CHAPTER 3

STUDIES ON STUB AND SHORT COLUMNS

3.1  GENERAL

The design of light gauge steel hollow members in-filled with ordinary concrete are normally carried out on the basis of provisions prescribed by different codes of practices. Due to huge variations in the properties of the in-filled concrete and the complexity in the interaction that occurs between the concrete and the outer shell, it frequently makes an uneconomical design of members and systems completely on theoretical basis. Also when adequate analytical design procedures are not available for the member or composites, the testing of the components are recommended. The behaviour of composite columns and beams can often be ascertained experimentally and suitable modifications can be incorporated, wherever necessary.

Current design theories assume full bond between the steel shell and the concrete core to simplify the determination of the ultimate moment of resistance. However, in reality slippage at the steel-concrete interface is inevitable after the tensile cracking of the core concrete. For a better understanding of the complex interface behaviour, experimental and analytical investigations are more effective. Push out resistance tests on concrete are used to determine the bond slip characteristics between outer steel shell and in-filled concrete through the load versus slip response, which can also be used to evaluate the overall performance of the concrete-filled
steel sections. A better understanding is needed on the force transfer mechanism between steel and concrete when large rotations and buckling occur.

In the present investigation, a series of experiments were carried out to study the behaviour of hollow, PCC in-filled and three varieties of SFRC in-filled stub, short, medium columns and beams with emphasizes on the ultimate strength, ductility, axial deformations, lateral deflections and post failure strength reserve in elastic as well as in the plastic ranges with a focus on the bond between steel and concrete. An effort has also been made to compare the experimental and theoretical results with the results obtained using numerical models.

Out of eighty eight specimens prepared for the experimental study, seventy eight tests are conducted on columns and ten tests are conducted on beams. The columns are subjected to axial load and uni-axial and bi-axial eccentric loads. The beams are subjected to one third point loads about its major and minor axes. The effects of provision of the in-fill, the eccentricity of the load, the flat width to thickness ratio and the slenderness ratio on the load carrying capacities of the columns and beams are studied.

### 3.2 MATERIAL CHARACTERISTICS

#### 3.2.1 Steel Section

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 3.1. The size of the steel sections are 100 mm × 50 mm and 2 mm thick.
In order to determine the actual material properties, three steel coupons were cut from all the four faces of these sections and tested to failure under tension as per ASTM-A370 specification. Figure 3.2 shows the details of the specimen for the tension coupons. The test set up is shown in Figure 3.3. The yield strength of the light gauge steel is determined either by the offset method or the strain under load method. Offset method is used generally for gradual yielding steel and it is used to calculate the yield strength of the material. In the offset method, the yield strength is the stress corresponding to the intersection of the stress-strain curve and a line parallel to the initial straight-line portion offset by a specified strain. The offset is usually specified as 0.2 percent. The typical stress-strain behaviour of the tensile coupon test is shown in Figure 3.4.

From the tests, the following average values were obtained. Yield stress $f_y = 270 \text{ N/mm}^2$, Ultimate stress $f_u = 410 \text{ N/mm}^2$, 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 to be used is given by $B/t \leq 52 \left(\frac{235}{fy}\right)^{\frac{1}{2}}$, as prescribed by Eurocode4 -1994. The specimens maximum $B/t$ ratio is 46 and the value of $52 \left(\frac{235}{fy}\right)^{\frac{1}{2}}$ for the test specimens are 48.51, hence the chosen sections satisfy the above requirements and therefore all the specimens can be classified as compact sections.

![Figure 3.2 Details of tension coupons](image)

**Figure 3.2** Details of tension coupons

![Figure 3.3 Test set up for tension coupons](image)

**Figure 3.3** Test set up for tension coupons
3.2.2 Plain Cement Concrete

The concrete mix was designed for a cube compressive strength of 20 MPa at 28 days. The design mix of 1:2.09:2.25 with a w/c ratio of 0.49, using 12.5 mm size (max.) coarse aggregate and 2.36 mm (max.) size fine aggregate was used as per ACI committee 211.1.1991 recommendations. The PCC and SFRC for the composite columns were mixed in two separate batches. From the concrete mix, concrete cubes and cylinders were prepared and tested in compression to obtain the actual material properties. The test setup is shown in Figure 3.5 and the stress-strain curves for all types of concrete are shown in Figure 3.6. The material properties of PCC and SFRC are listed in Table 3.1.

3.2.3 Steel Fiber Reinforced Concrete (SFRC)

To prepare the SFRC in-filled steel composite columns, three different volume fractions of steel fibers were chosen viz., 0.75%, 1.00% and 1.25%. Crimped steel fibers having an aspect ratio of 70 ($l_f \approx 30.80$ mm and
df = 0.44mm) were used for this purpose. The material properties of SFRC are listed in Table 3.1. It has been shown that it was possible to relate the mechanical properties of fiber reinforced concrete to the fundamental properties of the fibers, such as diameter, length, and percentage in the matrix material, shape, bond characteristics and tensile strength. The weight percentage of fibers in the matrix material ranges from 0 to 6%, which corresponds to about 0 to 150 kg of fibers per cubic meter of concrete. An attempt has been made to study the optimum volume fraction for the chosen type of fiber for in-filled columns. The experimental characteristics of SFRC are compared with the theoretical values.

The expression for the compressive strength of fiber-reinforced concrete $f_{cf}$ proposed by Nataraja et al is presented in equation (3.1).

$$f_{cf} = f_c + 2.1604 \left( \frac{W_f l_f}{d_f} \right)$$  \hspace{1cm} (3.1)

where $W_f =$ weight percentage of fibers

In contrast to plain cement concrete for which most structural codes assume negligible resistance in tension, fiber reinforced concrete has, at the ultimate stage, a residual tensile strength $f_{tu}$ given by Soroushan and Lee is

$$f_{tu} = 2 \eta_1 \eta_0 \tau V_f \frac{l_f}{d_f}$$  \hspace{1cm} (3.2)

where $\eta_1$ an effectiveness factor of the form of fibers given by

$$\eta_1 = \frac{\tau}{\sigma_{fu}} \left( \frac{l_f}{d_f} \right)$$  \hspace{1cm} (3.3)

$\eta_0 =$ distribution factor of fibers (= 0.41 for random distribution of fibers in a three dimensional space)

$\tau =$ bond strength of fibers (2 ~ 6 MPa)

$V_f =$ volume percentage of fibers
The average elastic tangent modulus in compression $E_{cf}$ of fiber reinforced concrete is given by Bentur and Mindess as,

$$E_{cf} = \gamma V_f E_f + (1 - V_f) E_c$$

(3.4)

$E_f$ = modulus of elasticity of fibers

where the correlation factor $\gamma = \eta \left[1 - \frac{\tanh(n_r l_f / d_f)}{n_r l_f / d_f}\right]$  

(3.5)

In the above, the dimensionless co-efficient $n_r$ is given by

$$n_r = \sqrt{\frac{2E_c}{E_f(1+\nu_c)\log_e(1+V_c)}}$$

(3.6)

The modulus of elasticity $E_c$ of confined concrete (Eurocode2) is

$$E_c = 9500 (f_c + 8)^{\nu_c}$$

(3.7)

Using equations (3.1), (3.2) and (3.3), the compressive and residual tensile strength of SFRC are calculated. The elastic tangent modulus and Modulus of elasticity of confined concrete are calculated using equations (3.4), (3.5), (3.6) and (3.7). All the values are presented in Table 3.1.
### Table 3.1. Material properties of PCC and SFRC

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Type of in-fill</th>
<th>Cube Strength N/mm²</th>
<th>Cylinder Strength N/mm²</th>
<th>Cylinder Strength (Nataraja et al.) N/mm²</th>
<th>Flexural Strength N/mm²</th>
<th>Spilt Tensile Strength N/mm²</th>
<th>Residual Tensile Strength (Soroushian et al.) N/mm²</th>
<th>Youngs Modulus (Ec) (by Test) (x10⁴) N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain Cement Concrete</td>
<td>32.44</td>
<td>26.65</td>
<td>--</td>
<td>4.24</td>
<td>4.10</td>
<td>--</td>
<td>2.968</td>
</tr>
<tr>
<td>2</td>
<td>0.75% Steel Fiber Reinforced Concrete</td>
<td>41.78</td>
<td>34.97</td>
<td>27.95</td>
<td>4.94</td>
<td>5.15</td>
<td>1.59</td>
<td>3.230</td>
</tr>
<tr>
<td>3</td>
<td>1.00% Steel Fiber Reinforced Concrete</td>
<td>57.78</td>
<td>47.19</td>
<td>28.34</td>
<td>5.86</td>
<td>6.55</td>
<td>2.13</td>
<td>3.800</td>
</tr>
<tr>
<td>4</td>
<td>1.25% Steel Fiber Reinforced Concrete</td>
<td>38.60</td>
<td>31.64</td>
<td>28.81</td>
<td>4.40</td>
<td>5.10</td>
<td>2.66</td>
<td>3.109</td>
</tr>
</tbody>
</table>

**Figure 3.5** Test set up for calculation of stress-strain behaviour of concrete
3.3 TEST SPECIMENS

The cross section of the steel sections is shown in Figure 3.7.

The thickness of the steel tube is 2 mm
The geometrical properties of the section are given below.

Area of steel tube $= 566.80 \text{ mm}^2$
Area of in-filled concrete $= 4433.20 \text{ mm}^2$
Moment of Inertia $(I_{xx}) = 73.1982 \times 10^4 \text{ mm}^4$
Moment of Inertia $(I_{yy}) = 25.194 \times 10^4 \text{ mm}^4$

For the composite sections equivalent areas are calculated and listed in Table 3.2.

Figure 3.7 Cross section details of the test specimen
## Table 3.2 Equivalent area of the test specimens

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Description</th>
<th>Concrete converted to Steel</th>
<th>Steel converted to Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PCC</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Area of Steel</td>
<td>1197.44 mm²</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>Area of Concrete</td>
<td>--</td>
<td>8619.63 mm²</td>
</tr>
<tr>
<td>3</td>
<td>$I_{xx}$</td>
<td>$1.246 \times 10^6$ mm$^4$</td>
<td>$8.9712 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>4</td>
<td>$I_{yy}$</td>
<td>$3.7199 \times 10^5$ mm$^4$</td>
<td>$2.6714 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>5</td>
<td>$r_{xx}$</td>
<td>32.26 mm</td>
<td>32.21 mm</td>
</tr>
<tr>
<td>6</td>
<td>$r_{yy}$</td>
<td>17.60 mm</td>
<td>17.60 mm</td>
</tr>
<tr>
<td><strong>0.75% SFRC</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Area of Steel</td>
<td>1281.73 mm²</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>Area of Concrete</td>
<td>--</td>
<td>8122.61 mm²</td>
</tr>
<tr>
<td>3</td>
<td>$I_{xx}$</td>
<td>$1.311 \times 10^6$ mm$^4$</td>
<td>$8.31 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>4</td>
<td>$I_{yy}$</td>
<td>$3.857 \times 10^5$ mm$^4$</td>
<td>$2.45 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>5</td>
<td>$r_{xx}$</td>
<td>31.98 mm</td>
<td>31.98 mm</td>
</tr>
<tr>
<td>6</td>
<td>$r_{yy}$</td>
<td>17.36 mm</td>
<td>17.36 mm</td>
</tr>
<tr>
<td><strong>1.00% SFRC</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Area of Steel</td>
<td>1402.57 mm²</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>Area of Concrete</td>
<td>--</td>
<td>7566.64 mm²</td>
</tr>
<tr>
<td>3</td>
<td>$I_{xx}$</td>
<td>$1.404 \times 10^6$ mm$^4$</td>
<td>$7.57 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>4</td>
<td>$I_{yy}$</td>
<td>$4.073 \times 10^5$ mm$^4$</td>
<td>$2.197 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>5</td>
<td>$r_{xx}$</td>
<td>31.98 mm</td>
<td>31.64 mm</td>
</tr>
<tr>
<td>6</td>
<td>$r_{yy}$</td>
<td>17.04 mm</td>
<td>17.04 mm</td>
</tr>
<tr>
<td><strong>1.25% SFRC</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Area of Steel</td>
<td>1255.23 mm²</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>Area of Concrete</td>
<td>--</td>
<td>8266.90 mm²</td>
</tr>
<tr>
<td>3</td>
<td>$I_{xx}$</td>
<td>$1.291 \times 10^6$ mm$^4$</td>
<td>$8.50 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>4</td>
<td>$I_{yy}$</td>
<td>$3.81 \times 10^5$ mm$^4$</td>
<td>$2.512 \times 10^6$ mm$^4$</td>
</tr>
<tr>
<td>5</td>
<td>$r_{xx}$</td>
<td>32.07 mm</td>
<td>32.07 mm</td>
</tr>
<tr>
<td>6</td>
<td>$r_{yy}$</td>
<td>17.42 mm</td>
<td>17.42 mm</td>
</tr>
</tbody>
</table>
3.4 TEST SERIES

The details of experiments conducted is presented in Figures 3.8 to 3.12.

**Figure 3.8** Detail of the experiments conducted on stub columns

**Figure 3.9** Detail of the experiments conducted on short column
Figure 3.10 Detail of the experiments conducted on axially and eccentrically loaded medium columns
Figure 3.11 Detail of the experiments conducted to study the effect of flat-width to thickness ratio on hollow and in-filled medium columns
Figure 3.12 Detail of the experiments conducted to study the flexural behaviour of hollow and in-filled beams
3.5 PUSH OUT TESTS

Push out tests were carried out to study the bond strength between the steel sections and the in-fill. The test specimens for this test was 550 mm in length and the concrete was poured to 500 mm length leaving a 50 mm gap in which a groove of 7 mm in the form of an equilateral triangle had been made at the end for relieving the air during testing. The test set-up is shown in Figure 3.13. The load was applied through a steel plate, which rests directly on concrete in small increments and the slip was observed using a deflectometer as shown in the test set-up.

The slip was found to be very minimal indicating excellent bond between the steel tube and the concrete up to the ultimate load. Beyond the ultimate load, there was a rapid increase in the slip, indicating the sudden reduction in bond. Due to increased characteristic compressive strength, the bond strength was high for SFRC in-filled specimens when compared to PCC in-filled specimens. The bond strength for PCC, 0.75% SFRC in-fill, 1% SFRC in-fill and 1.25% SFRC in-fill are found to be 0.282 N/mm², 0.389 N/mm², 0.461 N/mm² and 0.318 N/mm², respectively. Compared to PCC in-filled specimens, the SFRC in-filled specimens take about 63% more load before slip. Tested specimens are shown in the Figure 3.14. According to Eurocode4 (clause 4.8.2.7), the shear resistance provided by bond stresses and friction at the interface or by mechanical shear connection for the concrete in-filled hollow sections are given by 0.40 N/mm² and the 1% SFRC in-filled sections satisfy this condition well. The load - slip characteristic of the in-filled specimens are shown in Figure 3.15.
Figure 3.13 Test set up for push out test

Figure 3.14 Tested specimens of push out test
Figure 3.15 Load-slip characteristics of in-filled specimens
3.6 TESTS ON STUB COLUMNS

Stub columns are specimens whose height is not less than three times the largest dimension of the section and not more than twenty times the least radius of gyration as prescribed in the IS: 801-1975. Tests on stub columns were conducted on specimens to get their ultimate strength. In the present investigation five stub columns (with slenderness ratios of 17.50) were tested.

3.6.1 Test Procedure

Rectangular hollow, PCC in-filled, 0.75%, 1.00% and 1.25% SFRC in-filled stub columns were tested under axial load conditions. The test specimen was placed centrally. The verticality of the stub column was ensured. The test set up is shown in the Figure 3.16. Top and bottom fixture is shown in Figures 3.17 and 3.18. To make hinged end condition steel balls are placed on the grooved plates. Deflectometer is used to measure the axial shortening and strain gauges are fixed at the longer face of the specimen to measure strain.
For each of the five stub columns, the axial load was increased slowly till the ultimate loads are reached. Prior to the actual test, a load level of 10 kN was applied so that the platens of the testing machine were firmly attached to both ends of the specimen. The axial load was then applied at a loading rate of 0.30 mm/min. The axial shortening and longitudinal strain of the specimen were recorded at a load increment of 25 kN.

Figure 3.17 Bottom fixture

Figure 3.18 Top fixture
3.6.2 Failure Mode and Ultimate Loads

The failure mode is characterized by crushing of concrete and outward bulging of the steel tubes at the bottom of the columns. Figure 3.19 shows the tested specimens. It is also observed that the PCC in-filled columns are taking 53% more load than the hollow columns and SFRC in-filled columns are taking 85% more load than the hollow columns. Compared with PCC in-filled columns the SFRC in-filled columns are taking around 20% more load. In general, comparing the capacity of all the columns, the 1% SFRC in-filled columns have enhanced strength properties. The axial load carrying capacities calculated from the expressions given by the various codes of practice were compared with the experimental results for all the 5 specimens and are listed in Table 3.3.

3.6.3 Comparison of Test Results with the Design codes

In this section, the experimental data are compared with the values predicted by the design codes such as Eurocode4 and British code BS 5400. According to Eurocode4, the axial load capacity ($N_u$) of CFT stub columns are...
determined by summing up the strengths of the steel tube and the concrete core as in equation (3.8). The EC4 is applicable to CFT stub columns with concrete cylinder strength and steel yield stress not greater than 50 and 355MPa, respectively.

\[
N_u = A_s f_y + A_c f_c' \quad (3.8)
\]

As per the British code BS5400, the strength of the stub column is determined using the equation (3.9). A coefficient of 0.675 was included in the concrete cube strength to account for long-term and size effects of concrete cube.

\[
N_c = A_s f_y + 0.675 A_c f_{cu} \quad (3.9)
\]

The axial load carrying capacity calculated using the expressions given in codes are compared with the experimental results for all the 5 specimens and are listed in Table 3.3. It shows that the mean and standard deviation of the theoretical load \(P_{\text{the}}\) and experimental load \(P_{\text{exp.}}\) ratio for the different design codes. The results show that BS5400 predicts the column strength conservatively where as the EC4 predicts it slightly on the higher side. The EC4 predicted around 7% more ultimate load than the experimental results for in-filled columns and estimated 7.20% lower values for hollow columns. The design strength predicted by BS5400 is nearer to the experimental values. The code BS5400 gives a mean value of 0.969 and a standard deviation of 0.040 compared to the experimental values and is the best predictor and thus are acceptable for the calculation of axial strength of SFRC in-filled stub columns.

The comparison is also illustrated through Figure 3.20.
Table 3.3 Comparison of experimental and theoretical strengths of stub columns

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Specimen Type</th>
<th>Experimental Load (kN) $P_{\text{exp}}$</th>
<th>Euro code 4</th>
<th>BS5400</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$P_{\text{exp}}$</td>
<td>$P_{\text{the}}$</td>
<td>$P_{\text{the}}$</td>
</tr>
<tr>
<td>1</td>
<td>A</td>
<td>170.00</td>
<td>157.68</td>
<td>157.68</td>
</tr>
<tr>
<td>2</td>
<td>B</td>
<td>261.60</td>
<td>275.37</td>
<td>254.38</td>
</tr>
<tr>
<td>3</td>
<td>C</td>
<td>300.00</td>
<td>312.11</td>
<td>282.22</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>320.00</td>
<td>366.07</td>
<td>329.91</td>
</tr>
<tr>
<td>5</td>
<td>E</td>
<td>280.00</td>
<td>297.40</td>
<td>272.74</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>1.045</td>
<td>0.969</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td></td>
<td></td>
<td>0.077</td>
<td>0.040</td>
</tr>
</tbody>
</table>

Figure 3.20 Comparison of experimental loads and theoretical loads
3.6.4 Load vs. Axial Shortening Behaviour

Load versus axial shortening behaviour of all the five stub columns are shown in Figure 3.21. In the initial stages of loading, the load versus axial shortening plots showed linear variations. After attaining the peak value (the maximum load), the curve dropped slowly for hollow columns and sharply for in-filled columns showing the reduction in the column capacity with small increments in axial shortening. This is clearly indicated for 1% SFRC in-filled column, which exhibited superior performance than the other types.

![Figure 3.21 Load vs. Axial shortening behaviour of hollow and in-filled columns](image)

3.6.5 Load vs. Strain Behaviour

The loads versus microstrain plots for the stub columns are shown in Figure 3.22. All the columns are showing similar load-strain behaviour in the initial stages of loading and varied only beyond the yield point. The strain hardening portion is longer for the SFRC in-filled columns compared to PCC in-filled columns exhibiting the ductile nature of the in-fill. The type D column shows a better ductile region than other columns.
3.7 DISCUSSION OF RESULTS

An experimental program on 5 hollow and in-filled light gauge steel stub columns subjected to axial compression has been described. The strength increase resulting from confinement of high strength concrete by steel section was observed. Comparisons of failure loads, the PCC in-filled column were taking 1.65 times more load than the hollow columns. The SFRC in-filled columns were around 1.80 times and 1.15 times more loads than hollow and PCC in-filled columns respectively. The comparison of failure loads between the test loads and the design codes are presented. It indicates that EC4 is unsafe to predict the ultimate capacity of the in-filled columns. On the other hand BS5400 predicts the failure loads close to the experimental loads. The BS5400 method gives a mean value of 0.969 and a standard deviation of 0.040 compared with experimental value is the best predictor and thus are acceptable for the calculation of axial strength of SFRC in-filled stub columns. Test results also manifested favourable ductility performance for hollow and SFRC in-filled stub column specimens which was ascertained by the load versus strain curves. For the lower load levels i.e. up to 100kN the axial shortening was 10% less for 1% SFRC in-filled columns compared to other columns. This factor shows the impact of SFRC in-fill in the stub columns.
3.8 TESTS ON SHORT COLUMNS

These tests are conducted to study the behaviour of hinged short columns under axial load, which include rectangular hollow, PCC in-filled, 0.75%, 1.00% & 1.25% SFRC in-filled short columns whose slenderness ratio is 35. The test set up is shown in Figure 3.23.

![Test set up](image)

**Figure 3.23 Test set up**

3.8.1 Test Procedure

The tests were conducted on a column load frame. In order to apply truly axial load to the specimens, 22 mm thick plates were directly welded to the ends of the specimen. To simulate simply supported end condition two plates 30 mm thick and size 300 x 200 mm with a spherical groove at the center to accommodate a ball of 40 mm diameter was bolted to the end plates at either end by four 16 mm diameter bolts such that their centres coincide in plan. The load was applied through a calibrated proving ring of 100 kN capacity. 10 mm electrical strain gauges were used to measure the strains at
mid height of the columns. The axial shortenings and the strain gauge readings were taken for each increment of load up to the failure of the specimens. The specimens were loaded with a load increment of 10 kN in the elastic range and with 5 kN load increment after columns have began to yield. The ultimate loads were identified by rapid increase in the deflectometer and strain gauge readings and the subsequent drop in the load.

### 3.8.2 Failure Modes and Ultimate Loads

It was observed that all the in-filled columns were crushed under the axial load and no buckling failures or shear failures were observed. The typical failure appearances of the test specimens are shown in Figure 3.24. For the hollow columns alone, the failure was due to local buckling near the bottom support whereas, all the in-filled columns failed at their bottom due to crushing of in-fills at the verge of failure.

![Failure modes of short columns](image)

i) Crushing of in-fills and outward bulging of steel tube

ii) Local buckling of steel tube (Hollow)

**Figure 3.24** Failure modes of short columns
It is observed that, the strength of PCC in-filled columns is 112% more than the strength of the hollow columns. The SFRC in-filled columns with 1% volume fraction of the fibers had taken 27% more load than the corresponding PCC in-filled columns and 169% more load than the corresponding hollow columns. Even though the percentage of fibers is increased from 1.00% to 1.25%, there is no significant increase in the ultimate load.

### 3.9 THEORETICAL EXPRESSIONS

The loads were calculated from the provisions given in Eurocode 4 and BS 5400 and are listed in Table 3.4.

#### 3.9.1 Eurocode4: 1994

The nominal strengths of the in-filled columns \((N_{SD})\) were calculated from the expressions

\[
N_{PL, RD} = (A_a f_y / \gamma_{ma}) + (A_c f_{ck} / \gamma_c) \quad \text{where} \quad (\gamma_{ma} = 1.10 \text{ and } \gamma_c = 1) \quad (3.10)
\]

\[
N_{PL, R} = (A_a f_y / \gamma_{ma}) + (A_c f_{ck} / \gamma_c) \quad (\gamma_{ma} = \gamma_c = 1) \quad (3.11)
\]

\[
N_{cr} = \left[\pi^2 EI / (L^2)\right] \quad (3.12)
\]

where

- \(L\) is the buckling length of the column
- \(EI = (E_a I_a + 0.8E_{cd} I_c)\) \quad (3.13)
- \(E_a\) is Young’s Modulus of the steel forming the section and \(I_a\) is area of steel section
- \(E_{cd} = (E_{cm} / \gamma_c)\) \quad (3.14)
- \(E_{cm}\) is the Young’s modulus of concrete
\[ \bar{\lambda} = \left( \frac{N_{PL.R}}{N_{cr}} \right)^{0.5} \]  

(3.15)

for this value of \( \bar{\lambda} \), the reduction factor \( X \) is selected from the EC3.

The Strength of the in-filled column \( N_{SD} = (X) \times (N_{PL.RD}) \)  

(3.16)

### 3.9.2 BS 5400: Part 5: 1979

The load carrying capacity of concrete in-filled columns is calculated as per British code BS5400 from the equation

\[ N_U = 0.91A_sf_y + 0.45 A_c f_{cu} \]  

(3.17)

The loads calculated from the above expressions given in Eurocode4 and BS5400 are listed in Table 3.4. Comparison of the theoretical values, the BS5400 is estimating the strength of the in-filled columns

### Table 3.4 Comparison of experimental and theoretical strengths of short columns

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Specimen Type</th>
<th>Experimental Load (kN) ( P_{exp.} )</th>
<th>Eurocode 4 Theoretical Load(kN) ( P_{the} )</th>
<th>( \frac{P_{the.}}{P_{exp.}} )</th>
<th>BS5400 Theoretical Load(kN) ( P_{the} )</th>
<th>( \frac{P_{the.}}{P_{exp.}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>115.00</td>
<td>109.21</td>
<td>0.950</td>
<td>100.00</td>
<td>0.870</td>
</tr>
<tr>
<td>2</td>
<td>B</td>
<td>244.22</td>
<td>240.00</td>
<td>0.983</td>
<td>203.98</td>
<td>0.835</td>
</tr>
<tr>
<td>3</td>
<td>C</td>
<td>262.60</td>
<td>256.69</td>
<td>0.977</td>
<td>222.61</td>
<td>0.848</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>309.89</td>
<td>301.26</td>
<td>0.972</td>
<td>254.53</td>
<td>0.821</td>
</tr>
<tr>
<td>5</td>
<td>E</td>
<td>253.41</td>
<td>248.22</td>
<td>0.980</td>
<td>216.41</td>
<td>0.854</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td></td>
<td>0.972</td>
<td></td>
<td>0.846</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td></td>
<td></td>
<td></td>
<td>0.006</td>
<td></td>
<td>0.019</td>
</tr>
</tbody>
</table>
conservatively. The design strengths predicted by EC4 are nearer to the experimental values. The EC4 method gives a mean value of 0.972 and a standard deviation of 0.006 with reference to the experimental values and hence can be taken as the best predictor and thus acceptable for the calculation of axial strength of SFRC in-filled short columns. The experimental and the theoretical loads are given in the bar chart shown in Figure 3.25.

![Bar chart showing experimental and theoretical loads](image)

**Figure 3.25 Experimental and theoretical loads**

The behaviour of the short concrete composite columns is usually discussed in concurrence with the following terms.

### 3.9.3 Concrete Contribution Factor \((\alpha)\)

According to clause 11.1.4 in BS5400, the concrete contribution factor is defined as the ratio of the contribution of strength by concrete to the strength of the composite column.

\[
\text{Concrete Contribution factor (}\alpha\text{)} = 0.45 \frac{A_c f_{cu}}{N_u} \tag{3.18}
\]

\(N_u\) = Squash load
3.9.4 Constraining Factor ($\xi$)

The constraining factor is defined as the ratio of the strength of steel to the strength of concrete as given by Han et al.

\[
\text{Constraining factor } (\xi) = \frac{A_s f_y}{A_c f_{ck}}.
\]  

(3.19)

where $f_{ck}$ is the 67\% of the compression strength of concrete cubical blocks.

Table 3.5 shows the values of Concrete contribution factor ($\alpha$) and Constraining factor ($\xi$) for all the types of columns.

**Table 3.5 Values of concrete contribution factor and constraining factor for all the types of columns**

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Specimen Type</th>
<th>Experimental Load ($P_{exp}$) (kN)</th>
<th>Theoretical Load ($P_{the}$) (Eurocode4) (kN)</th>
<th>Concrete contribution Factor ($\alpha$)</th>
<th>Constraining factor ($\xi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>115.00</td>
<td>109.21</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>B</td>
<td>244.22</td>
<td>240.00</td>
<td>0.246</td>
<td>1.643</td>
</tr>
<tr>
<td>3</td>
<td>C</td>
<td>262.60</td>
<td>256.69</td>
<td>0.278</td>
<td>1.276</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>309.89</td>
<td>301.26</td>
<td>0.359</td>
<td>0.922</td>
</tr>
<tr>
<td>5</td>
<td>E</td>
<td>253.41</td>
<td>248.22</td>
<td>0.274</td>
<td>1.380</td>
</tr>
</tbody>
</table>

For higher compressive strength of confined concrete, the concrete contribution factor ($\alpha$) is higher and the constraining factor ($\xi$) is lower. For 1\% SFRC in-filled columns the value of concrete contribution factor is higher and constraining factor is lesser exhibiting the enhanced structural strength.
3.10 LOAD VS. AXIAL SHORTENING BEHAVIOUR

The axial shortening behaviour of all types of columns is illustrated in Figure 3.26. Hollow columns show larger axial shortening beyond the ultimate load, compared to the other types of columns. The axial shortening of SFRC in-filled columns are lesser due to the enhanced characteristic compressive strength of the in-filled concrete compared to PCC in-filled and hollow columns. Beyond the ultimate loads, all the columns show considerable axial shortening before failure. However, there is a notable difference in the behaviour of the hollow column and 1% SFRC in-filled column from the rest of the columns. For the 1.25% SFRC in-filled concrete columns, the descending curve is found to be critical showing the reduction in load was large compared to axial shortening.

![Figure 3.26 Load vs. axial shortening behaviour of hollow and in-filled column](image-url)

Figure 3.26 Load vs. axial shortening behaviour of hollow and in-filled column
3.11 LOAD vs. MICRO STRAIN BEHAVIOUR

Figure 3.27 shows the load micro strain behaviour of all the columns. All the columns exhibit considerable amount of plastic strains at the lower load levels. However, SFRC in-filled columns show lesser strains up to the elastic state and undergo considerable strain beyond the ultimate load compared to the plain concrete in-filled columns and hollow columns. The plots clearly indicate that, the ductility performance of SFRC in-filled columns is higher for 0.75%, 1% and 1.25% SFRC in-filled columns by 26%, 39% and 24% respectively compared to PCC in-filled column.

![Load vs. Micro strain behaviour of hollow and in-filled column](image)

3.12 DISCUSSION OF RESULTS

In this experimental program hollow and in-filled light gauge steel short columns subjected to axial compression has been described. There is a considerable increase in the strength of the in-filled columns, due to the confining effect of PCC and SFRC by the steel section compared to the
hollow columns. The PCC in-filled column has taken 2.20 times more load than the corresponding hollow columns. The SFRC in-filled columns have taken around 2.25 times and 1.15 times more loads than hollow and PCC in-filled columns respectively. Hollow columns showed large axial shortening beyond the ultimate load. The axial shortening of SFRC in-filled columns are lesser due to increased characteristic compressive strength of the in-fill compared to plain concrete in-filled and hollow columns. Beyond the ultimate loads, the axial shortenings are significant. All the columns exhibit larger amount of plastic strains at the lower loads. The SFRC in-filled columns show lesser strains up to the elastic state and considerable strains beyond the ultimate loads when compared with the PCC in filled columns and hollow columns. A comparison of the failure loads obtained from the experiments and from the design codes indicates that BS5400 is not safe to predict the ultimate loads of the in-filled columns. The EC4 predicts the ultimate loads closest to the experimental loads, whose mean value is 0.972 and the standard deviation is 0.006 compared to the experimental values. Hence EC4 is taken as the best predictor and is acceptable for calculation of the axial strength of SFRC in-filled short columns. Compared to stub columns, the short columns take lesser axial loads, due to their higher slenderness ratio. The column in-filled with 1% SFRC showing good performance than all other type of columns.