

FLEXURAL BEHAVIOUR OF SFRC IN-FILLED LIGHT GAUGE STEEL RECTANGULAR BOX SECTIONS

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ABSTRACT: This paper presents an experimental study on the flexural behaviour of plain cement concrete (PCC) and steel fiber reinforced concrete (SFRC) in-filled light gauge steel rectangular box sections under pure bending. Concrete in-fill provides internal support to the section and delays local buckling. It also contributes to the inertia of the section which increases the flexural strength and stiffness of the member. A reference hollow beam, plain concrete in-filled and SFRC in-filled (with three different volume fraction of fibers) beams are tested up to failure in pure bending with respect to their principal axes. Extensive strain and deformation measurements are taken at compression and tension zones. It is found that the SFRC in-filled sections take more load than hollow and plain concrete in-filled beams when it is subjected to loads about their major and minor axes. In this experimental study, it is observed that SFRC in-filled beams with 1% volume fraction of fibers have enhanced structural properties much more than other types of beams. The moment - strain plots show, a good ductility performance by the hollow and SFRC in-filled beams compared to the plain concrete in-filled beams. A comparison of the observed moment capacities and flexural stiffnesses is made, with the values calculated using the expressions recommended by the various codes of practice such as AIJ, BS 5400, Eurocode 4 & LRFD. It was found that the values predicted by the AIJ code agree reasonably well with the experimental results. The codes developed for ordinary cement concrete in-filled beams require modifications when they are applied to high performance concretes such as SFRC in-filled composite beams.

Keywords: Hollow; PCC and SFRC in-filled beams; pure bending; flexural stiffness; flexural strength; ductility

1. INTRODUCTION

Hybrid structures such as concrete-filled or Steel Fiber Reinforced Concrete (SFRC) in-filled steel hollow sections are very effective in special types of applications such as piles, poles, highway overhead sign structures, bridge components, etc. Concrete-filled steel hollow section (CFSHS) columns are widely used in the construction industry for the past few years owing to the advantages of combining two materials. The steel hollow sections in-filled with concrete have higher strength and greater stiffness than the conventional structural steel sections and reinforced concrete sections. Concrete being a brittle material, has low tensile strength and low ductility. Steel fiber reinforced concrete, a two phase composite material, having randomly distributed steel fibers, has higher resistance against cracking, improved strength in shear, tension, compression and flexure with better toughness and ductility as compared to plain concrete. The addition of randomly distributed fibers improves many properties of concrete such as fracture strength, toughness, impact resistance, flexural strength and fatigue resistance. Both ductility and structural performance factors depend on the structural form selected and the materials used. One of the suggestions often made to improve the failure strain of concrete is by effectively confining concrete and inclusion of fibers in concrete matrix, which improves the ductility of concrete elements. Hence, SFRC in-filled steel sections provide excellent seismic resistance in two orthogonal directions and higher load carrying capacities. Furthermore, the Hyogoken-Nanbu Great Earthquake in Japan, 1995 [1], showed the merits of CFSHS beams which had demonstrated better ductility performance and larger energy absorption capacity than the conventional reinforced concrete and steel beams. Therefore, CFSHS columns and beams are considered to be more favourable structural component for use in earthquake sensitive regions. The main advantage of CFSHS columns and beams are attributed to

the confining effect of core concrete by the steel hollow sections. The volume increase of the core concrete due to the development of cracks is constrained by the steel hollow section. As a result, the strength and the ductility performance of the core concrete are enhanced. It is also noted that more advantages can be achieved for CFSHS columns and beams with high strength in-filled material. High strength in-fill like SFRC provides better strength, greater confinement effect and ductility performance than ordinary in-fill. In spite of these advantages, the use of SFRC in-fill columns and beams in the construction industry is till scarce owing to the lack of understanding on their structural behaviour. Design codes such as AIJ [2], BS 5400 [3], Eurocode 4 [4] & LFRD [5] are applicable only to the design of normal strength CFSHS beams. Hence, there is a need for extensive investigations of the behaviour of SFRC in-filled beams under various types of loading so that guidelines and recommendation can be developed for design.

2. PREVIOUS RESEARCH

Composite members consisting of steel hollow sections filled with concrete are extensively used in structures, which are subjected to large applied moments, particularly in zones of high seismicity. Composite square or rectangular concrete-filled tubes have been used increasingly as columns and beam columns in braced and un-braced framed structures [6]. Their use has ranged from low rise compression members, open floor plan construction using cold formed steel circular or rectangular tubes filled with concrete to large diameter cast-in-situ members used as the primary lateral resistant columns in multi storied buildings. In addition, concrete fillings are often used in retrofitting of damaged steel bridge piers after the 1994 Northridge earth quake in USA [7].

In spite of the bulk literature written over the last four decades on the technique of concrete filling of steel hollow sections, only a little of it was devoted to the study of flexural behaviour of these members. A set of pure bending tests on twelve circular concrete filled steel hollow sections (CFSHS) with diameters vary from 34 to 110 mm and wall thickness between 1mm and 3 mm were conducted by Elchalakani et al. [8]. The test results revealed that filling of the voids in steel tube enhances the strength, ductility and energy absorption capacities of the thinner sections than the thicker sections. A simplified formula to determine the ultimate bending capacity of circular concrete filled tubes was also developed in this paper. Twelve rectangular CFSHS beams were tested under pure bending mode by Gho and Liu [9]. The test results show that all the specimens demonstrated favourable post-yield behaviour with good ductility performance. The test results were also compared with the values obtained using the formulae given in various codes. These codes considerably underestimate the moment capacities of the beams. The flexural behaviour of concrete –filled fiber-reinforced polymer circular tubes was studied by Fam and Rizkalla [10]. 20 beams were tested to study the flexural behaviour of concrete-filled glass fiber reinforced polymer tubes (GFRP). The concrete filled pultruded GFRP tubes show higher stiffness than concrete-filled filament-wound tubes of the same thickness. The load deflection behaviour of concrete-filled GFRP was almost linear. The cracking load was significantly less compared to the ultimate load. The stiffness after cracking was mainly governed by the laminate structure of the tube and the diameter-to-thickness ratio. Han [11] conducted a series of tests on the flexural behaviour of CFSHS beams. The parameters varied in these tests were depth to width ratio from 1 to 2 and tube depth to wall thickness ratio from 20 to 50. Due to the presence of the in-fill, the beams behaved in a ductile manner. The predicted load versus lateral deflection curves for the composite beams by AIJ [2], BS5400 [3], Eurocode 4 [4] and LFRD [5] codes have been found in good agreement with experimental values. The flexural behaviour of self-consolidating concrete in-filled steel tubes was studied by Han et al [12]. A total of 36 composite beams filled with self-consolidating concrete were tested to study their flexural behaviour. The test parameters were sectional types, steel yield strength, the ratio of tube diameter to wall thickness and the ratio of shear span to depth. The test

results were compared with the values predicted by the existing codes AIJ [2], BS5400 [3], Eurocode 4 [4] and LRFD [5]. The results revealed that the beams filled with self-consolidating concrete are very similar to those of the composite beams filled with normal concrete. It was also found that the shear span to depth ratio has a very little influence on the behaviour of the concrete in-filled steel tubular beams with circular and square sections. The Eurocode 4 [4] recommendations are found to be closest to the experimental values. Helena and Knight [13] studied the flexural behaviour of normal and high strength concrete in-filled light gauge steel beams. The test results showed that the non-linearity in strain started at 75% and 95% of the ultimate load in compressive and tensile strains of the beams respectively.

The behaviour of SFRC in-filled light gauge steel columns were experimentally studied by Senthil et al [14]. In this paper a total of 18 slender columns were tested on three different cross sectional shapes under axial and eccentric loads with 1% volume fraction of steel fibers on concrete. It was concluded in this paper that SFRC in-filled columns have better load carrying capacity and ductility than the plain concrete in-filled columns and the plain concrete in-filled columns are found to have higher stiffness than the SFRC in-filled columns. Mursi and Uy [15] presented a comprehensive experimental study of thin walled steel sections utilizing high strength steel of a thin walled nature and filled with normal strength concrete. A numerical model was also developed to study the behaviour of slender concrete filled high strength steel columns incorporating material and geometric non-linearities. The behaviour of concrete filled steel slender columns affected by elastic or inelastic local buckling was also investigated. Liu [16] experimentally investigated the strength of 12 high strength rectangular concrete-filled steel hollow columns subjected to eccentric loading. The primary test parameters were the cross sectional aspect ratio, slenderness ratio and load eccentricity. The experimental ultimate capacities of the specimens were compared with the design strengths predicted by the codes. In the comparison, Eurocode 4 [4] over estimates the results by 3% whereas the LRFD [5] conservatively predicted the failure loads. The ultimate capacity of high strength rectangular concrete filled steel hollow section stub columns were experimentally investigated by Liu, Gho and Yuan [17]. 22 rectangular concrete filled steel hollow sections specimen with cross sectionals aspect ratio of 1, 1.50 and 2 were tested to failure under axial concentrated loading. In this paper, the ultimate loads obtained from the experiment were compared with the values calculated from the design codes, Eurocode 4 [4], LRFD [5] & ACI [18]. The comparison of results showed that Eurocode 4 [4] closely predicts with a difference of 6% while LRFD [5] and ACI [18] underestimated the critical loads by 16% and 14%, respectively. It was also noted that the strength of specimens decreases with the increase of cross-sectional aspect ratios. The behaviour of circular concrete filled steel tubes with various concrete strength under axial load was presented by Giakoumelis and Lam [19]. In this paper the effect of steel tube thickness, the bond strength between concrete and steel tube and the confinement of concrete were examined. It was concluded that the effect due to concrete shrinkage is critical for high strength and negligible for normal strength concrete.

From the above review, it is clear that there were only a few significant research works have been reported on the flexural behaviour of concrete in-filled light gauge steel hollow beams and there exist the following areas that require further investigations: The flexural behaviour of SFRC in-filled rectangular steel hollow section beams with respect to its major and minor axes, the differences of the predicted moment carrying capacity and flexural stiffness using the existing codes such as AIJ [2], BS 5400 [3], Eurocode 4 [4] & LRFD [5] and the bond – slip behaviour of SFRC in-filled rectangular steel hollow sections.

Hence the objectives of this paper are

1. To study the bond slip behaviour of plain cement concrete and SFRC in-filled light gauge steel hollow sections,
2. To study the flexural behaviour of hollow, plain cement concrete and SFRC in-filled beams
3. To illustrate the difference between the predicted beam capacities and flexural stiffness with the existing codes such as AIJ [2], BS 5400 [3], Eurocode 4 [4] & LRFD [5].

3. EXPERIMENTAL PROGRAMME

3.1 Test Specimens

A total of 10 specimens are prepared to study the flexural behaviour of light gauge steel hollow rectangular sections with and without in-fill. The details of experiments conducted are presented in Figure 1. The dimension of the steel hollow sections are 100x50x2mm. All the specimens are 1474mm long and it was tested to the failure under pure bending about its major and minor axes.

3.2 Material Properties

3.2.1 Rectangular steel hollow sections

The hollow sections are made from light gauge steel sheets, continuously welded at the middle of the 100 mm face along its length. In order to determine the actual material properties, four steel coupons are cut from the four faces of these sections and tested to failure under tension according to the requirement of American Society for Testing and Materials Standard (ASTM) [20]. The following average values are obtained Yield stress $f_y = 270 \text{ N/mm}^2$; Ultimate stress $f_u = 415 \text{ N/mm}^2$; Percentage of elongation = 13%; and the Modulus of Elasticity = $2.05 \times 10^5 \text{ N/mm}^2$. To prevent the local buckling failure of the specimens, the allowable B/t ratio of the steel hollow sections is given by $B/t \leq 52 (235/f_y)^{1/2}$ prescribed by Eurocode 4 [4] has been used. The chosen specimen satisfies the above requirements and therefore all the specimens could be classified as compact sections. The B/t limits specified in Eurocode 4 [4] is conservative as it is derived without taking in to account the stability enhancement of the steel hollow section due to the in-fills.

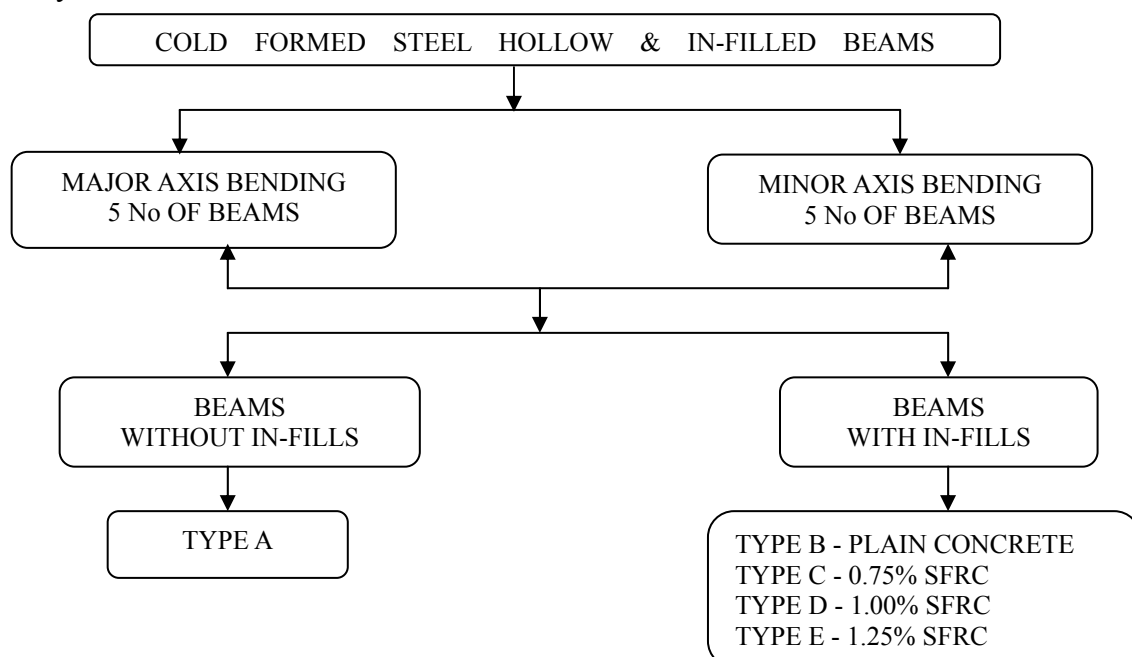


Figure 1. Details of Experiments Conducted

3.2.2 Plain Cement Concrete (PCC) and Steel Fiber Reinforced Concrete (SFRC)

The concrete mix is designed for a cube compressive strength (f_{ck}) of 20 MPa at 28 days. The designed mix proportion is 1:2.09:2.25 with 10mm size (max) coarse aggregate and 2.36mm (max) size fine aggregate with a water-cement ratio of 0.4875 based on ACI committee 211.1-1991 recommendations. Crimped steel fibers having an aspect ratio of 70 ($l_f=30.80\text{mm}$ and $d_f=0.44\text{mm}$) are used to prepare SFRC. The concrete for the eight rectangular SHS beams is mixed in four batches. The remaining two SHS beams are tested as hollow beams. Two SHS beams are filled with plain cement concrete prepared as per the above mix proportion. The remaining six beams are in-filled with SFRC having three different volume fractions of steel fibers viz. 0.75%, 1.00% and 1.25%. 8mm thick flat plates are welded on to the base of the steel hollow sections to support the wet concrete. The concrete is poured vertically in to the steel hollow section in four layers and each layer is compacted well by raising the specimen about 30cm and suddenly placing it on a rubber sheet. The specimens are cured for 28 days. Subsequently 8mm thick flat plates are welded at the top of the sections to complete the specimen. Table 1 shows the material properties of plain cement concrete (PCC) and steel fiber reinforced concrete (SFRC).

Table 1. Material Properties Concrete

Specimen label	Type of in-fill	Cube compressive strength (f_{ck}) (N/mm ²)	flexural strength (N/mm ²)		Young's Modulus (E_c) (N/mm ²)	
			test	IS:456 [21]	test	IS: 456 [21]
Type A	Hollow (without in-fill)	---	---	---	---	---
Type B	P.C.C	32.44	4.24	3.99	2.968×10^4	2.848×10^4
Type C	0.75% SFRC	41.78	4.94	---	3.230×10^4	---
Type D	1.00% SFRC	57.78	5.86	---	3.800×10^4	---
Type E	1.25% SFRC	38.60	4.40	---	3.109×10^4	---

3.2.3 Push out test procedure

Push out tests are carried out to study the bond strength between the steel sections and the in-fill. The test specimen for these test is 550mm in length and the concrete is poured to 500mm length leaving a 50mm gap in which a groove of 7mm in the form of an equilateral triangle is made at the end for relieving the air during testing [13]. The test set-up is shown in Figure 2. The load is applied through a steel plate, which rests directly on concrete in small increments and the slip is observed using a deflectometer as shown in the test set-up. The slip is very minimal indicating high bond - strength up to the ultimate load. Beyond the ultimate load, there is a rapid increase in slip, indicating the sudden decrease of bond. Due to increased characteristic compressive strength, the bond strength is high for SFRC in-filled specimens when compared to PCC in-filled specimens. Tested specimens are shown in the Figure 3 and the load - slip characteristic of the in-filled specimens are shown in Figure 4. The bond strength is found to be 0.282 N/mm², 0.389 N/mm², 0.461 N/mm² and 0.318 N/mm² respectively for PCC, 0.75% SFRC in-fill, 1% SFRC in-fill and 1.25% SFRC in-fill. When compared to PCC in-filled specimens, SFRC in-filled specimens takes about 63% more load before slip.

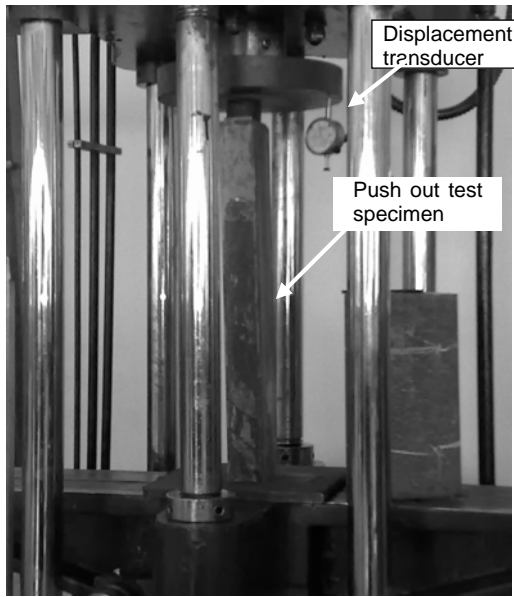


Figure 2. Test Set Up for Push Out Test



Figure 3. Tested Specimens of Push Out Test

3.2.4 Flexural stiffness (K_e)

The experimental flexural stiffness (K_{ee}) determined for the 10 tested beams are compared with the theoretical flexural stiffnesses (K_e) calculated from the expressions given in the codes and listed in Table 2. The expressions for the flexural stiffness by the different codes are given in the Eqn. 1 to Eqn. 4.

1. AIJ [2] Flexural Stiffness $K_e = E_s \cdot I_s + 0.20 E_c \cdot I_c$ (1)
where $E_s = 205,800$ MPa; $E_c = 21,000 (f_c'/19.60)^{1/2}$ MPa
2. BS 5400 [3] Flexural Stiffness $K_e = E_s \cdot I_s + E_c \cdot I_c$ (2)
where $E_s = 206,000$ MPa; $E_c = 450 \cdot f_{cu}$ MPa
3. Eurocode 4 [4] Flexural Stiffness $K_e = E_s \cdot I_s + 0.60 E_c \cdot I_c$ (3)
where $E_s = 206,000$ MPa; $E_c = 9500 (f_{ck} + 8)^{1/3}$ MPa
4. LRFD [5] Flexural Stiffness $K_e = E_s \cdot I_s + 0.80 E_c \cdot I_c$ (4)
where $E_s = 199,000$ MPa; $E_c = 4733 (f_c')^{1/2}$ MPa

Table 2 also shows the mean and standard deviation of the ratio (K_e / K_{ee}) obtained from different design codes. The results clearly show that the codes over estimate the flexural stiffness of the beams in general. The flexural stiffness predicted by AIJ [2] is around 6% higher than the experimental value about the major axis loading and 0.80% lower than the experimental value about the minor axis loading for in-filled beams. Eurocode 4 [4], BS 5400 [3] and LRFD [5] gave a flexural stiffness about 45% higher than the experimental flexural stiffness values about the major axis loading and 15% higher than the experimental flexural stiffness about the minor axis loading for in-filled beams. The AIJ [2] method gives a mean value of 1.056 and standard deviation of 0.022 and predicts a slightly higher stiffness than the test results and is found to be the best predictor. This clearly shows that the flexural stiffness for the in-filled beams are less than the theoretical prediction. Due to this less flexural stiffness, the specimens show high ductile behaviour. Comparing the flexural stiffness of all the in-filled beams, 1% SFRC in-filled beams shows less flexural stiffness about the major and the minor axes.

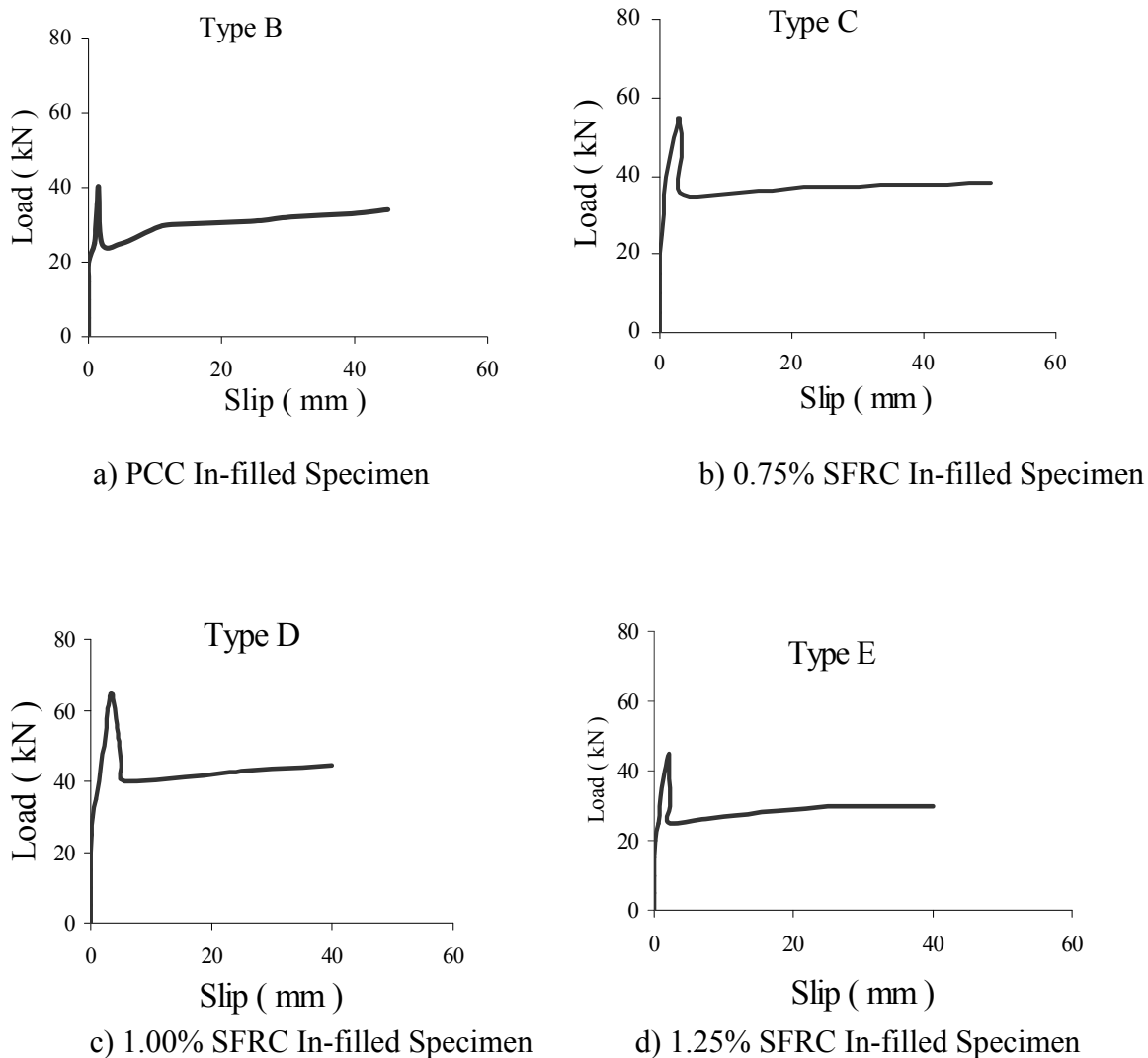


Figure 4. Load - Slip Characteristics of In-filled Specimens

3.3 Test Procedure

Rectangular hollow and SFRC in-filled beams are tested in the beam loading frame of 1000 kN capacity in the Advanced Structural Engineering Laboratory of B.S.A. Crescent Engineering College, Chennai, India. The beams are tested up to failure, five with longer side vertical (about major axis) and five with shorter side vertical (about minor axis). The objective is to find the ultimate moment of the beam along the stronger and weaker axes. Loading arrangements are made such that two point loads are applied at one-third spans. Above this loading arrangement a 1000 kN proving ring is placed and the load from the jack is applied through the proving ring. Strain gauges are fixed at six different test locations, three at top fiber of the beam and three at bottom fiber of the beam. Deflectometers are placed at the bottom at three different points to measure the vertical deflections, two deflectometer at $1/3^{\text{rd}}$ span points and one at mid span. Two more deflectometers are also placed at the two ends on sides to check whether the load is applied vertically without any lateral translation of the beam. The loading pattern and positions of the strain gauges and deflectometers are shown in Figure 5. The load is applied at equal intervals of 0.50 kN in the elastic

range and at shorter intervals in the inelastic range. Deflectometer readings and strain gauge readings are taken for every increment of load. Figure 6 shows the failure modes of the tested beams about major and minor axes.

Table 2. Comparison between Predicted Initial Section Flexural Stiffness and Test Results

Sl. No.	Specimen Type	Experiment (K _{ee})		AIJ [2]				BS 5400 [3]				Eurocode 4 [4]				LRFD [5]			
		kN m ²		Theoretical (K _e)		Theoretical / Experimental (K _e / K _{ee})		Theoretical (K _e)		Theoretical / Experimental (K _e / K _{ee})		Theoretical (K _e)		Theoretical / Experimental (K _e / K _{ee})		Theoretical (K _e)		Theoretical / Experimental (K _e / K _{ee})	
		Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis
1	A	139.53	46.05	150.06	51.65	1.075	1.122	150.06	51.65	1.075	1.122	150.06	51.65	1.075	1.122	150.06	51.65	1.075	1.122
2	B	149.25	56.82	152.50	51.93	1.022	0.914	239.36	61.68	1.604	1.086	229.09	66.44	1.535	1.169	248.91	62.75	1.668	1.104
3	C	144.23	54.04	152.08	51.88	1.054	0.960	209.72	58.35	1.454	1.080	220.51	64.83	1.529	1.200	230.86	60.72	1.601	1.124
4	D	141.11	51.37	151.91	51.86	1.077	1.010	200.20	57.28	1.419	1.115	217.26	64.22	1.540	1.250	224.13	59.97	1.588	1.167
5	E	144.93	54.35	152.17	51.88	1.050	0.955	214.63	58.90	1.481	1.084	222.08	65.13	1.532	1.198	234.12	61.09	1.615	1.124
Mean						1.056	0.992			1.407	1.097			1.442	1.188			1.510	1.128
Standard Deviation						0.022	0.080			0.198	0.020			0.206	0.047			0.245	0.023

Table 3. Comparison between Predicted Beam Strength and Test Results

Sl. No.	Specimen Type	Experiment (M _{ee})		AIJ [2]				BS 5400 [3]				Eurocode 4 [4]				LRFD [5]			
		kN – m		Theoretical (M _e)		Theoretical / Experimental (M _e / M _{ee})		Theoretical (M _e)		Theoretical / Experimental (M _e / M _{ee})		Theoretical (M _e)		Theoretical / Experimental (M _e / M _{ee})		Theoretical (M _e)		Theoretical / Experimental (M _e / M _{ee})	
		Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis	Major Axis	Minor Axis
1	A	2.457	1.474	3.953	2.721	1.609	1.846	4.946	3.146	2.013	2.134	3.953	2.721	1.609	1.846	3.953	2.721	1.609	1.846
2	B	6.550	3.439	5.713	2.880	0.872	0.837	5.427	3.318	0.829	0.965	5.266	2.767	0.804	0.805	5.123	3.542	0.782	1.030
3	C	7.370	4.422	6.460	2.989	0.877	0.676	5.527	3.345	0.750	0.756	5.594	3.268	0.759	0.739	6.068	3.660	0.823	0.828
4	D	7.861	5.405	7.694	3.173	0.979	0.587	5.671	3.379	0.721	0.625	6.150	3.540	0.782	0.655	6.978	3.774	0.888	0.698
5	E	6.592	3.931	6.054	2.941	0.918	0.748	5.009	3.336	0.760	0.849	4.942	3.216	0.750	0.818	5.798	3.627	0.880	0.923
Mean						1.051	0.939			1.015	1.066			0.941	0.973			0.996	1.065
Standard Deviation						0.315	0.515			0.560	0.610			0.374	0.492			1.129	0.424

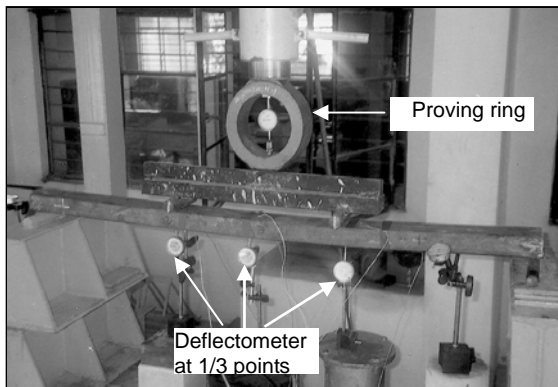


Figure 5. Test Set Up

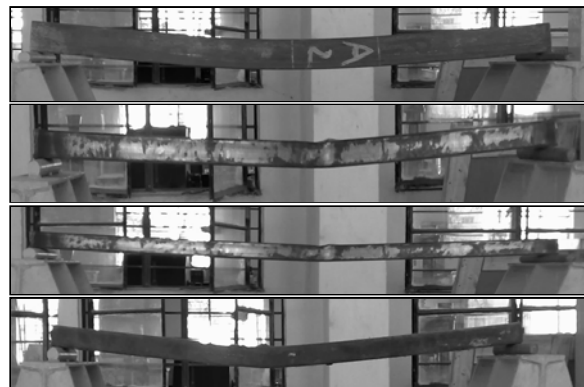


Figure 6. Failure Modes of the Tested Beam Specimen

4. RESULTS AND DISCUSSION

4.1 Ultimate Load

All the specimen are tested up to failure under pure bending about their major and minor axes. For each load interval, the loading is paused for a minute to allow for the plastic deformation of steel and for the concrete to crack. The cracking sound of concrete are heard at loads around 2 kN and 1.50kN for hollow beams and around 10 kN and 6 kN for beams with in-fills for loads applied about their major and minor axes respectively. All the in-filled beams failed only due to overall bending. After attaining the ultimate load levels the beams deflected more without any significant increase in the load. This shows the crushing of concrete at the point of maximum bending tensile stress and rapid yielding of outer steel section. The SFRC in-filled beams carried around 3 times more load than the hollow beams about their major axis loading and 3.66 times more load than the hollow beams about their minor axis loading. When compared with PCC in-filled beams the SFRC in-filled beams carried 1.20 times more load about their major axis and 1.55 times more load about their minor axis.

4.2 Provisions in Different Codes for the Determination of Moment Capacities

The expressions for determining the moment capacity of in-filled beams by the various codes are illustrated through Eqn. 5 to Eqn. 16. These equations are theoretically analysed for the present study of SFRC in-filled beams. The notations used in this paper for the expressions are the same as that used in the original code.

4.2.1 AIJ [2]

This AIJ [2] code predicts the theoretical moment based on the combined effect of steel and concrete taking into account of the plastic section modulus and angular location of the neutral axis. The property of the steel is fully utilized by taking into account the plastic section modulus of the steel tube. The ultimate confined concrete strength is considered here which will give the true value of the in-filled beams.

$$\text{Ultimate moment of CFT cross section } M_u = {}_sM_u + {}_cM_u \quad (5)$$

$$\text{Ultimate moment due to steel tube } ({}_sM_u) = {}_sZ_p \sigma_y \quad (6)$$

$$\text{Ultimate moment due to concrete } ({}_cM_u) = \frac{1}{12}({}_c r_o f_c) d^3 \sin^3 \theta \quad (7)$$

4.2.2 BS 5400 [3]

This code predicts the flexural behaviour by taking the average compression stress in the concrete at failure to the design yield strength of steel considering the composite action between steel and concrete. It also gives more importance to steel section rather than concrete core. So the theoretical moment calculated for SFRC in-filled beams will not exhibit the true moment.

$$\text{Ultimate moment} = 0.91f_y \left[A_s \frac{h - d_c}{2} + b_f t_f (t_f + d_c) \right] \quad (8)$$

4.2.3 Eurocode 4 [4]

The moment resistance of the CFT beams are calculated based on the plastic stress distribution and full strain compatibility of the cross section for the both steel and concrete. Here the strength of concrete in tension was omitted in the calculation and this code was applicable to the design of CFT

beams with concrete cylinder strength and steel yield stress of not higher than 50 and 355 MPa respectively.

$$\text{Ultimate flexural capacity of CFT section} \quad M_{pl,Rd} = M_{max} R_d - M_n R_d \quad (9)$$

$$\text{The maximum resistance about major axis } M_{max} R_d = w_{pa} \cdot f_{yd} + w_{ps} \cdot f_{sd} + w_{pc} \cdot F_{cd}/2 \quad (10)$$

$$\text{Combined plastic section modulus of structural steel reinforcement and concrete parts} \\ M_n R_d = w_{pan} \cdot f_{yd} + w_{psn} \cdot f_{sd} + w_{pcn} \cdot F_{cd}/2 \quad (11)$$

$$\text{Plastic section modulus of the concrete core} \quad w_{pcn} = (d-2t) h_n^2 \quad (12)$$

$$\text{Plastic section modulus of the steel tube} \quad w_{pan} = b h_n^2 - w_{pcn} - w_{psn} \quad (13)$$

$$\text{Neutral axis depth} \quad h_n = \frac{N_{pm,Rd} - A_{sn} (2f_{sd} - f_{cd})}{2b f_{cd} + 4t (2f_{yd} - f_{cd})} \quad (14)$$

Plastic section modulus of the steel reinforcement $w_{psn} = 0$ (for CFT beams)

$$\text{The compression resistance of the whole and concrete } N_{pm,Rd} = A_c f_{cd} \quad (15)$$

4.2.4 LRFD [5]

This code predicts the strength of CFT beams by the plastic stress distribution in the cross section for both steel and concrete as like Eurocode 4 [4]. However the flexural strength of the CFT columns were determined based on the steel hollow section. The AISC – LRFD method was limited to CFT beams with concrete cylinder strength and steel strength less than 55 and 380 MPa respectively. Ultimate moment of composite cross section is given by,

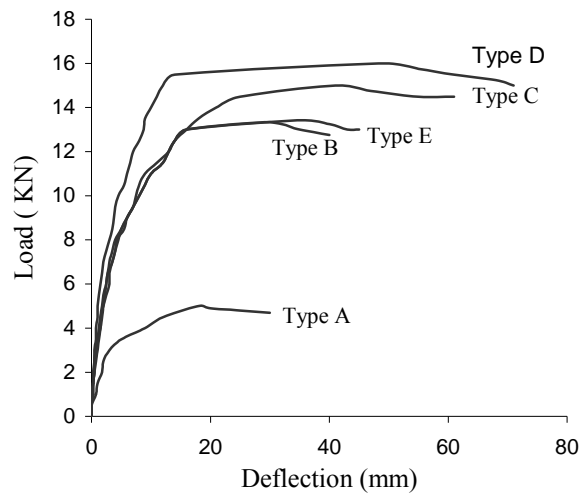
$$M_n = Z \cdot f_y + \frac{1}{3} (h_2 - 2C_r) A_r f_{yr} + \left(\frac{h_2}{2} - \frac{A_w \cdot f_y}{1.7 f_c h_1} \right) A_w f_y \quad (16)$$

The moment capacities (M_e) using the above expressions are computed for all the 10 specimens and compared with the experimental results (M_{ee}) in Table 3. It also shows the mean and the standard deviation of the ratio (M_e / M_{ee}) for different design codes. These results clearly indicate that all the codes are conservative. The moment capacities predicted by AIJ [2] and Eurocode 4 [4] are around 11 % lower than the experimental values, with respect to their major axis and around 30 % lower than the experimental values with respect to their minor axis for SFRC in-filled beams. Similarly the moment capacities predicted by BS 5400 [3] and LRFD [5] are around 16% lower than the experimental moment values about their major axis and around 19% lower than the experimental moment values about their minor axis for SFRC in-filled beams. AIJ [2] method gives a mean value of 1.051 and a COV of 0.315 and predicts 11% lower capacity than the corresponding test results and is the best predictor and thus are acceptable for the calculation of moment capacities of SFRC in-filled steel beams.

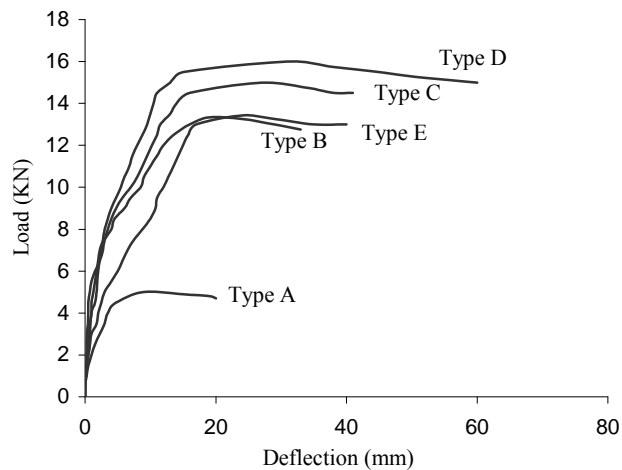
4.3 Load – Deflection

The load - deflection characteristics for all the beams tested are shown in Figure 7. The in-filled beams in general exhibit significant yield plateau with good ductility performance in the post yield region. In the initial stages of loading, all the beams exhibit almost the same load -deflection

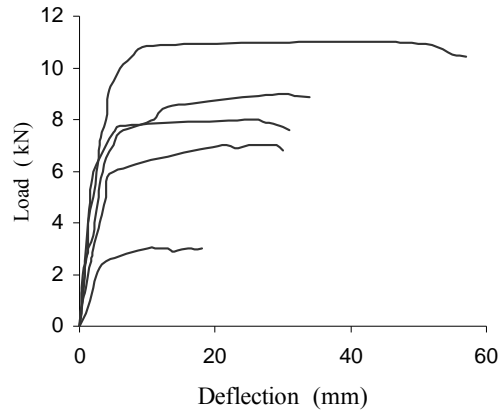
behaviour. Rapid deformations started first for type A beams followed by type B beams and type E beams. The low characteristic strength of concrete in type E beams has an impact on the load deflection plots also. In general the type D beams show better ductility performance compared with other specimens. The mid span deflection became larger for type D beam when the load crossed 16 kN about major axis and 11 kN about minor axis. The 1/3rd span deflection became rapid for type D beam when the load crossed 15 kN about major axis and 10.50 kN about minor axis. In the PCC in-filled beams, rapid deflection point started at lower load levels, which clearly show the brittle behaviour of concrete. In the SFRC in-filled beams, the rapid deflection started at higher load levels which show less stiffness compared to the PCC in-filled beams. Due to the in-fill, the lateral deflections have reduced and the moment capacities have increased. Thus longer span concrete in-filled specimens will show less deflections and greater safety against permissible lateral deflections. Beams with plain concrete in-fill are stiffer than other types of beams.



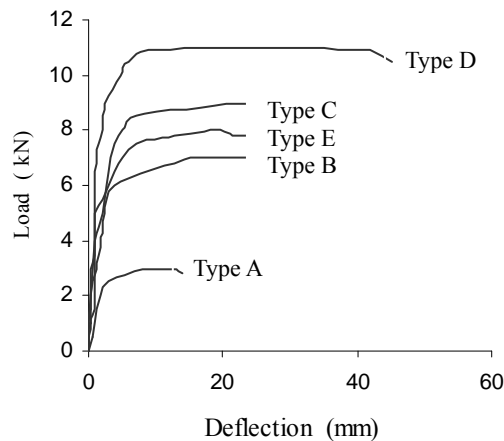
i) Load versus mid span deflection (major axis)



ii) Load versus 1/3 span deflection (major axis)



iii) Load versus mid span deflection (minor axis)



iv) Load versus 1/3 span deflection (minor axis)

Figure 7. Load - Deflection Plots of Hollow and In-filled Beams

4.4 Moment - Strain

Figure 8 & Figure 9 show the Moment – Micro strain behaviour of hollow and concrete in-filled beams under pure bending about major and minor axes respectively. Compressive strains are recorded from the top strain gauges and tensile strains are recorded from the bottom strain gauges. The Moment – Micro strain plots show an early elastic response and in-elastic behaviour with gradually decreasing stiffness thereafter, until the ultimate moments are reached asymptotically. For all practical considerations the moment corresponding to the maximum fiber strain of 0.20% is taken as the moment capacity of the composite beam as it is found that when the maximum fiber strain reach 0.20%, the moments tend to stabilize. The moment capacities of the test specimen are listed in Table 3. As illustrated in the Figures 8 and 9, both the compressive and tensile strains of the specimens are higher than 0.20% at the yield plateau, which indicates that both flanges of the specimen have yielded prior to their moment capacities. The non-linearity in strains starts at 85% and 95 % of the ultimate load for compressive and tensile strains respectively. The failure of the beams is identified by the overall bending of the beam followed by the localized buckling under the concentrated load in the case of hollow beam and by overall bending in the case of in-filled beams.

After comparing the moment strain plots of all the beams, the type D beams (1% SFRC in-filled) show a significant yield plateau beyond the yield point indicating good ductility performance than other type of beams about the major and minor axes.

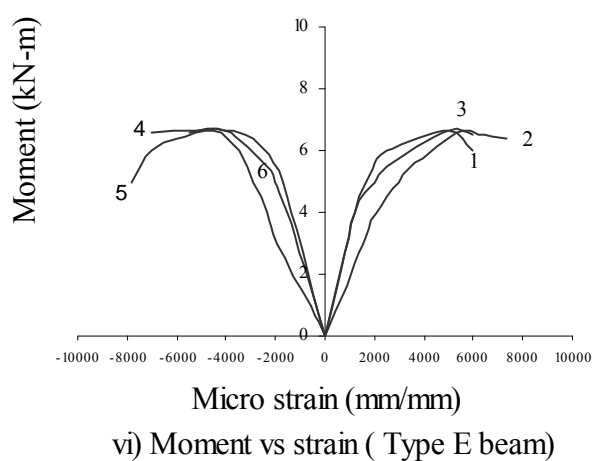
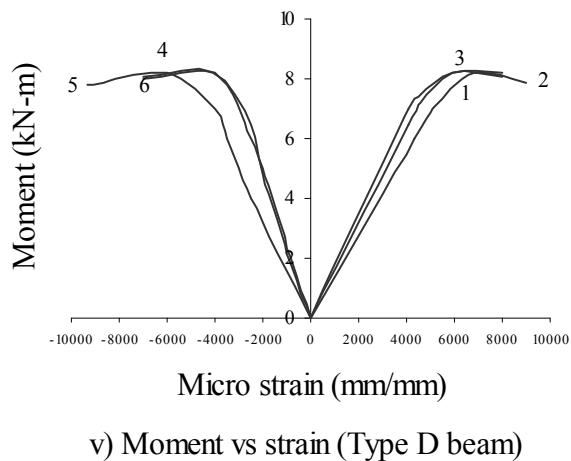
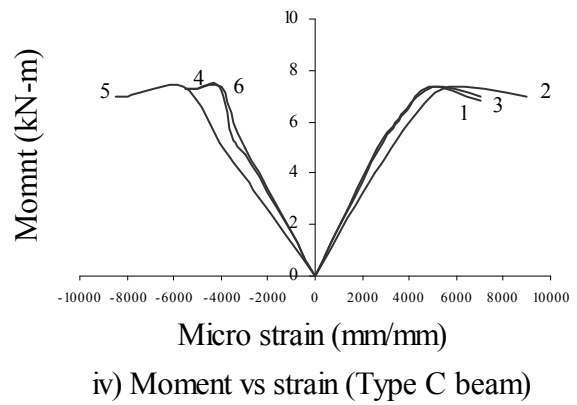
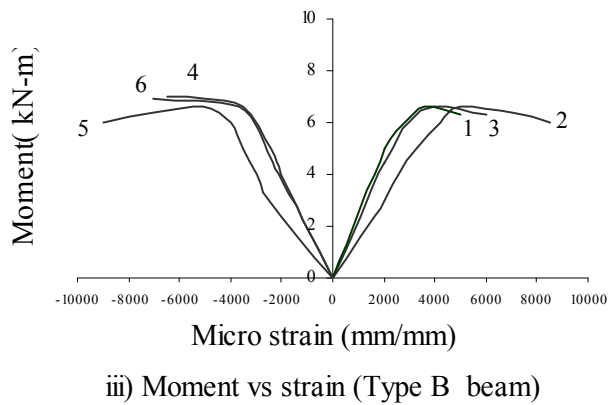
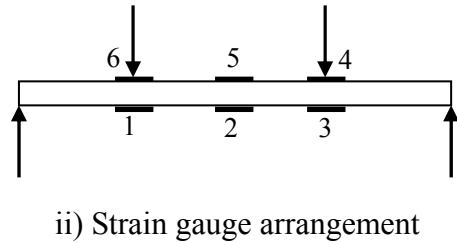
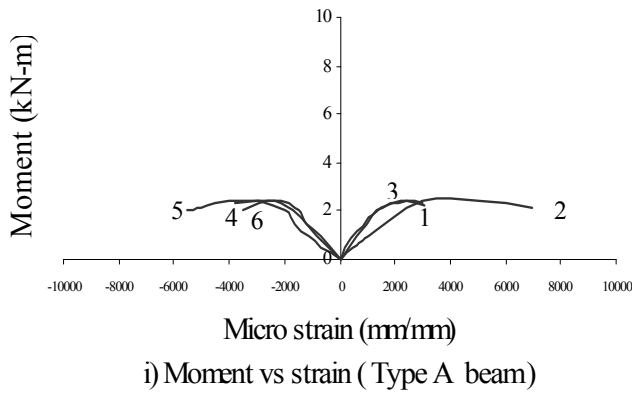
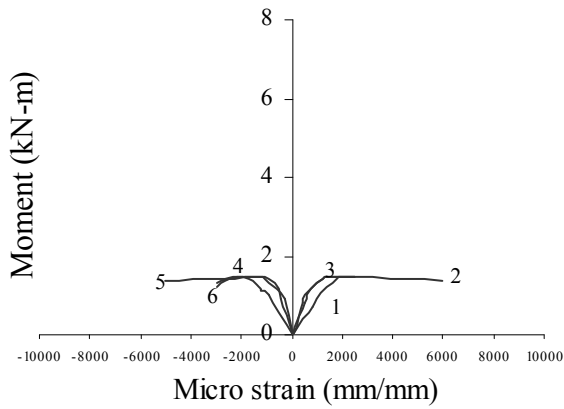
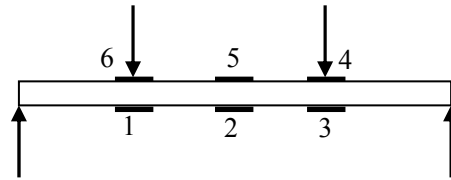


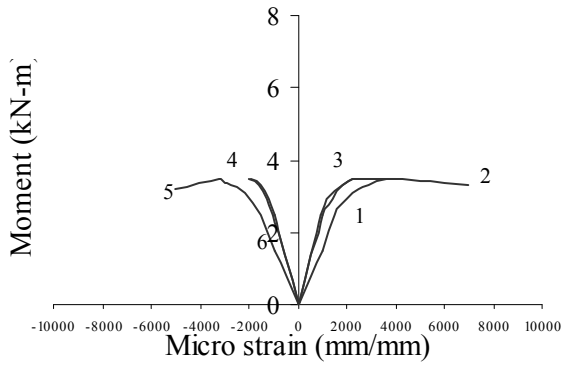
Figure 8. Moment vs Micro Strain Plots (About Major Axis)



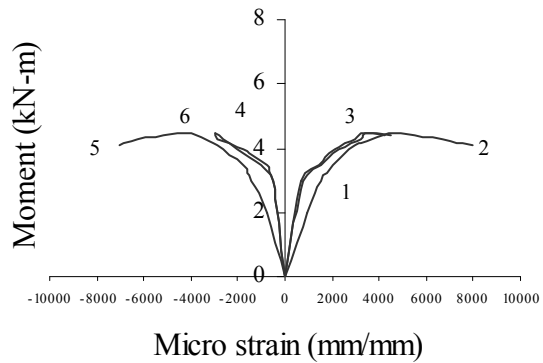
i) Moment vs strain (Type A beam)



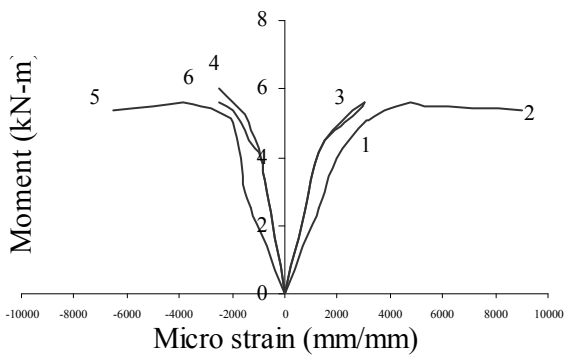
ii) Strain gauge arrangement



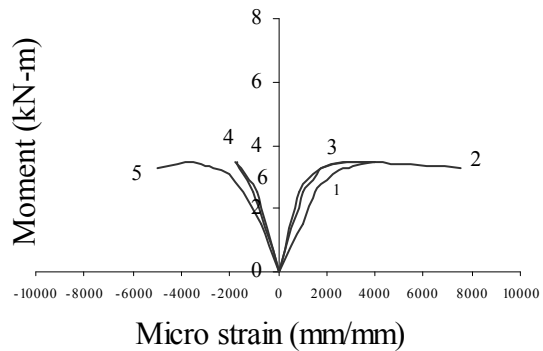
iii) Moment vs strain (Type B beam)



iv) Moment vs strain (Type C beam)



v) Moment vs strain (Type D beam)



vi) Moment vs strain (Type E beam)

Figure 9. Moment vs Micro Strain Plots (About Minor Axis)

5. CONCLUSIONS

The following observations and conclusions are drawn based on the present study on hollow and in-filled light gauge steel sections.

Due to the plain cement concrete in-fill and the SFRC in-fill, the light gauge steel hollow sections exhibit enhanced flexural performances of the beams compared to hollow beams. The bond strength of SFRC in-filled specimens is higher than PCC in-filled specimen. The SFRC in-filled specimen takes 63% more load than the PCC in-filled specimens before the slip. Because of the SFRC in-fill, the tested in-filled beams behaved in a relatively ductile manner. The enhanced ultimate load and ductility of the beams are due to the “composite action” between the steel tube and the concrete core. The theoretical flexural stiffness predicted based on the codes, AIJ [2], BS 5400 [3], Eurocode 4 [4] and LRFD [5] are compared with the experimental flexural stiffness. In general, all the codes predicted slightly higher values for the flexural stiffness. The AIJ [2] code shows closer values of the experimental results and can be accepted for calculation of the flexural stiffness of SFRC in-filled beams.

The SFRC in-filled beams show lesser deflection than the hollow and PCC in-filled beams and exhibit significant yield plateau. The moment capacities of the hollow, PCC & in-filled beams are experimentally calculated. The SFRC in-filled beams carry around 3 times more load than the hollow beams with respect to the major axis and 3.66 times more load than the hollow beams with respect to the minor axis. When compared with PCC in-filled beams, the SFRC in-filled beams carry about 1.20 times more load with respect to the major axis and 1.55 times more load with respect to the minor axis. The SFRC in-filled beams with 1% volume fraction of fiber is found to have higher flexural strength and less stiffness than the other two volume fraction of fibers (0.75% and 1.25%) studied and can be taken as optimum percentage of volume fraction fibers in the SFRC in-filled beams. The validity of different codes for determining the strength and stiffness of PCC in-filled beams and their applicability to SFRC in-filled beams have been discussed. The experimental moment capacities of the beams and the theoretical moment capacities calculated based on the codes AIJ [2], BS 5400 [3], Eurocode 4 [4] and LRFD [5] are compared. The AIJ [2] and Eurocode 4 [4] gave a moment capacity around 11 % lesser than the experimental values, with respect to the major axis and 30 % lesser than the experimental value with respect to the minor axis for SFRC in-filled beams. BS 5400 [3] and LRFD [4] gave a moment capacity about 16% lower than the experimental moment values about the major axis and 19% lower than the experimental moment values about the minor axis for SFRC in-filled specimens. The code provisions indicate plain cement concrete in-filled light gauge beams cannot be used directly for SFRC in-filled light gauge steel beams. Hence new provisions are required for composite beams with high performance concrete in-fills such as SFRC.

NOMENCLATURE

A_c	concrete cross - sectional area
A_s	steel cross - sectional area
A_{sn} / A_r	area of the reinforcement
A_w	web area of the encased steel
B	width of rectangular steel tube
b_f	external dimension of rectangular section
C_r	average distance to the reinforcement
D	depth of rectangular steel tube
d / b	out side diameter of steel tube
d_c	distance of neutral axis from the most compressed face of concrete
d_f	diameter of fiber
E_c	concrete modulus of elasticity
E_s	steel modulus of elasticity
f_c	concrete cylinder strength
f'_c	characteristic concrete cylinder strength
f_{cd}	design yield strength of the concrete
f_{ck} / f_{cu}	characteristic concrete strength
f_{sd} / f_{yr}	design yield strength of steel reinforcement
$f_y / \sigma_y / f_{yd}$	yield strength of the steel tube
h	depth of concrete in-fill
h_1	width of the member perpendicular to the plane of bending
h_2	width of the member parallel to the plane of bending
I_c	moment of inertia of concrete core
I_s	moment of inertia of steel tube
K_e	theoretical flexural stiffness of composite beam
K_{ee}	experimental flexural stiffness of composite beam
l_f	length of fiber
M_e	theoretical moment of composite beam
M_{ee}	experimental moment of composite beam
${}_c r_o$	outer radius of the concrete tube
t	thickness of the steel tube
${}_s Z_p / Z$	plastic section modulus of the steel tube
θ	angular location of the neutral axis of the beam

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