f_{ck} b_c} \tag{5}$$

$$N_s = \frac{2tf_y d - P_B}{4tf_y} \tag{6}$$

$$P_B = \frac{1.7\gamma f_{ck} b_c d t f_y}{0.85\gamma f_{ck} b_c + 4tf_y} \tag{7}$$

$$M = tf_y(d^2 + db_c - 2N_s^2) - 0.425\gamma^2 N_c^2 f_{ck} b_c \tag{8}$$

The analytical equations are unable to take into account the effect of shear connectors on the enhancement of shear bond. The effect of shear bond can be incorporated indirectly by including buckling of steel plates. It is known that buckling stress (σ_b) of steel plates can be calculated using Eq. (9).

$$\sigma_b = \frac{K_b \pi^2 E_s}{12(1-\mu^2)\left(\frac{d}{t}\right)^2} \tag{9}$$

where K_b is the bending buckling coefficient for different degrees of edge restraint (Є), Es is the modulus elasticity of cold rolled sheet and μ is the Poisson's ratio. K_b can be taken from the design chart proposed by Hossain (2003, 2005). For beams with yield stress less than buckling stress, moment carrying capacity can be predicted using yield stress of steel and for beams with yield stress greater than buckling strength, moment carrying capacity can be predicted using buckling stress.

3. Experimental Programme

Sixteen specimens consisting of three series *B*, *C* and *R* representing respectively box, channel and reinforced concrete beams of effective span 1440mm were considered in the investigation, the details of which are shown in Table 1. For *B* and *C* series: Breath b = 85 mm, Depth d = 85 mm, Lip length L = 15 mm and Thickness t = 2 mm. In composite sections of *B* and *C* series, few beams were provided with shear connectors. Two different orientations were considered for *C*-series of beams. In first case, specimens were oriented such that lip was on the compression side and in second case lip was on the tension side (Fig. 2). For *R* (*R*(*C*), *R*(*B*)) series, reinforcement consisting of 8 mm and 10 mm diameter HYSD bars were used at top and bottom respectively and 2 legged 8 mm diameter stirrups were provided at a spacing of 100 mm.

3.1 Shear connectors

Two types of shear connectors, bar type and bolt type were used. Bar type shear connectors were obtained from the left over pieces of the same sheet which was used for making the channel and box sections. The bar type shear connectors of size 10*30 mm obtained and mild steel bolts of 4mm diameter and 25 mm length, were welded to the beam. Fig. 3 shows the arrangement of shear connectors.

3.2 Materials used

To get actual yield stress of cold formed steel sheet out of which the channel and box sections were prepared, three coupons were cut out and prepared as per the Indian Standard IS 1608-2005. From the tensile test conducted using tensometer, the yield stress of 172.5 MPa and ultimate Stress of 280 MPa were obtained. Ordinary Portland cement of 53-grade, coarse aggregate 12 mm down size and natural river sand (medium) were used to obtain the concrete of grade M40, which was used to obtain the composite beams and equivalent reinforced concrete beam.

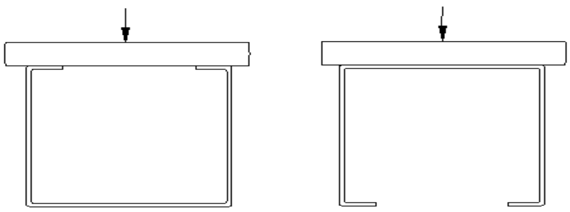
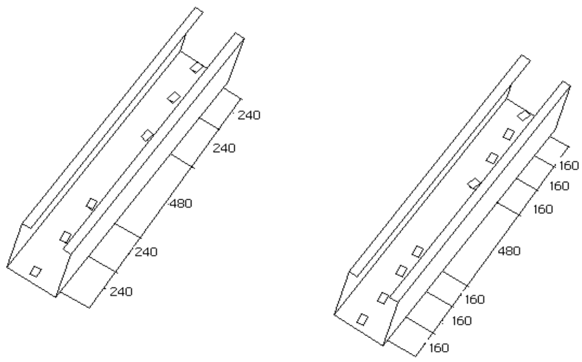


Fig. 2 Sectional view of orientation of *C*-series of beams



(a) (b)

Fig. 3 Arrangement of shear connectors (a) 6Nos; and (b) 8Nos

Table 1 Cross sectional details of beams tested

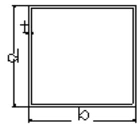
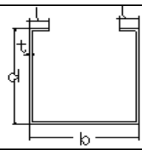
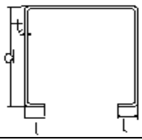
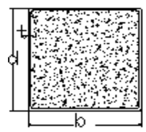
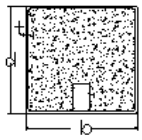
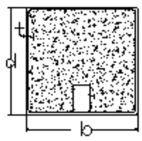
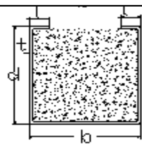
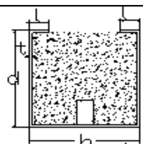
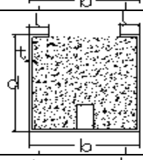
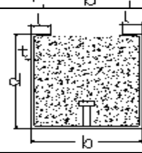
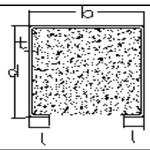
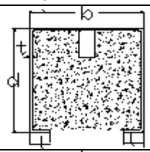
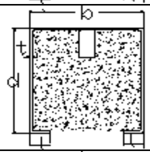
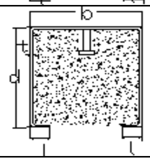
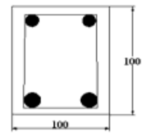
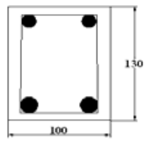
Sl No.	Description	Designations	Cross section
1	Box empty	B	
2	Channel top flange lip empty	Ct_f	
3	Channel bottom flange lip empty	Cb_f	
4	Box filled (without shear connector)	BF	
5	Box filled 6 bars (as shear connector)	BFS_6	
6	Box filled 8 bars (as shear connector)	BFS_8	
7	Channel filled top flange lip (without shear connector)	CFt_f	
8	Channel filled top flange lip 6 bars (as shear connector)	CFt_fS_6	
9	Channel filled top flange lip 8 bars (as shear connector)	CFt_fS_8	
10	Channel filled top flange lip 6 bolts (as shear connector)	CFt_fB_6	

Table 1 Continued

11	Channel filled bottom flange lip (without shear connector)	CFb_f	
12	Channel filled bottom flange lip 6 bars (as shear connector)	CFb_fS_6	
13	Channel filled bottom flange lip 8 bars (as shear connector)	CFb_fS_8	
14	Channel filled bottom flange lip 6 bolts (as shear connector)	CFb_fB_6	
15	Reinforced concrete beam having an equivalent theoretical moment carrying capacity as that of channel section composite beam	$R(C)$	
16	Reinforced concrete beam having an equivalent theoretical moment carrying capacity as that of box section composite beam	$R(B)$	

3.3 Procedure to obtain composite test specimen

The externally painted empty channel and box sections were kept in horizontal and vertical positions respectively. Concrete was poured from the open end. After 24 hours of Casting, all beams were covered with wet gunny bags for 28 days. Standard cubes and prisms were cast to determine the compressive strength of concrete. After 24 hours, the control specimens were de-molded and kept immersed in water for 28 days. The average value of compressive strength was found to be 51 MPa.

3.4 Testing procedure

A precession loading frame of 2000 kN capacity was used for testing the beams. Two point loads were applied transversely at one third distance from support using a cross beam to provide a pure bending region in the central portion of the beam.

Load was applied gradually using a hydraulic jack, in increments till failure of the beams. Dial gauges of sensitivity 0.01 mm were used to measure the deflections of the beams at the mid span and at the points of application of load as shown in Fig. 4. The behavior of beams was keenly

Fig. 4 Test setup

observed throughout the loading range till failure. Also the appearance of initial separation, buckling, propagation of cracks and slip were observed and recorded.

4. Test results and discussion

Load-mid span deflection curves are plotted for *B*, *C* and *R* series of beams as shown in Fig. 5 to 8**.** Ductility index based on load deflection curve for each series of beams *B*, *C* and *R* is shown in Fig. 9. Strength to weight ratio comparison for empty, composite and *RCC* beams is shown in Fig. 10. Typical failure patterns are shown in Figs. 11-14. The theoretical ultimate load computed using Eqs. (3) and (8) is compared with the experimental ultimate loads resisted are reported in Table 2.

4.1 Ultimate load

The experimental ultimate loads resisted by *B*, *C* and *R* series of beams are reported in Table 2. In *B* series of beams, load carrying capacity of composite box section without shear connector was found to be 38.77% more than empty box section. When 6 and 8 numbers of shear connectors were provided to composite box section beams, the load carrying capacity was increased to 53.06% and 48.98% respectively. On an average, the measured load carrying capacity of the entire composite beams is 46.94% more than the empty beam.

In *C* series, the experimental ultimate loads of all composite beams without shear connectors on an average was 33.3% more than empty beam. The composite beams without shear connectors with bottom flange lip (CFb_f) carried 14.63% more load than top flange lip (CFt_f). With shear connectors, on an average, the composite beam with top flange lip exhibited 43% more load carrying capacity than the composite beam with bottom flange lip.

It is also interesting to compare the behavior of channel section beams between bottom flange lip and top flange lip with and without shear connectors. In case of both empty and composite section without shear connectors the section with bottom flange lip (Cb_f and CFb_f) showed higher ultimate loads than the section with top flange lip (Ct_f and CFt_f). However the behavior is reversed when shear connectors are provided. Channel sections with top flange lip with 6 and 8 number of shear connector (CFt_fS_6-CFt_fS_8), showed tremendous increase in load of 126% and 113% and channel sections with bottom flange lip with 6 and 8 number of shear connector showed 40% (CFb_fS_6-CFb_fS_8) increase in load in comparison to empty sections. The higher load carrying capacity exhibited by Channel sections with top flange lip with shear connectors can be attributed to the availability of higher amount of steel on tension face. This observation can also be made referring the predicted ultimate load (P_{th}) reported in Table 2.

Comparing the composite sections, the increase in load carrying capacity with shear connectors is more in *C* series than in *B* series. Also it can be noticed that there is little reduction in load carrying capacity when the number of shear connectors increased from 6 to 8 in both the cases. No difference in the behavior between bar type and bolt type of shear connectors was observed.

4.2 Load-mid span deflection behavior and ductility index

The variations of measured deflection at mid span of each beam tested for known applied load for *B*, *C* and *R* series of beams are presented in Figs. 5-8.

In *B* series, the entire composite beams (*BF*, BFS_6 and BFS_8) follow the same load deflection curve (Fig. 5) up to 20 kN with a deflection of 14 mm, but later on exhibit slight change in load

Table 2 Comparisons between theoretical and experimental loads

Sections	Theoretical load P_{th} kN	Experimental load P_{exp} kN	P_{exp} / P_{th}
Empty Section	AS/NZS 4600-2005		
B	12.586	14.782	1.174
Ct_f	7.293	9.352	1.282
Cb_f	7.292	10.559	1.448
Composite section	Rigid plastic analysis 100% interaction (No interaction)		
BF	18.896	20.513	1.085
BFS_6	18.896	22.625	1.191
BFS_8	18.896	22.022	1.165
CFt_f	17.959 (11.759)	12.369	0.688 (1.051)
CFt_fS_6	17.959	21.117	1.176
CFt_fS_8	17.959	19.91	1.108
CFt_fB_6	17.959	21.117	1.176
CFb_f	12.456	14.179	1.138
CFb_fS_6	12.456	14.782	1.186
CFb_fS_8	12.456	14.782	1.186
CFb_fB_6	12.456	13.877	1.114
Reinforced beams	IS 456-2000		
R(*C*)	17.959	16.893	0.940
R(*B*)	18.896	30.77	1.628

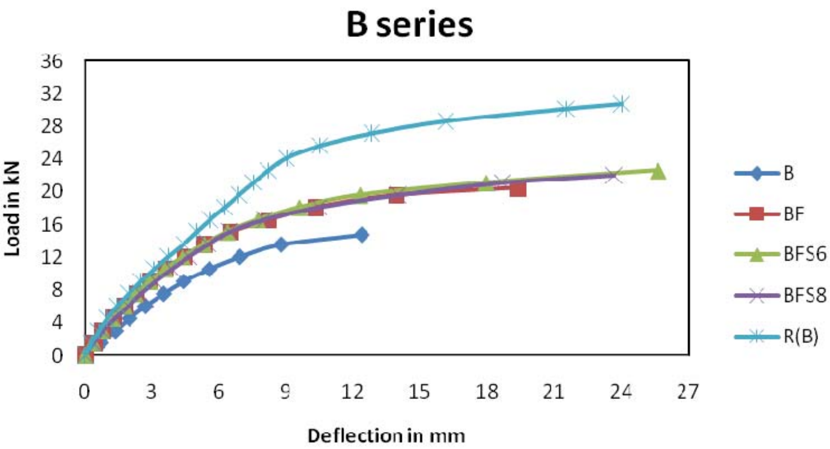


Fig. 5 Load vs. deflection graph for *B* Series of Beams and *R*(*B*)

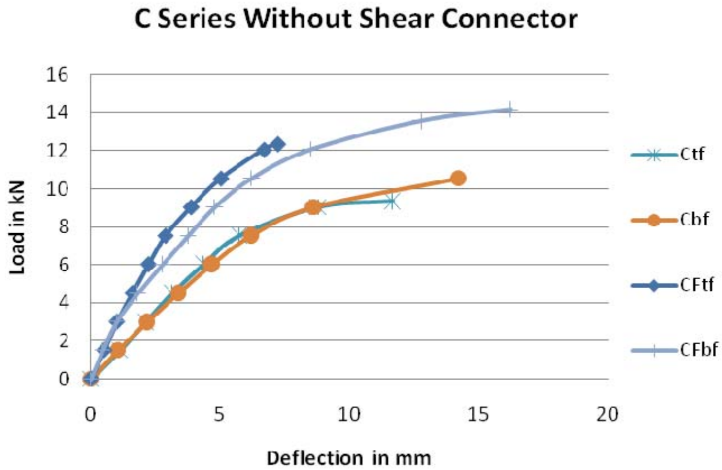


Fig. 6 Load vs. deflection graph for *C* series without shear connectors

deflection values up to failure. Composite beam with 6 numbers of shear connectors recorded 10.3% increase in load and 32% increase in deflection when compared with composite beam without shear connectors. Composite beams experienced superior load deflection behavior than empty (*B*) beams (Fig. 5). Composite beam with 6 number of shear connector recorded 36% decrease in load and 8% increase in deflection when compared with *RC* beam (*R*(*B*)).

In *C* series as shown in Fig. 6, channel section with bottom flange lip (CFb_f) carried maximum load and deflection both in case of empty and composite sections when shear connectors are not provided. When shear connectors are provided, composite section with top flange lip (CFt_fS_6) withstood maximum load and deflection as shown in Fig. 7. The increase in load and deflection at failure of composite beam is found to be 70.7% and 228.97% respectively more than composite beam without shear connectors. The behavior of all channel beams tested is shown in Fig. 8. The increase in load and deflection at failure of composite beam (CFt_fS_6) is found to be 25% and 45% respectively more than *RC* beam (*R*(*C*)).

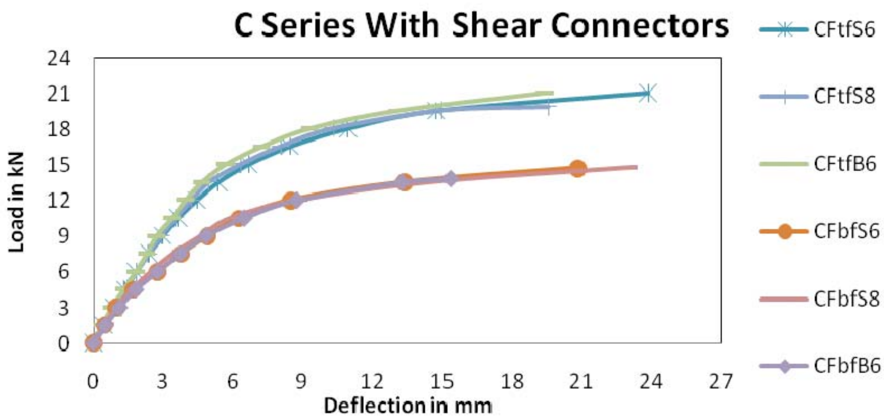


Fig. 7 Load vs. deflection graph for *C* series with shear connectors

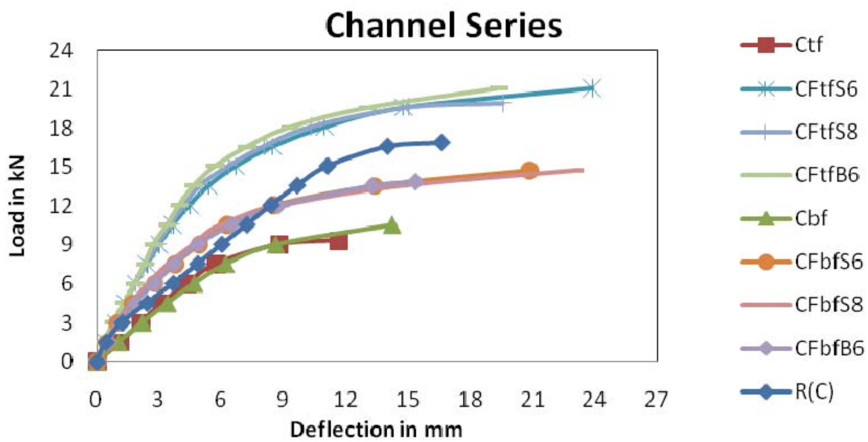


Fig. 8 Load vs. deflection graph for *C* series of beams and *R*(*C*)

Fig. 9 shows magnitudes of ductility index based on deflection for all series of beams. Ductility index is calculated as the ratio of ultimate deflection to the deflection at yield (taken as point at which non-linearity starts). Composite sections with shear connectors show higher ductility index than corresponding empty and *R* series of beams. The composite action increased the ductility index in the range of 2.37 to 3.32 for box and 2.37 to 4.44 for channel section. The shear connectors improved the behavior further. Composite beam with top flange lip having 6 number of shear connectors (CFt_fS_6) shows maximum ductility index of 4.4.

4.3 Strength to weight ratio

Fig. 10 shows the comparison of strength to weight ratio of *C*, *B* and *R* series of beams. Empty sections have maximum strength to weight ratio followed by composite and then *RCC* beams. Empty box section showed higher strength to weight ratio than empty channel section, though hardly any difference was recorded between the corresponding composite beams.

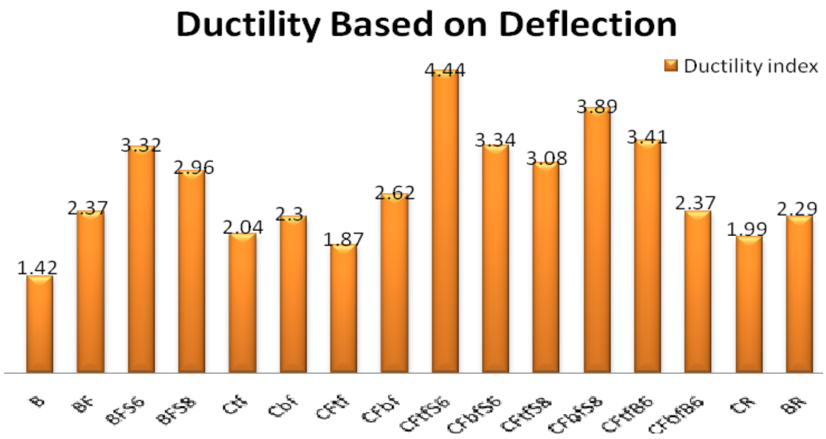


Fig. 9 Ductility of all series of beams

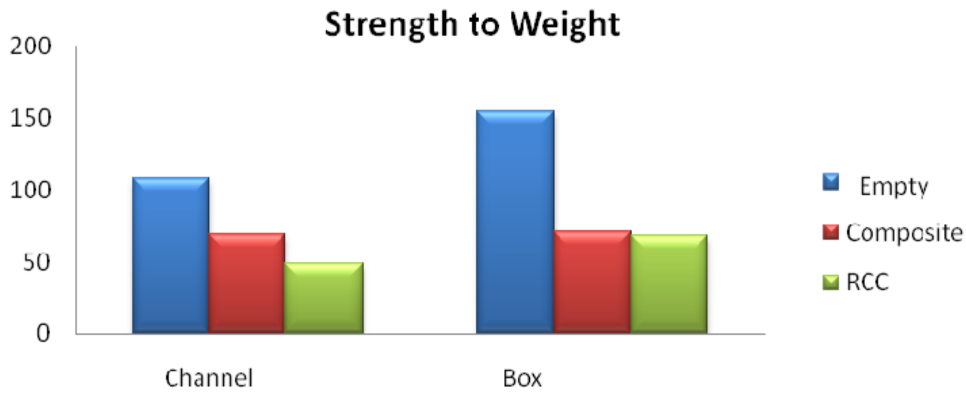


Fig. 10 Strength to Weight ratio of *C*, *B* and *R* series of beams

4.4 Failure pattern

It was observed that in case of empty beams, local buckling took place in the compression zone near point of application of load leading to failure. In composite beams also local buckling occurred in compression zone of the beams, but at a load higher than that of empty steel beams as shown in Fig. 11 which lead to failure. However the local buckling phenomena, in both cases took place only after yielding of steel.

4.5 Comparison of experimental and theoretical result

The theoretical ultimate load carrying capacity of each beam is compared with the experimental ultimate load resisted by each series of beams as given in Table 2. Theoritical ultimate loads (P_{th}) of composite channel and box beams are calculated using equation 5 and 8 respectively and those of empty beams using Australian/New Zealand code (AS/NZS 4600-2005). The experimental loads are generally higher than the theoretical loads but are close to the theoretical results.

(a)

(b)

(c)

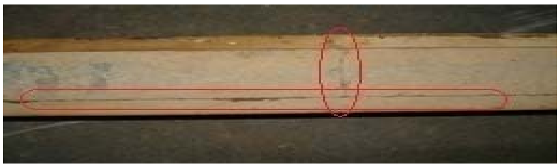
(d)

Fig. 11 Local buckling phenomena (a) *B*; (b) Ct_f; (c) Cb_f; (d) *BF*

Buckling stress of steel plates is calculated by Eq. (9). Taking percentage restraint (Є) as zero and $k_b = 24$, buckling stress obtained is 2400 N/mm^2. All series of beams have d/t ratio equal to 42.5. It is found that the buckling stress is much higher than the yield stress of 172.5 N/mm^2 even at 0% restraint. Hence yielding of steel plate occurs before buckling and analysis based on yielding of steel (rigid plastic theory) governs. The yielding of steel is also evident from the extended deflection depicted by load-deflection curves shown in Figs. 5-8 and higher values of ductility index. For all series of composite beams, good agreement between experimental and theoretical loads confirms the above statement.

It can be seen that the ratio P_{exp}/P_{th} is lower for composite sections than for empty sections. This proves the applicability of rigid plastic theory adopted for composite sections. For composite channel section without shear connectors (CFt_f) experimental load is closer to the theoretical result for no interaction. In the case of *RCC* beams, experimental load of *R*(*C*) beam was slightly less than the theoretical value, but for *R*(*B*) beam the experimental load was found to be much higher than the theoretical load.

(a)



(b)

(c)

Fig. 12 Failure pattern of CFt_f (a) Longitudinal view; (b) Crack and separation of concrete from steel; (c) slip of concrete

(a)

(b)

(c)

Fig. 13 Failure pattern of $CFtfS_6$ (a) longitudinal view; (b) separation of concrete from steel ; (c) slip of concrete

(a)

(b)

Fig. 14 Failure Pattern of *CFbf* (a) longitudinal view; (b) bottom view

5. Conclusions

1. Good agreement between proposed analytical model and the experimental results were observed in the present study. Hence analytical models based on rigid plastic theory developed are applicable for determining the flexural strength of cold formed steel concrete composite beams.
2. The load carrying capacities of composite box and channel sections without shear connectors were found to be 38.77% and 32.26% respectively more than empty box and channel sections.
3. Provision of shear connectors were found to substantially enhance the ultimate load carrying capacity of composite box and channel sections to an extent of about 51% and 120% respectively.
4. For empty and composite channel sections without shear connectors, the load carrying capacity is more when lip is on the tension side. Whereas for composite channel section with shear connectors, the behavior is reversed with load carrying capacity being more when lip is on compression side.
5. Magnitudes of ductility index of composite beams are considerably higher than *RCC* beam and empty beams .Composite channel beam with 6 numbers of bar type of shear connectors when top flange lip is under compression showed maximum ductility index of 4.44.
6. The Strength to weight ratio of composite channel section beams was 30% more than *RCC* (*CR*) beams.
7. Yielding of steel followed by local buckling, slip, cracking of concrete and separation of concrete from steel governed the failure of composite beams.

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