

Experimental Investigation on Reinforced Concrete Filled Steel Rectangular Fluted Columns

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Abstract— Behaviour of Concrete Filled Reinforced Steel Tubular Columns having rectangular Flutes have been investigated by conducting experiment on thirteen Columns having 2.5m length and different L/D ratios. Columns reinforced and filled with SCC were tested with three different L/D ratios of 15, 20 and 25 and varying the longitudinal reinforcements with 3,4,5 and 6 number of reinforcements. The columns were tested having its ends firmly held in position and analysed for its stiffness and strength. Though the columns were tested with L/D ratio of 25, columns did not buckle, failing at the ends showing failure is by local buckling. The results are compared with the existing empirical equations and have found that the secant equation compares comparatively well with 50 to 60 per cent. Increase in longitudinal reinforcement in the columns have shown a maximum of 32 % of increase in axial load in the case of Columns with L/D ratio of 25 with 4 number of bars.

Keywords: CFSFC, CFST, SCC, Composite, Circular Column, Rectangular Flutes.

1 INTRODUCTION

CFST (Concrete Filled Steel Tubes) Columns came into existence during early 1960. Substantial research has been made to understand the behaviour since then (1-21). The advantage of using these CFST Columns have been found by Japanese first and employed in the construction of multi-storeyed buildings effectively. Now, the analysis and Design of these CFST Columns have found place even in Codes and Specifications. It has been envisaged to study strength, stiffness and buckling characteristics by providing flutes to steel sheet of columns which enhances aesthetics of columns. Also, fluted columns enhances the strength and also stiffness as the surface area of steel sheet and moment of inertia of the column increases. The advantage of steel members having high tensile strength and ductility and concrete members having better compressive strength have been better made use as a composite member. Additional longitudinal reinforcement in the columns makes the columns still stronger. Hence, it has been envisaged to check whether such a columns would act as a slender.

Research has been in progress around the world on experimental and analytical studies on Concrete Filled Steel Tubular Columns for more than four decades. Substantial contribution has been made since then in understanding the behaviour of CFST columns and to arrive at a design procedure. Quite few countries have incorporated the design procedure in their respective codes also.

Most of the researchers (1-21) have considered the contribution of geometric properties like shape, L/D ratio, t/D ratio,

boundary conditions, strength of materials and the loading conditions. It has been found that generally the failure occurs by either local buckling or yield failure. It has been found that Euro code gives a better design method which yields values nearer to experimental values.

Studies performed on different L/D ratios with small eccentricities have yielded that the degree of confinement offered by a thin walled circular steel tube to the internal concrete is dependent on the load conditions.

2 EXPERIMENTAL INVESTIGATION

2.1 Preparation of Specimen

Mild steel sheet having thickness 0.8 mm has been pressed in a mill to obtain five rectangular fluting of 40 mm width 10 mm at apex uniformly along the length. These sheets were given a tubular shape and tacked along the edges at an interval of 250 mm along the length of the column. The number and the size of the flutes remained same irrespective of the diameter of the column i.e., for different L/D ratios. The development length of the width of each of these columns with different L/D ratios as compared to a circular column of the same diameter is 24, 29 and 34 % for L/D ratios of 15, 20 and 25 respectively. Reinforcement cage is then placed inside these fluted tubes taking care to maintain the necessary cover. The five types of columns have been shown in Fig 2.1(a) & Fig 2.1(b). Though regular ties have not been used, however four ties have been provided at equal distances to keep the reinforcement in position. A Self Compacting Concrete of design mix M20 designed as per Nan Su method(22) and tested for conformity as per IS specifications is poured into the fluted steel tube. These columns were cured for 28 days by frequently pouring water over top of the column. Pilot specimens cured in a similar manner were tested to know the basic properties and are entered in Table 1.

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TABLE 1.
Details of testing of the materials from experimental results

Materials	Concrete	Reinforcement	Steel Sheet
Poisson's ratio (μ)	0.16	0.28	0.26
Modulus of Elasticity (E) N/mm ²	0.223×10^5	0.21×10^5	0.723×10^5
Remarks	Split tensile strength test	Tensile test	Tension coupon test



Fig 2.1.(a) Fluted steel tube without reinforcement



Fig 2.1.(b) Fluted steel tube with reinforcement

2.2 Experimental Setup

The tests were conducted using a 2000 kN capacity hydraulic jack placing the specimen in the testing machine as shown in Fig.2.2. The bearing surfaces of the testing machine and the bearing plates were wiped clean and any loose sand or other material removed from the surface of the specimen. Which were to be in contact with the bearing plates. The specimen was placed between the bearing plates in such a manner that the upper bearing plate was directly in line with the lower plate and the bearing plates extend at least 25 mm from

each end of the specimen. The columns were at placed restraining rotation at both ends. Care was taken to ensure that truly axial load was transformed to each of the columns. This was achieved by using plumb bob and Theodolite.



Fig. 2.2. Experimental Set Up.



Fig 2.3. Connection of Dial Test Indicators Laterally

2.3 Instrumentation

Foil strain gauge (8mm x 8mm) $350 + 0.5 \Omega$ has been used to measure the strains at the center of the steel tube and center of the reinforcement (core) of the specimens. Three numbers 50 mm dial test indicator with a least count of 0.01 mm one for axial and the other two for lateral were used to measure axial and lateral deformations upon loading as shown in Fig 2.3. Apart from these instruments plumb bob and linear scales have been used.

3 RESULTS AND DISCUSSION

Behaviour of the columns has been studied with respect to deformation characteristics in the axial direction and in the transverse direction. Strains have been measured on the steel sheet in two perpendicular directions.

Generally the columns have shown linear behaviour up to about one third of the total load that is about 200 kN axial compressive load. It was envisaged to study the buckling characteristic because of the less width to length of column. None of the columns have shown buckling, near the mid portion of the column. The confinement of the column is so large and even the columns without the longitudinal reinforcement have not shown any buckling. All the columns have failed near supports of column showing local buckling. Columns with L/D ratio of 25 also have not buckled. Behaviour of each column and its characteristics have been explained in subsequent articles. Among various number of reinforcements, the ultimate load has been found to be maximum for L/D ratio of 15, 20 and 25 to be in 4 number of reinforcements as shown in Fig.3.

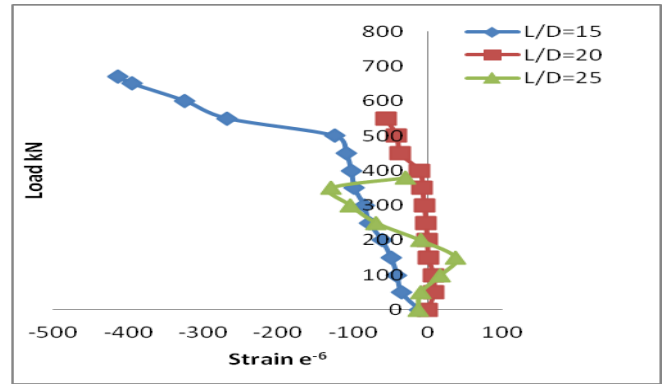


Fig 3.1 (a) Load Vs Strain

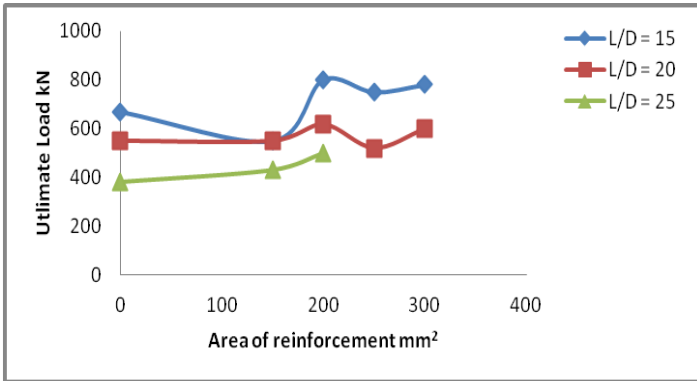


Fig 3 Ultimate load v/s Area of reinforcement

3.1 Columns Without Reinforcement (CFSFC-NR)

Columns without reinforcement have been tested with L/D ratios of 15, 20 and 25 to compare with the columns having reinforcement. For columns with L/D ratio of 15 and 20 the yield load with 500 kN & 400 kN where as for the other column with L/D ratio of 25, the yield load got reduced by 70 & 63 % . As the L/D ratio increased the maximum load has reduced by 18 and 43 % as compared to the column with L/D ratio of 15, showing that the column strength reduces to three fourths and to half as the L/D ratio is reduced from 15 to 20 and 25 respectively. The lateral deformation has increased by 54% and 42% as L/D ratio increased from 15 to 20 and 25 respectively. The maximum strains recorded were 414, 55 and 129 microstrains along the axial direction on the steel sheet. The variation of strains for various loads is depicted in Fig 3.1 (a). Similar readings were observed in the other dial gauge kept perpendicular. The load Vs axial deformation and lateral deformation is shown in Fig 3.1.(b) and 3.1(c). The stress Vs strain is shown in Fig 3.1 (d). The column failed due to local buckling, the failure being at the bottom of the column at a distance of 230 mm , 360 mm and 180 mm at top for L/D ratio of 15,20 and 25 respectively. The sheets have opened up at the bottom of the column for L/D ratio of 15 and the sheets have opened up at the top of the column for L/D ratios of 20 and 25 respectively.

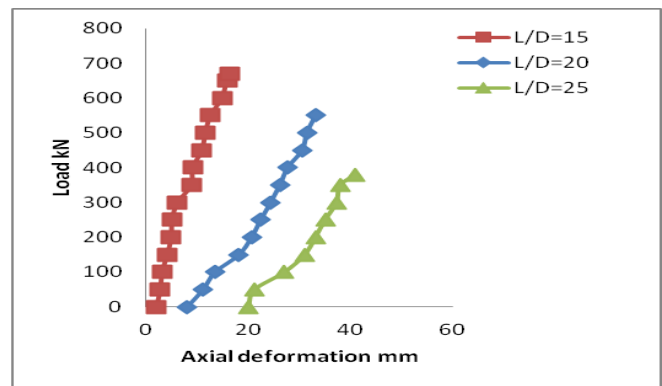


Fig 3.1 (b) Load Vs Axial deformation

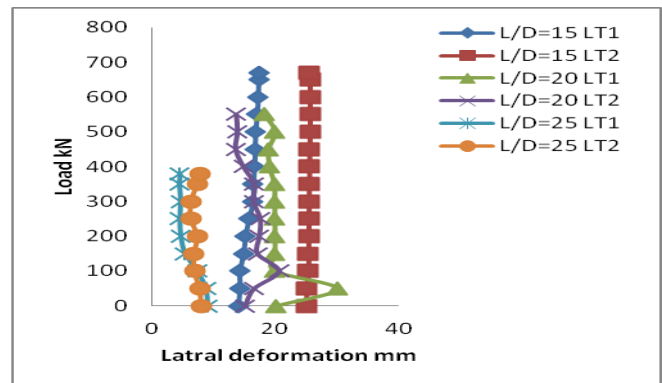


Fig 3.1 (c) Load Vs Lateral deformation

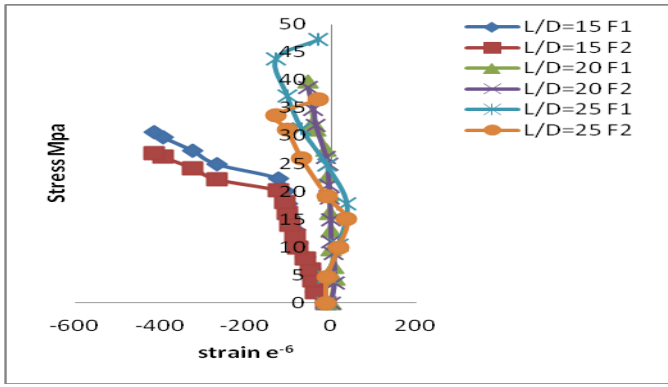


Fig 3.1 (d) Stress Vs strain

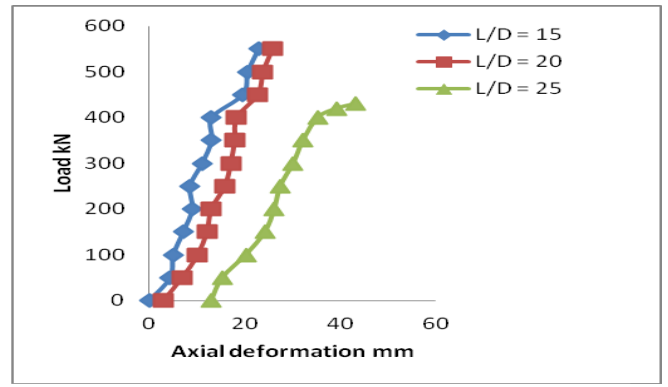


Fig 3.2 (b) Load Vs Axial deformation

Fig 3.1 Behaviour of columns CFSFC-NR

3.2 With Reinforcement (CFSFC-3/#8)

Testing of columns having reinforcements of three number of 8 mm rods with three different L/D ratios have been reported here. Both yield loads and the maximum loads have reduced as the L/D ratio increased. The yield load was 450 kN for the column with L/D ratio of 15 and reduces to 22 % and 56% for L/D ratio of 20 and 25. The maximum load is 550 kN for L/D ratio of 15 and reduces to 0 % and 22 % for L/D ratios of 20 and 25 showing that the strength reduces to almost quarter as the L/D ratios reduces from 15 to 25. The distance at which the failure occurred by opening up of sheet is at a distance of 330 mm at bottom, 250 mm from bottom and 230 mm from top for L/D ratios of 15, 20 and 25 respectively, showing that the failure can either be at top or bottom of the column. Strains have almost remained same as the L/D ratio increases as shown in Fig 3.2(a). The maximum deflection decreases as the L/D ratio increases as can be seen in Fig 3.2 (b). Load Vs maximum lateral deformation has been shown in Fig 3.2 (c). Which shows that though, not much of deformation exists, slight lateral deformation has been observed for the column with L/D ratio of 25. The maximum lateral deformation has been observed to be 3.04, 8.38 and 2.02 mm respectively. For L/D ratio of 15, 20 and 25. It can also be observed that the strains are more in the reinforcement and not on the steel sheet. The stress Vs strain is shown in Fig 3.2 (d).

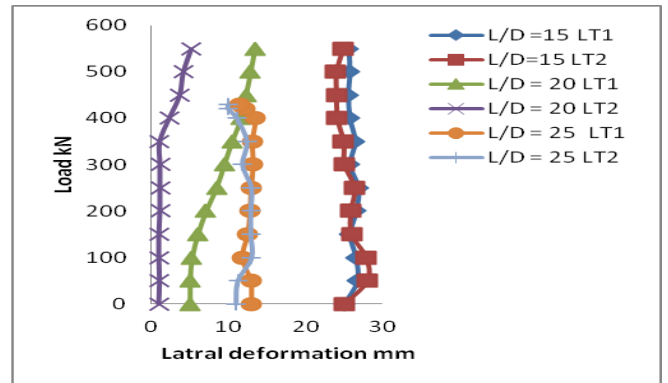


Fig 3.2 (c) Load Vs Lateral deformation

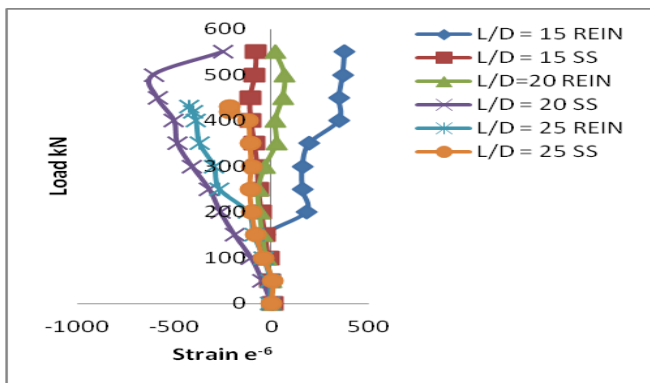


Fig 3.2 (a) Load Vs Strain

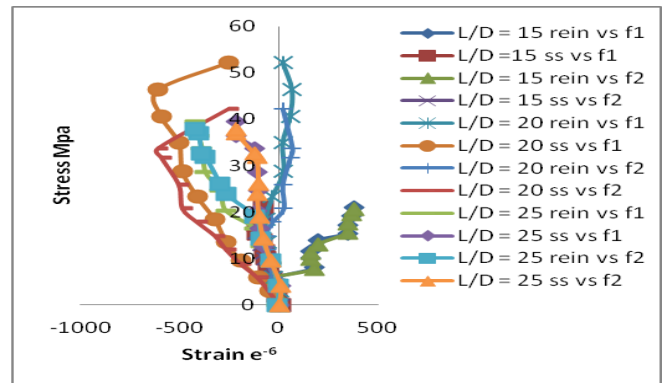


Fig 3.2 (d) Stress Vs strain

Fig 3.2 Behaviour of columns CFSFC -3/#8

3.3 CFSFC-4/#8

For the dimension of column chosen four number of reinforcements appears to be optimum as the yield and ultimate load is maximum for the parameter that have been chosen. The maximum load resisted by the column with L/D ratio of 15 is 800 kN. It has been observed that as the L/D ratio reduces from 15, ultimate load reduces to 23 % and 38 % for L/D ratio of 20 and 25 respectively. The maximum strains recorded were 527,632 and 337 microstrains along the axial direction on the steel sheet. The variation of strains for various loads is depicted in Fig 3.1 (a). The maximum deflection both in longitudinal direction and lateral direction have been ob-

served to be 14.8, 26.88 & 26 and 6.94, 4.06 and 4.67 respectively for the column with L/D ratio of 15, 20 and 25. The graph of load Vs deformation have been shown in Fig 3.3 (b) and (c). As has been in earlier cases, little more horizontal deformation has been observed for columns with L/D ratio of 25. The stress Vs strain is shown in Fig 3.3 (d). The distances at which the failure occurred are 280 mm from bottom 210 mm from top and 260 mm from bottom support showing that the fracture location gets nearer to the bottom as L/D ratio reduces.

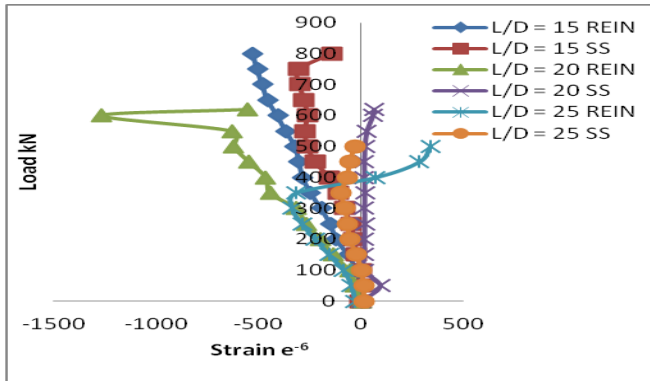


Fig 3.3 (a) Load Vs Strain

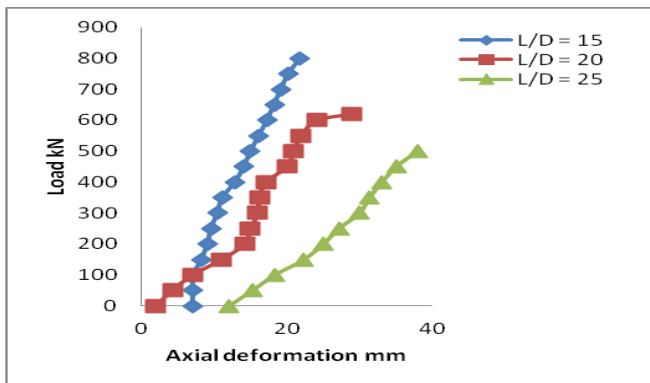


Fig 3.3 (b) Load Vs Axial deformation

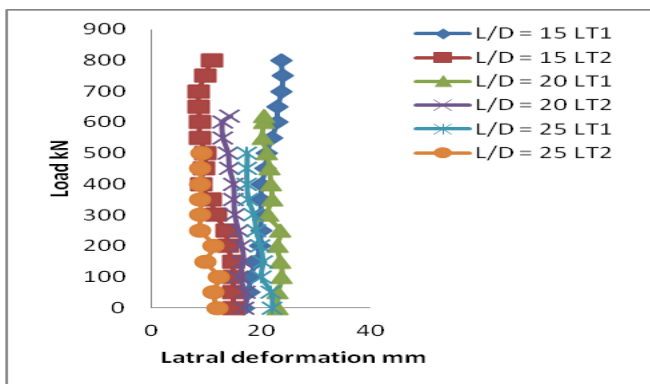


Fig 3.3 (c) Load Vs Lateral deformation

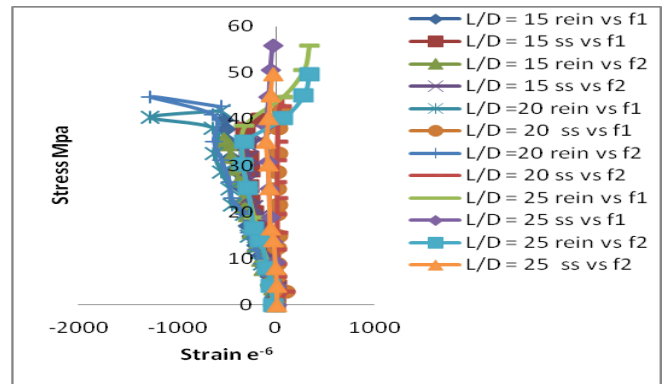


Fig 3.3 (d) Stress Vs strain

Fig 3.3 Behaviour of columns CFSFC-4/#8

3.4 CFSFC-5/#8

For the columns with 5 bars, L/D ratio considered are 15 and 20 only as the five numbers of reinforcement could not be placed in the column with an L/D ratio of 25 i.e., with the column diameter of 100 mm. Hence only two columns have been tested in this category along with the column having no reinforcement. Columns with L/D ratios of 15 and 20 has resisted similar loads as that of column with four number of bars. The yield load is 10 % and 40 % less as compared to non reinforced column.

As can be seen from Fig.3.4 (a), Load Vs Strain, the strain has increased constantly up to a load earlier to yield load and later has remained constant where as for L/D 20, the strain has increased up to about 300kN i.e yield load and later it has drastically changed. The axial deformation is about 27% more and 1 % less as compared to the column with same L/ D ratio without reinforcement as shown in Fig 3.4 (b). Fig. 3.4(c) depicts behaviour of load vs. lateral deformation, the lateral deformation remained almost same as compared to that of column without reinforcement, showing stiffness remained same even with reinforcement of 5 longitudinal bars in the column. The stress Vs strain is shown in Fig 3.4 (d).

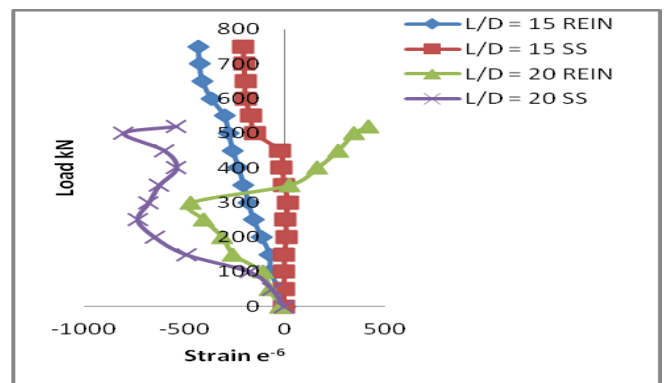


Fig 3.4 (a) Load Vs Strain

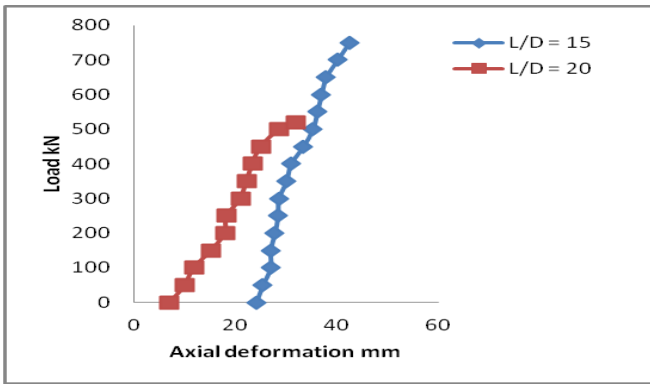


Fig 3.4 (b) Load Vs Axial deformation

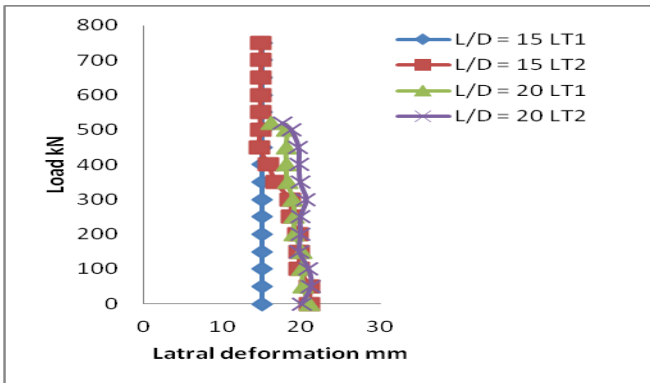


Fig 3.4 (c) Load Vs Lateral deformation

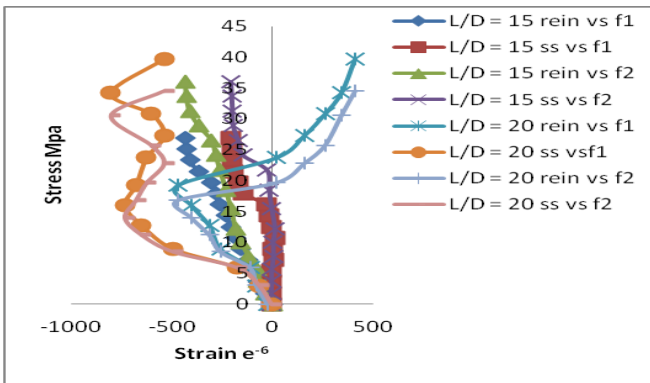


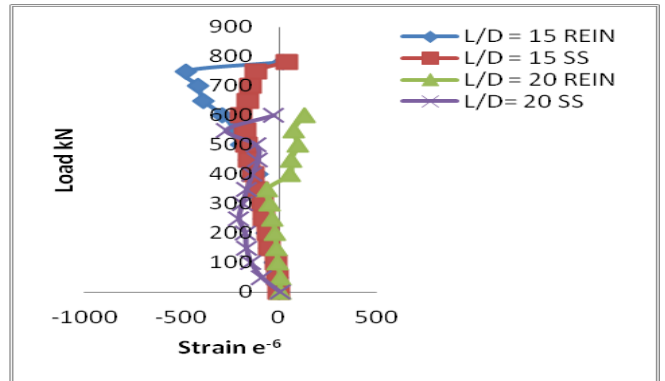
Fig 3.4 (d) Stress Vs strain

Fig 3.4 Behaviour of columns CFSFC-5/#8

3.5 CFSFC-6/#8

As has been indicated earlier, because of the constraint of placing bars L/D ratio of 15 and 20 have been considered here. Both the yield loads for L/D 15 and 20 have increased by 10 % and decreased by 38 % where as the ultimate load increased by 16% and 9% compared to that of no reinforcement column. When compared to columns with 5 bars in columns,, the yield load has increased by about 22% and 0% for L/D ratio of 15 and 20 and ultimate load increased by 4% and 15%. The maximum load reduces by about 10 % when the column dimension increased from L/D 15 to 20. The

failure occurred at top and bottom for L/D ratio 15 and 20 respectively at a distance of 330 mm and 300 mm. The maximum strains recorded were 478 and 269 microstrains along the axial direction on the reinforcement. The variation of strains for various loads is depicted in Fig 3.1(a). The axial deformation is about 62% more and 8 % less as compared to the column with same L/ D ratio without reinforcement as shown in Fig 3.4(b). Fig 3.4(c) depicts behaviour of load vs. lateral deformation, the lateral deformation by 44% more as compared to that of column without reinforcement for L/D ratio 15 and 20 respectively, showing stiffness remained same even with reinforcement of 5 longitudinal bars in the column. The stress Vs strain is shown in Fig 3.5 (d).



3.5 (a) Load Vs Strain

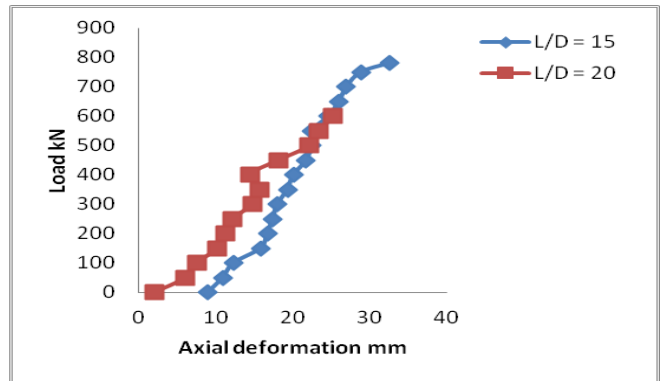


Fig 3.5 (b) Load Vs Axial deformation

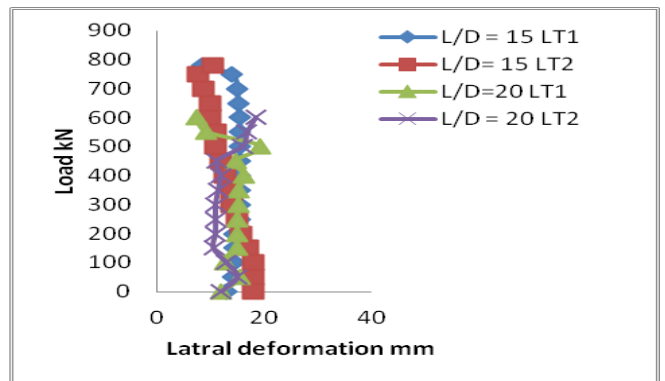


Fig 3.5 (c) Load Vs Lateral deformation

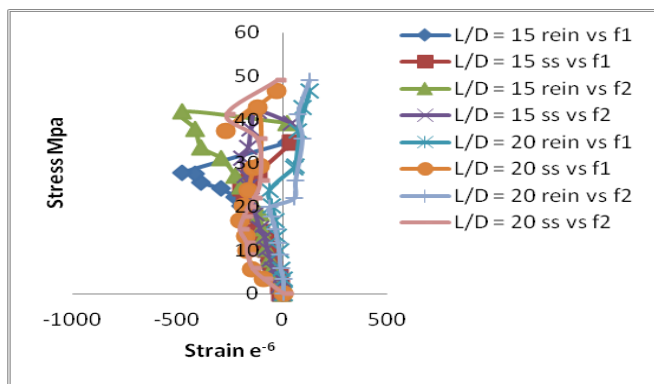


Fig 3.5 (d) Stress Vs strain

Fig 3.5 Behaviour of columns CFSFC-6/#8



4(c). Failure of column L/D 25 at top with 3/#8



4(d) Failure of column CFSFC L/D 20 at bottom with 3/#8



4(a). Failure of all columns (buckling does not exist)



4 (b). Failure of columns at edges for L/D 15

Fig 4. Failure of columns

4 THEORETICAL EQUATIONS

These results of tests of CFSFC have been compared with the results of various existing theoretical equations given here under.

4.1 Euler's equations

$$P_{cr} = (\pi^2 EI) / l^2 = f_c A$$

4.2 Rankine-Gorden's formula

$$P = (f_c \times A) / ((1 + \alpha(l/k)^2))$$

4.3 Secant formula

$$P = (f_c \times A) / (1 + e(y_c/k^2)) \times \{1 + \alpha(l^2/k^2)\}$$

4.4 Tangent Modulus equation

$$(P/A)_{cr} = (\pi^2 E_t) / (l/k)^2 = f_c \times A$$

4.5 Rankine-Merchant's formula

$$f_{ac} = \{0.66 \{f_{cc} \times f_y\} / [f_{cc}^n + f_y^n]^{(1/n)}\}$$

4.6 Infilled columns

$$P_u = 1.1 A_c f_{cc} [1 - ((0.000025 L^2) / (r_c)^2)] + A_s f_{sc} + A_{st} f_{st}$$

4.7 IS: 456-2000

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

Composite section

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} + 0.67 f_{ss} A_{sc}$$

Reduction factor C_r has not been considered as none of the columns have shown any sort of buckling.

4.8 Nominal Design

$$p = p1 + p2 + p3$$

Where $p1 = L \times t \times f_y$ (Load taken by steel sheet)

$$p2 = A_c \times f_{ck} = \left[\frac{\pi(d)^2}{4} + 5(1 \times h) \right] \times f_{ck}$$

(Load taken by concrete)

$$p3 = A_{st} \times f_y = \left[\frac{\pi(d)^2}{4} \times n \right] \times f_y$$

(Load taken by reinforcement)

These equations have been modified to consider equivalent area, moment of inertia and young's modulus of that of CFSFC members. The values obtained from these equations have been compared with the experimental values and are tabulated in table 2. It can be observed that the values obtained by tangent equation are about 40% conservative as compared to that of no reinforcement column for the column with L/D ratio of 15, 20 and 25.

5 CONCLUSION

- The reinforcement improves the axial strength of the column by about 19%, 13% and 32% for 4 number of reinforcements with L/D ratio of 15, 20 and 25 as compared to the column with no reinforcement.
- The strength of Column reduces by 25% and 50% when L/D ratio reduces from 15 to 20 and 15 to 25.
- All the columns have failed near the supports of the column.
- As the reinforcement in the column increased the distance of failure point moves towards centre of the column and as L/D reduces the failure point moves towards the support.
- As number of reinforcement increases the distance of failure due to buckling has also increased by 43%, 22%, 65% & 43% respectively. For 3,4,5 & 6 number of reinforcements for L/D = 15. Similar results have also been found for other L/D ratios.
- No buckling has been observed for any of the columns even with higher L/D ratio of 25
- The lateral deformation is more in the case of 4 number of reinforcement with L/D ratio of 25.

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TABLE 2.
 Details Of Load Carrying Capacities Of CFSFC From Experimental And Empirical Values.

SI No	Name of the Specimen (L/D C No of rein)	Expt load kN	Nominal design		Infilled column		IS 456-2000		Euler's		Rankine's Gorden's		Secant Equation		Tangent		Rankin's Merchant	
			Load kN	%decrease	Load kN	%decrease	Load kN	%decrease	Load kN	%decrease	Load kN	%decrease	Load kN	%decrease	Load kN	%decrease	Load kN	%decrease
1	15 C 0	670	546	18.50	241	64.02	233	65.22	511	23.73	392	41.29	336	49.85	511	23.73	299	55.37
2	15 C 3	550	610	-10.91	282	48.72	274	50.18	540	1.81	413	24.90	375	31.81	540	1.81	315	42.72
3	15 C 4	800	631	21.12	296	63.00	288	64.00	549	31.37	420	47.50	307	61.62	549	31.37	320	60.00
4	15 C 5	750	652	13.06	310	58.66	302	59.73	559	25.46	427	43.06	318	57.60	559	25.46	326	56.53
5	15 C 6	780	673	13.71	324	58.44	316	59.48	568	27.17	435	44.23	304	61.02	568	27.17	331	57.56
6	20 C 0	550	339	38.36	145	73.63	136	75.27	311	43.45	207	62.36	184	66.54	311	43.45	163	70.36
7	20 C 3	550	402	26.90	187	66.00	174	68.36	340	38.18	225	59.09	139	74.72	340	38.18	178	67.63
8	20 C 4	620	423	31.77	200	67.74	186	70.00	350	43.54	231	62.74	192	69.03	350	43.54	183	70.48
9	20 C 5	520	444	14.61	214	58.84	199	61.73	359	30.96	237	54.42	173	66.73	359	30.96	187	64.03
10	20C6	600	466	22.33	227	62.16	212	64.66	369	38.50	244	59.33	161	73.16	369	38.50	192	68.00
11	25 C 0	380	241	36.57	101	73.42	91	76.05	219	42.36	127	66.57	94	75.26	219	42.36	102	73.15
12	25 C 3	430	305	29.06	141	67.20	125	70.93	248	42.32	143	66.74	126	70.69	248	42.32	115	73.25
13	25 C 4	500	326	34.80	154	69.20	137	72.60	257	48.60	147	70.60	103	79.40	257	48.60	118	76.40