Structural Engineering and Mechanics, *Vol. 70, No. 3 (2019) 303-310*
DOI: https://doi.org/10.12989/sem.2019.70.3.303

Strengthening of capacity deficient RC beams - An experimental approach

M. Adil Dar[1], N. Subramanian[2], A.R. Dar*[3], Amer Iliyas Rather[3], Mir Atif[3] and Sayeeda Syed[3]

[1]*Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi, India*
[2]*Consulting Engineer, Maryland, USA*
[3]*Department of Civil Engineering, National Institute of Technology Srinagar, J&K, India*

(Received October 26, 2018, Revised February 13, 2019, Accepted February 24, 2019)

Abstract. Any revision of seismic codes usually demands a higher capacity from structural members, making existing structures unsafe particularly from strength considerations. Retrofitting of capacity deficient members is very suitable for tackling such situations. This paper presents an experimental study on different retrofitting measures adopted for strengthening a series of reinforced concrete (RC) beams. Four identical RC beam specimens were casted, out of which three specimens were strengthened by different schemes (viz., bolted hot rolled flat, bolted cold-formed steel channel, and carbon fibre reinforced polymer (CFRP) laminate, respectively) on their tension face and tested under four-point monotonic loading. This study focuses on the investigation of the flexural behaviour of these retrofitted beams, observed in terms of strength and stiffness. It was concluded that all retrofitting measures improved the structural performance of these beams. However, the cost involved with each strengthening mode was proportional to the improvement in the performance achieved.

Keywords: flexural members; experimental investigation; cold-formed steel; carbon fibre reinforced polymer laminate

1. Introduction

Structures all over the world are susceptible to increasing load demands due to up-gradation of existing codes or unplanned increase in the number of storeys. Although structures were safe under their pre-revision loading, they might become unsafe under post-revision loading. These structures are required to maintain a certain performance level, which includes load carrying capacity from strength and serviceability considerations in addition to durability and aesthetic appearance. Structural failure occurs when a structure, or a part of it, loses the ability to support the load acting upon it. In view of the existing design deficiencies and the performance of faulty construction, there is an urgent need to look for appropriate strengthening measures to ensure safety of structures. Maintaining the desired performance levels in these structures (which were safe earlier but became deficient due to codal revision) becomes a challenge. Such structures can be kept in service either by demolition of the capacity deficient structural members and replacing them with adequately strong new members, or by restricting the maximum load on such members. Since replacement of such capacity deficient new members, or by restricting the maximum load on such members. Since replacement of such capacity deficient members incurs huge amount capital and time, thus, retrofitting becomes the adoptable way of improving their load carrying capacity and extend their service life. In addition, retrofitting will ensure in sustainability of materials. However, increasing the load carrying capacity of the structure has to be complimented by fulfillment of serviceability criteria at these loads (Dhanoa *et al.* 2016, Dar *et al.* 2017a). During the planning stage of structural strengthening, the most crucial decision is of the choice of an appropriate strengthening material which should result in convenient strengthening as well as its durability, at minimum cost (Vasudeva and Kaur 2016). Hot rolled steel flats have been conventionally adopted for the strengthening of capacity deficit beams. They may be attached to the beams by bolts grouted in the soffit of the beam or by the application of epoxy resins (Alam *et al*. 2016). They improve the load carrying capacity of the beam viz., improves its ductility and stiffness.

Su *et al.* (2010) conducted four-point bending tests on simply supported RC and bolted side-plated (BSP) specimens. The test results imply that the strength of the bolts and plates greatly influence the two structural performance criteria of the specimens: post-elastic strength enhancement and displacement ductility. The specimen strengthened by strong bolt arrangement and weak steel plate had sufficient strength enhancement and ductility. The beam strengthened by strong bolt arrangement and strong steel plate experienced brittle and undesirable failure. The depth of steel plates should be controlled, while sufficient bolts should be used to ensure the desirable ductile beam failure. The cost of strengthening arrangement for 'strong bolt-weak plate' is also reduced since the depth of the plate is half the depth required for strong plate arrangement. However, lower depth of steel plate is not as effective as higher depth in enhancing the shear capacity of the beams. Vinay *et al.* (2015) carried out an experimental investigation on simply supported composite beam specimens to understand their flexural performance. The

*Corresponding author, Professor
E-mail: abdulrashid@nitsri.net

http://www.techno-press.com/journals/sem&subpage=7
ISSN: 1225-4568 (Print), 1598-6217 (Online)

beam specimens were tested by subjecting them to four-point loading. The cracking load, load *vs.* deflection behaviour, ultimate load, and failure pattern of the beam specimens were studied. The experimental results indicate that the load carrying capacity of the composite beams increased by up-to 215%. The experimental results also indicated that, the span to depth ratio and shear span to depth ratio influences the rate of improvement in the load carrying capacity of these beams. The mid-span deflections at ultimate load for the composite beams reduced by 50% when compared to the control beams.

Cold-formed steel (CFS) sections are becoming more and more popular since through the continuous research on CFS, highly efficient profiles have been developed to obtain desired properties. They also result in reduction in thickness of the CFS sections as well as cost while ensuring the desired properties in the structure (Anbarasu and Sukumar 2013, 2014, Dar *et al.* 2019a, b, c, d, e, 2018a, b, c, d, Valse *et al.,* 2013). Wehbe *et al*. (2011) developed concrete-CFS composite flexural members through experimental and analytical studies in order to assess their structural performance and failure modes, and to develop optimum beam configurations for the use in light-gauge steel (LGS) construction. The flexural and shear strengths, flexural stiffness, and interface shear transfer were investigated. In their research, only the flexural strength/stiffness characteristics was reported. The results showed that concrete-CFS composite beams can be designed for ductile flexural failure and that the degree of composite action is dependent upon the stand-off screws rather than the configuration.

Recently, carbon fibre reinforced polymer has been used for strengthening purposes. It has high tensile strength in addition to very low weight to volume ratio. As such, it has a high potential in the manufacturing of effective strengthening systems, to improve the flexural strength of RC beams. However, compared to the conventional strengthening techniques, cost of CFRP is relatively high and as such they may prove to be more suitable in special conditions only. Osman *et al*. (2016) reviewed more than sixty papers on reinforced concrete beams with opening and with and without strengthening by fiber reinforcement polymers (FRP) reinforcement. They concluded that the contribution of strengthening materials such as FRP and steel plate to provide additional safety depends on the required design life, environmental and stress conditions, and the FRP type used. Yasmeen *et al*. (2011) experimentally investigated the structural behaviour of damaged full-scale reinforced concrete beams retrofitted with CFRP laminates in shear/flexural zones. The internal reinforcement ratio, position of retrofitting and the length of CFRP were the primary variables considered. Their experimental results indicated that the beams retrofitted by using CFRP laminates in shear and flexural zones are structurally efficient. Also, the laminates restored the stiffness as well as the strength nearly equal to or even greater than those of the control beams. They even found that the efficiency of flexural strengthening varied with thelength of the CFRP laminates adopted. Plate de-bonding in retrofitted beams was the main failure mode observed.

All the previous studies have incorporated different retrofitting approaches for different capacity deficit RC beams, where all of them have performed well. However, the comparison of various retrofitting schemes in terms of performance and cost on similar capacity deficient RC beams has not been conducted.

2. Objectives of this study

This study was undertaken to compare the structural behaviour of retrofitted RC beams using three strengthening measures (viz., bolted hot rolled flat, bolted cold-formed steel channel and carbon fibre reinforced polymer (CFRP) laminate respectively) on their tension face. These beam specimens were tested under four-point monotonic loading. Strength and stiffness of these beams were the main parameters investigated in order to evaluate the efficiency of each strengthening measure.

In order to evaluate the effectiveness of a strengthening scheme, it was essential to obtain the desired benchmark of relevant parameters. Since strength and stiffness of the beams were the main parameters, an RC beam without any strengthening scheme was also tested to quantify its strength and stiffness. This model is named benchmark beam and is shown in Fig. 1. For future references this model is referred as BMB. Three beams, which are identical to BMB were also prepared and later strengthened through the three schemes, mentioned earlier; viz., strengthened by bolted hot rolled flat, bolted cold-formed steel channel, and carbon fibre reinforced polymer (CFRP) laminate respectively, on their tension face. For future references hot rolled flat strengthened beam, CFS channel strengthened beam and CFRP strengthened beam are referred as HRSB, CFSB and FRPSB respectively. The improvement in strength and stiffness of strengthened beams were studied in terms of the percentage enhancement in order to evaluate the efficiency of the strengthening measure adopted.

3. Experimental study

This section describes the preparation of the four models, testing of the materials involved and the test set-up adopted to study their behaviour.

3.1 Model preparation

The cross section of the beam was fixed at 175 mm×250 mm. The total length of the beam was 1700 mm with an effective span of 1500 mm. The longitudinal reinforcement comprised of three bars of 16mm diameter both at tension and compression faces of the beam as shown in Fig. 1. Shear reinforcement was provided using 2-legged stirrups of 8 mm diameter, spaced uniformly at 125 mm centre to centre throughout the span of the beam. An effective cover of 30 mm was adopted on all the four sides in all beams. For HRSB and CFSB two bolts of 10mm diameter at a spacing of 125 mm centre to centre were welded to all the stirrups for holding the hot rolled plate and cold-formed steel channel beam (in real retrofitting, the bolts have to be placed by drilling the concrete and then properly grouting the holes with epoxy or concrete slurry). The predicted

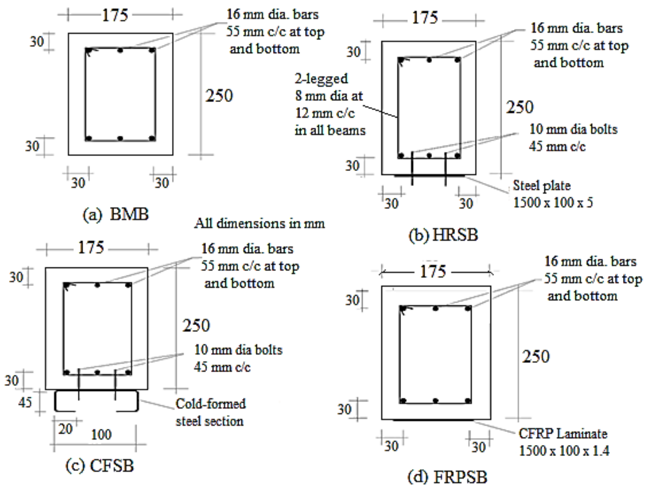


Fig.1 Strengthening Schemes used

Table 1 Material property of steel and concrete

Material	A (mm^2)	f_y (MPa) A	f_y (MPa) B	f_u (MPa) A	f_u (MPa) B	E (GPa) A	E (GPa) B	f_{ck} (MPa) A	f_{ck} (MPa) B
Concrete	-	-	-	-	-	-	-	20	23
Steel reinforcement	-	500	507.6	565	572.3	200	197.2	-	-
Hot rolled steel flat	500	250	290.7	410	430.6	200	198.5	-	-
Cold-formed section	460	350	405.4	500	565.2	200	199.4	-	-
CFRP laminate	140	-	-	2800	2892.5	165	163.3	-	-

A: Nominal values, B: Measured values

design strength of BMB as per Indian Standard (IS 456) is 190 kN. For HRSB, a steel strip 100 mm wide, 5 mm thick and 1500 mm long was attached to the soffit of the beam by bolts spaced uniformly at 125 mm c/c. For CFSB a cold-formed steel C-section 2 mm thick, 100 mm wide and 1500 mm long was attached to the tension face of the beam by bolts spaced uniformly at 125 mm c/c. For FRPSB a CFRP laminate 100 mm wide, 1.4 mm thick and 1500 mm long was attached to the soffit of the beam by epoxy adhesives (Araldite). Concrete of grade M20 was prepared by mixing cement, fine aggregate and coarse aggregate in the ratio of 1:1.5:3 respectively with a water/cement ratio of 0.55, at ambient temperature in the Structural Engineering Laboratory of National Institute of Technology Srinagar. Ordinary Portland cement of grade 43 was used as a binder.

3.2 Material testing

Tests were carried out on different materials used for strengthening in order to determine relevant properties for predicting design strengths. Tensile coupon tests conforming to the ASTM Standards (ASTM E8/E8M-13a) were used to determine the mechanical properties of steel and FRP used. Cubes conforming to the ASTM Standards (ASTM C109/C109M) were cast and tested on 28^{th} day to determine the compressive strength of concrete. Computerized universal testing machine was used for conducting both tensile and compression tests of coupons and cubes. The relevant material properties of the steel and concrete obtained from the material testing are given in Table 1.

4. Experimental setup

The model testing was performed on a 500 kN capacity testing rig 4 m long, 1.2 m wide, and 2.2 m high. The four-point loading as shown in Fig. 2 was applied by means of a hydraulic jack of 500 kN capacity at the rate of 0.20kN/s, which was transferred to the beams through a proving ring of 500 kN capacity. Displacements produced under corresponding loads were recorded by dial gauges of least count 0.01 mm mounted at appropriate locations (See Fig. 3). Since load applied through the hydraulic jack will act at a single point only, a spreader beam (ISMB 175) was used to achieve four-point loading. All the beams were tested under simply supported end conditions. The loading arrangement is shown in Fig. 3. The load was applied in small increments up to the failure of each beam specimen. At every load increment, propagation of cracks in the beams were also observed and recorded by using a crack detection microscope of least count 0.01 mm.

Before carrying out the serious experimental work for achieving well defined objectives from high precision experimental testing, it is essential to critically evaluate the performance of the experimental set-up being used for this

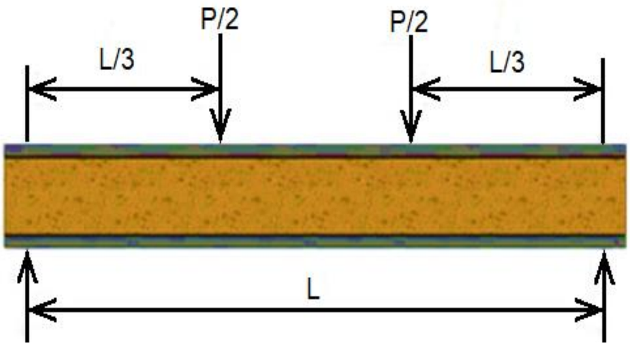


Fig. 2 Four-point loading

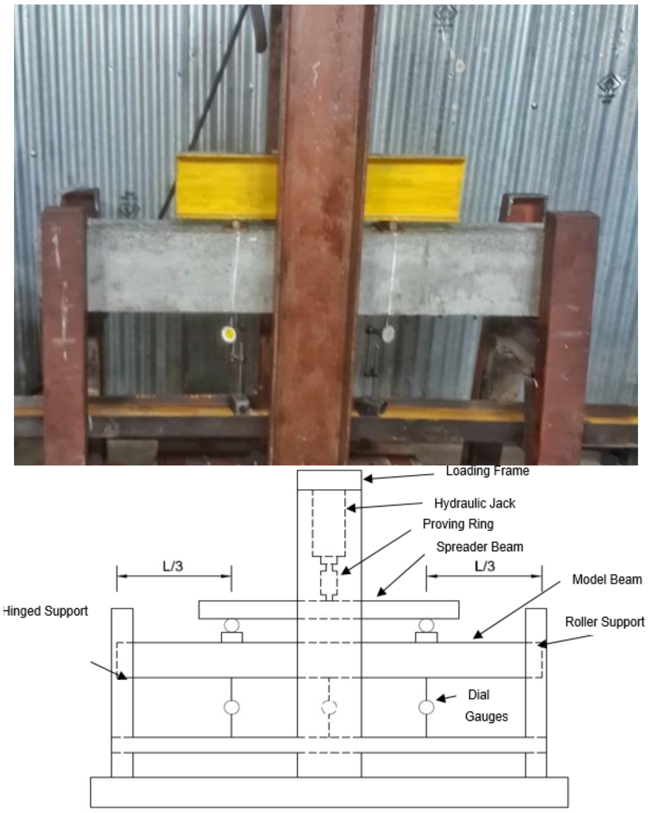


Fig. 3 Loading arrangement

purpose. This is necessary to have confidence in the accuracy and reliability of experimentally measured data. For checking the performance of the experimental set-up, the best course of action is to perform preliminary testing on a trial model. This will not only help in checking the performance of loading frame but also help in identifying the shortcomings (if any) in the trial model, and also provide clues to make necessary changes in the beam models for obtaining better results (Dar *et al.* 2017b, 2015a, b). Hence, a trial model was set-up and loaded up to 10 kN for testing the performance of the loading frame and hydraulic jack.

Since no shortcomings were found in the trial testing, all the four specimens were mounted for testing one after the other. The age of the specimens on the day of testing was 30 days. Mid-span deflections at corresponding loads were recorded for all the model.

Table 2 Comparison of load carrying capacities

Specimen	Ultimate Load Carrying Capacity		
	P_{Des}(kN)	P_{Exp}(kN)	P_{Exp}/ P_{Des}
BMB	191.18	195	1.01
HRSB	313	290	0.93
CFSB	321.6	320	0.99
FRPSB	351	360	1.02
		Ave.	0.98
		Std. Dev.	0.04

Table 3 Comparison of test results for all specimens

Specimen	Test Results					
	δ (mm)	P_{Exp} (kN)	η (%)	K kN/mm	η (%)	M (kNm)
BMB	6.98	195	-	27.93	-	48.75
HRSB	8.54	290	49	33.95	21.55	72.50
CFSB	9.33	320	64	34.29	22.77	80.00
FRPSB	12.02	360	85	29.95	7.23	90.00

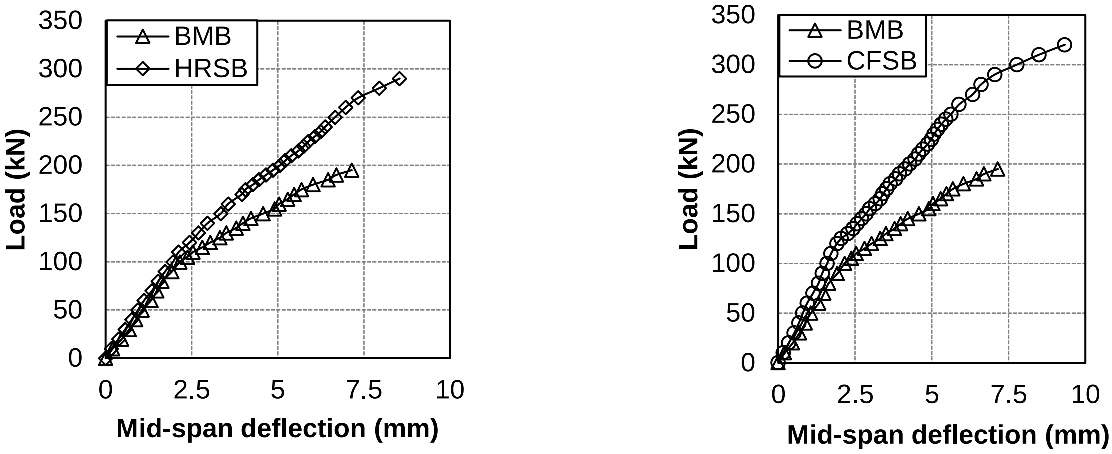
5. Test results and discussions

The performance and effectiveness of the various strengthening measures employed to augment the flexural strength and stiffness of RC beam specimens were compared. A comparison between the theoretical and experimental load carrying capacity of the beam specimens is shown in Table 2.

The results from the testing of all specimens are presented in Table 3. The applied load *vs.* mid-span deflection plots of the specimens are shown in Figs. 4 and 7. The BMB failed at a load of 195 kN by crushing of concrete after the steel reinforcement had yielded. It followed a crack pattern characteristic to flexural member. The first visible crack was observed below the loading point at a load of approximately 65 kN. The crack originated near the soffit of the beam and later propagated upwards vertically. This crack progressed steadily along its length and width up to a load of around 140 kN. Thereafter, the width of the crack started increasing at a higher rate. This is attributed to loss of bond bet-ween concrete and reinforcing steel. Similar cracks surf-aced throughout the length of beam as load was incremented further.

All the strengthened specimens behaved similar to the BMB up to initiation of cracking. However, after cracking, the strengthened specimens showed less deflection at a given load as compared to the BMB.

In the case of HRSB, a vertical flexural crack was observed at mid-span near the soffit of the beam at a load of around 65 kN. The crack originated near the soffit of the beam and propagated upwards. The crack showed a steady progress in its length and width up to a load of around 190kN. Crack width increased at a relatively higher rate as the load was increased beyond 190 kN on account of loss of bond between concrete and reinforcing steel. The load *vs.* mid-span deflection curve (Fig. 4(a)) is fairly linear up to a load of about 110 kN (deflection = 2.14 mm). This initial



(a) BMB and HRSB Load displacement curve for Model A

(b) BMB and CFSB

Fig. 4 Load vs deflection curve

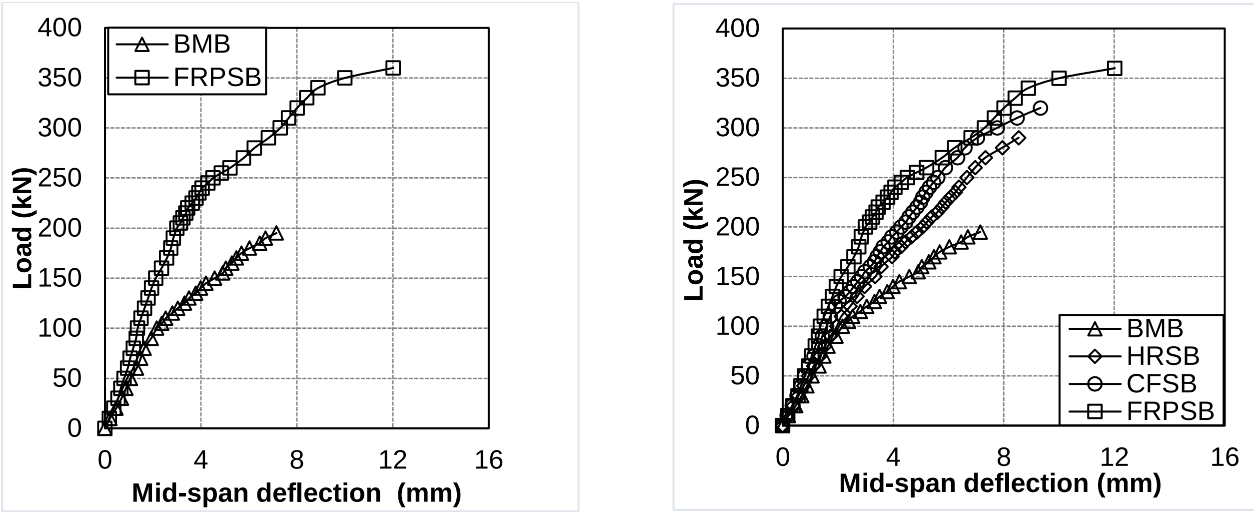
Fig. 5 Emanation of shear crack in CFSB

Fig. 6 De-bonding of CFRP laminate in FRPSB

portion shows that the load is carried mainly by the RC beam. However, a change in slope thereafter indicates attainment of yield with increased contribution towards strength and stiffness from hot rolled steel flat (as slope of the curve is steeper than that for BMB). The curve, although less steep, was nearly linear up to a load of 260 kN (plastic stage followed by strain hardening) and flatter thereafter (ultimate strength of the beam). The specimen failed at a load of 290 kN (deflection = 8.54 mm) by a combination of loss of bond between concrete and bolts (pull out of embedded bolts). The crack width was less than that of the BMB due to the presence of steel flat. It was observed that there was 49% increase in the ultimate load carried by HRSB in comparison to BMB. The maximum value of mid-span deflection in HRSB at failure was 8.54 mm.

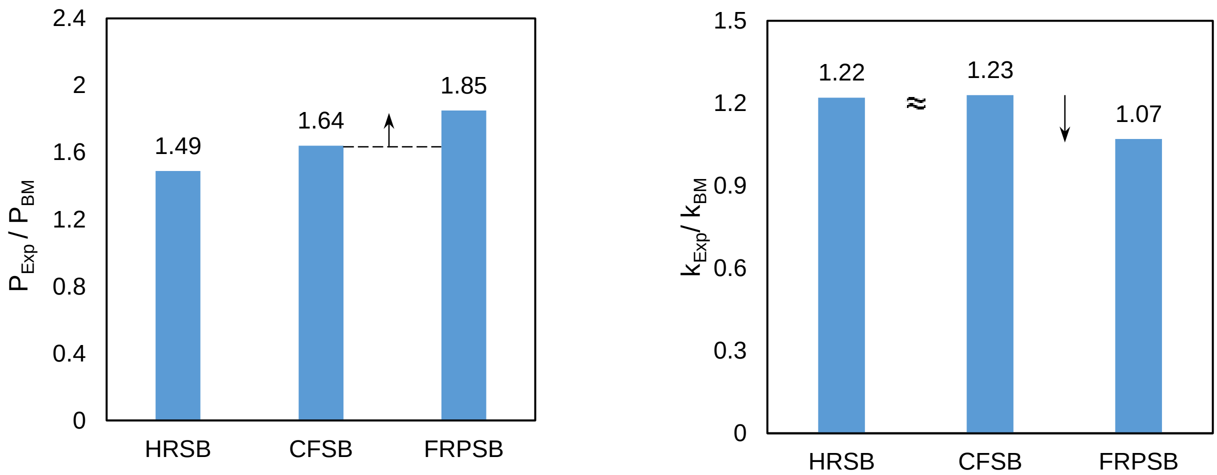
For CFSB, the formation and propagation of cracks was similar to HRSB. The load *vs.* deflection curve (Fig. 4(b)) is a fairly straight line up to a load of 120 kN (deflection = 1.91 mm). With further increase in load, load *vs.* deflection curve flattened towards the x-axis (implying that the specimen crossed elastic limit), however, the slope was steeper than that of HRSB. Thus, the incorporation of CFS section has improved the stiffness better than that of by the addition of steel plate. A fine shear crack originated close to the neutral axis of the beam specimen near the support at a load of around 210 kN. With increase in the loading, up to 250 kN, a flexural crack started originating in the concrete beam, near the CFS section and joined the diagonal crack. With further increase in load, the diagonal crack started widening with minimal increase in its length towards the top of the specimen. Finally, the crack width increased considerably leading to bearing failure in the specimen. The flexural crack that originated near the CFS section, maybe due to the loss of bond between concrete and embedded bolts (pulling out of bolts), and is the zone attracting the maximum shear force. However, the failure here occurred due to emanation of a diagonal crack near the support as seen in Fig. 5 and its rapid widening followed by crushing of concrete at compression face of the beam specimen at a load of 320 kN. The ultimate load carried by CFSB was observed to be 64% higher than that of BMB. The maximum recorded deflection in the specimen was 9.33 mm. On comparing the results of CFSB with those of HRSB, the area of structural steel used for strengthening HRSB and CFSB was 500 mm^2 and 460 mm^2 respectively. However, CFSB resisted a 10% higher load as compared to HRSB. At a given load, stiffness of CFSB was higher compared to HRSB.

In case of FRPSB, flexural cracks started originating around the loading point of the beam specimen. With further increase in load, cracks were observed in the



(a) BMB and FRPSB (b) All specimens

Fig. 7 Comparison of load vs. deflection curve



(a) Strength gain in various specimens (b) Stiffness gain in various specimens

Fig. 8 Comparison of gain in strength and stiffness

middle-third span of the beam. The load *vs.* deflection curve (Fig. 7(a)) is a nearly straight up to a load of 140 kN (deflection = 1.94 mm) implying that the specimen yielded at this load. With further increase in load, slope of the load *vs.* deflection curve dropped, however, it was greater than that of for both HRSB and CFSB. Thus, it is concluded that improvement in stiffness is higher in FRPSB as compared to other two retrofitted specimens. At a load of about 170 kN, a crack originated between the supports and loading point of the beam specimen. The crack width of this crack increased more rapidly as compared to other cracks initiating de-bonding at the epoxy concrete interface (as shown in Fig. 6) which progressed towards the centre on the specimen and finally lead to failure at a load of 360 kN with extensive cracking throughout the specimen. Maximum deflection recorded at failure was 12.02 mm.

Amongst all the specimens, it was observed that the ultimate load resisted by FRPSB was the highest as shown in Fig. 7(b). The ultimate load, in this case, was 85% more than that of BMB, 24% and 12.5% more than that of HRSB and CFSB respectively. However, the cost of strengthening using different materials was also found to match with the strength achieved. A comparison of the strength and stiffness gain is shown in Fig. 8.

It must be noted that the various strengthening schemes of the beam specimens have other implications as well. The CFSB doesn't look aesthetically pleasing, and the CFS part protrudes out and does not look pleasing. Furthermore, CFS sections due to their thin-walled nature may be prone to buckling (if subjected to compressive loads). Therefore, CFS sections with large geometric imperfections might not prove to be advantageous. A comparison of stiffness characteristics indicated that FRPSB did not perform satisfactorily. Hence, in cases where stiffness is a governing criterion, FRPSB should not be used.

6. Summary and conclusions

An experimental investigation was carried out to study the performance and effectiveness of various retrofitting schemes which may be adopted for flexural strengthening of capacity deficit RC beams. The improvements achieved in terms of strength and stiffness were compared to an un-strengthened benchmark beam specimen (BMB). Based on this study, the following conclusions were drawn:

• The design strength prediction of beam specimens as per the Indian code, IS 456, matched well with the experimental results with an average of 0.98 and a standard deviation of 0.04 for P_{Exp}/P_{Des}.

• In terms of strength, FRPSB performed the best, followed by CFSB and HRSB. The percentage strength improvement was 85%, 64% and 49% respectively with respect to the BMB. This was primarily due to the composite action in these beam specimens, which resulted in change in the geometrical properties resulting in significant strength improvement.

• In terms of stiffness, CFSB and HRSB performed nearly the same, followed by FRPSB. The percentage stiffness improvement was 23%, 22% and 7% respectively with respect to the BMB. For CFSB, the increase in moment of inertia of the specimen due to the flanges of the cold-formed steel section (being along the depth of beam) explains the higher improvement in its stiffness when compared to HRSB. The increase in stiffness of FRPSB is attributed to high tensile strength of CFRP laminate.

• Failure in CFSB occurred due to emanation of shear crack at the support and its diagonal propagation towards the neutral axis. This can be prevented by taking suitable measures to check bearing stress in the beam. The failure of FRPSB was characterized by relatively sudden de-bonding of epoxy resin, whereas failure in the case of HRSB and CFSB was characterized by crushing of concrete in compression zone vis-a-vis wide diagonal cracks at ends of beam span.

• Despite using around 10% more steel in HRSB compared to CFSB, the later showed an extra effectiveness of 10% in strength compared to the former. This was mainly due to increased moment of inertia in CFSB due to the flanges of the cold-formed steel section (being along the depth of beam).

7. Recommendations and future scope

From the results of this experimental study, the following recommendations are made:

• From strength perspective, HRSB can be used for a strength improvement of up to 50%, CFSB, for a strength improvement of up to 65% and FRPSB for a strength improvement of up to 85%.

• From stiffness perspective, FRPSB can be used for a stiffness improvement of up to 7%, HRSB and CFSB for a stiffness improvement of up to 20%.

• From economic considerations, HRSB should be preferred over CFSB and FRPSB. From aesthetics considerations, FRPSB should be preferred over CFSB and HRSB.

This study presented the bending behaviour of strengthened RC beam specimens. Further research needs to be carried out on the shear behaviour of these specimens. Three-point loading tests are recommended for studying the shear behaviour of these beam specimens. A parametric study needs to be carried out on width, length and thickness of the strengthening material to optimize the same for improved performance. Furthermore, the effect of the connection spacing between the strengthened material and the RC beams can be studied. Also, FRP laminates of varying thicknesses can be studied to optimize its size for improved performance. The effect of using different epoxy adhesives on the performance of FRP laminates can also be studied. Lastly, there is a need to make strengthening of capacity deficit members easy, affordable and convenient. This can prevent many disasters that occur mainly due to under-performance of structurally deficit members.

References

Alam, A., Alriyami, K., Muda, Z.C. and Jumaat, M.Z. (2016), “Hybrid kenaf fibre composite plates for potential application in shear strengthening of reinforced concrete structure”, *Ind. J. Sci. Technol.*, **9**(6), 1-7. https:// 10.17485/ijst/2016/v9i6/77483.

Anbarasu, M. and Sukumar, S. (2013), “Study on the effect of ties in the intermediate length cold formed steel (CFS) columns”, *Struct. Eng. Mech.*, **46**(3), 323-335. https://doi.org/10.12989/sem.2013.46.3.323.

Anbarasu, M. and Sukumar, S., (2014), “Influence of spacers on ultimate strength of intermediate length thin walled columns”, *Steel Compos. Struct.*, **16**(4), 37-454. https://doi.org/10.12989/scs.2014.16.4.437.

ASTM (2013), *Standard Test Methods for Tension Testing of Metallic Materials*, E8/E8M-13a, American Society for Testing and Materials, West Conshohocken, U.S.A.

ASTM (2013), *Standard Test Method for Compressive Strength of Hydraulic Cement Mortars*, C109/C109M-16a, American Society for Testing and Materials, West Conshohocken, U.S.A.

Dar, M.A., Subramanian, N., Dar, A.R., Anbarasu, M. and Lim, J.B.P. (2018a), “Structural performance of cold-formed steel composite beams”, *Steel Compos. Struct.*, **27**(5), 545-554. https://doi.org/10.12989/scs.2018.27.5.545

Dar, M.A., Sahoo, D.R., Pulikkal, S. and Jain, A.K. (2018b), “Behavior of laced built-up cold-formed steel columns: Experimental investigation and numerical validation”, *Thin Wall. Struct.*, **132**(11), 398-409. https://doi.org/10.1016/j.tws.2018.09.012.

Dar, M.A., Sahoo, D.R. and Jain, A.K. (2018c), “Monotonic compression behaviour of cold-formed steel built-up laced columns”, *Proceedings of the 7th Asia Conference on Earthquake Engineering*, Bangkok, Thailand.

Dar, M.A., Sahoo, D.R. and Jain, A.K. (2018d), “Ultimate strength of cold-formed steel built-up columns: Effect of lacing slenderness”, *Proceedings of the 9th International Conference on Advances in Steel Structures*, Hong Kong, China, December.

Dar, M.A., Mir, A., Dar, A.R. and Rather, A.I. (2017a), “Strengthening of RC beams: A study”, *Proceedings of International Technology Congress*, Pune, India.

Dar, M.A., Subramanian, N., Dar, A.R. and Raju, J. (2017b), “Rehabilitation of a distressed steel roof truss-a study”, *Struct. Eng. Mech.*, **62**(5), 567-576. https://doi.org/10.12989/sem.2017.62.5.567.

Dar, M.A., Subramanian, N., Dar, A.R. and Raju, J. (2015a), “Experimental investigations on the structural behaviour of a distressed bridge”, *Struct. Eng. Mech.*, **56**(4), 695-705. https://doi.org/10.12989/sem.2015.56.4.695.

Dar, M.A., Yusuf, M., Dar, A.R. and Raju, J. (2015b).

"Experimental study on innovative cold-formed steel beams", *Steel Compos. Struct.*, **19**(6), 1599-1610. https://doi.org/10.12989/scs.2015.19.6.1599.

Dar, M.A., Subramanian, N. Dar, A.R., Anbarasu, M., Lim, J.B.P. and Mir, A. (2019a), "Behaviour of partly stiffened cold-formed steel built-up beam: Experimental investigation and numerical validation", *Adv. Struct. Eng.*, **22**(1), 172-186. https://doi.org/10.1177/1369433218782767.

Dar, M.A., Subramanian, N., Dar, A.R., Majid, M., Haseeb, M. and Tahoor, M. (2019b), "Structural efficiency of various strengthening schemes for cold-formed steel beams: Effect of global imperfections", *Steel Compos. Struct.*, **30**(4), 393-403. https://doi.org/10.12989/scs.2019.30.4.393.

Dar, M.A., Sahoo, D.R. and Jain, A.K. (2019c), "Axial resistance of cold-formed steel built-up columns with intermediate slenderness", *Proceedings of the 4th Indian Conference on Applied Mechanics*, IISc Bangalore, Karnataka, India.

Dar, M.A., Sahoo, D.R. and Jain, A.K. (2019d), "Axial compression behavior of laced CFS built-up columns with unstiffened angle sections", *J. Constr. Steel Res.*, Communicated.

Dar, M.A., Sahoo, D.R. and Jain, A.K. (2019e), "Compression capacity of short cold-formed steel built-up columns with double-lacing configuration and low sectional compactness", *Proceedings of Annual Stability Conference on Structural Stability Research Council*, St. Louis, Missouri, U.S.A., April.

Dhanoa, G.S., Singh, J. and Singh, R. (2016), "Retrofitting of reinforced concrete beam by ferrocement technique", *Ind. J. Sci. Technol.*, **9**(15), 1-9.

Indian Standard Plain and Reinforced Concrete-Code of Practice (2000), Bureau of Indian Standards, New Delhi, India.

Osman, B.H., Wu, E., Ji, B. and Abdelgader, A.M.S. (2016), "A state of the art review on reinforced concrete beams with openings retrofitted with FRP", *Int. J. Adv. Struct. Eng.*, **8**(3), 253-267. https://doi.org/10.1007/s40091-016-0128-7.

Su, R.K.L., Siu, W.H. and Smith, S.T. (2010), "Effects of bolt-plate arrangements on steel plate strengthened reinforced concrete beams", *Eng. Struct.*, **32**(6), 1769-1778. https://doi.org/10.1016/j.engstruct.2010.02.028.

Valsa Ipe, T., Sharada Bai, H., Manjulavani, K. and Iqbal, M.M.Z. (2013), "Flexural behaviour of cold-formed steel-concrete composite beams", *Steel Compos. Struct.*, **14**(2), 105-120. https:// 10.12989/scs.2013.14.2.105.

Vasudeva, R. and Kaur, M. (2016), "Retrofitting of RC beams using glass fiber reinforced polymer sheets: An experimental study", *Ind. J. Sci. Technol.*, **9**(44), 1-11.

Vinay, N., Harish, M.L. and Prabhakara, R. (2015), "Experimental investigation on the flexural behavior of the steel-concrete composite beams", *Int. Res. J. Eng. Technol.*, **2**(7), 1293-1301.

Wehbe, N., Wehbe, A., Dayton, L. and Sigl, A. (2011), "Development of concrete/cold-formed steel composite flexural members", *Proceedings of the Structures Congress*, Las Vegas, Nevada, U.S.A., April.

Obaidat, Y.T., Heyden, S., Dahlbloma, O., Abu-Farsakhb, G. and Abdel-Jawad, J. (2011), "Retrofitting of reinforced concrete beams using composite laminates", *Constr. Build. Mater.*, **25**(2), 591-597. https://doi.org/10.1016/j.conbuildmat.2010.06.082.

CC

Nomenclature

f_y	Yield strength
f_u	Ultimate strength
E	Modulus of Elasticity
A	Area of cross section
f_{ck}	Compressive strength of concrete
Z	Section modulus
P_{Des}	Design strength predicted by IS 456
P_{Exp}	Test strength
P_{BM}	Strength of benchmark beam
k_{Exp}	Stiffness of the beams
k_{BM}	Stiffness of benchmark beam
M	Moment in the central mid-third portion
δ	Deflection at the mid-span
η	Strength effectiveness ratio
η	Stiffness effectiveness ratio
CFS	Cold-formed steel
BMB	Benchmark beam
HRSB	Hot rolled steel strengthened beam
CFSB	Cold-formed steel strengthened beam
FRPSB	Carbon fibre reinforced polymer laminate strengthened beam