

STRUCTURAL RESPONSE OF EXPANSIVE CONCRETE FILLED STAINLESS STEEL TUBES SUBJECTED TO AXIAL COMPRESSION

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ABSTRACT

The usage of calcium-sulfoaluminate (CSA) cement is gaining popularity in construction of concrete filled steel tubular columns or composite columns due to its various properties such as early age expansion and low carbon emission. But addition of expansive cements like CSA also induces the brittle behaviour in CFST columns. This study experimentally examines the performance of circular and square composite columns with stainless steel tubes and different percentages of CSA cement to understand their axial compression behaviour. The effect on ductility of these composite columns was also analysed. Variables involved for this study were a) percentage of CSA cement; b) grades of concrete; c) cross sectional shape and d) length to diameter ratio. The results showed that the tested expansive concrete filled stainless-steel tubular (ECFSST) columns had higher axial compression strength and excellent ductility when compared with that of normal concrete of same grade. It was also observed that the confinement effect of stainless steel on expansive concrete is as reliable as that of normal concrete. The ultimate compression strength of all the ECFSST specimens determined experimentally in lab were compared to the predicted maximum strength determined using the current standards AISC360-10 (2010), EC4 (2004), and GB 50936-2014 (2010). These codes are shown to be appropriately cautious in estimating the sectional strength of expansive concrete-filled stainless steel tubular stub columns.



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1. INTRODUCTION

Concrete-filled steel tube (CFST) members have emerged as contemporary structural components, commonly applied in high-rise construction projects, industrial facilities, and arch bridges (Khusru et al., 2021; Liew et al., 2019). Their popularity roots from the superior strength, rigidity, ductility, and energy-dissipating capacity they deliver through effective composite interaction (Karimi & Nematzadeh, 2020). Steel tube and concrete interaction in such composite columns not only compensate for each other's

shortcomings but also give full play to each other's advantage (Wang et al., 2022). However, due to shrinkage of concrete core, separation between steel and concrete occurs and this debonding effect between steel and concrete delays their interaction under loading, hampering the performance of CFST (Ho et al., 2021; F. Y. Liao et al., 2011).

A solution to this matter is usage of expansive concrete which is also known as self-stressing concrete which is formulated by adding expansive agents. There are mainly three products such as calcium sulfoaluminates (Type K),

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calcium oxide (Type G) and magnesium oxide (Type M) that assist expansive property during hydration stage by forming ettringite, portlandite and magnesium hydroxide respectively. Calcium-sulfoaluminate (CSA) cement mixes are getting increasing popularity as expansive concrete owing to their lower carbon footprint (Shakouri et al., 2023). These concrete mixes expand during the earliest stages of hydration, as opposed to normal concrete made using Ordinary Portland Cement (OPC) (C. Xu et al., 2009), there are a number of studies suggesting the use of expansive concrete or self-stressing concrete in CFST (Carballosa et al., 2015; Huang et al., 2020; H. Li et al., 2020; N. Li et al., 2020a; Liu et al., 2022a; L. Xu et al., 2021).

Li et al. (2020b) carried out axial compression experiments on eighteen CFST columns considering higher values of self-stress. The findings indicated that higher values of self-stress enhanced the ultimate load-carrying capacity by 27.5%, while it reduced the deformation capacity of CFST columns by nearly 70%. Huang et al. (2020) observed improvement in bonding properties and axial load carrying capacity of UHP-CFST column when expansive agent was added in mix (0% to 12%). N. Li et al. (2020a) observed thirty six SCCFST columns with self-stressing concrete (self-stress magnitude ranging 3.31 MPa to 6.66 MPa) with and without steel fibres. It was observed from this study that with addition of self-stress the ultimate load (P_U) and .85ultimate load ($0.85P_U$) were achieved at lower axial displacements, but has little impact on ductility of columns with thin steel tubes. Liu et al. (2022b) observed the behaviour of self-stressing concrete-filled steel tube (SCFST) with slender section and varying self-stressing properties. The study concluded that higher load carrying capacity of about 31% average was achieved while negatively affecting the deformability simultaneously. 25.1% of reduction in ductility index was also observed in the same study which is remarkable. It was also observed in their study that the advantage of self-stress diminishes columns with higher length to diameter ratios. Carballosa et al. (2015) worked on two types of expansive additives in concrete mixes: types K (calcium sulfoaluminate) and G (calcium oxide). It was found that type G additive provides better and stable expansion however, type K experienced certain shrinkage.

On the other hand, H. Li et al. (2020a) investigated the effect of Type M and Type G expansive additives in core concrete and on mechanical properties of CFST. It was shown in the study that both of these expansive agents in core if CFST columns may successfully prevent concrete and steel tube gap. However, selection of CaO-MgO ratios will affect the overall performance of core concrete. Adding to its benefits expansive concrete mixes, expansive concrete when confined between steel inherits densified pore arrangement, reduced porosity, and greater compressive strength (Hu et al., 2024)

Despite the expansive concrete has the ability to add on so many benefits in CFST columns, such effects may manifest as brittle response and diminished post-peak characteristics (J. Liao et al., 2021). Moreover, there are studies predicting more chloride diffusivity in CSA cements than normal Portland cements leading to corrosion of steel (Quillin, 2001; Shakouri et al., 2023).. To address these issues related to CSA cements in CFST columns, usage of stainless-steel tubes can be a practical solution as it is corrosion resistant and provides better ductility as compared to conventional steel tubes. There are studies related to concrete-filled stainless steel tubes (Dai et al., 2020; Gunawardena & Aslani, 2019; He et al., 2021; Zhang et al., 2022) however, none of them uses expansive concrete as the core concrete. Insufficient findings as well as a lack of evaluation in this area indicate that it is desirable to be looked up.

This research is experimental investigation on the mechanical behaviour of expansive concrete-filled stainless-steel tubular (CFSST) columns subjected to uniaxial compression, A total of sixteen specimens were prepared and tested, consisting of eight specimens each for circular and square cross-sections. Two grades of concrete (M30 & M50) with 3 different percentages of expansive concrete (8,12,16) were used as core along with stainless steel tubes of two L/D ratios (3,8) for comparison. The compressive behaviour of the tested ECFSST specimens employing these variables was then compared and analysed on the basis of testing data and observations. The influence caused by various parameters is studied, including sectional profile and L/D ratio of steel tube, and the constitutive material properties of concrete core of composite column. Finally, the reliability of established design provisions, including AISC 360-10 (2010), EC4 (2004), and DBJ/T13-51-2010 (2010), was assessed with respect to predicting the ultimate load capacity of the developed composite columns.

2. EXPERIMENTAL PROGRAM

2.1 Casting of specimen

For the present investigation, sixteen column specimens were produced in the laboratory, including eight circular and eight square members, and examined under axial compression loading. For concrete core two grades of concrete M30 & M50 were designed with three different percentages of expansive agent (8%, 12%, 16%). Calcium-sulfoaluminate (CSA) based expansive agent was used for this study. To make comparisons normal concrete core samples were also prepared for each corresponding grade of concrete. Circular samples had outer diameter of 114mm and thickness 3.2mm while the square samples were made with B= 100mm & thickness = 2mm. Two different L/D ratios (3,8) were selected for both the cross sections and various concrete mixes. One end of the columns was welded with stainless steel plate only. After pouring the concrete, other end of the

stainless-steel pipe was welded with stainless steel plate of 120x120x10 size. Labelling of the samples was done in the following manner:

- The first letter of the label stand for the cross-sectional shape: ‘C’ for circular cross-section and ‘S’ for square cross-section.
- First number of the label represents grade of concrete: C-30, C-50.
- Second letter of the label is “E” denoting expansive concrete which is followed by number 0,1,2 and 3 representing the 0, 8, 12, 16 percentage of expansive cement in the mix respectively.
- Third letter “L” followed by number 1 & 2 represents two lengths. Here “1” represents the length for L/D ratio 3 and “2” represents the length for L/D ratio 8.

2.2 Materials

Steel Tube

Factory manufactured austenitic stainless-steel pipe with grade AISI 304 was used for CFSST specimens for both

circular and square cross-sections. Prior to concrete placement, the stainless-steel tubes were cut to the required lengths, and an endplate made of stainless-steel was welded to one end of each tube. Mechanical properties of stainless-steel pipes were tested using coupon test in conformity with ASTM standards (ASTM E8, 2010). Coupons were extracted from the original stainless-steel tubes, and their dimensions are illustrated in Figure 1.

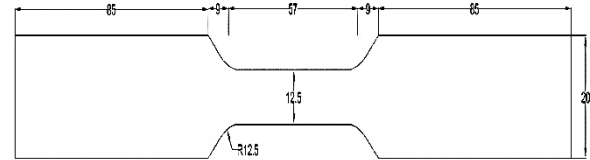


Figure 1. Representing the sample dimension for coupon testing of stainless steel tube as per ASTM

For both the cross-sections three coupons were cutout from the tubes and average results of coupon test are presented in Table 1.

Table 1. Material characteristics of stainless-steel tubes determined by coupon testing as per ASTM.

Cross-sectional profile	Yield stress (f_y) MPa	Poisson's ratio	Elastic modulus (E) MPa	Tensile strength (Min) MPa
Circular	208	.29	193860	515
Square	210	.31	198950	535

Design Mix and Raw Materials

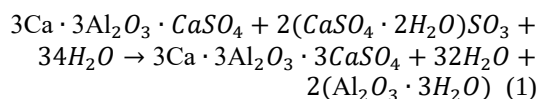
Portland cement is used as binder for normal concrete mix and it is replaced with calcium-sulfoaluminate

cement in above mentioned percentages for expansive concrete mix. The composition of OPC 43 grade cement and CSA cement is given in Table 2.

Table 2. Compositional breakdown of CSA agent and Portland cement of grade OPC 53.

Cement	CaO (%)	Al ₂ O ₃ (%)	SiO ₂ (%)	MgO (%)	Fe ₂ O ₃ (%)	CO ₂ (%)	SO ₃ (%)	Ig. loss (%)
CSA	36.76	34.73	5.02	2.5	1.1	1	18.91	8.4
Portland	62.5	4.4	20.8	2.6	3.2	-	3	2

It can be observed from the table that there exists higher percentage of Al₂O₃ and SO₃ in CSA cement which serves as main source in formation of ettringite. Equation 1 demonstrates the formation of ettringite as shown below.



In order to understand the difference of performance between Portland cement and self-stressing calcium Sulfoaluminate cement in CFSST, design mix were

prepared for grade M30 and M50 with normal Portland cement only and 3 different percentages of expansive concrete (8%, 12%, 16%). Three concrete cubes (150 × 150 × 150 mm) were cast for each concrete mix to check for their design strength and elastic modulus. River sand with fineness modulus 2.9 was used as fine aggregate for the concrete mix. Coarse aggregate of size 10mm to 20 mm were used. The cubes were removed from the mould after 8 hours and then cured in water with temperature range of (25°C to 30°C) to replicate the curing conditions for concrete inside steel tubes. Compression test of cubes and prisms were done at 28 days as per IS 456-2000. The details of mix design, compressive strength and elastic modulus for each grade is given in Table 3.

Table 3. Mix design for concrete core with various percentage of expansive agent.

concrete grade	concrete id	content kg/m ³						superplasticizer %	f _{cu}
		cement	flyash	sand	aggregate	water	expansive agent		
M30	E0	440	-	728	1162	153	0	0.9	35
	E1	404.8	-	728	1162	153	35.2	0.9	37
	E2	387.2	-	728	1162	153	52.8	0.9	39
	E3	369.6	-	728	1162	153	70.4	0.9	40
M50	E0	450	110	656	1046	178	0	0.6	58
	E1	414	110	656	1046	178	36	0.6	61
	E2	396	110	656	1046	178	54	0.6	63
	E3	378	110	656	1046	178	72	0.6	66

2.3 Experimental Setup and Test Procedure

$$\sigma_z = \frac{E_s}{1-\mu^2} (\epsilon_z + \mu_s \epsilon_\theta) \quad (3)$$

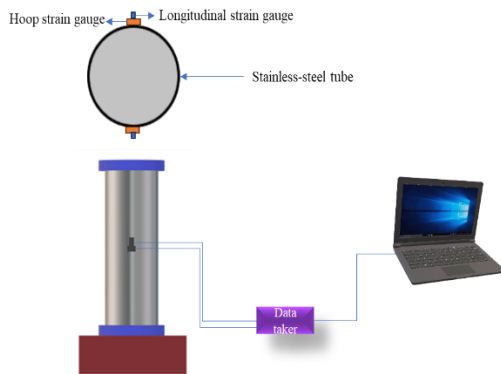
Set up for pre-stress calculation

Before the measurement of axial compression capacity of ECFSST, it is preferred to measure the pre-stress or self-stress generated due to expansion of concrete inside the stainless-steel tube for complete analysis. Measurement of prestress is done using strain gauges attached in longitudinal and circumferential direction at mid height of the column as depicted in Figure 2. The column specimens were left for 30 days and strains were observed using a data taker. The process to measure and calculate prestress is followed as described N. Li et al. (2020b) and Liu et al. (2022).

Here σ_θ and σ_z represent the hoop and longitudinal stresses in the tube, while ϵ_θ and ϵ_z denote the measured hoop and longitudinal strain in tube. μ_s and E_s refer to the Poisson's ratio and elastic modulus of steel, respectively. The hoop stress and longitudinal stress produced in concrete can be calculated as given in Equation 4 and 5.

$$q = \frac{t}{r} \sigma_\theta \quad (4)$$

$$\sigma_{CZ} = -\frac{2t}{r} \sigma_z \quad (5)$$


Figure 2. Presentation of the procedure of prestress calculation

Going by Newtonian mechanics, In CFST columns the tubes are anticipated to bear biaxial stresses. A uniform distribution of hoop strain along the wall is considered, and radial strain is disregarded owing to the tube's small thickness. The equations to calculate hoop stress σ_θ and axial stress σ_z are given as equation 2 and 3.

$$\sigma_\theta = \frac{E_s}{1-\mu^2} (\epsilon_\theta + \mu_s \epsilon_z) \quad (2)$$

Set up for axial compression test

The behaviour of ECFSST specimens under axial compression was observed with 1000kN UTM (universal testing machine) in laboratory as shown in Figure 3. A linear variable displacement transducer (LVDTs) was employed to calculate the downward displacement of the specimens due to compression. At middle of column height, two pairs of strain gauges were affixed over the stainless-steel tubes opposite to each other. Both the pair included two strain gauges, which were used to compute the longitudinal and transverse strains of the steel tubes. Before loading application, the column was pre-loaded with a small load to fix alignment of the column and proper operation of testing and measuring equipments. Then, all the loads were released and values of strain gauges and LVDT were set to zero. The final loading was then put on the specimens at rate of displacement 0.3mm/min. The testing of each specimen was stopped when the load was dropped to 75% of the ultimate loading capacity which was determined according to Eurocode 4 (Johnson & Anderson, 2001). A 10% increase in the predicted load was applied to determine the ultimate strength of the counterpart ECFSST columns.



Figure 3. Depiction of experimental setup for axial compression testing of ECFSSST columns

3. FINDINGS AND DISCUSSION

3.1 Self-stress

The observation of self-stress in ECFSSST showed that the patterns of increment of axial and longitudinal strains were similar in columns with E1, E2 & E3 concrete mixes. The hoop strain in core concrete increased rapidly in first week of curing and after that there was steady increase as observed till 30 days. The longitudinal strains also increased rapidly by 12-14 days of curing after that very little increment was observed. The calculated radial self-stresses are shown in Figure 4 -11 for circular and square column specimens.

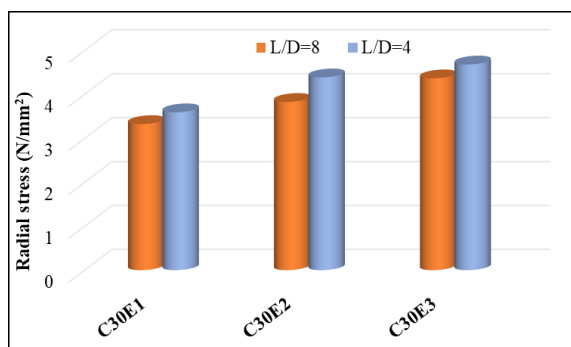


Figure 4. Radial stress in M30 grade concrete with different percentages of expansive cement for circular columns

The results indicated a subtle variation in the development of self-stress with an increasing L/D ratio in both square and circular columns. Columns with higher L/D ratios exhibited up to 13% lower radial stress and similar variations were seen for axial stresses as well. The reason for the same could be distribution of expansive hoop stress and axial stress along the length in case of columns with higher L/D ratio. Effect of cross-sectional shape can also be seen on development of self-stress. Square columns were observed to exhibit marginally lower self-stress levels compared to their circular counterparts, though the difference is minimal and practically insignificant.

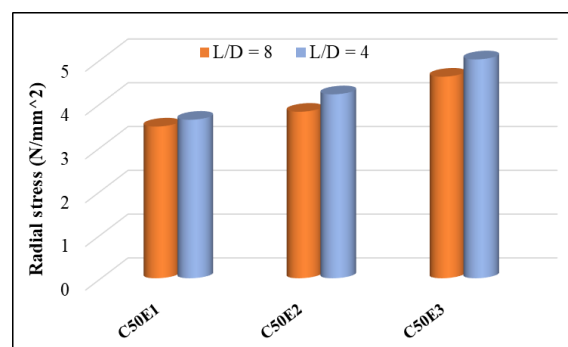


Figure 5. Radial stress in M50 grade concrete with different percentages of expansive cement for circular columns

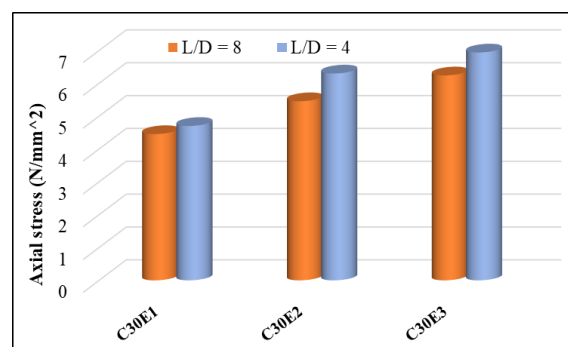


Figure 6. Axial stress in M30 grade concrete with different percentages of expansive cement for circular columns

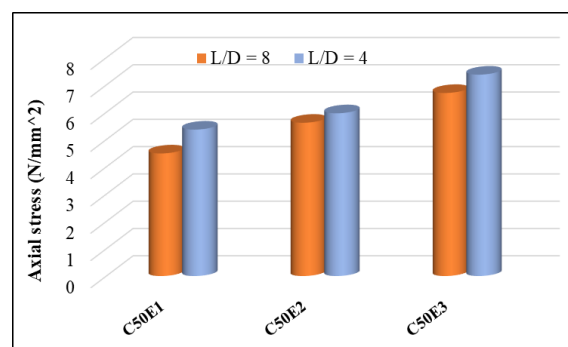


Figure 7. Axial stress in M50 grade concrete with different percentages of expansive cement for circular columns

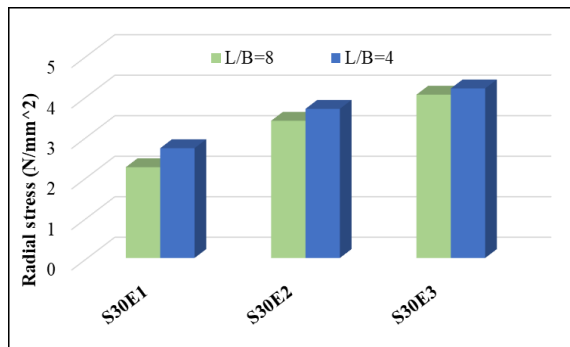


Figure 8. Radial stress in M30 grade concrete with different percentages of expansive cement for square columns

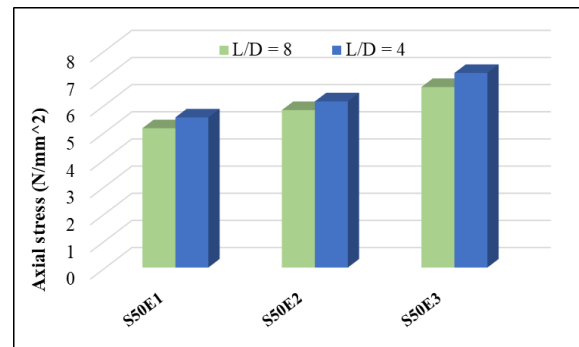


Figure 11. Axial stress in M50 grade concrete with different percentages of expansive cement for circular columns

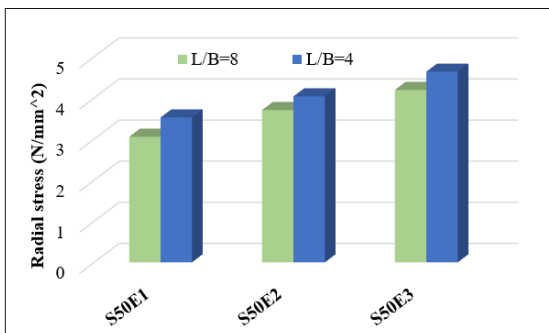


Figure 9. Radial stress in M50 grade concrete with different percentages of expansive cement for square columns

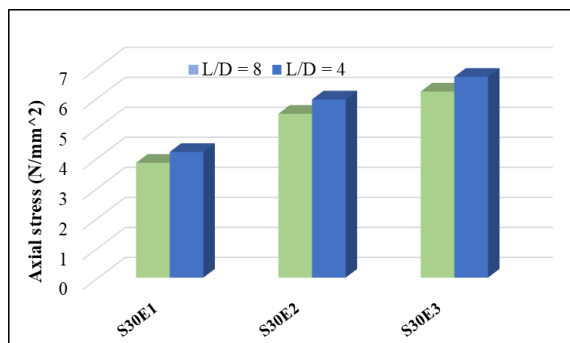


Figure 10. Axial stress in M30 grade concrete with different percentages of expansive cement for square columns

3.2 Modes of Failure of Specimens

In case of L1 group of CFSST columns with or without expansive agent comparable failure behaviour was observed. At the onset of loading, the specimens exhibited no apparent visual alterations. However, as the axial load reached near its peak value, outward buckling started developing at the ends and mid-height regions of the steel tubes which is due to shear cracks in concrete core. The peak value of load was accompanied by significant bulging developed as a consequence of crushing of core concrete causing local buckling of steel tube. The axial deformation of the column continued to increase swiftly until the test was terminated. In some samples outward bulging was so pronounced specially in square ones that it turned into fractures. It may be due to less thickness of the column and comparatively less confinement from core concrete. For the L2 group of columns, the failure mechanism was primarily governed by global buckling of the tube due to the reduced influence of hoop stress. Majorly, columns observed global buckling failure along with local buckling. The failure patterns of square columns were similar to L1 type of columns with shear failure at the ends. The failure patterns of tested specimens are shown in Figure 12. Most of the samples showed maximum lateral deflection at the middle span however some at either of the ends.



Figure 12. Demonstration of failure modes of various tested samples

3.3 Graphical Observations

The curves representing the axial load versus axial shortening for circular L1 stub columns are shown in Figure 13 (a) and 13 (b). The response of all the circular L1 columns were similar in terms of their profile. Typically, all the curves showed an initial linear-elastic phase. All the curves demonstrated a gradual decline in post-peak stage leading into a horizontal plateau. On

addition of self-stress an obvious increase in peak load can be seen. As the percentage of expansive agent increases the peak is obtained at lower axial deformation. It can be depicted that with addition of expansive agent, steeper ascending branch is achieved. However, the post peak response is not much affected by expansive agent except in M30E3L1 & M50E3L1 the descending branch of the curve is steeper owing to the increased brittleness of concrete core.

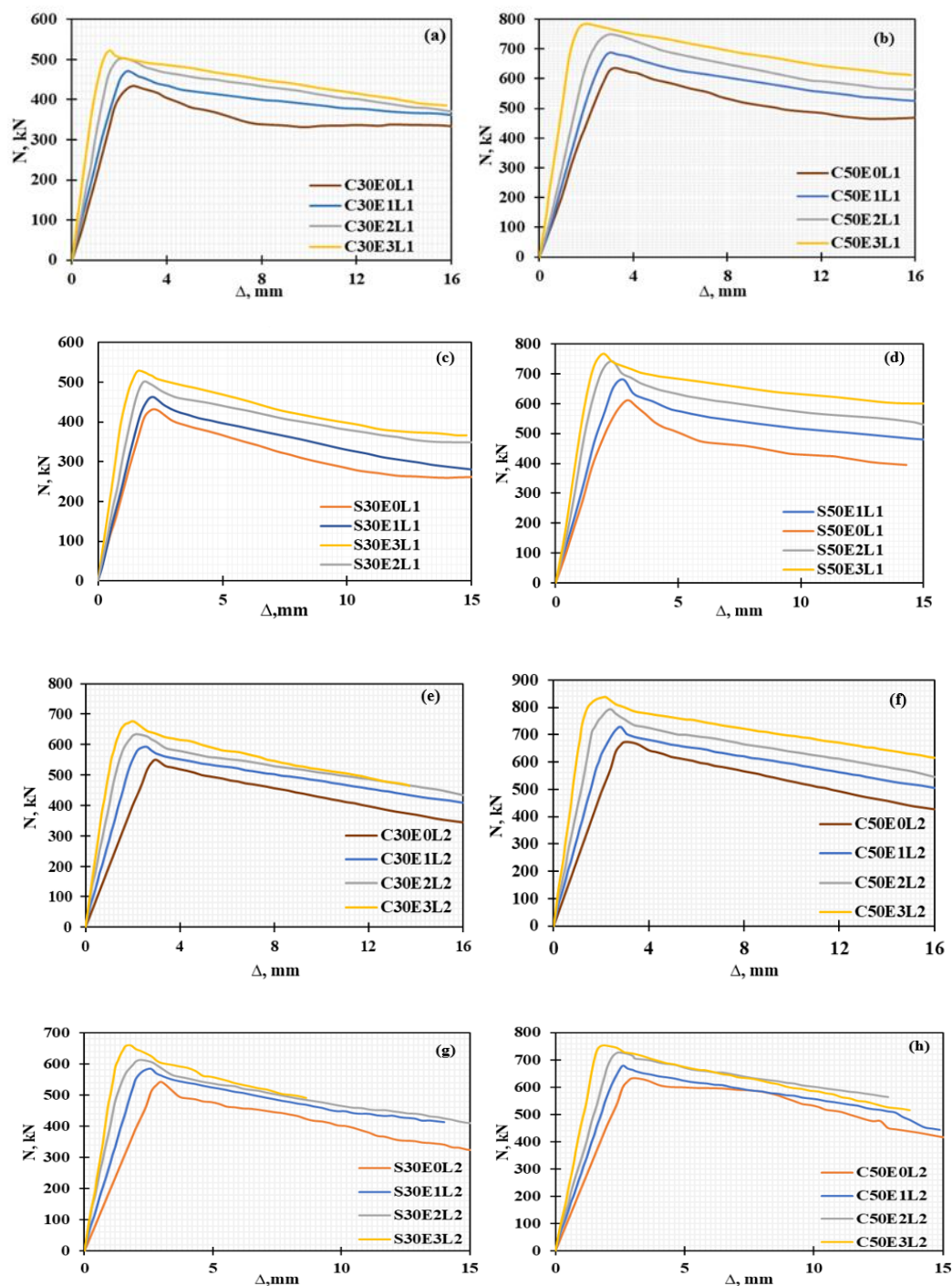


Figure 13. Axial Load vs deflection curves for tested ECFSST specimens.

Figure 13 (c), (d) represents the response of square L1 stub columns with different percentages of expansive agent. Square columns also showed an increased peak strength with addition of expansive agent. The initial response is similar to circular stub columns but post-peak there is sudden degradation of strength. It is evident from Figure 13 that the core concrete strength does not impact the profile of axial load-shortening curves of the columns.

Figure 13 (e), (f) gives the response of circular L2 type slender columns which were similar to stub columns up till the peak. The increase in self-stress can also be seen to give higher peak and steeper ascending branch. The distinction of response can be seen in descending branch of the curve. The L2 type columns showed reduction in deformability in post-peak stage with increase in self-stress. Similar observations were seen Liu et al. (2022). Figure 13 (g), (h) provides the deformation response of square L2 type columns. The impact of self-stress can be observed through increased peak strength of columns.

3.4 Ultimate load/capacity

The effect of self-stress on ultimate load in CFSST columns is presented in Figure 14 and 15 along with detailed results of the specimens in Table 4.

Improvement in ultimate capacity with addition of self-stress is observed. Figure 14 and 15 show that round stub CFSST column ultimate load capacity increased by 8.7%, 16.4%, and 20.7% with 8%, 12%, and 16% expansive agent, respectively.

The square stub columns also brought about corresponding increases of 6.9%, 15.4%, and 22.1%. Differences in ultimate load capacity were governed by how axial and radial stresses influenced the concrete core. Radial self-stress improved compressive strength and increased capacity, whereas axial self-stress, through pre-compression, reduced it. In order to better understand the increment of ultimate capacity of CFSST with increase in self-stress a strength enhancement factor Δf_c as defined in by Chen et al. (2018) is evaluated using Equation 6.

$$\Delta f_c = \frac{N_{u,s} - N_{u,0}}{A_c} \quad (6)$$

Here, $N_{u,s}$ and $N_{u,0}$ is the ultimate capacity of the ECFSST and the counterpart CFSST columns, respectively.

Calculated Δf_c for all samples are given in Table 4.

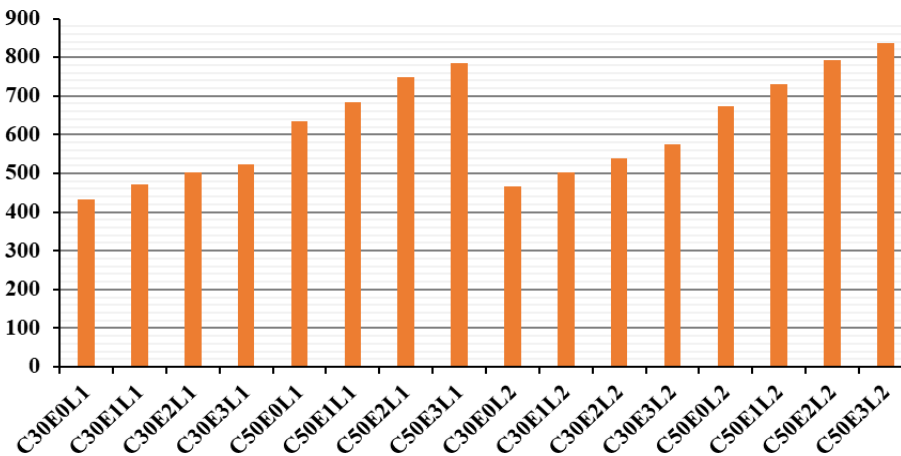


Figure 14. Comparative description of ultimate load of circular CFSST columns

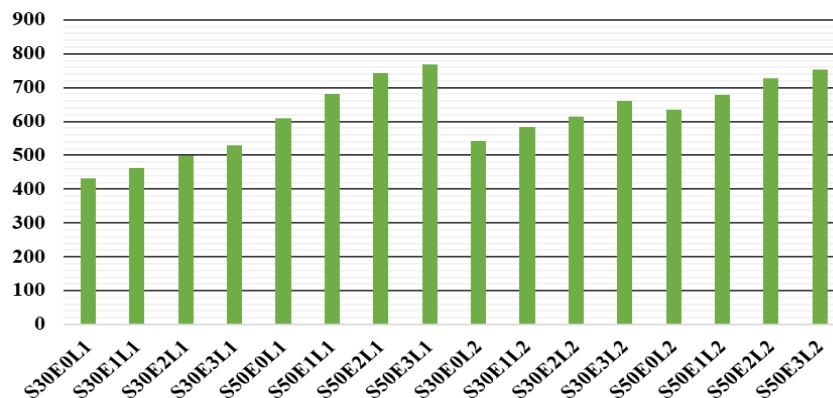


Figure 15. Comparative description of ultimate load of square ECFSST columns

Table 4. Details of ECFSSST samples with test results.

Specimen ID	D	L	t	Δf_c (MPa)	N_{85} (MPa)	DI	SI	N_U (MPa)	N_{EC4}	N_{AISC}	N_{GB}
C30E0L1	114	400	2	-	346.8	3.2	1.00	432.9	442.0	415.3	434.5
C30E1L1	114	400	2	3.9	376.5	4.0	1.09	470.4	492.1	459.7	463.0
C30E2L1	114	400	2	7.4	410.5	4.2	1.16	503.7	512.6	477.6	487.8
C30E3L1	114	400	2	9.5	430.9	5.5	1.21	522.7	542.2	495.3	512.6
C50E0L1	114	400	2	-	519.35	3.2	1.02	635.44	626.3	621.2	616.4
C50E1L1	114	400	2	5.0	558.45	3.5	1.10	684.216	624.1	665.5	674.6
C50E2L1	114	400	2	11.0	611.15	3.1	1.20	747.7749	718.8	674.4	686.2
C50E3L1	114	400	2	15.0	640.798	5.4	1.26	784.041	736.0	683.0	697.9
C30E0L2	114	916	2	-	467.5	2.4	0.97	550	419.6	401.3	434.4
C30E1L2	114	916	2	4.4	503.2	3	1.04	592	492.3	443.2	462.9
C30E2L2	114	916	2	8.9	539.75	3.3	1.12	635	522.5	459.9	487.8
C30E3L2	114	916	2	13.3	575.025	3.6	1.19	676.5	563.4	476.6	512.6
C50E0L2	114	916	2	-	571.9	2.6	0.9	672.8	649.6	592.2	682.0
C50E1L2	114	916	2	8.12	620.4	2.9	1.0	729.9	771.1	627.9	743.9
C50E2L2	114	916	2	17.14	674.3	3.1	1.1	793.3	795.3	635.9	756.3
C50E3L2	114	916	2	23.3	711.4	4.1	1.1	836.9	863.7	644.2	768.7
S30E0L1	100	400	2	-	367.8	2.2	0.95	432.7	453.0	422.6	437.9
S30E1L1	100	400	2	3.1	393.3	2.4	1.01	462.7	531.4	465.9	462.7
S30E2L1	100	400	2	7.0	424.3	3.5	1.09	499.2	549.3	483.0	487.6
S30E3L1	100	400	2	8.1	449.2	3.6	1.16	528.5	558.7	500.4	512.4
S50E0L1	100	400	2	-	610.0	2.2	0.98	596.845	630.1	621.0	597.9
S50E1L1	100	400	2	7.0	681.9	2.4	1.09	671.8767	724.1	655.5	654.4
S50E2L1	100	400	2	14.0	743.1	3.5	1.19	723.14	758.9	664.0	665.6
S50E3L1	100	400	2	16.0	768.2	4.6	1.23	768.2	766.5	672.8	676.9
S30E0L2	100	800	2	-	542.85	2.0	1.2	542.85	637.2	408.2	437.8
S30E1L2	100	800	2	4.3	584.304	2.6	1.2	584.304	678.7	450.2	462.7
S30E2L2	100	800	2	7.4	613.58	2.8	1.2	613.58	706.5	466.6	487.5
S30E3L2	100	800	2	5.0	590.97	4.4	1.1	660	724.9	482.8	512.3
S50E0L2	100	800	2	-	539.1	3.2	1.02	634.2	635.8	603.2	685.9
S50E1L2	100	800	2	4.0	577.2	3.2	1.09	679.0	687.4	636.9	747.8
S50E2L2	100	800	2	9.7	618.8	3.7	1.17	728.0	706.3	645.3	760.2
S50E3L2	100	800	2	12.4	640.3	3.8	1.21	753.3	726.4	652.2	772.6

Note: Here D = diameter, L= length of column, t = thickness of stainless-steel tube, N_{ULT} = peak strength of column, N_{85} = 85% of ultimate strength on degradation curve, N_{EC4} = predicted strength from Eurocode 4, N_{AISC} = predicted strength from American code, N_{GB} = predicted strength from Chinese code.

It is evident from the table that the value of Δf_c increases with the addition of expansive agent. The trend further reveals that the rate of increase in Δf_c is more pronounced when the expansive agent content is raised from E0 to E1 than from E1 to E2; however, a decline in the rate of increase is observed beyond this point. This pattern closely aligns with the radial and axial stress development noted for the E1, E2, and E3 mixes in circular stub columns, as illustrated in the Figure 16.

Furthermore, a comparative analysis of Δf_c values between square and circular columns indicates that circular columns exhibit marginally higher improvements. This superior behaviour is primarily linked to the enhanced confinement capacity naturally associated with circular cross-sections, relative to square counterparts.

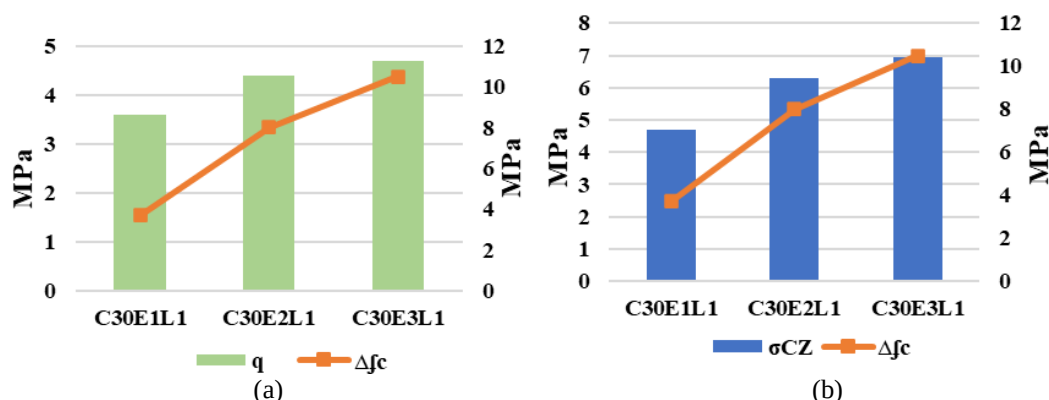


Figure 16. Depiction of similarity in development of Δf_c , radial stress (q) and axial stress (σ_{CZ}) in CFSST samples

3.5 Ductility index analysis

In order to better understand the deformability of CFST columns, analysis of ductility index is very important. This is a metric that has been defined as the ratio of Δ_{85} to Δ_u (Equation 7). Calculation of DI for all samples is given in Table 4.

$$DI = \frac{\Delta_{85}}{\Delta_u} \quad (7)$$

The results indicate that increase in expansive cement content contributes to a higher ductility index, likely due to enhanced internal confinement and micro-expansion effects. Conversely, as the L/D ratio increases, the ductility index decreases, reflecting the slenderness-related reduction in the confinement effect and energy absorption capacity.

3.6 Strength enhancement index

To understand the confinement effect in ECFSST columns, strength enhancement index (SI) was analysed and studied. Strength enhancement index (SI) is defined as the ratio of ultimate strength of ECFSST to the combined strength of tube and concrete core as given in Equation 8. The calculated SI values of all samples are specified in Table 4. SI values increase as the percentage of expansive concrete increases. It is also observed that SI values are slightly higher in M50 grade columns than M30 grade. Also, in square columns slightly lower values are observed than circular columns.

$$SI = \frac{N_{U,CFST}}{A_s f_y + A_c f'_{ck}} \quad (8)$$

3.7 Code Based Analysis

To effectively assess the experimental results an analysis has been done with standards AISC360-10 (2010), EC4 (2004), and GB 50936-2014 (2010). Various equations associated with these codes have also been provided below.

EC 4, European code

According to European code when $\lambda \leq 0.5$ and eccentricity ratio $e/D \leq 0.1$, confinement is considered in strength evaluation. The ultimate strength is given as N_{us} :

$$N_{us} = A_s f_y \eta_s + A_c f'_c \left(1 + \eta_c \frac{t}{d} \frac{f_y}{f'_c} \right) \quad (9)$$

where,

$$\eta_s = \eta_{so} + (1 - \eta_{so}) \frac{10e}{D} \quad (10)$$

$$\eta_c = \eta_{c0} \left(1 - \frac{10e}{D} \right) \quad (11)$$

$$\eta_{so} = 0.25(3 + 2\bar{\lambda}) \leq 1 \quad (12)$$

$$\eta_{c0} = 4.9 - 18 \cdot 9\bar{\lambda} + 17\bar{\lambda}^2 \geq 0 \quad (13)$$

Here, η_s and η_c are the coefficients reflecting upon the confinement effects of CFST columns. The factor η_{so} represents the weakening effect of concrete's lateral dilation on the yield strength of the steel tube. Conversely, coefficient η_{c0} signifies the beneficial confinement effect, which elevates the compressive strength of the concrete core.

Euro code also includes the effect of slenderness of CFST columns where it is calculated as relative slenderness value $\bar{\lambda}$ as follows.

$$\bar{\lambda} = \sqrt{\frac{N_{ns}}{N_{cr}}} \quad (14)$$

where N_{ns} is the squash strength and N_{cr} is the buckling strength calculated as,

$$N_{ns} = A_s f_y + A_c f'_c \quad (15)$$

$$N_{cr}^{EC4} = \frac{\pi^2 (EI)_{eff}^{EC4}}{L_{eff}^2} \quad (16)$$

GB 50936-2014, Chinese code

The GB 50936-2014 specification provides a design formula for circular CFST columns, where the confinement effect is considered in determining the improved strength. The equation is expressed as:

$$P_{U,GB} = A_{sc} \cdot f_{sc} \quad (17)$$

$$f_{sc} = (1.212 + B\theta + C\theta^2) f_c \quad (18)$$

$$\theta = \frac{A_s f_y}{A_c f'_c} \quad (19)$$

$$B = \frac{0.176}{f_y} + 0.974 \quad (20)$$

$$C = -\frac{0.104 f'_c}{14.4} + 0.031 \quad (21)$$

AISC-360–16, American code

According to the AISC provisions, the composite effect between steel and concrete in CFST columns is not considered. The ultimate capacity is derived by combining the individual strength contributions from the steel tube and concrete core. The corresponding equation is formulated as:

$$P_{U,AISC} = A_s f_y + C_2 f'_c A_c \quad (22)$$

Where C_2 is a safety factor which is taken as 0.95.

It can be observed through the Figure 17, 18 and 19 that European code EC4 is better in predicting the ultimate capacity of tested ECFSST columns. American code AISC-360–16 and Chinese code GB 50936-2014 both are predicting well for smaller L/D but for higher L/D ratio it is underestimates the ultimate capacity of ECFSST columns. Ratios of codal and experimental results are given in Table 5.

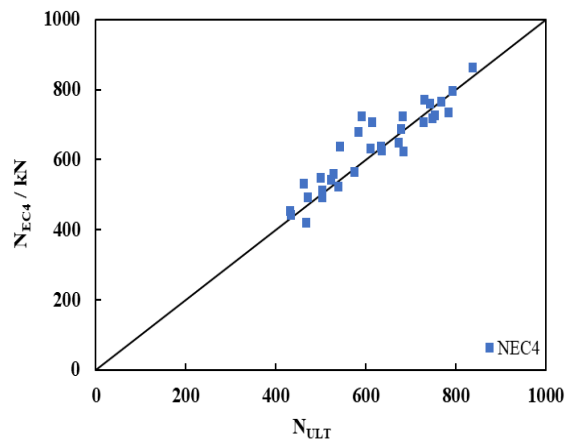


Figure 17. Graphical comparison of ultimate capacity from experimental results and codal prediction from Eurocode, EC4.

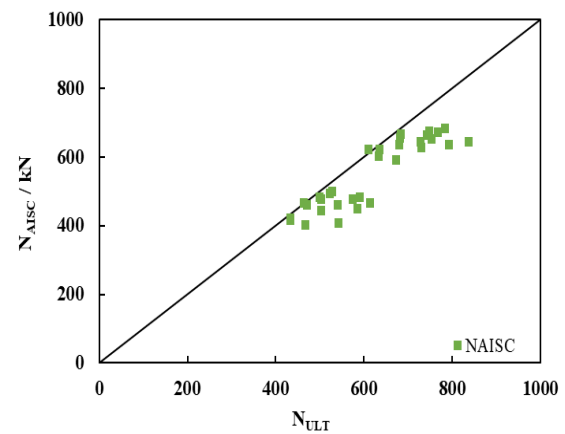


Figure 18. Graphical assessment of ultimate capacity from experimental results and codal prediction from American code, AISC-360-16.

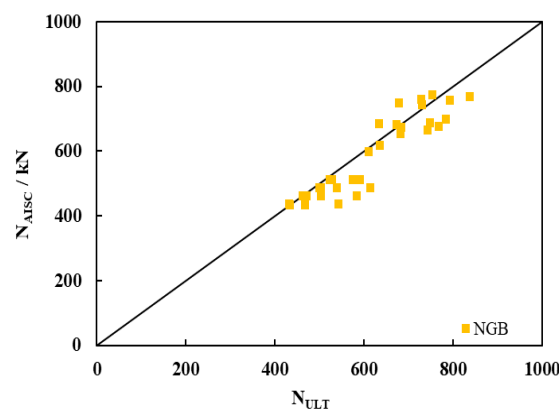


Figure 19. Graphical assessment of ultimate capacity from experimental results and codal prediction from Chinese code, GB 50936-2014.

Table 5. Comparison of codal prediction and experimental results for ECFSSST samples.

	N_{ULT} / N_{EC4}	N_{ULT} / N_{AISC}	N_{ULT} / N_{GB}
C30E0L1	0.98	1.04	1.00
C30E1L1	0.96	1.02	1.02
C30E2L1	0.98	1.05	1.03
C30E3L1	0.96	1.06	1.02
C50E0L1	1.01	1.02	1.03
C50E1L1	1.10	1.03	1.01
C50E2L1	1.04	1.11	1.09
C50E3L1	1.07	1.15	1.12
C30E0L2	1.11	1.16	1.08
C30E1L2	1.02	1.14	1.09
C30E2L2	1.03	1.17	1.11
C30E3L2	1.02	1.21	1.12
C50E0L2	1.04	1.14	0.99
C50E1L2	0.95	1.16	0.98
C50E2L2	1.00	1.25	1.05
C50E3L2	0.97	1.30	1.09
S30E0L1	0.96	1.02	0.99
S30E1L1	0.87	0.99	1.00
S30E2L1	0.91	1.03	1.02
S30E3L1	0.95	1.06	1.03
S50E0L1	0.97	0.98	1.02
S50E1L1	0.94	1.04	1.04
S50E2L1	0.98	1.12	1.12
S50E3L1	1.00	1.14	1.13
S30E0L2	0.99	1.55	1.44

S30E1L2	0.97	1.47	1.43
S30E2L2	0.98	1.49	1.42
S30E3L2	1.03	1.55	1.46
S50E0L2	1.00	1.05	0.92
S50E1L2	0.99	1.07	0.91
S50E2L2	1.03	1.13	0.96
S50E3L2	1.04	1.15	0.98
Mean	0.99	1.15	1.08
Standard Deviation	0.05	0.16	0.15

4. CONCLUSION

Through this study it is evident that expansive concrete filled stainless-steel tubular (ECFSST) columns are provide better strength than normal CFSST. However, the percentage of expansive agent, L/D ratio of column and cross-sectional shape may affect the results. Following points have been concluded through this study.

1. The strength enhancement through expansive concrete is effectively marked between 10%-12% of expansive agent addition, also, evident through other studies. However, the ductility of columns were not affected much in columns with 16% expansive agent owing to high ductility of stainless-steel.
2. Square shaped columns also showed higher ultimate strength with addition of Type K expansive agent. For square columns ultimate capacity improved by 6%-11% with 8% EA content , 15%-20% improvement in capacity with 12% addition of EA , 20%-22% on 16% of EA. Moreover, post peak behaviour was affected in square columns.
3. The columns with higher L/D ratio also showed improvement in axial deformation capacity. Axial and radial prestress was slightly lower in specimens with higher L/D ratio.
4. Ductility index of columns were also improved on increasing EA percentage. Large axial deflections were observed without fracture of stainless-steel tubes. Except in some square columns higher L/D ratio.
5. The ultimate strength values estimated using EC4 (European Code) and GB 50936-2014 (Chinese Code) closely matched the

experimental results. In contrast, the AISC 360-16 (American Code) tended to underestimate the strength, particularly in specimens with larger length-to-diameter (L/D) ratios.

Statements and Declarations

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Author Contributions

- *Isha Rohilla*: Experimental work, methodology, conceptualization, analysis, original draft preparation, and editing.
- *S. M. Gupta*: Visualization, conceptualization, reviewing, and editing.

Data Availability: The raw and processed data supporting this study are not publicly available at present, as they form part of an ongoing investigation.

Ethics Approval: Not applicable, as the study did not involve human participants, animals, or related data/tissues.

Consent to Participate: All authors provided informed consent to participate in this study.

Consent for Publication: All authors consent to the publication of this manuscript.

Conflict of Interest: The authors declare that they have no competing interests.

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