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# PERFORMANCE OF SFRC INFILLED LIGHT GAUGE STEEL BOX COLUMN UNDER AXIAL AND ECCENTRICAL LOADS

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***Abstract:*** The concrete-filled steel columns are gain importance in several applications like buildings, shopping malls, bridges, and heavy apartment structures, etc. The steel box as a part of a composite column completely encases the concrete core so that the ductility of the encased concrete core is highly improved enhances the seismic- resistant property. The enhancement of concrete-filled steel columns in structural properties is due to the composite action between the constituent elements. The main objective of the study is to investigate the behavior of hollow, Plain Cement Concrete (PCC), and Steel Fiber Reinforced Concrete (SFRC) in-filled medium columns subjected to axial and eccentric loads. It was evaluated experimentally and compared to numerical analysis via Ansys.

**Keywords: Steel column: Axial and Eccentric load; ANSYS.**

## I.INTRODUCTION

The Euler's pioneering work on elastic buckling of columns presented in 1759, extensive studies concerning the behavior of columns have been conducted worldwide. The current trend is to reduce costs, improve productivity, quality and take full advantage of information technology and benefits from the new economy. As a result, enhanced research effort has focused attention on developing techniques for combining steel and concrete effectively, called steel-concrete composite structures. The application of composite members consisting of concrete-filled steel hollow sections has become increasingly popular in Civil Engineering structures in the last two decades. This is due to their advantages over the conventional structural sections in terms of strength, ductility, stiffness, energy absorption capacity, easy construction procedure, and the overall economy. The steel and the concrete element in a composite member complement each other ideally. While the steel confines the concrete laterally allowing it to develop the optimum compressive strength and ductility, the concrete, in turn, supports the steel shell laterally to prevent elastic local buckling.

## II. EXPERIMENTAL INVESTIGATION

The design theories assumed a full bond interaction between the steel shell and the concrete core for the simplicity of the calculation and the ultimate moment resistance. However, in reality, slippage at the steel-concrete interface is inevitable after the tensile cracking of concrete. For a better understanding of the complex interface behavior experimental and analytical investigation is a more effective method. The hollow sections were made from light gauge steel sheets, continuously welded at the middle along its length. The light gauge steel sections are shown in figure 1.

a) Specimens b) Cleaning for oil & Grease

Figure.1 Test Specimens

In order to determine the actual material properties, three steel coupons are cut from the four faces of these sections and tested to failure under tension as per ASTM A 370 specification. Figure 2.

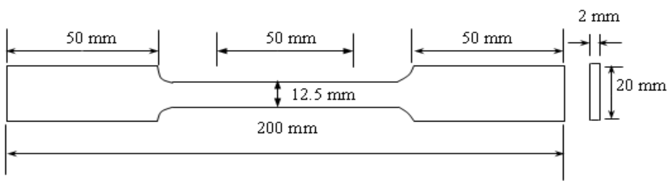



Figure .2 Details of Tension Coupons

## A. Test Specimens

The cross section of the steel sections is shown in Figure 3 and its geometrical properties are given below

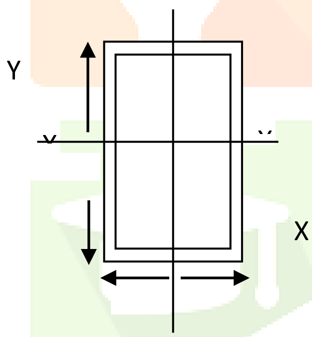



**Figure. 3 Cross Section Details of Test Specimen**

Area of steel tube = 566.80 mm2

Area of concrete = 4433.20 mm2

Moment of Inertia(Ixx) =73.1982 x 104mm4

Moment of Inertia(Iyy) = 25.194 x 104 mm4

All the values are listed in Table 1.1

For the composite sections equivalent areas are calculated and listed in Table 1

| Sl. No. | Description | Concrete converted to Steel | Steel converted to Concrete |
|---|---|---|---|
| PLAIN CEMENT CONCRETE | | | |
| 1 | Area of Steel | 1197.44 mm$^2$ | ---- |
| 2 | Area of Concrete | ---- | 8619.63 mm$^2$ |
| 3 | $I_{xx}$ | 1.246 x 10$^6$ mm$^4$ | 8.9712 x 10$^6$ mm$^4$ |
| 4 | $I_{yy}$ | 3.7199 x 10$^5$ mm$^4$ | 2.6714 x 10$^6$ mm$^4$ |
| 5 | $r_{xx}$ | 32.26 mm | 32.21 mm |
| 6 | $r_{yy}$ | 17.60 mm | 17.60 mm |
| 0.75% STEEL FIBER REINFORCED CONCRETE | | | |
| 1 | Area of Steel | 1281.73 mm$^2$ | ---- |
| 2 | Area of Concrete | ---- | 8122.61 mm$^2$ |
| 3 | $I_{xx}$ | 1.311 x 10$^6$ mm$^4$ | 8.31 x 10$^6$ mm$^4$ |
| 4 | $I_{yy}$ | 3.857 x 10$^5$ mm$^4$ | 2.45 x 10$^6$ mm$^4$ |
| 5 | $r_{xx}$ | 31.98 mm | 31.98 mm |
| 6 | $r_{yy}$ | 17.36 mm | 17.36 mm |
| 1.00% STEEL FIBER REINFORCED CONCRETE | | | |
| 1 | Area of Steel | 1402.57 mm$^2$ | ---- |
| 2 | Area of Concrete | ---- | 7566.64 mm$^2$ |
| 3 | $I_{xx}$ | 1.404 x 10$^6$ mm$^4$ | 7.57 x 10$^6$ mm$^4$ |
| 4 | $I_{yy}$ | 4.073 x 10$^5$ mm$^4$ | 2.197 x 10$^6$ mm$^4$ |
| 5 | $r_{xx}$ | 31.98 mm | 31.64 mm |
| 6 | $r_{yy}$ | 17.04 mm | 17.04 mm |
| 1.25% STEEL FIBER REINFORCED CONCRETE | | | |
| 1 | Area of Steel | 1255.23 mm$^2$ | ---- |
| 2 | Area of Concrete | ---- | 8266.90 mm$^2$ |
| 3 | $I_{xx}$ | 1.291 x 10$^6$ mm$^4$ | 8.50 x 10$^6$ mm$^4$ |
| 4 | $I_{yy}$ | 3.81 x 10$^5$ mm$^4$ | 2.512 x 10$^6$ mm$^4$ |
| 5 | $r_{xx}$ | 32.07 mm | 32.07 mm |
| 6 | $r_{yy}$ | 17.42 mm | 17.42 mm |

Table 1 Equivalent Area of the Test Specimens

## *B.* Test on Columns

To simulate the simply supported end conditions two plates 30mm thick and size 300x200mm with a spherical groove at the centre to accommodate a ball of 40mm diameter was bolted to the end plates on either ends by four 16mm diameter bolts such that their centers coincide is shown in figure.4. Above this, a proving ring of capacity 1000 KN was placed and the load from the jack was applied through the proving ring. The columns were tested up to failure. Prior to the actual test, a load level of 20 KN was applied so that the platens of the testing machine were firmly attached to both ends of the testing specimen. The axial and eccentric loads were slowly applied to the specimen by carefully controlling the loading and unloading valves in the jack. During the test, the longitudinal strain, axial shortening as well as in plane and out of plane deflections of the specimen were recorded at load increment of 10 KN in the elastic range and with 5 KN load increments after the columns began to yield, in order to have sufficient data points to delineate the "knee" of the load-strain curve. Deflect meter readings and strain-gauge readings were taken for every increment of load. All the operations and the change of loading rates were controlled manually. All the readings were recorded after the loads and the strains are stabilized.

Figure.4 Test column setup

*C.* Failure Mode and Ultimate Loads

Three modes of failure modes were observed for the specimens during the test are shown in figure 5. (**Double Curvature buckling of columns).** The first mode of failure occurred for the axially loaded columns and for columns loaded at 0.1D eccentricity. This was identified by local buckling at the longer face of the cross section and an overall single curvature bending about minor axis. The second mode of failure occurred for column loaded at 0.3B and 0.5B eccentricities. This was identified by single curvature bending showing insignificant sign of local buckling at mid heights of the columns. The third mode of failure were identified by the double curvature bending as shown in Figure 3.13, for column loaded at 0.3D and 0.5D eccentricities, which shows the effect of eccentricity about the major and minor axes .

Figure. 5 Double Curvature buckling of columns

| Sl. No. | Specimen label | e (mm) | Test Load $P_{exp}$ (kN) | Moment $M_e$ (kNm) | Theoretical Load Eurocode 4 | | Theoretical Load BS 5400 | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $P_{the}$ | $P_{the}$ / $P_{exp}$ | $P_{the}$ | $P_{the}$ / $P_{exp}$ |
| 1 | Type A | 0 | 182.11 | 0 | 175.41 | 0.963 | 169.37 | 0.930 |
| 2 | Type B | 0 | 294.30 | 0 | 270.40 | 0.919 | 264.33 | 0.898 |
| 3 | Type C | 0 | 313.920 | 0 | 300.470 | 0.957 | 295.456 | 0.941 |
| 4 | Type D | 0 | 372.780 | 0 | 327.240 | 0.878 | 320.311 | 0.859 |
| 5 | Type E | 0 | 362.970 | 0 | 312.180 | 0.860 | 305.412 | 0.841 |
| Mean | | | | | | 0.915 | | 0.894 |
| Standard Deviation | | | | | | 0.046 | | 0.043 |

Table 2 Comparison of the Results between the Test and Codes ( Load case: Axial )

| Sl. No. | Specimen label | e (mm) | Test Load $P_{exp}$ (kN) | Moment $M_e$ (kNm) | Theoretical Load Eurocode 4 (1994) | | Theoretical Load (BS 5400-1979) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $P_{the}$ | $P_{the}$ / $P_{exp}$ | $P_{the}$ | $P_{the}$ / $P_{exp}$ |
| 1 | Type A | 10 | 124.68 | 1.25 | 97.89 | 0.785 | 154.31 | 1.238 |
| 2 | | 30 | 115.00 | 3.45 | 112.64 | 0.979 | 106.31 | 0.924 |
| 3 | | 50 | 104.00 | 5.20 | 102.43 | 0.985 | 95.312 | 0.916 |
| 4 | Type B | 10 | 264.87 | 2.65 | 256.70 | 0.969 | 249.416 | 0.942 |
| 5 | | 30 | 244.68 | 7.34 | 238.60 | 0.975 | 232.312 | 0.949 |
| 6 | | 50 | 221.27 | 11.06 | 242.41 | 1.096 | 237.312 | 1.072 |
| 7 | Type C | 10 | 274.680 | 2.750 | 217.190 | 0.791 | 272.412 | 0.992 |
| 8 | | 30 | 273.800 | 8.210 | 268.220 | 0.980 | 262.432 | 0.958 |
| 9 | | 50 | 247.610 | 12.380 | 250.510 | 1.012 | 244.312 | 0.987 |
| 10 | Type D | 10 | 294.300 | 14.720 | 311.510 | 0.962 | 305.312 | 0.943 |
| 11 | | 30 | 266.000 | 3.990 | 275.190 | 0.969 | 269.312 | 0.948 |
| 12 | | 50 | 247.000 | 6.180 | 260.270 | 0.993 | 254.331 | 0.971 |
| 13 | Type E | 10 | 304.110 | 3.040 | 301.720 | 0.992 | 296.312 | 0.974 |
| 14 | | 30 | 279.000 | 8.370 | 271.310 | 0.972 | 263.192 | 0.943 |
| 15 | | 50 | 253.000 | 12.650 | 254.910 | 1.008 | 249.312 | 0.985 |
| Mean | | | | | | 0.965 | | 0.983 |
| Standard Deviation | | | | | | 0.079 | | 0.080 |

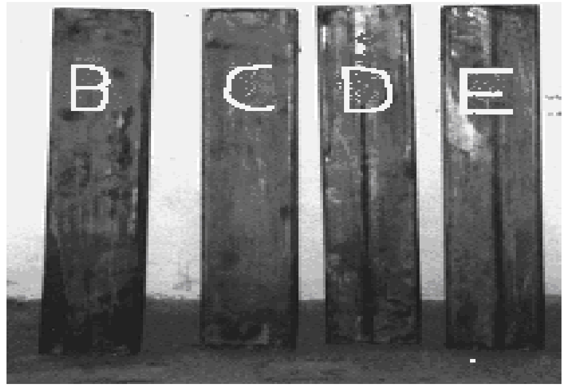
Table 3 Comparison of the Results between the Test and Codes (Load case: Uni-axial Eccentric along Y Axis)

## IV. DISCUSSION OF RESULTS

In this chapter an experimental study was made to study the possibility of using light gauge hollow, PCC in-filled and SFRC in-filled steel columns in practice. Based on the results of this study, the following results were discussed with in the scope of the tests:

- In the experimental programme three failure modes were observed during loading: For the axially loaded column failure is by over all buckling with significant sign of local buckling. For the eccentric loaded columns the failure is by single curvature and double curvature bending failure with insignificant sign of local buckling
- While applying the load for in-filled columns, the cracking sound of concrete occurred at 280 kN load for PCC in-filled columns and around 350 kN load for SFRC in-filled columns which indicates the enhanced structural performance of SFRC in-filled columns.
- Comparing the load carrying capacity, the SFRC in-filled columns carried around 2 times more load than the hollow columns for axial loads and 2.55 times more load for eccentrically loaded columns.
- In the load deflection curve, the non-linearity behaviour started at the on-set of loading for all columns loaded eccentrically with an eccentricity ratio of 0.30 and 0.50 irrespective of the axis.

- By comparing the stiffness of the SFRC in-filled column, the stiffness of PCC in-filled columns were on higher side which indicates the ductile behaviour of SFRC in-fill.



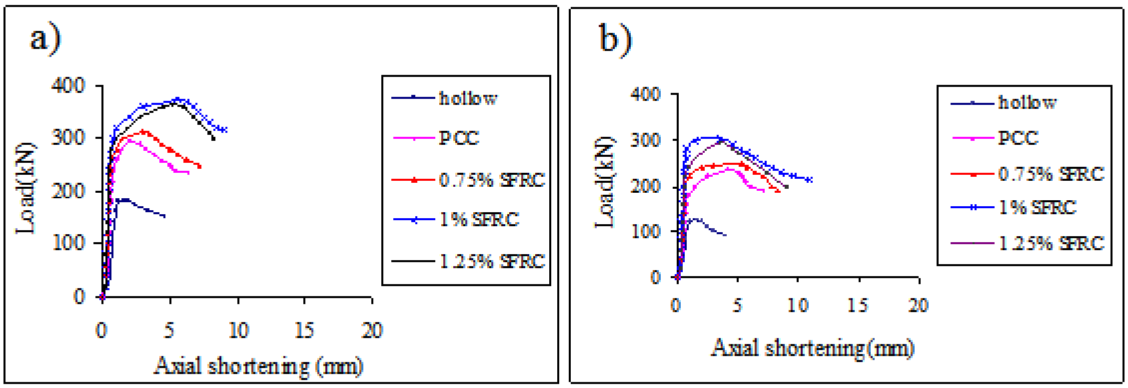




Figure.6 a) Axial load Vs axial shortening b) 0.1$e_x$ Eccentric load Vs axial shortening

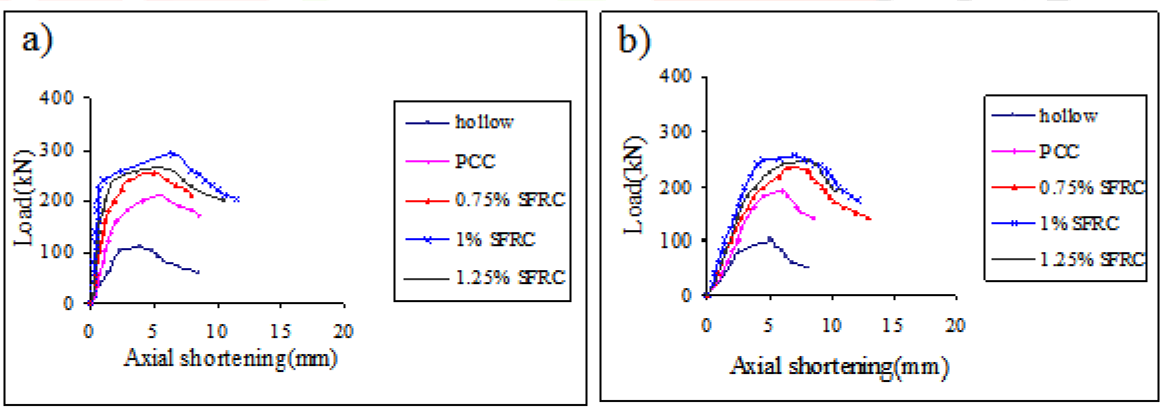



Figure.7 a) 0.3ex Eccentric load Vs axial shortening b) 0.5ex Eccentric load Vs axial shortening

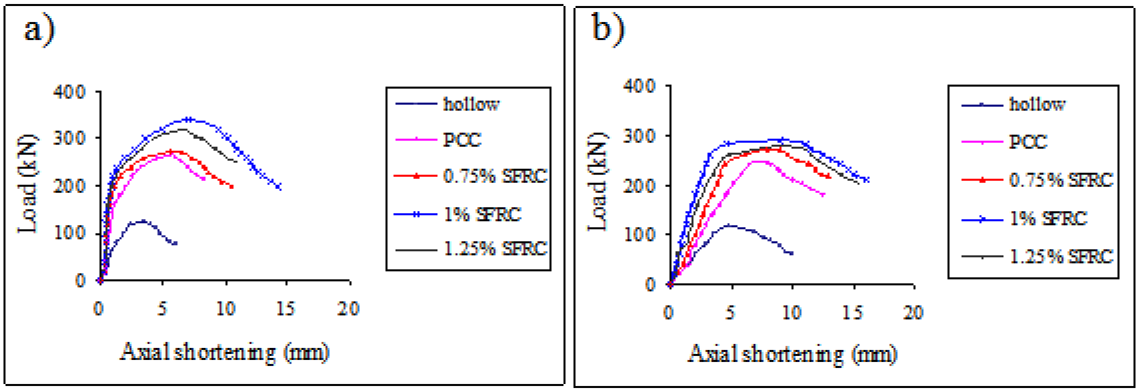



Figure.8 a)0.1ey Eccentric load Vs axial shortening b) 0.3ey Eccentric load Vs axial shortening

Figure.6-8 Load Vs axial shortening plots

## V. CONCLUSION

In this thesis an study was made to study the possibility of using light gauge hollow, PCC in-filled and SFRC in-filled steel columns in practice. Based on the study, the following results were discussed with in the scope:

- In the experimental programme three failure modes were observed during loading: For the axially loaded column failure is by over all buckling with significant sign of local buckling. For the eccentric loaded columns the failure is by single curvature and double curvature bending failure with insignificant sign of local buckling
- While applying the load for in-filled columns, the cracking sound of concrete occurred at 280 kN load for PCC in-filled columns and around 350 kN load for SFRC in-filled columns which indicates the enhanced structural performance of SFRC in-filled columns.
- Comparing the load carrying capacity, the SFRC in-filled columns carried around 2 times more load than the hollow columns for axial loads and 2.55 times more load for eccentrically loaded columns.
- In the load deflection curve, the non-linearity behaviour started at the on-set of loading for all columns loaded eccentrically with an eccentricity ratio of 0.30 and 0.50 irrespective of the axis.
- By comparing the stiffness of the SFRC in-filled column, the stiffness of PCC in-filled columns were on higher side which indicates the ductile behaviour of SFRC in-fill.
- The Strength Gain Index (SGI) was maximum for 1% SFRC in-filled column and this report suggested that the optimum percentage of SFRC may be taken as 1% for in-fill of crimped shape fibers.
- The ductility index value was on the higher side for the SFRC in-filled columns by more than 70% when compared to plain concrete in-filled columns.

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