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# Behaviour of Shear Connector in CFST for Axial Strength

Seema Subhash Rathod<sup>1</sup>, Dr. S. A. Bhalchandra<sup>2</sup>

<sup>1</sup>PG Scholar, <sup>2</sup>Dr. of Department of Civil Engineering, Government College of Engineering, Aurangabad, India

**Abstract:** Concrete Filled Steel structures (CFST) offers wide benefits like high strength, ductility, and energy absorption with the combined benefits of steels and concrete. It also reduces the complexity of the production, as it does not require the shuttering of work, and so it is not commonly used. In addition to CFST elements, are more efficient, and allow for rapid construction and cost savings due to the elimination of the shape and material of component part. Concrete-filled-steel-tube is currently gaining more and more popularity in the construction industry. Concrete-filled-steel-tube it is a component of a good performance, as a result of the impact of the steel and holds it with concrete, and the question of structural diversity. In this paper, it presents a study of the evolution of the load carrying capacity, used for the connection of a variety of sizes and shapes, with a different position. The composite action of steel and concrete there is a need for a strong bond between the steel and concrete interface. Analysis of CFST column using the Finite element method and the numerical study is done on the selected case under axial loading condition.

**Keywords:** CFST Column, Stiffener, ABAQUS software

## I. INTRODUCTION

Steel components have the advantages of high tensile strength, and yield strength, while the concrete elements have the advantages of high compressive strength and stiffness. The compound of the elements are connected to each other by steel and concrete, resulting in an element that has all the useful properties of both materials. The two main types of composite column are the steel-reinforcement concrete (SRC) column shown in Fig. 1, which consists of a steel section encased in reinforced or unreinforced concrete, and the concrete-filled steel tube (CFT) columns shown in Fig.no 2 consist of concrete filled composite column. CFT columns have many advantages over SRC columns. The major benefits of concrete filled columns are:

- 1) Steel column acts as permanent and integral formwork
- 2) The steel column provides external reinforcement, and
- 3) The steel column support several levels of construction prior to concrete being pumped

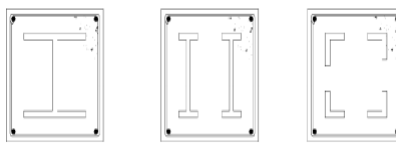


Fig.1: Concrete-encased Composite column

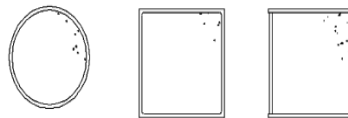


Fig.2: Concrete-filled Composite Column

### A. Concrete Filled Steel Tube

The use of CFTs provides great cost savings by increasing the area to be leased, at the same time, the required cross-sectional dimensions. This is particularly important for the design of high-rise buildings in the towns and cities, where the cost of providing the space for rent is a lot higher. It is particularly noticeable in the lower floors of high-rise buildings, which are usually of a short column. CFTs can provide a good monotonicity, and the buildings in the two orthogonal directions. The use of the elements of the composite CFT in every major area of small and medium-sized building with a seismic reduce the time in order to take full advantage of the capabilities in a two-way CFT.

## II. LITERATURE REVIEW

### A. Behavior Of Concrete-Filled Steel Tubes Subjected To Axial Impact Loads (2020)

The paper presents an experimental investigation of the dynamic behavior of a twelve-square-and rectangular-shaped CFST stub columns subjected to axial shock-loading with a weight testing machine. The study of concrete-filled steel tube elements (CFST) under the static loads are commonly used. However, limited research has been done on the behavior of CFST columns subjected to an axial impact load. At the same time, the moment of the forces, and deformational states of America was in for the shock waves. The models of the last element (FE) has been defined and tested, and with the help of the experimental results and the latest research, the data do not fully account for the effects of the deformation of the steel and the concrete. Then, with the help of a FE model, we study the influence of the cross-sectional shape, the steel strength, concrete compressive strength, steel-to-the-point energy-is the speech, the strike of the dynamic characteristics of CFST contact of the columns.

### B. Flexural Behavior of Circular Concrete Filled Steel Tubes (CFST) Under Sustained Load and Chloride Corrosion

In this paper, we study the bending behavior of the strength of the concrete and the steel tube (CFST) under long-term loading and chloride corrosion. 7 CFST specimens were tested under four-point bending load. It was found that corrosion will be a noticeable deterioration in the strength in bending, and the ductility of the CFST is in very good condition. In order to determine the full-scale behavior of the CFST under the corrosion of the terms and conditions, a newly-element analysis model analysis was developed. A parametric study was carried out in order to find the most important parameters affecting the residual strength under bending, on the basis of a simplified model for the calculation of the residual strength in the circular bending of the CFST under long-term loading and corrosion have been proposed.

### C. Study on Shear Connectors of Square Concrete Filled Steel Tubes

The author carried out a study on the intermediate plate (steel ring), and the square CFST. In order to compare the results obtained with the time-tested methods and reviews to FIVE, steel coils CFST. This parameter was the ratio, B/t, the tubes, the number of steel coils, of a thickness of the steel ring, and the distance between the steel rings. The complex column-beam (CFST) filled with concrete, the steel, the pipes, the stress of the beam, the first devoted to the steel in the pipe, and then to the side of the steel tube to the concrete, contacts, link, and the cut, so the CFST cut the contact is a very important part of it. The results of the analysis are in good agreement with the test results, the effect of the length of the two steel rings, and the thickness of the steel rings were studied.

## III.METHODOLOGY

### A. Confinement Effects of Binding Bars

The confinement effects of binding bars on the behavior of CFT stub columns with binding bars can be summarized in two aspects as follows.

- 1) *Constraining Lateral Deformation Of Concrete Core:* The interaction among the components of CFT stub column with binding bars is illustrated in Fig. Under the assumption that it is the depth along its longitudinal axis, which is equal to the distance between the longitudinal axes of the connecting bars (bs), which you can get from the Fig. 4 according to the equilibrium of forces as follows

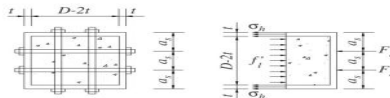


Fig.3: Lateral Confinement Of Concrete

$$f_1' h_2 (D - 2r) - \sigma_h h_2 \cdot 2r - \left(\frac{D}{2r} - 1\right) F_2 = 0 \dots\dots\dots (1)$$

$$F_2 = E_s \epsilon_s A_s \dots\dots\dots (2)$$

Substituting Eq.(2) into Eq.(1) produces

$$F_1' = \frac{\sigma_h \cdot \frac{D-2r}{2r} \cdot \frac{A_s}{D} \cdot E_s \cdot \epsilon_s}{2r-1} \dots\dots\dots (3)$$

It was observed that the binding bars yielded when the specimens came to ultimate strength. So Eq. (3) can be rewritten as

$$F_1' = \frac{\sigma_h}{2r-1} + \frac{\frac{D-2r}{2r} \cdot \frac{A_s}{D} \cdot E_s \cdot \epsilon_s \cdot F_y}{2r-1} \dots\dots\dots (4)$$

Where the second part on the right hand side indicates the contribution of binding bars to lateral pressure on concrete, which shows that the lateral confining stress of the concrete core  $F_1'$  increases while the spacing  $a_s$  and  $b_s$  decreases.

2) *Effect on Local Buckling of Steel Tube:* A local metal bending, steel column, with or without a binding alone is shown in the figure. 5. The concrete in-fill is considered as a rigid medium, restraining the free formation of buckles and forcing them to form away from the concrete. He et al applied an energy formulation to calculate the critical stress in a plate assuming that the displacement is a cosine function. The critical stress is expressed as

$$\sigma_{cr} = k \frac{\pi^2 E_a}{12(1-\mu^2)} \left(\frac{t}{b}\right)^2 \dots\dots\dots (5)$$

Where the buckling parameter k is given by

$$k = \frac{\lambda}{2} \left( \frac{a}{b} + 6\psi^2 + 4 \right) \dots\dots\dots (6)$$

$\psi$  Wavelength parameter defined as

$$\psi = \frac{a}{L}$$

It is pointed out that up to half of the wavelength of the local bending, and it is almost the same as the width of the w. Therefore, it is reasonable to assume that a = b, for the specimens without binding bars. Substituting a = b into Eq. (6) But for specimens with binding bars, it can be observed from this experiment that all of the buckling takes place at the spacing between two adjacent rows of level binding bars. It is considered clamped where the rows of level binding bars are located, i.e., a = bs. For specimens with binding bars in this experimental program, bs = D/2 and b = D. apply  $hs = \frac{D}{2}$  and  $h = D$  In eq.(5) calculate results in k

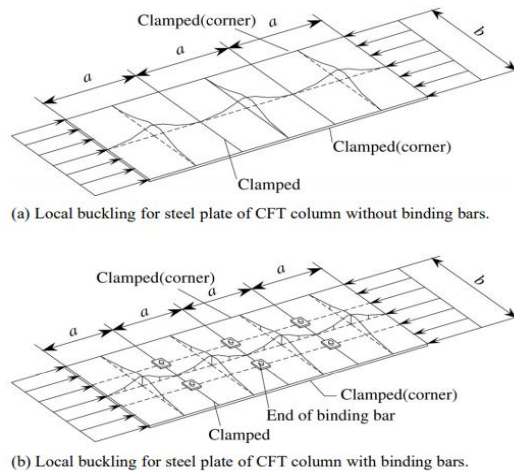


Fig.4: Local Buckling For Steel Plates of CFT Column

**B. Calculation of Ultimate Strength**

1) *Primary Methods In Current Codes and Specifications :* Methods for calculating the ultimate strength of normal sectioned square CFT stub columns under axial compressive loading are included in many current codes and specifications such as AIJ, EC4, LRFD and GJB4142- 2000. In the limit, the effect of the steel tube on the concrete core, this is ignored in the AIJ, EC4 and LRFD methods. GJB4142 2000, to take account of the importance of the seal of the remarkable endurance of square METERS, the plug of the columns in relation to the casting of the concrete slab and the steel tube column as an entity and taking into account concrete confinement with the so-called constraining factor  $\xi$ . The ultimate strength can be expressed as

$$N_u = A_{sc} f_{scy} \dots\dots\dots (7)$$

$$A_{sc} = A_c + A_e \dots\dots\dots (8)$$

$$f_{scy} = (2.212 + B\xi + C\xi^2) f_{ck} \dots\dots\dots (9)$$

Where,

$$B = \frac{0.138 f_y}{235} + 0.7646$$

$$C = \frac{-0.0727 f_{ck}}{20} + 0.0216$$

2) *Model:* It is noted that the methods existing in assume that the steel tube is in a state of uniaxial compressive stress and the concrete obtains its uniaxial ultimate strength while steel yields. Actually the steel tube is subjected to lateral compression after the Poisson's ratio of concrete exceed that of strengthen in the rising loading phase, which leads to the steel tube being in a state of hoop tension and longitudinal compression and the concrete core creature in a triaxial compressive state. The longitudinal compressive strength of the concrete center increases due to such a triaxial compression state. However, the increase in concrete strength outweighs the lessening in the yield strength of steel in vertical compression due to the confinement tension desirable to contain the concrete. The formula for calculating the strength of square CFT stub columns with binding bars are spoken as

$$N_u = A_s f_s + A_c f_{cc} \dots \dots \dots (10)$$

Where,

$f_s$  = Longitudinal strength of steel and  $f_{cc}$  the longitudinal strength of concrete.

In order to determine  $f_{cc}$ , the constitutive model of confined concrete proposed by Mander et al. is modified and applied herein. The ultimate in axial strength of confined concrete is known by

$$f_{cc} = f_{ck} \left( -1.254 - 2.254 \sqrt{1 + \frac{7.94 f_l}{f_{ck}}} - 2 \frac{f_l}{f_{ck}} \right) \dots (11)$$

Where,

$f_l$  is equivalent lateral confined stress, given herein by

$$f_l = k_e f'_l \dots \dots \dots (12)$$

In which  $f'_l$  is calculated from Eq. (4). The confinement effectiveness coefficient  $k_e$  is divided into  $k_{e1}$ , the level confinement effectiveness coefficient, and  $k_{e2}$ , the longitudinal confinement coefficient, i.

$$k_e = k_{e1} \cdot k_{e2} \dots \dots \dots (13)$$

Where,

$k_{e1}$  and  $k_{e2}$  Take the form as

$$k_{e1} = \frac{A_{e1}}{A_{cc1}} \dots \dots \dots (14)$$

$$k_{e2} = \frac{A_{e2}}{A_{cc2}} \dots \dots \dots (15)$$

Where,

$A_{e1}$  &  $A_{e2}$  is the level area of effectively confined concrete along with the longitudinal area of profitability confined concrete core while acting on  $Acc1$  &  $Acc2$  evel area of confined concrete core & longitudinal area of core respectively.

The arching action shown in Fig. 6t is assumed that a 6 will be the second stage of a parabola with a first tangent line at angle  $\theta$  So that  $A_{e1}$  and  $Acc1$  are given by

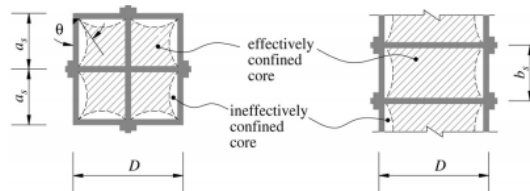


Fig.5: Effectively Concrete Confined Region Square CFT With Binding Bars

$$A_{e1} = (D - 2t)^2 - \sum_{i=1}^n \frac{[(D-2t) \tan \theta]^2}{6} \dots \dots \dots (16)$$

$$A_{cc1} = (D - 2t)^2 \dots \dots \dots (17)$$

And applying Eqs. (16) and (17) to Eq. (14) produces

$$k_{e1} = \left[ 1 - \frac{\sum_{i=1}^n \left( \frac{a_{e1}}{D} \right)^2 \tan \theta}{6} \right] \dots \dots \dots (18)$$

Similarly it can be deduced that

$$k_{c2} = \left[ 1 - \frac{b_s \tan \theta}{2(b-2t)} \right]^2 \dots\dots\dots (19)$$

Then, from Eq. (13) the confinement effectiveness coefficient is given by

$$k_p = \left[ 1 - \sum_{i=1}^n \frac{(c_s/r)^2 \tan^2 \theta}{6} \right] \left[ 1 - \frac{b_s \tan \theta}{2D-2(D-2t)} \right]^2 \dots\dots\dots (20)$$

It is found that the initial tangent angle  $\theta$  is mainly sensitive to level spacing of binding bars. Base on a linear regression, a simple shape of expression is in employ as

$$\theta = \frac{\pi}{100} \left( 13 + \frac{9.22s}{100} \right) \text{ (rad)} \dots\dots\dots (21)$$

It is shown that specimens with binding bars failed in postlocal buckling mode. The steel yielded as this category of column reached final strength, hypothetical to obey the Von Mises criteria given by

$$f_a^2 + f_a \sigma_h + \sigma_h^2 = f_{ay}^2 \dots\dots\dots (22)$$

For the local buckling strength of a steel plate  $f_b$  in CFT columns, Ge and Usami [19] proposed a relationship as

$$\frac{f_b}{f_{ay}} = \frac{1.2}{R} = \frac{0.3}{R^2} \leq 1.0 \dots\dots\dots (23)$$

This is ensured by keeping  $R \geq 0.85$ ; where  $R$  is the width–thickness ratio parameter, defined as

$$R = \frac{D}{t} \sqrt{\frac{12(1-\nu^2)}}{4u^2} \sqrt{\frac{f_{ay}}{E_s}} \dots\dots\dots (24)$$

Then from Eq. (23), the stress states of steel tubes can be determined in the two cases as follows. For  $R \geq 0.85$  applying  $f_a = f_b$  to Eqs. (23) and (22) results in

$$f_a = \left( \frac{1.2}{R} - \frac{0.3}{R^2} \right) f_{ay} \dots\dots\dots \text{(In Compression)} \dots\dots\dots (25)$$

$$\sigma_h = \frac{f_a - \sqrt{4f_{ay}^2 - 3f_a^2}}{2} \dots\dots\dots \text{(In tension)} \dots\dots\dots (26)$$

For  $R < 0.85$ , the effects of local buckling can be ignored [19]. Sakino et al. [20] deduced a relationship between stress coefficient  $au$  and  $bu$ , which represent hoop stress to yield strength of steel ratio and longitudinal stress to yield strength of steel ratio respectively, based on a large number of results of experiments for circular CFT short columns. The value of coefficients  $au$  and  $bu$  given as  $-0.19$  and  $0.89$  respectively are adopted herein to assume the value of hoop stress and longitudinal stress for steel in the case of  $R < 0.85$ . Therefore,  $\sigma_h$  and  $f_a$  are expressed as follow

$$\sigma_h = 0.19 f_{ay} \dots\dots\dots \text{(In tension)} \dots\dots\dots (27)$$

$$f_a = 0.89 f_{ay} \dots\dots\dots \text{(In Compression)} \dots\dots\dots (28)$$

3) *Simplification of the Method Proposed* : The method presented above is somewhat complicated and a simplification is proposed below

$$N_u = \phi_1 f_{ay} A_c + \phi_2 f_{ck} A_c \dots\dots\dots (29)$$

Where,

$\phi_1$  and  $\phi_2$  are the strength coefficient of steel and that of concrete, defined as

$$\phi_1 = \frac{f_a}{f_{ay}}, \quad \phi_2 = \frac{f_{ck}}{f_{ck}} \dots\dots\dots (30)$$

Based on regression from this program is show fig

. 12,  $\phi_1$  is given as

$$\phi_1 = \begin{cases} 0.09 & R < 0.05 \\ 0.897 - 0.7407 R & R > 0.85 \end{cases} \dots\dots (31)$$

It is found that the uniaxial strength of concrete filled in a square steel tube with binding bars primarily depends on the constraining factor of the binding bars  $\zeta$  and the width-to-thickness ratio parameter  $R$ .

$$\phi_2 = 1.03R^{-0.0261}(7.3836\zeta + 1.0588) \dots\dots (32)$$

C. Calculation Of Ultimate Bearing Capacity

In this paper, I have analysed the mechanical behaviour of the reinforcements, which are represented by the effective limit, the coefficient of the concrete. Non-load-bearing capacity is derived based on a unified theory of the strength, in order to analyze the factors that affect the good of the century.

1) *The Unified Theory of the Double-Slip Effect:* Yu Maohong propose a new, unified the income criterion for a double slip, which is based on the income criteria for the double-slip-and-body-double cut block, taking into account the effect of pre-stretching and compression, and the degree of damage to property, under the influence of the principal stress, and its expression is given by equation 1 and equation 2:

$$\begin{cases} F = \sigma_1 - \frac{1}{1+b}(b\sigma_2 + \sigma_3) = \sigma_s & \sigma_2 \leq \frac{\sigma_s + a\sigma_3}{1+a} \\ F' = \frac{1}{1+b}(\sigma_1 + b\sigma_2) - \sigma_3 = \sigma_s & \sigma_2 \geq \frac{\sigma_s + a\sigma_3}{1+a} \end{cases} \dots\dots\dots (1)$$

$$a = \frac{\sigma_s}{\sigma_c} \quad h = \frac{2\sigma_s - \sigma_c}{\sigma_s \tau_s} \dots\dots\dots (2)$$

Where,

F and F' = Strength theory function of Principal stress;

$\sigma_1, \sigma_2$  and  $\sigma_3$  = Principal stress;

$\sigma_s, \sigma_c$  and  $\tau_s$  = Stretching, compression and shear yield stress of material respectively;

B = Degree coefficient of the material fracture influenced by intermediate principal stress;

A = Tension and compression ratio of the material.

2) *Ultimate Bearing Capacity Of Axial Compression:* The cross-section of the concrete filled square steel tubular stiffened is shown in Figure

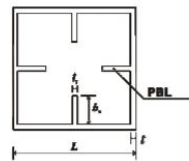


Fig.6: Cross section of Specimen

3) *The Equivalent Transformation Between Square-Circular Concrete Filled Steel Tubular:* There are a large number of experimental studies, which show that the concrete, which is filled by the square steel is a high impact, keeping in a more concrete filled with steel tube, primarily, on their own, to make concrete. This is because, rectangular, steel doesn't have a consistent a binding to the underlying concrete. The installed pipe, the wall shear connectors, the more anchor points, then the local bending can be prevented by effectively and efficiently. In this case, the applications of effective, practical barrier, the coefficient, differential stress, create a level playing field for the thrill. The process in which the equation by 3:

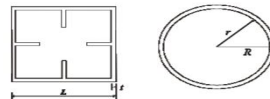


Fig.7: Equivalent Section

$$\begin{aligned} (L^2 - \pi R^2) & \\ (L - 2t)^2 &= \pi R^2 \\ r &= (L - 2t) / \sqrt{\pi} \\ t' &= 2t / \sqrt{\pi} \dots\dots\dots (3) \end{aligned}$$

Where,

L is side of square steel tube

t is wall thickness of square steel tube

R is outside diameter of equivalent circular steel

r is inside diameter of equivalent circular steel

t is wall thickness of equivalent circular steel

D. The Effective Constraint Coefficient Of Concrete

The effective restriction-area and ineffective restriction-area of core-concrete are separated in Figure 3.17

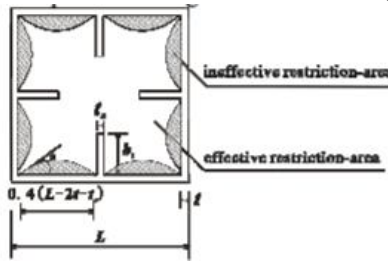


Fig.8: Effective and Ineffective Confined Areas Of Core Concrete

Where  $\theta$  is corner cut

$$\tan \theta = \frac{3.6 t f_y}{\left(\frac{L}{t} - t\right)^2}$$

Suppose ineffective restriction area is  $A_1$ , effective restriction area is  $A_s$ , concrete area is

$$A_c = (L - 2t)^2 - 4b_s t_s$$

$$A_s = 8 \times \frac{(0.4(L-2t-t_s))^2 \tan^2 \theta}{5} \dots \dots \dots (6)$$

$$A_e = A_c - A_1 \dots \dots \dots (7)$$

Define the effective constraint factor is  $k_e$

$$k_e = \frac{A_e}{A_c} = 1 - \frac{A_1}{A_c} = 1 - \frac{1.2 t f_y (L-2t-t_s)^2}{\left(\frac{L}{t} - t\right)^2 [(L-2t)^2 - 4b_s t_s]} \dots \dots \dots (6)$$

E. Binding Effect From Square Steel Tube To Core Concrete

Reference to literature, assuming square steel force diagram is shown in figure 3.19

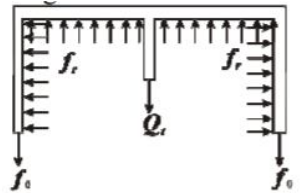


Fig.9: Force Diagram of Steel Tube

Obtained by the mechanical balance

$$2f_s t H + n Q_c = f_r (L - 2t - t_s) \dots \dots \dots (7)$$

Where,

- H is the height of the steel tube;
- n is the number of the holes ;
- Qt is the force suffered from shear connectors,
- Hu Jianhua suggest that  $Q_x = 1.95 A \sqrt{f_c}$
- A is area of the stiffener

The results show that, the effect of the shear connectors has related to s b and d ,

When  $d = bs/2$  it work best. So introduce impact factors

$$\gamma = 1 \frac{|b_s - 2d|}{b_s}$$

$$f_r = \frac{2f_s t + \gamma(1.95 n A \sqrt{f_c})/H}{L - 2t - t_s} \dots \dots \dots (8)$$

So the radial stress of the concrete is given by Eq 9

$$f_r^* = k_e f_r \dots \dots \dots (9)$$



**F. Bearing Capacity of Core Concrete**

Core concrete is in three-dimensional stress state, so  $\sigma_1 = \sigma_2 > \sigma_3$ , suppose  $\sigma_1 = \sigma_2 = f_c'$ , we can get the axial compressive stress of the core concrete from the unified strength theory

$$\sigma_3 = f_c + k\sigma_1 \dots\dots\dots (10)$$

Where  $\sigma_3$  is axial compressive stress of concrete;  $f_c$  is compressive strength of concrete prism;  $k = (1 + \sin \varphi) / (1 - \sin \varphi)$ ,  $k$  changes between 1.0 to 7.0

Axial compressive strength of effectively confined concrete is taken eq 11

$$f_{ce} = f_c + k f_c' \dots\dots\dots (11)$$

So the carrying capacity of effectively confined concrete is given by Eq. 12

$$N_{ce} = A_s f_{ce} - k_s [(L - 2t)^2 - 4b_s t_s] (f_c + k f_c') \dots\dots\dots (12)$$

For the non-effectively confined concrete, because of the decline of the constraints, introduce into the concrete strength reduction factor

$$\gamma_u = 1.67D^{-0.112}$$

Where,

$D$  is outside diameter of equivalent circular pipe. Its axial compressive strength is give by Eq.13:

$$f_{cd} = f_c + \gamma_u k f_c' \dots\dots\dots (13)$$

So the carrying capacity of non-effectively confined concrete is give by Eq.14:

$$N_{cd} = A_s f_{cd} = (1 - k_s) [(L - 2t)^2 - 4b_s t_s] (f_c + \gamma_u k f_c') \dots\dots\dots (14)$$

**G. Ultimate Bearing Capacity Of Concrete Filled Stiffened Square Steel Tubular Short Column**

The axial bearing capacity of square concrete-filled steel tubular stub column with PBL was constitutive of the bearing capacity of square steel tube, PBL stiffeners and core concrete, given by Eq.15:

$$N = A_s f_y + N_{ce} + N_{cd} \dots\dots\dots (15)$$

Where  $A_s$  is area of the square steel tube and the PBL stiffeners. Because there are openings on PBL stiffeners, the carrying capacity will be reduced, so introduce into the bearing capacity reduction factor

$$\gamma_s = \left(1 - \frac{d}{b_s}\right)$$

Where,  $d$  is the hole diameter,  $b_s$  is height of rib So,

$$A_s = 4 \left[ t(L - t) + \left(1 - \frac{d}{b_s}\right) t_s b_s \right]$$

Finally, the axial bearing capacity of square concrete-filled steel tubular stub column is given by Eq. 16:

$$N = 4 \left[ t(L - t) + \left(1 - \frac{d}{b_s}\right) t_s b_s \right] \left[ (L - 2t)^2 - 4b_s t_s \right] \left[ f_c + k f_c' \left[ \gamma_u + k_s (1 - \gamma_u) \right] \right] \dots\dots\dots (16)$$

**IV.RESULT AND DISCUSSION**

**A. Effect of Stiffening Scheme**

**1) Effect of L/B**

**Table 1: Load Carrying Capacity of Square Column**

Sr. No	Specimen (mm)	Size of Square (mm)		Length (mm)	L/B	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		Predicted ultimate strength (KN)	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		B	t			L	as	bs	ts	hs				
1	US 1	80	5	650	8.125						333.75		353.51	
2	SSOS 2	80	5	650	8.125	40	650	5	5	5	483.48	30.97	440.42	19.73
3	SSOS 3	80	5	650	8.125	40	650	5	10	5	512.87	5.73	522.26	15.67
4	SSOS 4	80	5	650	8.125	40	650	5	15	5	541.85	5.35	567.13	7.91
5	SSES 5	80	5	650	8.125	40	650	5	5	5	512.87	5.35	463.49	18.27
6	SSES 6	80	5	650	8.125	40	650	5	10	5	570.42	10.09	583.54	20.57
7	SSES 7	80	5	650	8.125	40	650	5	15	5	626.33	8.93	651.07	10.37
8	FSOS 8	80	5	650	8.125	14	650	5	5	5	713.66	12.24	765.88	14.99
9	FSOS 9	80	5	650	8.125	14	650	5	10	5	954.89	25.26	1010.1	24.18
10	FSOS 10	80	5	650	8.125	14	650	5	15	5	1177.36	18.90	1202	15.97
11	QPS 11	80	5	650	8.125	35	650	5	5	5	502.47	57.32	526.74	56.18
12	QPS 12	80	5	650	8.125	35	650	5	10	5	550.20	8.68	530.24	0.66
13	QPS 13	80	5	650	8.125	35	650	5	15	5	596.88	7.82	630.42	15.89
14	SVSES 14	80	5	650	8.125	40	650	5	5	5	512.87	14.07	543.5	13.79
15	SVSES 15	80	5	650	8.125	40	650	5	10	5	570.42	10.09	592.9	8.33
16	SVSES 16	80	5	650	8.125	40	650	5	15	5	626.33	8.93	652.19	9.09
17	ISEC 17	80	5	650	8.125	28.78	650	5	5	5	528.23	15.66	593.14	9.05
18	ISEC 18	80	5	650	8.125	28.78	650	5	10	5	600.49	12.03	647.06	8.33
19	ISEC 19	80	5	650	8.125	28.78	650	5	15	5	670.47	10.44	711.77	9.09
20	US 20	80	5	850	10.625						333.75		347.72	
21	SSOS 21	80	5	850	10.625	40	850	5	5	5	478.80	30.29	474.02	26.64
22	SSOS 22	80	5	850	10.625	40	850	5	10	5	503.61	4.93	507.88	6.67
23	SSOS 23	80	5	850	10.625	40	850	5	15	5	528.10	4.64	580.65	12.53
24	SSES 24	80	5	850	10.625	40	850	5	5	5	503.61	4.64	520.17	10.42
25	SSES 25	80	5	850	10.625	40	850	5	10	5	552.27	8.81	529.62	1.78
26	SSES 26	80	5	850	10.625	40	850	5	15	5	599.68	7.91	571.2	7.28
27	FSOS 27	80	5	850	10.625	14	850	5	5	5	661.81	9.39	636.98	10.33
28	FSOS 28	80	5	850	10.625	14	850	5	10	5	855.60	22.65	828.08	23.08
29	FSOS 29	80	5	850	10.625	14	850	5	15	5	1035.04	17.34	1035.1	20.00
30	QPS 30	80	5	850	10.625	35	850	5	5	5	494.49	52.23	510.75	50.66
31	QPS 31	80	5	850	10.625	35	850	5	10	5	534.48	7.48	586.19	12.87
32	QPS 32	80	5	850	10.625	35	850	5	15	5	573.67	6.83	585.24	0.16
33	SVSES 33	80	5	850	10.625	40	850	5	5	5	503.61	12.21	502.2	14.19
34	SVSES 34	80	5	850	10.625	40	850	5	10	5	552.27	8.81	561.28	10.53
35	SVSES 35	80	5	850	10.625	40	850	5	15	5	599.68	7.91	596.36	5.88
36	ISEC 36	80	5	850	10.625	28.78	850	5	5	5	515.35	14.06	535.51	10.20
37	ISEC 37	80	5	850	10.625	28.78	850	5	10	5	575.27	10.42	571.21	6.25
38	ISEC 38	80	5	850	10.625	28.78	850	5	15	5	633.44	9.18	639.41	10.67
39	US 39	80	5	1050	13.125						333.75		382.04	
40	SSOS 40	80	5	1050	13.125	40	1050	5	5	5	475.90	29.87	451.08	15.31
41	SSOS 41	80	5	1050	13.125	40	1050	5	10	5	497.87	4.41	482.75	6.56
42	SSOS 42	80	5	1050	13.125	40	1050	5	15	5	519.58	4.18	538.52	10.36
43	SSES 43	80	5	1050	13.125	40	1050	5	5	5	497.87	4.18	424.77	21.12
44	SSES 44	80	5	1050	13.125	40	1050	5	10	5	541.04	7.98	561.34	24.33
45	SSES 45	80	5	1050	13.125	40	1050	5	15	5	583.19	7.23	631.34	11.09
46	FSOS 46	80	5	1050	13.125	14	1050	5	5	5	629.71	7.39	624.08	1.15
47	FSOS 47	80	5	1050	13.125	14	1050	5	10	5	794.13	20.70	802.38	22.22
48	FSOS 48	80	5	1050	13.125	14	1050	5	15	5	946.94	16.14	936.11	14.29
49	QPS 49	80	5	1050	13.125	35	1050	5	5	5	489.54	48.30	454	51.50
50	QPS 50	80	5	1050	13.125	35	1050	5	10	5	524.75	6.71	512.91	11.49
51	QPS 51	80	5	1050	13.125	35	1050	5	15	5	559.30	6.18	546.22	6.10
52	SVSES 52	80	5	1050	13.125	40	1050	5	5	5	497.87	10.98	488.56	10.56
53	SVSES 53	80	5	1050	13.125	40	1050	5	10	5	541.04	7.98	547.52	10.77
54	SVSES 54	80	5	1050	13.125	40	1050	5	15	5	583.19	7.23	577.39	5.17
55	ISEC 55	80	5	1050	13.125	28.78	1050	5	5	5	507.37	13.00	504.01	12.71
56	ISEC 56	80	5	1050	13.125	28.78	1050	5	10	5	559.65	9.34	556.37	9.41
57	ISEC 57	80	5	1050	13.125	28.78	1050	5	15	5	610.52	8.33	612.01	9.09
<b>Average Error</b>												<b>13.48</b>		<b>14.29</b>

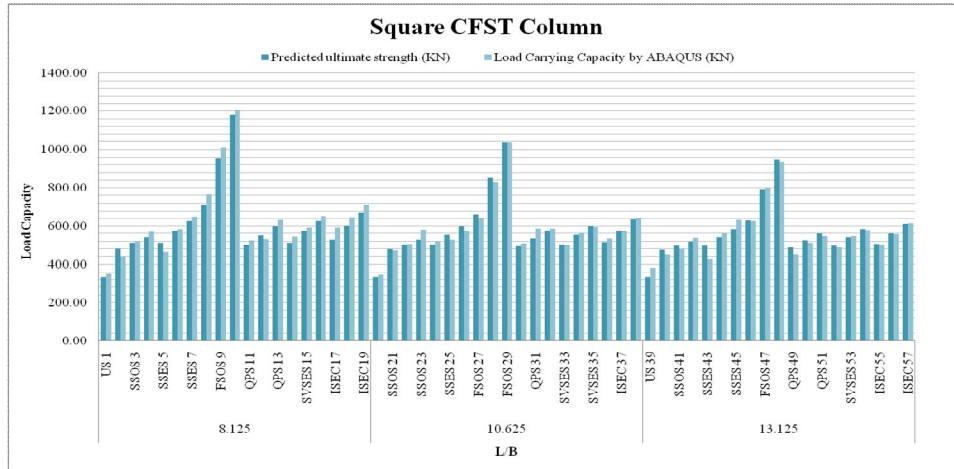


Fig.10: Load Carrying capacity of Square Section

Table 2: Load Carrying Capacity of Rectangular Column

Sr. No	Specimen (mm)	Size of Square (mm)			Length (mm)	L/B	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		Predicted ultimate strength (KN)	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		B	D	t			as	bs	ts	hs	ts				
1	US 1	50	100	5	650	13						311.5		335.25	
2	SSES 2	50	100	5	650	13	50	650	5	5	5	421.17	26.04	484.68	30.83
3	SSES 3	50	100	5	650	13	50	650	5	10	5	442.39	4.80	489.09	0.90
4	SSES 4	50	100	5	650	13	50	650	5	15	5	463.29	4.51	525.61	6.95
5	TSOS 5	50	100	5	650	13	41	650	5	5	5	447.44	3.42	493.21	6.16
6	TSOS 6	50	100	5	650	13	41	650	5	10	5	493.66	9.36	502.29	1.81
7	TSOS 7	50	100	5	650	13	41	650	5	15	5	538.27	8.29	506.99	0.93
8	QPS 8	50	100	5	650	13	30	650	5	5	5	471.89	12.33	489.00	3.55
9	QPS 9	50	100	5	650	13	30	650	5	10	5	540.75	12.73	569.95	14.20
10	QPS 10	50	100	5	650	13	30	650	5	15	5	606.19	10.80	644.32	11.54
11	PRS 11	50	100	5	650	13	35	650	5	5	5	502.99	17.02	510.36	20.79
12	PRS 12	50	100	5	650	13	35	650	5	10	5	598.86	16.01	589.61	13.44
13	PRS 13	50	100	5	650	13	35	650	5	15	5	687.23	12.86	694.24	15.07
14	SVSES14	50	100	5	650	13	50	650	5	5	5	442.39	35.63	450.37	35.13
15	SVSES15	50	100	5	650	13	50	650	5	10	5	483.86	8.57	502.14	10.31
16	SVSES16	50	100	5	650	13	50	650	5	15	5	524.01	7.66	538.17	6.69
17	ISEC17	50	100	5	650	13	62.93	650	5	5	5	437.67	16.48	449.54	16.47
18	ISEC18	50	100	5	650	13	62.93	650	5	10	5	474.69	7.80	487.36	7.76
19	ISEC19	50	100	5	650	13	62.93	650	5	15	5	510.66	7.04	524.62	7.10
20	US 20	50	100	5	850	17						311.50		394.81	
21	SSES 21	50	100	5	850	17	50	850	5	5	5	418.43	25.55	432.65	8.75
22	SSES 22	50	100	5	850	17	50	850	5	10	5	436.99	4.25	459.41	5.82
23	SSES 23	50	100	5	850	17	50	850	5	15	5	455.30	4.02	506.02	9.21
24	TSOS 24	50	100	5	850	17	41	850	5	5	5	440.84	3.17	497.48	1.69
25	TSOS 25	50	100	5	850	17	41	850	5	10	5	480.85	8.32	512.64	2.96
26	TSOS 26	50	100	5	850	17	41	850	5	15	5	519.63	7.46	538.63	4.83
27	QPS 27	50	100	5	850	17	30	850	5	5	5	460.71	11.34	464.94	13.68
28	QPS 28	50	100	5	850	17	30	850	5	10	5	519.19	11.26	508.61	8.59
29	QPS 29	50	100	5	850	17	30	850	5	15	5	575.05	9.71	584.02	12.91

30	PRS 30	50	100	5	850	17	35	850	5	5	5	487.98	15.14	485.30	16.90
31	PRS 31	50	100	5	850	17	35	850	5	10	5	570.62	14.48	562.42	13.71
32	PRS 32	50	100	5	850	17	35	850	5	15	5	647.51	11.88	650.36	13.52
33	SVSES33	50	100	5	850	17	50	850	5	5	5	436.99	32.51	432.47	33.50
34	SVSES34	50	100	5	850	17	50	850	5	10	5	473.35	7.68	484.31	10.70
35	SVSES35	50	100	5	850	17	50	850	5	15	5	508.72	6.95	516.01	6.14
36	ISEC36	50	100	5	850	17	62.93	850	5	5	5	433.38	14.81	425.61	17.52
37	ISEC37	50	100	5	850	17	62.93	850	5	10	5	466.34	7.07	472.78	9.98
38	ISEC38	50	100	5	850	17	62.93	850	5	15	5	498.51	6.45	503.14	6.03
39	US 39	50	100	5	1050	21						311.50		372.16	
40	SSES 40	50	100	5	1050	21	50	1050	5	5	5	416.73	25.25	446.26	16.60
41	SSES 41	50	100	5	1050	21	50	1050	5	10	5	433.64	3.90	476.14	6.28
42	SSES 42	50	100	5	1050	21	50	1050	5	15	5	450.35	3.71	501.36	5.03
43	TSOS 43	50	100	5	1050	21	41	1050	5	5	5	436.76	3.02	452.51	9.74
44	TSOS 44	50	100	5	1050	21	41	1050	5	10	5	472.92	7.65	475.04	4.74
45	TSOS 45	50	100	5	1050	21	41	1050	5	15	5	508.08	6.92	516.20	7.97
46	QPS 46	50	100	5	1050	21	30	1050	5	5	5	453.79	10.69	450.61	12.71
47	QPS 47	50	100	5	1050	21	30	1050	5	10	5	505.84	10.29	510.69	11.76
48	QPS 48	50	100	5	1050	21	30	1050	5	15	5	555.78	8.99	560.32	8.86
49	PRS 49	50	100	5	1050	21	35	1050	5	5	5	478.70	13.87	482.14	13.95
50	PRS 50	50	100	5	1050	21	35	1050	5	10	5	553.13	13.46	547.34	11.91
51	PRS 51	50	100	5	1050	21	35	1050	5	15	5	622.92	11.20	630.15	13.14
52	SVSES52	50	100	5	1050	21	50	1050	5	5	5	433.64	30.39	419.17	33.48
53	SVSES53	50	100	5	1050	21	50	1050	5	10	5	466.85	7.11	460.36	8.95
54	SVSES54	50	100	5	1050	21	50	1050	5	15	5	499.25	6.49	504.20	8.69
55	ISEC55	50	100	5	1050	21	62.93	1050	5	5	5	430.72	13.73	435.08	13.71
56	ISEC56	50	100	5	1050	21	62.93	1050	5	10	5	461.17	6.60	471.14	7.65
57	ISEC57	50	100	5	1050	21	62.93	1050	5	15	5	490.99	6.07	501.36	6.03
Average Error												11.35		11.25	

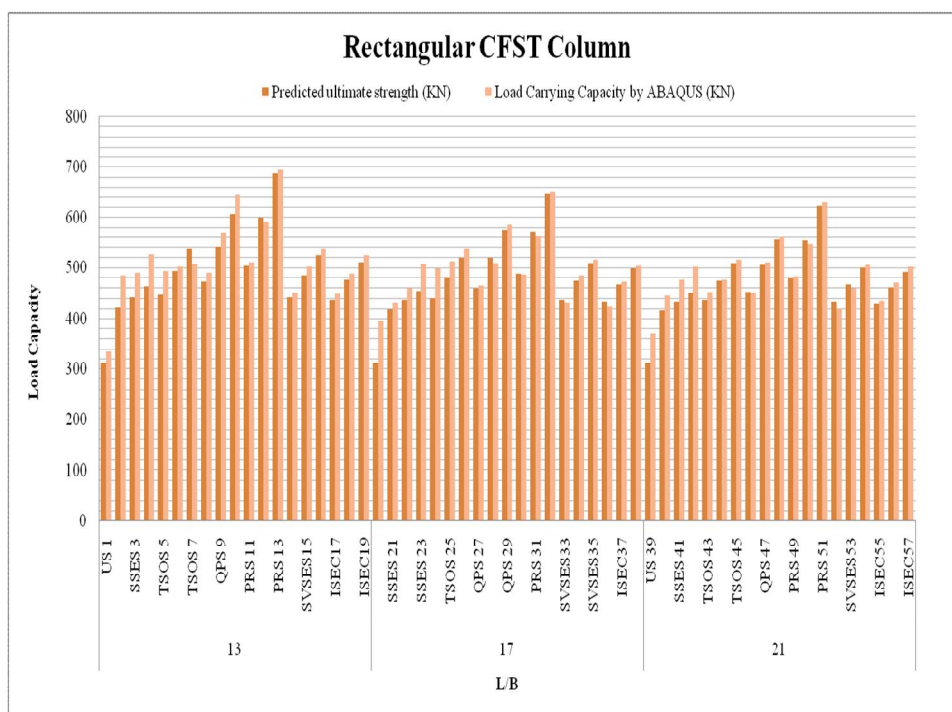


Fig.11: Load Carrying Capacity of Rectangular Section

Table 3: Load Carrying Capacity of Circular Column

Sr. No	Specimen	Size of Circle (mm)		Length (mm)	D/L	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		Predicted ultimate strength (KN)	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		D	t			L	as	bs	ts	hs				
1	US 1	80	5	650	8.125						261.99		331.61	
2	SSOS 2	80	5	650	8.125	60	650	5	5	5	396.44	33.91	428.30	22.58
3	SSOS 3	80	5	650	8.125	60	650	5	10	5	435.64	9.00	462.14	7.32
4	SSOS 4	80	5	650	8.125	60	650	5	15	5	473.75	8.04	510.93	9.55
5	DSEC 5	80	5	650	8.125	65	650	5	5	5	432.57	8.69	507.01	0.77
6	DSEC 6	80	5	650	8.125	65	650	5	10	5	504.96	14.34	551.35	8.04
7	DSEC 7	80	5	650	8.125	65	650	5	15	5	573.32	11.92	613.00	10.06
8	SVSES 8	80	5	650	8.125	60	650	5	5	5	396.44	30.85	399.78	34.78
9	SVSES 9	80	5	650	8.125	60	650	5	10	5	435.64	9.00	464.13	13.86
10	SVSES 10	80	5	650	8.125	60	650	5	15	5	473.75	8.04	520.90	10.90
11	US 11	80	5	850	10.625						261.99		248.08	
12	SSOS 12	80	5	850	10.625	60	850	5	5	5	391.62	33.10	387.06	35.91
13	SSOS 13	80	5	850	10.625	60	850	5	10	5	426.25	8.13	447.07	13.42
14	SSOS 14	80	5	850	10.625	60	850	5	15	5	460.06	7.35	455.48	1.85
15	DSEC 15	80	5	850	10.625	65	850	5	5	5	423.91	7.86	450.77	1.03
16	DSEC 16	80	5	850	10.625	65	850	5	10	5	488.58	13.24	497.36	9.37
17	DSEC 17	80	5	850	10.625	65	850	5	15	5	550.17	11.19	587.30	15.31
18	SVSES 18	80	5	850	10.625	60	850	5	5	5	391.62	28.82	417.10	28.98
19	SVSES 19	80	5	850	10.625	60	850	5	10	5	426.25	8.13	446.28	6.54
20	SVSES 20	80	5	850	10.625	60	850	5	15	5	460.06	7.35	494.88	9.82
21	US 21	80	5	1050	13.125						261.99		300.56	
22	SSOS 22	80	5	1050	13.125	60	1050	5	5	5	388.63	32.59	366.70	18.04
23	SSOS 23	80	5	1050	13.125	60	1050	5	10	5	420.44	7.57	489.41	25.07
24	SSOS 24	80	5	1050	13.125	60	1050	5	15	5	451.58	6.89	452.53	7.54
25	DSEC 25	80	5	1050	13.125	65	1050	5	5	5	418.54	7.32	402.41	11.08
26	DSEC 25	80	5	1050	13.125	65	1050	5	10	5	478.45	12.52	466.28	13.70
27	DSEC 27	80	5	1050	13.125	65	1050	5	15	5	535.85	10.71	545.35	14.50
28	SVSES 28	80	5	1050	13.125	60	1050	5	5	5	388.63	27.47	409.82	24.85
29	SVSES 29	80	5	1050	13.125	60	1050	5	10	5	420.44	7.57	437.93	6.42
30	SVSES 30	80	5	1050	13.125	60	1050	5	15	5	451.58	6.89	457.53	4.28
Average Error												14.02		13.54

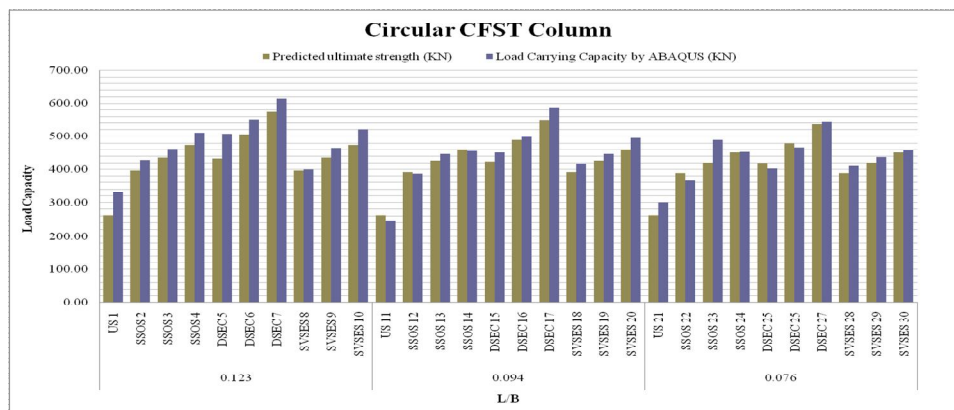


Fig.12: Load Carrying Capacity of Circular Section

2) Effect of HS/TS

Table 4: Load Carrying Capacity of Square Column

Sr. No	Specimen (mm)	Size of Square (mm)		Length (mm)	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		hs/ts	Predicted ultimate strength (KN)	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		B	t		L	as	bs	ts	hs					
1	SSOS 2	80	5	650	40	650	5	5	5	1	483.48		440.42	
2	SSES 5	80	5	650	40	650	5	5	5	1	512.87	5.73	463.49	4.98
3	FSOS 8	80	5	650	14	650	5	5	5	1	713.66	28.14	765.88	39.48
4	QPS 11	80	5	650	35	650	5	5	5	1	502.47	29.59	526.74	31.22
5	SVSES 14	80	5	650	40	650	5	5	5	1	512.87	2.03	543.5	3.08
6	ISEC 17	80	5	650	28.78	650	5	5	5	1	528.23	2.91	593.14	8.37
7	SSOS 21	80	5	850	40.00	850	5	5	5	1	478.80	9.36	474.02	20.08
8	SSES 24	80	5	850	40	850	5	5	5	1	503.61	4.93	520.17	8.87
9	FSOS 27	80	5	850	14	850	5	5	5	1	661.81	23.90	636.98	18.34
10	QPS 30	80	5	850	35	850	5	5	5	1	494.49	25.28	510.75	19.82
11	SVSES 33	80	5	850	35	850	5	5	5	1	503.61	1.81	502.2	1.67
12	ISEC 36	80	5	850	28.78	850	5	5	5	1	515.35	2.28	535.51	6.22
13	SSOS 40	80	5	1050	40	1050	5	5	5	1	475.90	7.65	451.08	15.77
14	SSES 43	80	5	1050	40	1050	5	5	5	1	497.87	4.41	424.77	5.83
15	FSOS 46	80	5	1050	14	1050	5	5	5	1	629.71	20.94	624.08	31.94
16	QPS 49	80	5	1050	35	1050	5	5	5	1	489.54	22.26	454	27.25
17	SVSES 52	80	5	1050	40	1050	5	5	5	1	497.87	1.67	488.56	7.07
18	ISEC 55	80	5	1050	28.78	1050	5	5	5	1	507.37	1.87	504.01	3.07
19	SSOS 3	80	5	650	40	650	5	10	5	2	512.87	1.07	522.26	3.49
20	SSES 6	80	5	650	40	650	5	10	5	2	570.42	10.09	583.54	10.50
21	FSOS 9	80	5	650	14	650	5	10	5	2	954.89	40.26	1010.1	42.23
22	QPS 12	80	5	650	35	650	5	10	5	2	550.20	42.38	530.24	47.51
23	SVSES 15	80	5	650	40	650	5	10	5	2	570.42	3.54	592.9	10.57
24	ISEC 18	80	5	650	28.78	650	5	10	5	2	600.49	5.01	647.06	8.37
25	SSOS 22	80	5	850	40	850	5	10	5	2	503.61	16.13	507.88	21.51
26	SSES 25	80	5	850	40	850	5	10	5	2	552.27	8.81	529.62	4.10
27	FSOS 28	80	5	850	14	850	5	10	5	2	855.60	35.45	828.08	36.04
28	QPS 31	80	5	850	35	850	5	10	5	2	534.48	37.53	586.19	29.21
29	SVSES 34	80	5	850	40	850	5	10	5	2	552.27	3.22	561.28	4.25
30	ISEC 37	80	5	850	28.78	850	5	10	5	2	575.27	4.00	571.21	1.74
31	SSOS 41	80	5	1050	40	1050	5	10	5	2	497.87	13.45	482.75	15.49
32	SSES 44	80	5	1050	40	1050	5	10	5	2	541.04	7.98	561.34	14.00
33	FSOS 47	80	5	1050	14	1050	5	10	5	2	794.13	31.87	802.38	30.04
34	QPS 50	80	5	1050	35	1050	5	10	5	2	524.75	33.92	512.91	36.08
35	SVSES 53	80	5	1050	40	1050	5	10	5	2	541.04	3.01	547.52	6.32
36	ISEC 56	80	5	1050	28.78	1050	5	10	5	2	559.65	3.33	556.37	1.59
37	SSOS 4	80	5	650	40	650	5	15	5	3	541.85	3.18	567.13	1.90
38	SSES 7	80	5	650	40	650	5	15	5	3	626.33	13.49	651.07	12.89
39	FSOS 10	80	5	650	14	650	5	15	5	3	1177.36	46.80	1202	45.83
40	QPS 13	80	5	650	35	650	5	15	5	3	596.88	49.30	630.42	47.55
41	SVSES 16	80	5	650	40	650	5	15	5	3	626.33	4.70	652.19	3.34
42	ISEC 19	80	5	650	28.78	650	5	15	5	3	670.47	6.58	711.77	8.37
43	SSOS 23	80	5	850	40	850	5	15	5	3	528.10	21.24	580.65	18.42
44	SSES 26	80	5	850	40	850	5	15	5	3	599.68	11.94	571.2	1.63
45	FSOS 29	80	5	850	14	850	5	15	5	3	1035.04	42.06	1035.1	44.82
46	QPS 32	80	5	850	35	850	5	15	5	3	573.67	44.57	585.24	43.46
47	SVSES 35	80	5	850	40	850	5	15	5	3	599.68	4.34	596.36	1.86
48	ISEC 38	80	5	850	28.78	850	5	15	5	3	633.44	5.33	639.41	6.73
49	SSOS 42	80	5	1050	40	1050	5	15	5	3	519.58	17.98	538.52	15.78
50	SSES 45	80	5	1050	40	1050	5	15	5	3	583.19	10.91	631.34	14.70
51	FSOS 48	80	5	1050	14	1050	5	15	5	3	946.94	38.41	936.11	32.56
52	QPS 51	80	5	1050	35	1050	5	15	5	3	559.30	40.94	546.22	41.65
53	SVSES 54	80	5	1050	40	1050	5	15	5	3	583.19	4.10	577.39	5.40
54	ISEC 57	80	5	1050	28.78	1050	5	15	5	3	610.52	4.48	612.01	5.66
Average Error											16.34		17.52	

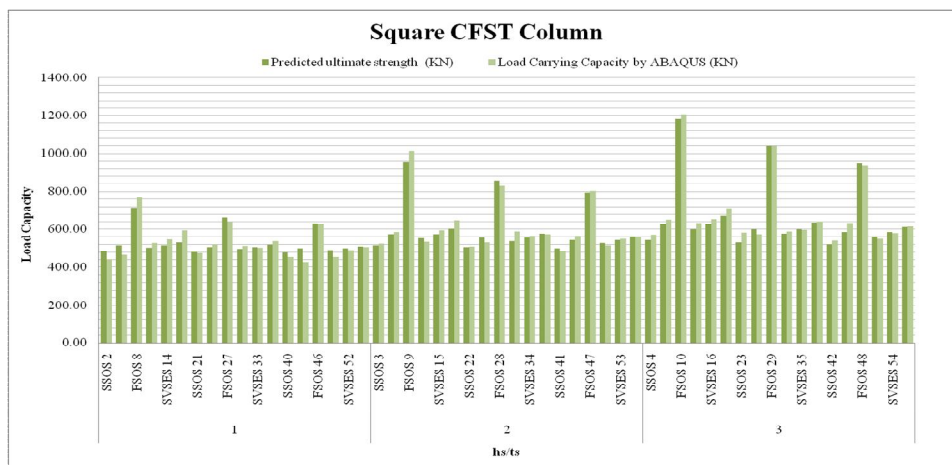


Fig.13: Load Carrying Capacity of Square Column

Table 5: Load Carrying Capacity of Rectangular Column

Sr. No	Specimen (mm)	Size of Square (mm)			Length h (mm)	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		hs/ts	Predicted ultimate strength (KN) Nuc	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		B	D	t		L	as	bs	ts	hs					
1	SSES 2	50	100	5	650	50	650	5	5	5	1	421.17		484.68	
2	TSOS 5	50	100	5	650	41	650	5	5	5	1	447.44	5.87	493.21	1.73
3	QPS 8	50	100	5	650	30	650	5	5	5	1	471.89	5.18	489.00	0.85
4	PRS 11	50	100	5	650	35	650	5	5	5	1	502.99	6.18	510.36	4.19
5	SVSES14	50	100	5	650	50	650	5	5	5	1	442.39	12.05	450.37	11.75
6	ISEC17	50	100	5	650	62.93	650	5	5	5	1	437.67	1.07	449.54	0.18
7	SSES 21	50	100	5	850	50	850	5	5	5	1	418.43	4.40	432.65	3.76
8	TSOS 24	50	100	5	850	41	850	5	5	5	1	440.84	5.09	497.48	13.03
9	QPS 27	50	100	5	850	30	850	5	5	5	1	460.71	4.31	464.94	6.54
10	PRS 30	50	100	5	850	35	850	5	5	5	1	487.98	5.59	485.30	4.20
11	SVSES33	50	100	5	850	50	850	5	5	5	1	436.99	10.45	432.47	10.89
12	ISEC36	50	100	5	850	62.93	850	5	5	5	1	433.38	0.83	425.61	1.59
13	SSES 40	50	100	5	1050	50	1050	5	5	5	1	416.73	3.84	446.26	4.63
14	TSOS 43	50	100	5	1050	41	1050	5	5	5	1	436.76	4.59	452.51	1.38
15	QPS 46	50	100	5	1050	30	1050	5	5	5	1	453.79	3.75	450.61	0.42
16	PRS 49	50	100	5	1050	35	1050	5	5	5	1	478.70	5.20	482.14	6.54
17	SVSES52	50	100	5	1050	50	1050	5	5	5	1	433.64	9.41	419.17	13.06
18	ISEC55	50	100	5	1050	62.93	1050	5	5	5	1	430.72	0.67	435.08	3.66
19	SSES 3	50	100	5	650	50	650	5	10	5	2	442.39	2.64	489.09	11.04
20	TSOS 6	50	100	5	650	41	650	5	10	5	2	493.66	10.38	502.29	2.63
21	QPS 9	50	100	5	650	30	650	5	10	5	2	540.75	8.71	569.95	11.87
22	PRS 12	50	100	5	650	35	650	5	10	5	2	598.86	9.70	589.61	3.33
23	SVSES15	50	100	5	650	50	650	5	10	5	2	483.86	19.20	502.14	14.84
24	ISEC18	50	100	5	650	62.93	650	5	10	5	2	474.69	1.90	487.36	2.94
25	SSES 22	50	100	5	850	50	850	5	10	5	2	436.99	7.94	459.41	5.73
26	TSOS 25	50	100	5	850	41	850	5	10	5	2	480.85	9.12	512.64	10.38
27	QPS 28	50	100	5	850	30	850	5	10	5	2	519.19	7.38	508.61	0.79
28	PRS 31	50	100	5	850	35	850	5	10	5	2	570.62	9.01	562.42	9.57
29	SVSES34	50	100	5	850	50	850	5	10	5	2	473.35	17.04	484.31	13.89
30	ISEC37	50	100	5	850	62.93	850	5	10	5	2	466.34	1.48	472.78	2.38

31	SSES 41	50	100	5	1050	50	1050	5	10	5	2	433.64	7.01	476.14	0.71
32	TSOS 44	50	100	5	1050	41	1050	5	10	5	2	472.92	8.31	475.04	0.23
33	QPS 47	50	100	5	1050	30	1050	5	10	5	2	505.84	6.51	510.69	6.98
34	PRS 50	50	100	5	1050	35	1050	5	10	5	2	553.13	8.55	547.34	6.70
35	SVSES53	50	100	5	1050	50	1050	5	10	5	2	466.85	15.60	460.36	15.89
36	ISEC56	50	100	5	1050	62.93	1050	5	10	5	2	461.17	1.22	471.14	2.29
37	SSES 4	50	100	5	650	50	650	5	15	5	3	463.29	0.46	525.61	10.36
38	TSOS 7	50	100	5	650	41	650	5	15	5	3	538.27	13.93	506.99	3.54
39	QPS 10	50	100	5	650	30	650	5	15	5	3	606.19	11.20	644.32	21.31
40	PRS 13	50	100	5	650	35	650	5	15	5	3	687.23	11.79	694.24	7.19
41	SVSES16	50	100	5	650	50	650	5	15	5	3	524.01	23.75	538.17	22.48
42	ISEC19	50	100	5	650	62.93	650	5	15	5	3	510.66	2.55	524.62	2.52
43	SSES 23	50	100	5	850	50	850	5	15	5	3	455.30	10.84	506.02	3.55
44	TSOS 26	50	100	5	850	41	850	5	15	5	3	519.63	12.38	538.63	6.05
45	QPS 29	50	100	5	850	30	850	5	15	5	3	575.05	9.64	584.02	7.77
46	PRS 32	50	100	5	850	35	850	5	15	5	3	647.51	11.19	650.36	10.20
47	SVSES35	50	100	5	850	50	850	5	15	5	3	508.72	21.43	516.01	20.66
48	ISEC38	50	100	5	850	62.93	850	5	15	5	3	498.51	2.01	503.14	2.49
49	SSES 42	50	100	5	1050	50	1050	5	15	5	3	450.35	9.66	501.36	0.35
50	TSOS 45	50	100	5	1050	41	1050	5	15	5	3	508.08	11.36	516.20	2.87
51	QPS 48	50	100	5	1050	30	1050	5	15	5	3	555.78	8.58	560.32	7.87
52	PRS 51	50	100	5	1050	35	1050	5	15	5	3	622.92	10.78	630.15	11.08
53	SVSES54	50	100	5	1050	50	1050	5	15	5	3	499.25	24.77	504.20	19.99
54	ISEC57	50	100	5	1050	62.93	1050	5	15	5	3	490.99	1.68	501.36	0.56
Average Error													8.27		7.01

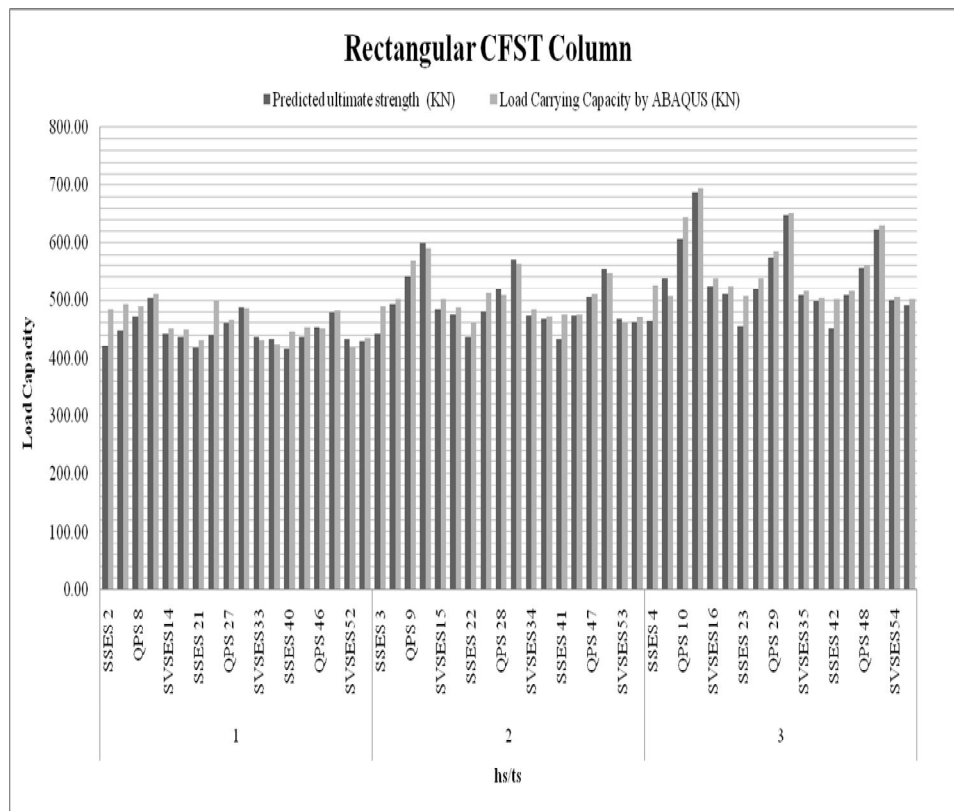


Fig.14: Load Carrying Capacity of Rectangular Column



Table 6: Load Carrying Capacity of Circular Column

Sr. No	Specimen	Size of Circle (mm)		Length (mm)	Space of Binding Arrangement (mm)			Size of Stiffener (mm)		h s/ ts	Predicted Ultimate Strength (KN)	% Error	Load Carrying Capacity by ABAQUS (KN)	% Error
		D	t		L	as	bs	ts	hs					
		D	t	L	as	bs	ts	hs	ts		Nu		KN	
1	SSOS 2	80	5	650	60	650	5	5	5	1	396.44		428.3	
2	DSEC 5	80	5	650	65	650	5	5	5	1	432.57	8.35	507.01	15.52
3	SVSES 8	80	5	650	60	650	5	5	5	1	396.44	8.35	399.78	21.15
4	SSOS 12	80	5	850	60	850	5	5	5	1	391.62	1.22	387.06	3.18
5	DSEC 15	80	5	850	65	850	5	5	5	1	423.91	7.62	450.77	14.13
6	SVSES 18	80	5	850	60	850	5	5	5	1	391.62	7.62	417.1	7.47
7	SSOS 22	80	5	1050	60	1050	5	5	5	1	388.63	0.76	366.7	12.08
8	DSEC 25	80	5	1050	65	1050	5	5	5	1	418.54	7.15	402.41	8.87
9	SVSES 28	80	5	1050	60	1050	5	5	5	1	388.63	7.15	409.82	1.81
10	SSOS 3	80	5	650	60	650	5	10	5	2	435.64	10.79	462.14	11.32
11	DSEC 6	80	5	650	65	650	5	10	5	2	504.96	13.73	551.35	16.18
12	SVSES 9	80	5	650	60	650	5	10	5	2	435.64	13.73	464.13	15.82
13	SSOS 13	80	5	850	60	850	5	10	5	2	426.25	2.15	447.07	3.68
14	DSEC 16	80	5	850	65	850	5	10	5	2	488.58	12.76	497.36	10.11
15	SVSES 19	80	5	850	60	850	5	10	5	2	426.25	12.76	446.28	10.27
16	SSOS 23	80	5	1050	60	1050	5	10	5	2	420.44	1.36	489.41	8.81
17	DSEC 25	80	5	1050	65	1050	5	10	5	2	478.45	12.12	466.28	4.73
18	SVSES 29	80	5	1050	60	1050	5	10	5	2	420.44	12.12	437.93	6.08
19	SSOS 4	80	5	650	60	650	5	15	5	3	473.75	11.25	510.93	14.29
20	DSEC 7	80	5	650	65	650	5	15	5	3	573.32	17.37	613	16.65
21	SVSES 10	80	5	650	60	650	5	15	5	3	473.75	17.37	520.9	15.02
22	SSOS 14	80	5	850	60	850	5	15	5	3	460.06	2.89	455.48	12.56
23	DSEC 17	80	5	850	65	850	5	15	5	3	550.17	16.38	587.3	22.45
24	SVSES 20	80	5	850	60	850	5	15	5	3	460.06	16.38	494.88	15.74
25	SSOS 24	80	5	1050	60	1050	5	15	5	3	451.58	1.84	452.53	8.56
26	DSEC 27	80	5	1050	65	1050	5	15	5	3	535.85	15.73	545.35	17.02
27	SVSES 30	80	5	1050	60	1050	5	15	5	3	451.58	15.73	457.53	16.10
Average Error											9.79		11.91	

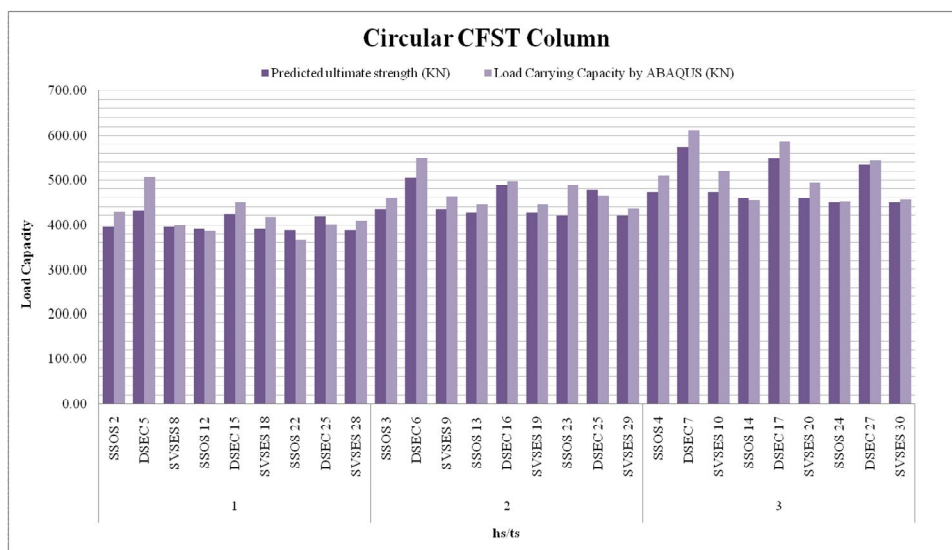


Fig.15: Load Carrying Capacity of Circular Column

## V. CONCLUSIONS

The axial compression 57-square, 57, rectangular, and 30 of circular CFST specimens are conducted. The largest width to thickness ratio reaches 3. Finite element analysis was subsequently carried out to determine the effect of stiffening scheme on the axial load distribution of concrete in steel tube. The following Results are concluded in influence of shear connector.

### A. Effect of L/B

- 1) The local buckling appearance of the stiffened columns is relatively different from those of the unstiffened columns. The tube buckling is less pronounced when the number of stiffeners or the stiffener height increases.
- 2) The L/B ratio increase axial load capacity reduces rapidly.
- 3) In this study observed analytical maximum load carrying capacity of 650 mm length four stiffener on opposite side on stiffener size 15x5 mm in square CFST column are 1202 KN as compare to other size of stiffener and length of column.
- 4) In rectangular 650mm length CFST column are maximum load capacity of PBL rib stiffener in 15x5 mm size stiffener is 694.24KN as compare to other.
- 5) In circular 650mm length CFST column are maximum load capacity of double stiffener on each side in 15x5 mm size stiffener is 613 KN as compare to other.
- 6) The Average % error in square, rectangular, circular CFST column is predicted by analytical models is 14.29% 11.25%, 13.54% respectively.

### B. Effect of hs/ts

- 1) The maximum axial load increases stably as the stiffener height and the number of stiffeners increase.
- 2) In this study observed analytical maximum load carrying capacity of 650 mm length four stiffener on opposite side on stiffener size 15x5 mm in square CFST column are 1202 KN as compare to other size of stiffener and length of column.
- 3) In rectangular 650mm length CFST column are maximum load capacity of Quarterly placed stiffener in 15x5 mm size stiffener is 644.32 KN as compare to other.
- 4) In circular 650mm length CFST column are maximum load capacity of double stiffener on each side in 15x5 mm size stiffener is 613 KN as compare to other.
- 5) The Average % error in square, rectangular, circular CFST column is predicted by analytical models is 17.52% 7.01%, 11.91% respectively.

### C. Comparison of Numerical and Analytical Result

- 1) Using the Numerical results and FEA results through regression analyses, a new model for predicting the axial load capacity is developed for stiffened CFST columns.
- 2) The average ratio of predicting strength to nominal strength of composite column is measured ultimate strength reaches 1.12, 1.02, 1 square, rectangular, circular column respectively.
- 3) Comparison of Average % error in numerical result and analytical result in square, rectangular, circular CFST column is predicted by analytical models is 4% 4.08%, 6.07% respectively.
- 4) The proposed model has low accuracy and the variability in the prediction of the ultimate strength as compared to the existing model.
- 5) Numerical and analytical result results shows good agreement as the % error is within limit less than 10%

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