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# Load Carrying Capacity of CFST Column Element With & Without Shear Studs Under Axial Compression

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*Abstract* - **Concrete Filled Steel Structures (CFST) offers wide benefits like high strength, ductility, energy absorption with the combined benefits of steels and concrete. Normally the concrete columns subjected to major loads will buckle and fail; to avoid this buckling phenomenon the use of concrete filled steel tube columns are required. Because they do not require formwork. When we provide the shear connectors in the concrete filled steel tube column the lateral displacement of CFST columns is ignored and the bond strength between the concrete and steel is increased. So the investigation is to be carried out by using L/D ratios and by different shear connectors, and finding out the load carrying capacity of that column. This Paper present a review the behavior of CFST comparing the models with & without shear studs differentiating position of shear studs. The composite actions of steel and concrete to occur there need a strong bond between steel and concrete interface. Analysis of CFST column by means of the Finite element method (ABAQUS) software and the experimental study is done on the selected case under concentric loading condition.**

*keywords* - **CFST column, Finite Element (ABAQUS), Shear Studs.**

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## I. INTRODUCTION

Concrete-filled steel tube (CFST) members are composite structures which evolved on the basis of the hollow steel tube (HST). For the hollow steel tube, local buckling of the flange may occur when the width-to-thickness ratio (B/t) be larger than a certain value. Thus, the plastic bending moment of an HST beam may not be achieved or maintained. Filling concrete is an efficient way to prevent local buckling and to enhance performance of HST beams. Although the filling concrete would increase the dead weight to a certain level, it is still considered an efficient way to enhance strength, stiffness and ductility of HST members. It proved that the increase in rotation angles of CFST members at ultimate moment can be three times larger than that of HST beams. CFST members can provide an excellent seismic resistance in two orthogonal directions as well as show good damping characteristics. They also show an excellent hysteresis behavior under cyclic loading when compared with HST tubes. CFST members have been used in tall structures and in retrofitting damage bridge piers. The use of CFST members in moment resisting frames eliminates the need for additional stiffness elements in panel zones and zones of high strain demand. Bridges with CFST members are expected to reduce noise and vibration levels when compared to ones with pure steel members. Moreover, CFST members have been proven to be cost effective in building structures.

The example of Aurora pedestrian arch bridge does demonstrate that the CFST is an appealing modular system and is easy to fabricate and erect. Many research efforts on the compressive behavior of CFST have been carried out in the past decades, however, the flexural performance of CFST is still very limited. It found that the flexural capacity of CFSTs was increased by 49% compared to the bare steel tube beam. Bridge also observed that the core concrete can provide approximately 7.5% more bending capacity than the hollow steel section. After these earlier studies, investigations were mainly focused on the depth-to-width ratio, shear span-to-depth ratio, and width-to-thickness ratio. Similarly, a favorable post-yield behavior of CFST member was reported. The distinguishing feature of ductile collapse and smooth loading process was further studied based on the unified theory. Finite element method, simplified analytical method and cross-sectional fiber analysis were proposed to predict the stiffness and bending strength. Found that the static strength of CFST member was significantly influenced by cyclic loading. Although previous studies have proved that the CFSTs had outstanding strength capacity, ductility, and seismic performance, the noted that the composite bending stiffness of CFST was similar to the theoretical stiffness of the bare steel tube due to the concrete cracking, also found that the concrete cracking in tension zone in the early loading stage would significantly decrease the ultimate capacity to a value extremely close to the stiffness of bare steel section.

## II. LITERATURE REVIEW

- ***Experimental and analytical investigation***

An experimental and numerical research on the seismic performance of semi-rigid concrete-filled steel tubular (CFST) frames with external sandwich composite wall panels (SCWPs) was reported. Four specimens of partially rigid CFST frames with external SCWPs and one specimen of pure semi-rigid CFST frame subjected to low-cyclic loading were conducted. Failure modes, horizontal load versus displacement relation curves were analyzed. The test specimens exhibited good hysteretic behavior, energy dissipation and ductility. Finite element (FE) study modeling was developed and the results obtained from the FE model matched well with the experimental results. Extensive parametric studies have been carried out to investigate the effect of steel strength, column slenderness ratio and steel wire diameter of wall, etc. on the strength and stiffness of the typed composite frames. The opening ratio and location of the SCWPs were also discussed. The experimental study and mathematical analysis will provide the scientific basis for design theory and application of the SCWPs in fabricated steel structure building.

- ***Compressive behavior of circular concrete filled steel tubes***

This paper presents an experimental study to investigate the compressive behavior of circular concrete filled steel tubes (CFSTs) when subjected to pure axial loading at a low rate of 0.6 kN/s. CFSTs of three different diameter-to-thickness (D/t) ratios of 54, 32, and 20 are measured in this study filled with two concrete's compressive strengths of 44 MPa and 60 MPa. The measured compressive axial capacities are compared to their corresponding theoretical values predicted by four different international codes and standards: the American Institute of Steel Construction (AISC), the American Concrete Institute (ACI 318), the Australian Standard (AS), and Eurocode 4. Result comparisons also included some recommended equations found in the literature. It was found that the effect of (D/t) ratio on the compressive behavior of the CFST specimens is larger than the effect of the other factors. The underestimation of the axial capacities calculated by most of these codes reduces as the D/t ratio increases as verified by the experimental results. A nonlinear finite element (FE) numerical model using the commercial software package ABAQUS is also developed and verified using the presented experimental results.

- ***Effect of carbon fibre reinforced polymer***

In this study author has described, numerical simulations are carried out to evaluate the effect of carbon fiber reinforced polymer (CFRP) strengthening of full scale concrete-filled steel tubular (CFST) columns under vehicular impact. In recent years, the risk of damage or failure of axial load bearing structural members has increased rapidly due to increase of accidental vehicle/ship collision events. Therefore, suitable strengthening technique requirements to be developed to minimize the casualty and economic loss caused by vehicular collisions with structural columns. In this study, numerical simulations are carried out to evaluate the effect of carbon fiber reinforced polymer (CFRP) strengthening of full scale concrete-filled steel tubular (CFST) columns under vehicular impact. Numerical models of bare and CFRP strengthened CFST columns were first developed and validated in a recent study of the authors. The validated finite element (FE) models are extended to full scale columns. Realistic vehicle behaviour is simulated with simplified mass spring vehicle model. The outer diameter of steel section is kept same and the wall thicknesses are changed to account the slenderness effects of hollow steel sections. Both vehicle and column deformations are considered during the impact simulation as observed in practical situation. The dynamic impact study results show that adhesively bonded CFRP sheets provide enhanced impact resistance capacity of strengthened columns by reducing lateral displacement about 40% compared to ordinary CFST columns. A comprehensive parametric study is conducted by varying the vehicle velocity, vehicle mass, axial static loading, vehicle stiffness and CFRP bond length to examine the effects of these parameters on the structural responses of bare and wrapped columns. CFRP wrapping is found to be a promising strengthening technique to control global failure of full scale CFST columns subjected to vehicular impact.

- ***Flexural behavior of circular concrete filled steel tubes***

This paper studies the flexural behavior of circular concrete filled steel tubes (CFST) under sustained load and chloride corrosion. 7CFST specimens were tested under a four-point bending load. It was found that corrosion causes noticeable deterioration to the flexural strength, while the ductility of CFST keeps well. A finite element analysis (FEA) model was developed to study the full-range behavior of CFST under corrosion. A parametric study was conducted to find the main parameters that influence the residual flexural strength, based on which a simplified model was proposed to calculate the residual flexural strength of circular CFST under long-term load and corrosion.

## III. MATERIAL AND M ETHODOLOGY

- ***Design steps using Erocode 4 (EC4)***

The design of composite columns and composite compression members with concrete fully and partially encased H-sections, and concrete filled rectangular and circular hollow sections. It is applicable to columns and compression members with steel all grades and normal weight concrete of strength classes.

In general, a composite column should be checked at the ultimate limit state for:

- Geometric limits of various elements of the steel sections against local buckling under compression.
- Resistances of cross-sections and members to internal forces and moments.
- Buckling resistance of the members, depending on their effective slenderness.
- Local resistances to interfacial shear forces between the steel sections and the concrete.
- Local resistances of the cross-sections at load introduction points.

- ***Design methods***

The code gives two methods for isolated composite columns in braced or non-sway Frames:

- General design method for composite columns applicable to both prismatic and non-prismatic members with either symmetrical or non-symmetrical cross-sections.
- Simplified design method specifically developed for prismatic composite columns with doubly symmetrical cross-sections.

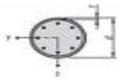
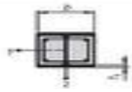
The use of the simplified design method is presented in detail. It should be noted that when the limits of applicability of this method are not satisfied, the general design method should be used.

- ***Local buckling***

The effects of local buckling may be neglected for a steel section fully encased and for other types of cross-section provided the maximum values are not exceeded:

Table 1. Maximum Values (D/T), (H/T) and (B/T) With Fy N/Mm$^2$

| Cross Section | Max (d/t), max(h/t) & max(b/t) |
|---|---|

| | | |
|---|---|---|
| Circular hollow steel section |  | $Max\ (d/t)\ =\ 90\frac{235}{f_y}$ |
| Rectangular hollow steel sections |  | $Max\ (h/t)\ =\ 52\sqrt{\frac{235}{f_y}}$ |
| Partially encased I sections |  | $Max\ (b/t_f)\ =\ 44\sqrt{\frac{235}{f_y}}$ |

## IV. RESULTS AND ANALYSIS

➢ ***Analytical Load Carrying Capacity***

Table 2. Load carrying capacity of circular section (KN)

| Sr. No | Outer Dia(D) in mm | Thickness (T) | Inner Dia(D) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 4 | 72 | 600 | without stud | 291.20 |
| 2 | 80 | 4 | 72 | 600 | 1 stud | 426.81 |
| 3 | 80 | 4 | 72 | 600 | 2 stud | 527.35 |
| 4 | 80 | 4 | 72 | 600 | 3 stud | 784.05 |
| 5 | 80 | 4 | 72 | 600 | 4 stud | 927.20 |

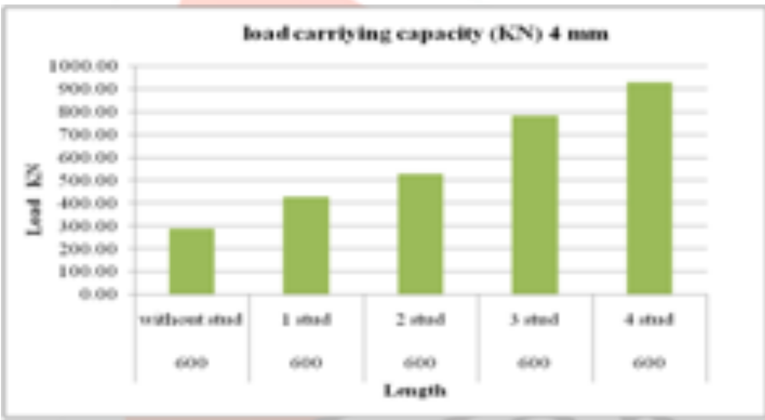



Figure 1. Load carrying capacity of circular section (KN)

Table 3. Load carrying capacity of circular section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (T) | Inner Dia(D) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 3 | 74 | 600 | without stud | 290.85 |
| 2 | 80 | 3 | 74 | 600 | 1 stud | 389.35 |
| 3 | 80 | 3 | 74 | 600 | 2 stud | 477.50 |
| 4 | 80 | 3 | 74 | 600 | 3 stud | 620.75 |
| 5 | 80 | 3 | 74 | 600 | 4 stud | 751.57 |

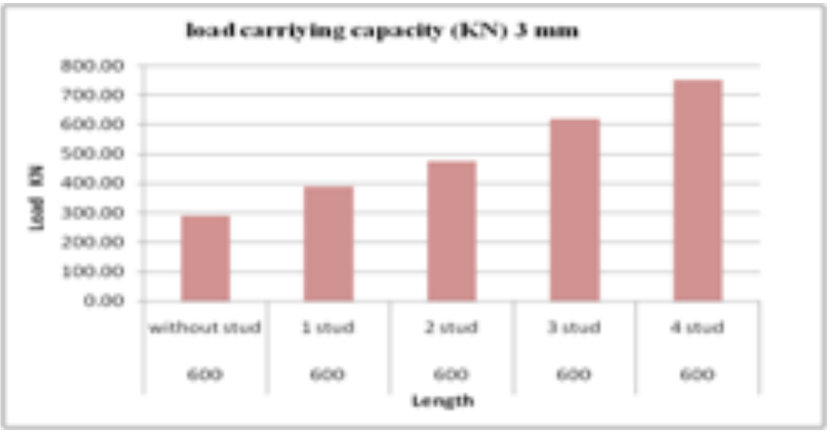



Figure 2. Load carrying capacity of circular section (KN)

Table 4. Load carrying capacity of circular section (KN)

| Sr. No | Outer Dia(D) in mm | Thickness (t) | Inner Dia(d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 2 | 76 | 600 | without stud | 280.27 |
| 2 | 80 | 2 | 76 | 600 | 1 stud | 495.95 |
| 3 | 80 | 2 | 76 | 600 | 2 stud | 526.06 |
| 4 | 80 | 2 | 76 | 600 | 3 stud | 647.30 |
| 5 | 80 | 2 | 76 | 600 | 4 stud | 798.00 |

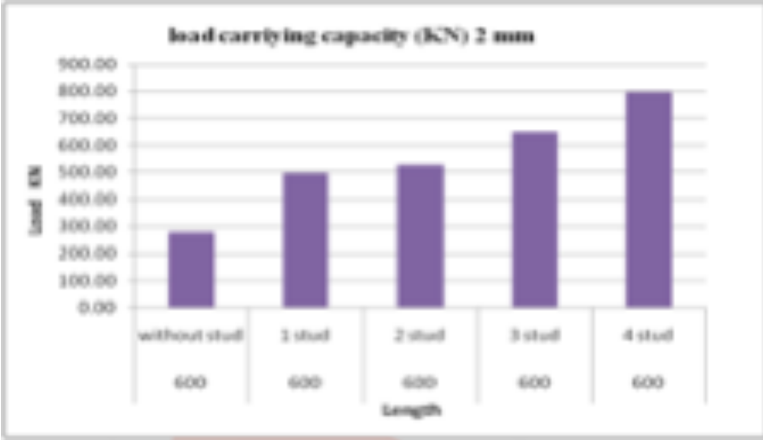



Figure 3. Load carrying capacity of circular section (KN)

Table 5. Load carrying capacity of Circular section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 4 | 72 | 650 | without stud | 280.56 |
| 2 | 80 | 4 | 72 | 650 | 1 stud | 472.66 |
| 3 | 80 | 4 | 72 | 650 | 2 stud | 540.26 |
| 4 | 80 | 4 | 72 | 650 | 3 stud | 640.24 |
| 5 | 80 | 4 | 72 | 650 | 4 stud | 745.18 |

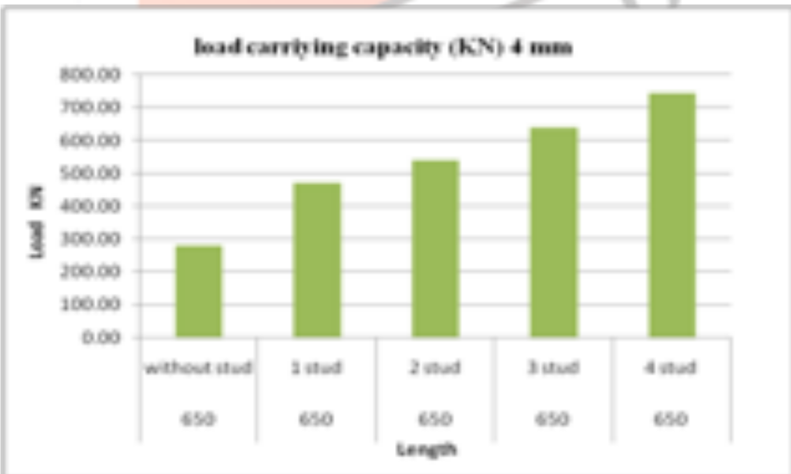



Figure 4. Load Carrying Capacity of Circular Section (KN)

Table 6. Load carrying capacity of Circular section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 3 | 74 | 650 | without stud | 320.70 |
| 2 | 80 | 3 | 74 | 650 | 1 stud | 356.65 |
| 3 | 80 | 3 | 74 | 650 | 2 stud | 398.93 |
| 4 | 80 | 3 | 74 | 650 | 3 stud | 498.62 |
| 5 | 80 | 3 | 74 | 650 | 4 stud | 687.89 |

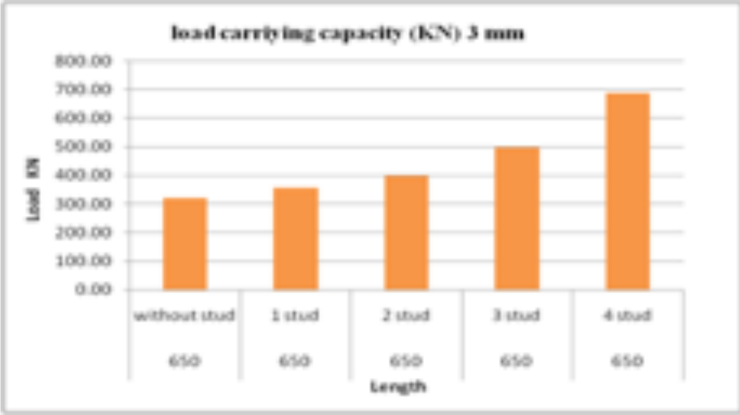



Figure 5. Load Carrying Capacity of Circular Section (KN)

Table 7. Load Carrying Capacity of Circular Section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 2 | 76 | 650 | without stud | 387.03 |
| 2 | 80 | 2 | 76 | 650 | 1 stud | 528.48 |
| 3 | 80 | 2 | 76 | 650 | 2 stud | 428.96 |
| 4 | 80 | 2 | 76 | 650 | 3 stud | 692.50 |
| 5 | 80 | 2 | 76 | 650 | 4 stud | 705.32 |

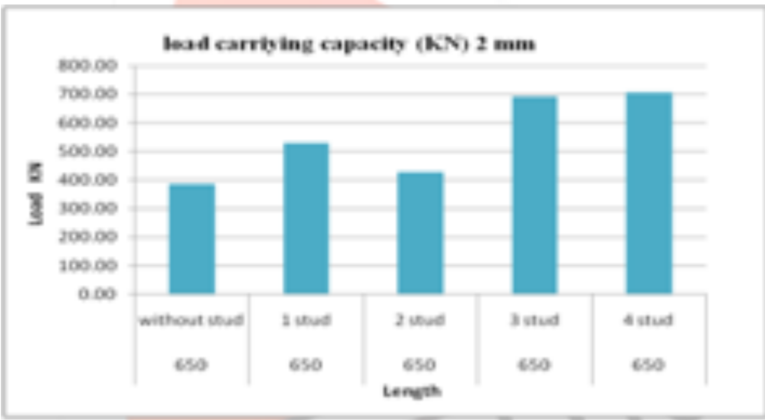



Figure 6. Load Carrying Capacity of Circular Section (KN)

Table 8. Load Carrying Capacity of Circular Section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 4 | 72 | 500 | without stud | 305.56 |
| 2 | 80 | 4 | 72 | 500 | 1 stud | 505.63 |
| 3 | 80 | 4 | 72 | 500 | 2 stud | 648.89 |
| 4 | 80 | 4 | 72 | 500 | 3 stud | 450.00 |
| 5 | 80 | 4 | 72 | 500 | 4 stud | 558.36 |

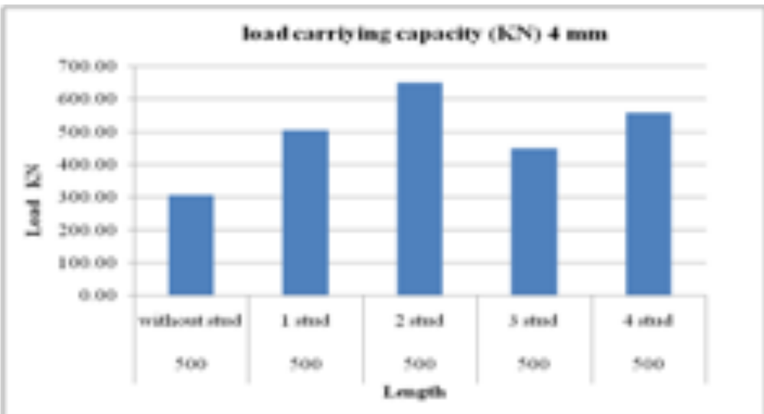



Figure 7. Load Carrying Capacity of Circular Section (KN)

Table 9. Load Carrying Capacity of Circular Section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 3 | 74 | 500 | without stud | 235.84 |
| 2 | 80 | 3 | 74 | 500 | 1 stud | 358.21 |
| 3 | 80 | 3 | 74 | 500 | 2 stud | 415.23 |
| 4 | 80 | 3 | 74 | 500 | 3 stud | 548.96 |
| 5 | 80 | 3 | 74 | 500 | 4 stud | 745.65 |

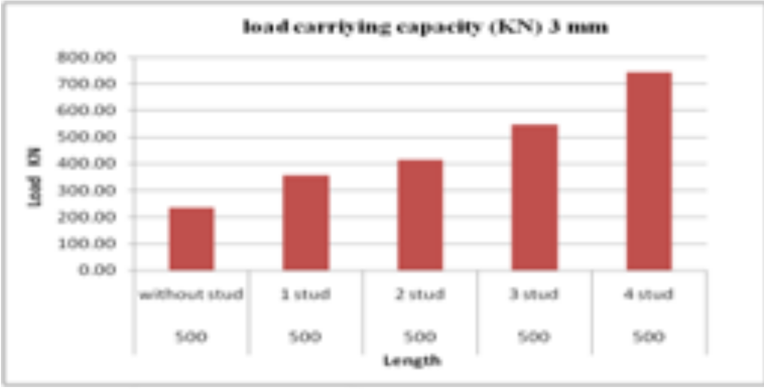



Figure 8. Load Carrying Capacity of Circular Section (KN)

Table 10. Load Carrying Capacity of Circular Section (KN)

| Sr. No | Outer Dia (D) in mm | Thickness (t) | Inner Dia (d) in mm | Length | Studs | Load Carrying Capacity (KN) |
|---|---|---|---|---|---|---|
| 1 | 80 | 2 | 76 | 500 | without stud | 264.51 |
| 2 | 80 | 2 | 76 | 500 | 1 stud | 415.45 |
| 3 | 80 | 2 | 76 | 500 | 2 stud | 520.41 |
| 4 | 80 | 2 | 76 | 500 | 3 stud | 715.00 |
| 5 | 80 | 2 | 76 | 500 | 4 stud | 755.72 |

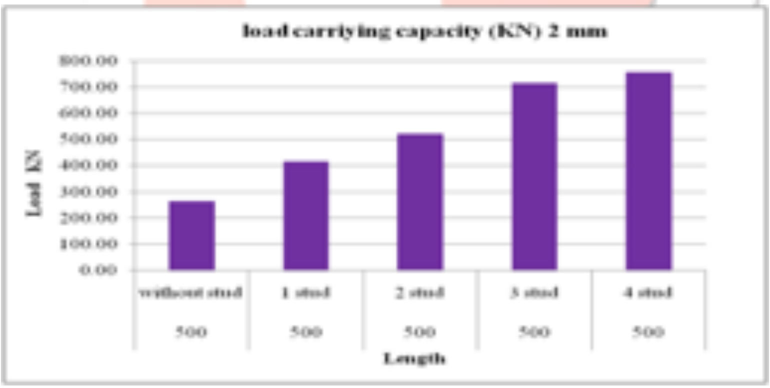



Figure 9. Load Carrying Capacity of Circular Section (KN)

- ***Comparison***

Table 11. Comparison Load Carrying Capacity of without Studs

| Sr. No | Outer Dia (D) in mm | Length | Thickness (t) | Inner Dia (d) in mm | Studs | load Carrying Capacity (KN) | % Difference w.r.t Without Studs |
|---|---|---|---|---|---|---|---|
| 1 | 80 | 600 | 4 | 72 | without stud | 291.20 | |
| 2 | | | 3 | 74 | without stud | 290.85 | 0.12 |
| 3 | | | 2 | 76 | without stud | 280.27 | 3.77 |
| 4 | 80 | 650 | 4 | 72 | without stud | 280.56 | 0.10 |
| 5 | | | 3 | 74 | without stud | 320.70 | 14.31 |
| 6 | | | 2 | 76 | without stud | 387.03 | 20.68 |
| 7 | 80 | 500 | 4 | 72 | without stud | 305.56 | 26.66 |
| 8 | | | 3 | 74 | without stud | 235.84 | 29.56 |
| 9 | | | 2 | 76 | without stud | 264.51 | 12.16 |


Figure 10. Comparison Load Carrying Capacity of without Studs

Table 12. Comparison Load Carrying Capacity of with Similar Length and Studs

| Sr. No | Outer Dia (D) in mm | Inner Dia (d) in mm | Length | Thickness (t) | Studs | Load Carrying Capacity (KN) | % Difference w.r.t Without Studs |
|---|---|---|---|---|---|---|---|
| 1 | 80 | 72 | 600 | 4 | without stud | 291.20 | |
| 2 | | | | | 1 stud | 426.81 | 46.57 |
| 3 | | | | | 2 stud | 527.35 | 81.10 |
| 4 | | | | | 3 stud | 784.05 | 169.25 |
| 5 | | | | | 4 stud | 927.20 | 218.41 |
| 6 | 80 | 74 | 600 | 3 | without stud | 290.85 | |
| 7 | | | | | 1 stud | 389.35 | 33.87 |
| 8 | | | | | 2 stud | 477.50 | 64.17 |
| 9 | | | | | 3 stud | 620.75 | 113.43 |
| 10 | | | | | 4 stud | 751.57 | 158.40 |
| 11 | 80 | 76 | 600 | 2 | without stud | 280.27 | |
| 12 | | | | | 1 stud | 495.95 | 76.95 |
| 13 | | | | | 2 stud | 526.06 | 87.70 |
| 14 | | | | | 3 stud | 647.30 | 130.96 |
| 15 | | | | | 4 stud | 798.00 | 184.73 |
| 16 | 80 | 72 | 650 | 4 | without stud | 280.56 | |
| 17 | | | | | 1 stud | 472.66 | 68.47 |
| 18 | | | | | 2 stud | 540.26 | 92.56 |
| 19 | | | | | 3 stud | 640.24 | 128.20 |
| 20 | | | | | 4 stud | 745.18 | 165.60 |
| 21 | 80 | 74 | 650 | 3 | without stud | 320.70 | |
| 22 | | | | | 1 stud | 356.65 | 11.21 |
| 23 | | | | | 2 stud | 398.93 | 24.39 |
| 24 | | | | | 3 stud | 498.62 | 55.48 |
| 25 | | | | | 4 stud | 687.89 | 114.50 |
| 26 | 80 | 76 | 650 | 2 | without stud | 387.03 | |
| 27 | | | | | 1 stud | 528.48 | 36.55 |
| 28 | | | | | 2 stud | 428.96 | 10.84 |
| 29 | | | | | 3 stud | 692.50 | 78.93 |
| 30 | | | | | 4 stud | 705.32 | 82.24 |
| 31 | 80 | 72 | 500 | 4 | without stud | 305.56 | |
| 32 | | | | | 1 stud | 505.63 | 65.48 |
| 33 | | | | | 2 stud | 648.89 | 112.36 |
| 34 | | | | | 3 stud | 450.00 | 47.27 |
| 35 | | | | | 4 stud | 558.36 | 82.73 |
| 36 | 80 | 74 | 500 | 3 | without stud | 235.84 | |
| 37 | | | | | 1 stud | 358.21 | 51.89 |
| 38 | | | | | 2 stud | 415.23 | 76.06 |
| 39 | | | | | 3 stud | 548.96 | 132.77 |
| 40 | | | | | 4 stud | 745.65 | 144.03 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 41 | | | | | without stud | 264.51 | |
| 42 | | | | | 1 stud | 415.45 | 57.06 |
| 43 | 80 | 76 | 500 | 2 | 2 stud | 520.41 | 96.75 |
| 44 | | | | | 3 stud | 715.00 | 170.31 |
| 45 | | | | | 4 stud | 755.72 | 185.71 |

Figure 11. Comparison Load Carrying Capacity of with Similar Length and Studs

Table 13. Comparison Load Carrying Capacity of with Similar Studs

| Sr. No | Outer Dia (D) in mm | Inner Dia (d) in mm | Thickness (t) | Length | Studs | Load Carrying Capacity (KN) | % Difference w.r.t Studs |
|---|---|---|---|---|---|---|---|
| 1 | | | | 600 | without stud | 291.20 | |
| 2 | 80 | 72 | 4 | 650 | without stud | 280.56 | 3.79 |
| 3 | | | | 500 | without stud | 305.56 | 8.91 |
| 4 | | | | 600 | 1 stud | 426.81 | |
| 5 | 80 | 72 | 4 | 650 | 1 stud | 472.66 | 10.74 |
| 6 | | | | 500 | 1 stud | 505.63 | 6.98 |
| 7 | | | | 600 | 2 stud | 527.35 | |
| 8 | 80 | 72 | 4 | 650 | 2 stud | 540.26 | 2.45 |
| 9 | | | | 500 | 2 stud | 648.89 | 20.11 |
| 10 | | | | 600 | 3 stud | 784.05 | |
| 11 | 80 | 72 | 4 | 650 | 3 stud | 640.24 | 22.46 |
| 12 | | | | 500 | 3 stud | 450 | 42.28 |
| 13 | | | | 600 | 4 stud | 927.20 | |
| 14 | 80 | 72 | 4 | 650 | 4 stud | 745.18 | 24.43 |
| 15 | | | | 500 | 4 stud | 558.36 | 33.46 |
| 16 | | | | 600 | without stud | 290.85 | |
| 17 | 80 | 74 | 3 | 650 | without stud | 320.7 | 10.26 |
| 18 | | | | 500 | without stud | 235.84 | 35.98 |
| 19 | | | | 600 | 1 stud | 389.35 | |
| 20 | 80 | 74 | 3 | 650 | 1 stud | 356.65 | 9.17 |
| 21 | | | | 500 | 1 stud | 358.21 | 0.44 |
| 22 | | | | 600 | 2 stud | 477.5 | |
| 23 | 80 | 74 | 3 | 650 | 2 stud | 398.93 | 19.70 |
| 24 | | | | 500 | 2 stud | 415.23 | 4.09 |
| 25 | | | | 600 | 3 stud | 620.75 | |
| 26 | 80 | 74 | 3 | 650 | 3 stud | 498.62 | 24.49 |
| 27 | | | | 500 | 3 stud | 548.96 | 10.10 |
| 28 | | | | 600 | 4 stud | 751.57 | |
| 29 | 80 | 74 | 3 | 650 | 4 stud | 687.89 | 9.26 |
| 30 | | | | 500 | 4 stud | 745.648 | 8.40 |
| 31 | | | | 600 | without stud | 280.27 | |
| 32 | 80 | 76 | 2 | 650 | without stud | 387.025 | 38.09 |
| 33 | | | | 500 | without stud | 264.51 | 46.32 |
| 34 | 80 | 76 | 2 | 600 | 1 stud | 495.95 | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 35 | | | | 650 | 1 stud | 528.48 | 6.56 |
| 36 | | | | 500 | 1 stud | 415.45 | 27.21 |
| 37 | 80 | 76 | 2 | 600 | 2 stud | 526.056 | |
| 38 | | | | 650 | 2 stud | 428.96 | 22.64 |
| 39 | | | | 500 | 2 stud | 520.412 | 21.32 |
| 40 | 80 | 76 | 2 | 600 | 3 stud | 647.3 | |
| 41 | | | | 650 | 3 stud | 692.5 | 6.98 |
| 42 | | | | 500 | 3 stud | 715 | 3.25 |
| 43 | 80 | 76 | 2 | 600 | 4 stud | 798 | |
| 44 | | | | 650 | 4 stud | 705.32 | 13.14 |
| 45 | | | | 500 | 4 stud | 755.72 | 7.15 |

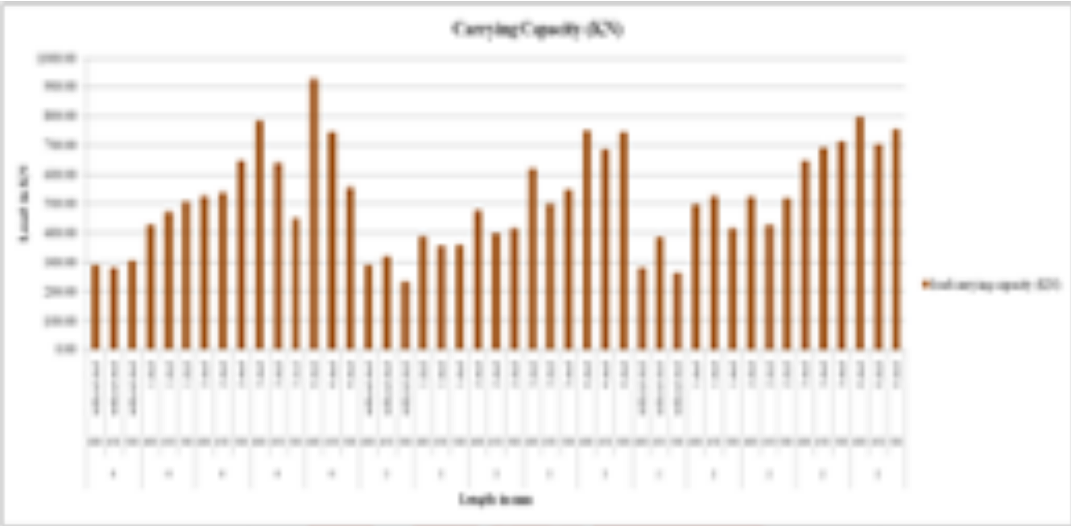


Figure 12. Comparison Load Carrying Capacity of with Similar Studs

- ***Experimental Load Carrying Capacity***

Table 14. Details of Tests Specimens and Results

| Section | Length (mm) | Ultimate Load(KN) | Ultimate Deflection (mm) |
|---|---|---|---|
| CFST Column with 4 Studs | 600 | 156 | 30.75 |
| CFST Column with 2 Studs | 600 | 165 | 25.24 |
| CFST Column without Studs | 650 | 181 | 17.85 |
| CFST Column with 1 Studs | 650 | 173 | 12.9 |
| CFST Column with 1 Studs | 500 | 220 | 20.3 |

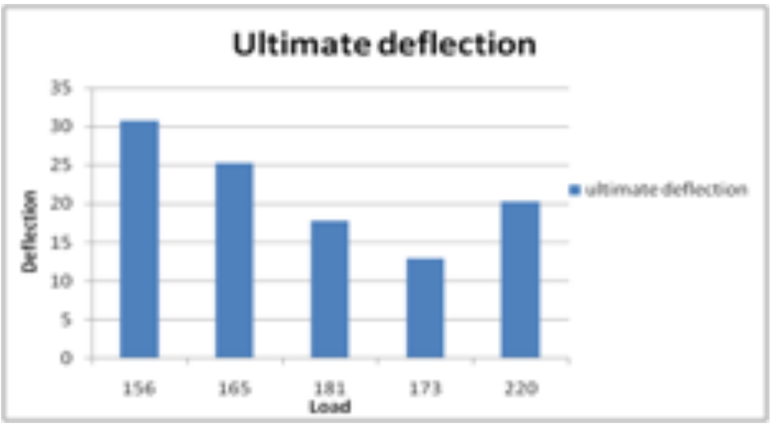



Figure 13. Load v/s Deflection

- ***Experimental Performance***

Figure 14. Experimental Performance

- ***Buckling Patterns***

The buckling patterns and the failure patterns seen during the test were given in Figure demonstrated the buckling behavior of all specimens. For the CFST segments without shear connectors, it could be seen from Figure that nearby buckling happened at the base and top of the segment, which was due to the end impact. For the segments with shear connectors, in addition to end impact, the shear connectors prevented the strain development along the height of the specimen which in turns restricted the outward buckling

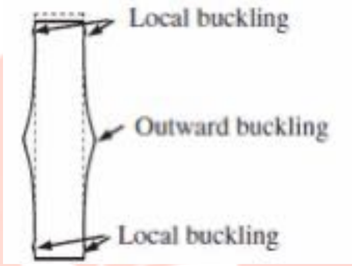



Figure 15. Failure Pattern

- ***Comparison of Experimental & Analytical result***

Table 15. Comparison Results

| Section | Length (mm) | Analytical Ultimate Load(KN) | Experimental Ultimate Load (KN) |
|---|---|---|---|
| CFST Column with 4 Studs | 600 | 927.20 | 156 |
| CFST Column with 2 Studs | 600 | 527.35 | 165 |
| CFST Column without Studs | 650 | 387.03 | 181 |
| CFST Column with 1 Studs | 650 | 472.66 | 173 |
| CFST Column with 1 Studs | 500 | 505.63 | 220 |

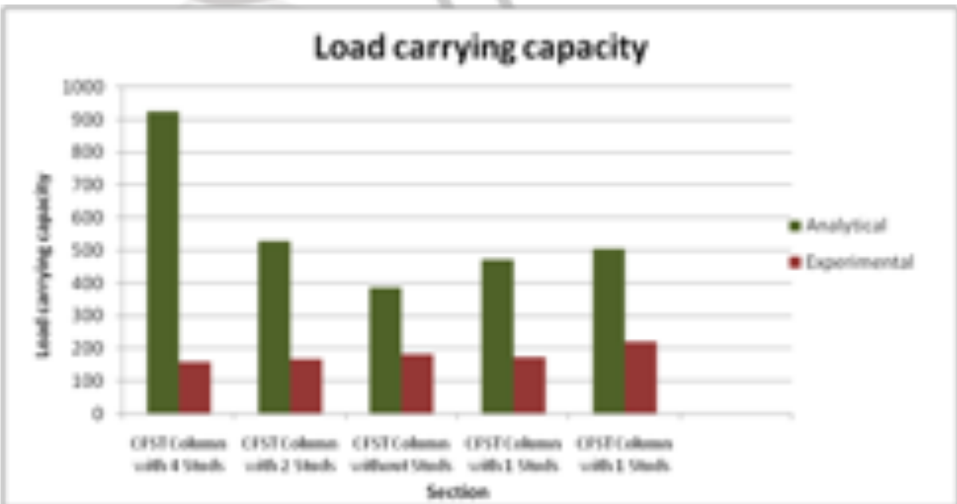



Figure 16. Load Carrying Capacity Of Analytical &Experimental Result

## V. CONCLUSION

The behavior of concrete filled steel tube columns provided with and without shear studs has been studied in respect of load carrying capacity and reduction of local buckling. The variation in load carrying capacities as obtained by software analysis is as given below:

a. The circular CFST column are optimized section 600 mm length and 80 mm diameter in circular concrete filled steel tube columns without shear studs analyzed in which optimum the load carrying is 291.20 KN.
b. The circular CFST column are optimized section 500 mm length and 80 mm diameter in circular concrete filled steel tube columns without shear studs analyzed in which optimum the load carrying is 305.56 KN.
c. The circular CFST column are optimized section 650 mm length and 80 mm diameter in circular concrete filled steel tube columns without shear studs analyzed in which the optimum load carrying is 387.03 KN.

The behavior of concrete filled steel tube columns provided with shear studs has been studied in respect of load carrying capacity and reduction of local buckling. The variation in load carrying capacities as obtained by software analysis is as given below:

a. The circular CFST column are optimized section 500 mm length and 80 mm diameter in circular concrete filled steel tube columns with shear studs analyzed in which 4mm thick, 500 mm length, 2 studs of section is optimum the load carrying is 648.89 KN.
b. The circular CFST column are optimized section 650 mm length and 80 mm diameter in circular concrete filled steel tube columns with shear studs analyzed in which 2mm thick, 650 mm length, 3 studs of section is optimum the load carrying is 692.50 KN.
c. The circular CFST column are optimized section 600 mm length and 80 mm diameter in circular concrete filled steel tube columns with shear studs analyzed in which 4mm thick, 600mm length, 4 studs of section is optimum the load carrying is 927.20 KN.

The behavior of concrete filled steel tube columns provided with or without shear studs comparison has been studied in respect of load carrying capacity and reduction of local buckling. The variation in load carrying capacities as obtained by software analysis is as given below:

a. In CFST column without shear studs provided has been studied in which 80 mm diameter, 500 mm length and 3 mm thickness column is maximum percentages of difference are 29.56 than the with respect to other without studs.
b. In circular concrete filled steel tube columns provided with similar length and different thickness with no of studs has been studied in which 80 mm diameter, 600 mm length, 4 mm thickness and 4 studs provided column is maximum percentages of difference are 218.41 than the with respect to other studs.
c. In circular concrete filled steel tube columns provided with similar thickness and different length and with same studs has been studied in which 80 mm diameter, 500 mm length, 2 mm thickness and without studs provided column is maximum percentages of difference are 46.32 than the with respect to studs.

The behavior of concrete filled steel tube columns provided with and without shear studs has been studied in respect of load carrying capacity, reduction of local buckling and deflection. The variation in load carrying capacities and deflection as obtained by experimental analysis as given below:

a. CFST Column with 4 Studs section is give minimum load 156 KN and maximum deflection as such as 30.75 mm as compared to the other CFST section.
b. CFST Column with 1 Studs section is give maximum load 220 KN and minimum deflection as such as 20.3 mm compared to the other CFST section.
c. From load versus deflection curve, the proportional increase in load carrying capability of CFSTC(without shear connectors)increased in load carrying capacity and minimum deflection as such as 181 KN and 17.85 mm respectively than the with shear connector CFST columns.
d. The comparative study reveals that analytical test result is maximum than experimental test result.

Comparison of Analytical and experimental result proved that the analytical result is increase than the experimental results. The experimental study using circular CFST with and without shear connectors showed comparable results with those obtained in analytical studies.

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