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Experimental Study on Load-Carrying Capacity of Double-Skinned Concrete-Filled-Tubular-Steel (CFST) Columns under Axial Compressive Load

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ABSTRACT
The focus of Paper is,investigate the ultimate-load carrying capacity and behaviour of double-skinned concrete filled tubular steel (DSCFT) columns under axial-compression. Total of 15 specimens are considered for the study which consists of 9 double-skinned concrete filled tubular steel (DSCFT) column specimens, 3 concrete-filled-steel-tubular (CFST) column specimens and 3 hollow double-skinned steel tubular (HDSST) column specimens. Specimens are divided into 3 groups based on slenderness ratio having lengths 0.75 m, 1.0 m and 1.5 m. For each length, three hollowness ratios such as 0.39, 0.56 and 0.58 are adopted for DSCFT column specimens.

KEYWORDS: double-skinned concrete filled tubular steel (DSCFT), concrete filled steel-tubular (CFST), hollow double skinned steel tubular (HDSST), ultimate load , slenderness ratio.

I. INTRODUCTION

Combining two distinct materials to work in tandem and bear the anticipated loads. The key benefit of utilizing such members is the ability to fully leverage the strengths of both materials, leading to more efficient and effective structural component.CFST columns can attain greater strength for a given cross-sectional area when compared to reinforced concrete columns because they harness the advantageous properties of both concrete and steel to a greater degree.

CFST columns has advantages such as increased availability of floor space due to their superior compressive stiffness and load carrying capacity, allowing for effective reduction in column size when compared to reinforced concrete columns. Additionally, these columns enable faster construction rates and eliminate need for formwork, making it ideal for high-rise structures. The confinement provided from the steel tube in CFST columns is another key benefit, as it increases strength of concrete core and enhances the column's resistance to local buckling of steel tube, thereby delaying buckling. Consequently, these columns enable the creation of more efficient and load-bearing column sections.

[1] The paper is on the introduction of a novel type of hybrid structural column, known as the multi-tube concrete column (MTCC) with a combination of fibre-reinforced polymer (FRP), concrete, and steel. The MTCC design incorporates an outer FRP tube with several steel tubes situated within, and all the tubes are filled with concrete.The testconducted with keeping a control on deflection rate of 0.6 mm/min. They have found that stress-strain curves of MTCC were higher than CFFT columns.

[2] The present literature is a expermental study on the effects of some key parameters on behaviour of fibre-reinforced polymer (FRP) concrete-steel double skinned tubular columns under compressive load. The study findings reveal that as the dia of the inner steel-tube in the double-skin tubular columns (DSTC) increases, the strain and stress of concrete significantly rise. In DSTC structures with a constant confinement ratio, an increase the strength of concrete results in decrease in the ultimate axial strain. However, strength of inner steel tube has no effect on ultimate conditions of DSTC.

[3] Study on hybrid DSTC under axial compression with concentrating on three main issues: use of high-strength concrete (HSC), filament wound tubes and the usage of large-scale specimens.Results show the use of high-strength concrete, there is an improvement of ductility of

hybrid DSTC specimens. The axial stress-strain curves of concrete within the hybrid DSTC specimens exhibit an increasing trend when the FRP tube effectively confines the concrete. However, if the concrete is not properly confined, the stress-strain curves display fluctuations or even sudden stress drops. In cases where high-strength concrete was used, damage localization was observed.

[4] A total of 32 confined concrete columns reinforced with fibre-reinforced polymer (FRP) were fabricated, consisting of 8 solid columns with FRP (FCSC), 4 CFST columns with FRP (CCFT), and 20 double-skin tubular columns reinforced with a combination of FRP, concrete, and steel (DSTC).In the study, its observed that ultimate stress of the steel-tube was not achieved upon FRP rupture. The locations of FRP rupture were consistent with the positions whereinner steel-tube buckled. In the DSTC specimens where the inner-tube was filled with concrete, the damage to FRP and concrete occurred in a wider zone than in the hollow specimens.

[5] In this paper, FEM was employed to investigate the behaviour of confined concrete within tubes. The model considered the stiffness of the FRP tube, stiffness of steel tube, and size of inner void as key parameters. This model wasused in a parametric study to evaluate impact of significant parameters on the confined concrete behaviour. Furthermore, a stress-strain model aimed at design was developed based on the outcomes from finite element analysis, combined with result of previous experimental studies.

II. EXPERIMENTAL PROGRAMME

2.1 MATERIALS

In this experimental study, The hollow structural steel-tubes with circular cross-section having a yield-stress of 240 N/mm2, Birla Super Ordinary Portland Cement (OPC) of grade 53,M-sand used as fine aggregates, Size of coarse aggregates of 12.5 mm are used. Concrete mix-design for M20 is done using IS 10262:2009, mix proportioning guidelines. The slump and water-cement ratio are considered according to standards mentioned in IS 10262:2009

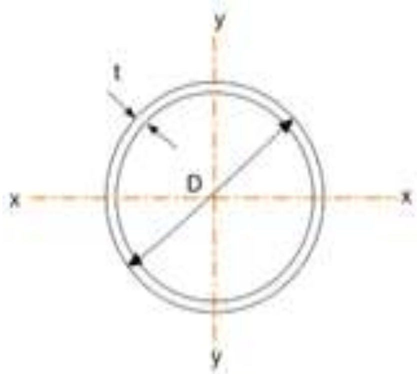

FIG-1: Cross sectional details of hollow circular sections

Particular	**Fineaggregate**	**Coarseaggregate**
SpecificGravity	2.6	2.6
Waterabsorption-(%)	8	0.6

TABLE-1: Physical-properties of aggregate

Sl No.	**PROPERTIES**	**RESULT**
1	Specific gravity	2.91
2	Initial setting-time	120 minutes
3	Final setting-time	225 minutes

TABLE-2: Physical-properties of cement

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2.2 SPECIMEN DETAILS

The details of all 15 specimens are tabulated in the table 3 below:

Length (m)	Specimen	Outer-tube		Inner-tube		Hollowness ratio	L/D_o
		D_o (mm)	t_o (mm)	D_i (mm)	t_i (mm)		
0.75	DSCFT 1	114.3	3.6	42.4	2.6	0.39	6.56
	DSCFT 2	114.3	3.6	60.3	3.6	0.56	
	DSCFT 3	114.3	5.4	60.3	3.6	0.58	
	HDSST 1	114.3	3.6	60.3	3.6	0.56	
	CFST 1	114.3	5.4	-	-	0	
1.0	DSCFT 4	114.3	3.6	42.4	2.6	0.39	8.75
	DSCFT 5	114.3	3.6	60.3	3.6	0.56	
	DSCFT 6	114.3	5.4	60.3	3.6	0.58	
	HDSST 2	114.3	3.6	60.3	3.6	0.56	
	CFST 2	114.3	5.4	-	-	0	
1.5	DSCFT 7	114.3	3.6	42.4	2.6	0.39	13.12
	DSCFT 8	114.3	3.6	60.3	3.6	0.56	
	DSCFT 9	114.3	5.4	60.3	3.6	0.58	
	HDSST 3	114.3	3.6	60.3	3.6	0.56	
	CFST 3	114.3	5.4	-	-	0	

2.3 PREPARATION OF SPECIMENAND TESTING

The hollow steel tubes, which are commonly available in the market, have a standard length of 6 meters. To obtain desired lengths of 0.75 meters, 1 meter, and 1.5 meters, the tubes were placed against a rotating metal cutter and cut accordingly. Following this, the smaller diameter tubes were concentrically placed inside the larger diameter tube to form a double-skinned steel tubular section. To ensure concentricity, the steel-tubes were connected by welding with steel connecting rods shown in fig. 2. The epoxy acts as a bonding agent between the concrete and with the steel such that any slippage between concrete &steel is avoided. The concrete is placed in annular space between outer and inner tube of fabricated double skinned steel tube specimens making the section as DSCFTcolumn specimens.

FIG-2: double-skinned concrete filled tubular steel (DSCFT) column

The specimens underwent axial compression loading to assess their load-carrying-capacity, with axial deflections being measured at regular load intervals. The experimental setup consisted of a 2.4 m high frame with a total load resistance capacity of 2000 kN, and spacers used to accommodate different column heights. A hydraulic jack equipped with a 1000 kN loading cell employed to applyload from the bottom, with dial gauges used to measure axial deflections.

Deflection measurements taken with respect to a stationary diaphragm, the top-surface of the lower spacer was used as the reference point. Axial deflections were recorded at 10 kN intervals during loading, with the column being placed between the lower and upper spacers and supported at both ends with hinged conditions.

FIG-3: Experimental setup for axial-compressive loading

III. RESULTS AND DISCUSSION

In this section, the ultimate-load and the deflection of 15 specimens are mentioned below.

DSCFT 1 -The failure axial compressive load is 620 kN. The axial deflection at 350 kN is 5.414 mm.

DSCFT 2 -The failure axial compressive load is 630 kN. The axial deflection measured at 350 kN was 7.743 mm.

DSCFT 3-The failure axial compressive load is 705 kN. The axial deflection measured at 350 kN was 7.915 mm.

CFST 1-The failure axial compressive load is 640 kN. The axial deflection measured at 350 kN was 6.724 mm.

HDSST 1 -The failure axial compressive load is 680 kN. The axial deflection measured at 300 kN was 6.423 mm.

DSCFT 4 -The failure axial compressive load is 510 kN. The axial deflection at 300 kN is 5.743 mm.

DSCFT 5 -The failure axial compressive load is 520 kN. The axial deflection measured at 300 kN was 6.624 mm.

DSCFT 6 -The failure axial compressive load is 550 kN. The axial deflection measured at 300 kN was 8.884 mm.

CFST 2 -The failure axial compressive load is 553 kN. The axial deflection measured at 300 kN was 5.402 mm.

HDSST 2 - The failure axial compressive load is 440 kN. The axial deflection measured at 300 kN was 5.402 mm.

DSCFT 7 -The failure axial compressive load is 420 kN. The axial deflection at 250 kN is 8.743 mm.

DSCFT 8 -The failure axial compressive load is 450 kN. The axial deflection measured at 250 kN was 4.524 mm.

DSCFT 9 -The failure axial compressive load is 485 kN. The axial deflection measured at 250 kN was 4.524 mm.

CFST 3 -The failure axial compressive load is 465 kN. The axial deflection measured at 250 kN was 5.428 mm.

HDSST 3 - The failure axial compressive load is 380 kN. The axial deflection measured at 200 kN was 5.428 mm.

3.1 Comparison of experimental results of the specimens

The ultimate axial-load-carrying-capacity of the CFST and HDSST specimen compared with DSCFT specimen. The comparison of ultimate-load carrying capacities of DSCFT and CFST Is depicted in fig 4 andcomparison of ultimate-load-carrying-capacity of DSCFT and HDSST Is depicted infig 5. Effect of varying hollowness ratio and slenderness ratio are represented in fig 6 and 7.

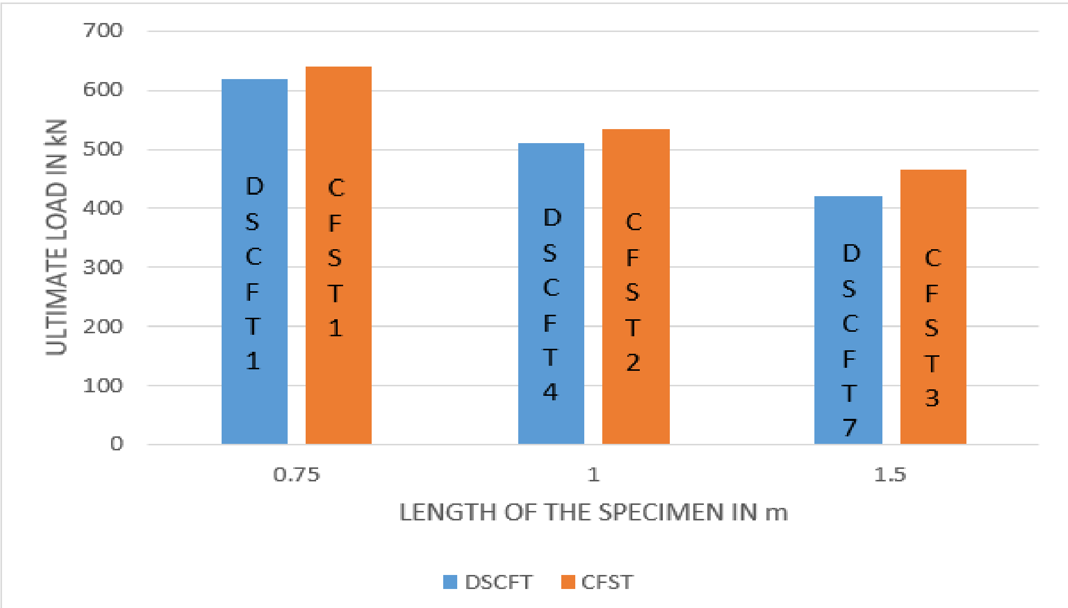

FIG-4: Comparison of DSCFT with CFST

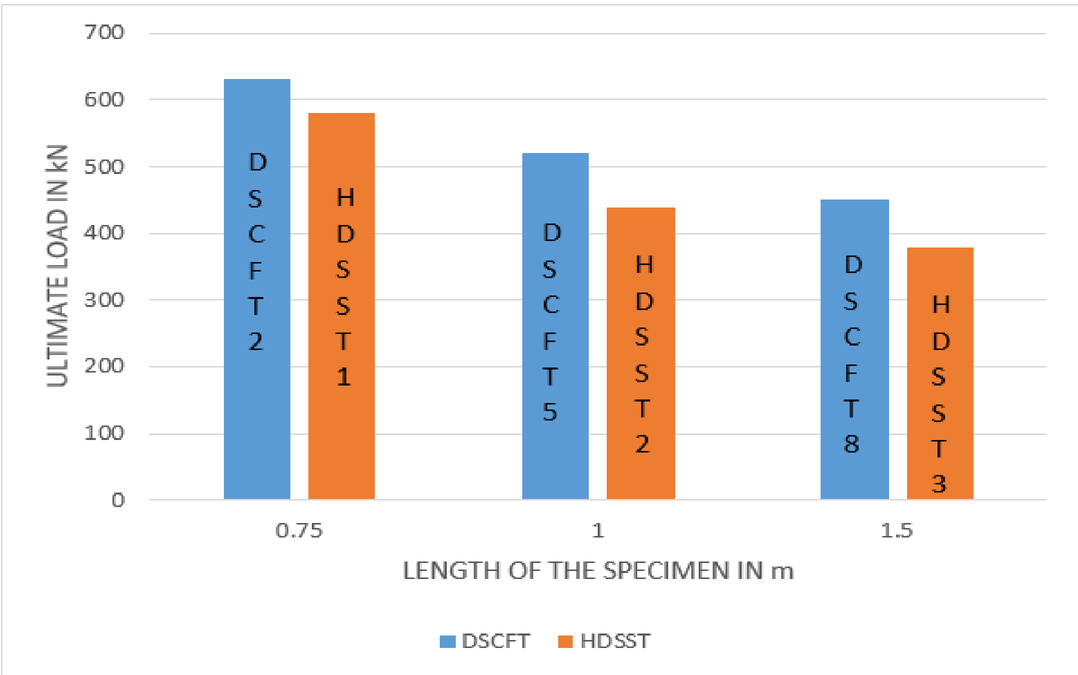

FIG-5: Comparison of DSCFT with HDSST

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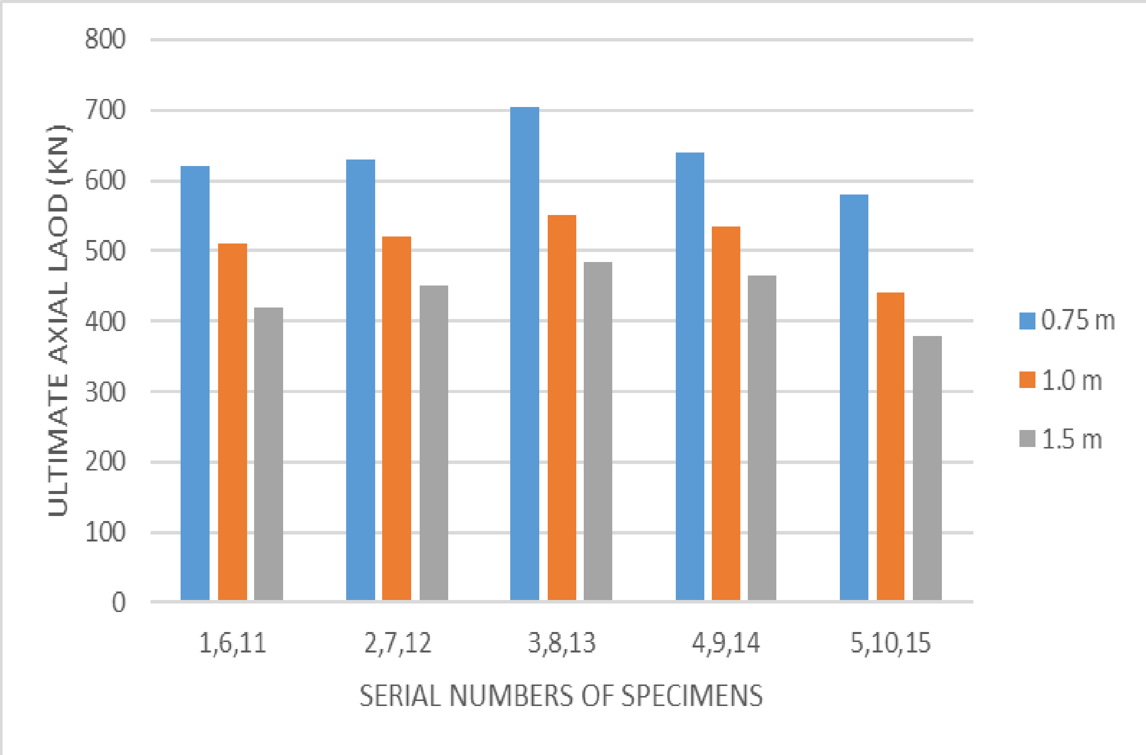

FIG-6: Variation of load carrying capacity with varying slenderness ratio

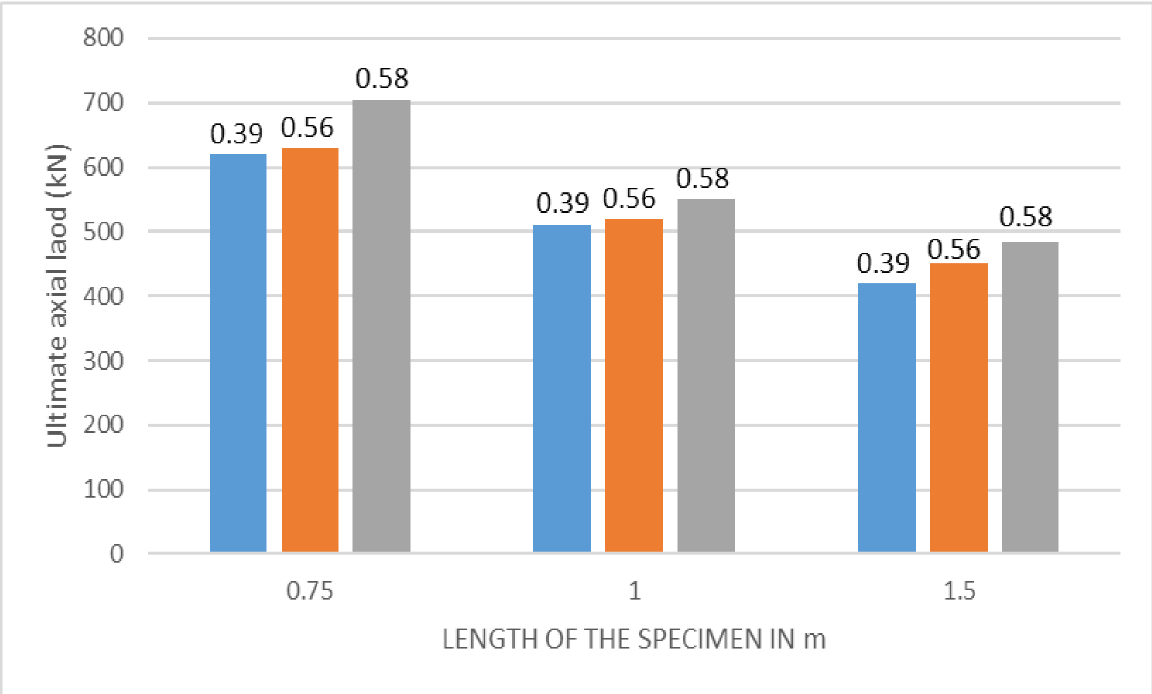

FIG-7: Variation of load carrying capacity with varying hollowness ratio

IV. CONCLUSION

- Ultimate axial-load carrying capacity of double-skinned steel tubular columns are found to increase with increase in hollowness ratio.

- Ultimate axial-load carrying capacity of all specimens found to decrease with increase in slenderness ratio.
- Axial-load-carrying-capacity of DSCFT columns and CFST columns are similar for all the three heights (0.75 m, 1.0 m, and 1.5 m).
- Load v/s deflection of both DSCFT columns and CFST columns found to be similar.
- All the DSCFT columns performed better than HDSST columns.
- The failure of specimens with height 0.75 m and 1.0 m were due to local bulging of columns and failure of specimens with 1.5 m was due to buckling of columns. In hollow double-skinned steel tubular columns, the buckling of inner steel-tube was observed.
- All the concrete infilled specimens failed mainly due to failure of concrete infill since concrete reached its ultimate-capacity and steel has reached yield strength but not its ultimate capacity.
- In case of DSCFT columns, the failure of connection between the inner tube and outer tube was not observed.
- The results for ultimate-load of specimens found to be similar with the theoretical load carrying capacities calculated from Euro code 4.
- Overall, the performance of DSCFT columns found to be similar to that of CFST columns and traditional CFST columns can be replaced with DSCFT columns.

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