Structural Behavior & Design of Concrete Filled Steel Tube as Column as Per EC-4

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Abstract - An efficient method of resisting lateral loads for buildings in the 60-plus-story range is to position columns farthest from the building center with shear-resisting elements in between. This idea has given rise to a whole new category of composite systems characterized by their use of super columns interconnected across the building with a shear-resisting web-like framing. Large-diameter pipes or tubes filled with high-strength concrete in the range of 41 to 138 MPa. Generally, neither longitudinal nor transverse reinforcement is used within the steel pipe or tube. So this paper is based upon the design of CFST as per EUROCODE -4 & by using Ansys'11 as a analysis tool, the structural behavior of CFST (designed by myself) is shown in this paper. So from this it concluded that double skin concrete filled steel tube is efficient one.

Index Terms - Concrete Filled Steel Tube, EuroCode-4, DSCFT, ANSYS'11, Axial loading, behaviors & comparison.

◆

1 INTRODUCTION

With the increasing use of composite construction worldwide, there is a growing interest in utilizing Concrete - Filled Tubes (CFTs) as a primary column member. The interest develops from the fact that properties of steel and concrete in the CFTs are fully utilized, so that the strength, stiffness and ductility of the structures constructed from CFTs can be enhanced simultaneously. Since the function of longitudinal reinforcement and transverse confinement can be achieved due to presence of the steel tubes, the traditional longitudinal and transverse reinforcement may be eliminated. This type of Column also maintains sufficient ductility when high strength concrete is used. CFT columns can replace conventional structural columns like reinforced concrete, structural steel with reinforced concrete and structural steel alone with enhanced performance and at the same time reducing costs to a minimum. It is especially useful in high-rise buildings where high strength is required [6]. Table 1 shows cost comparison of different CFT.

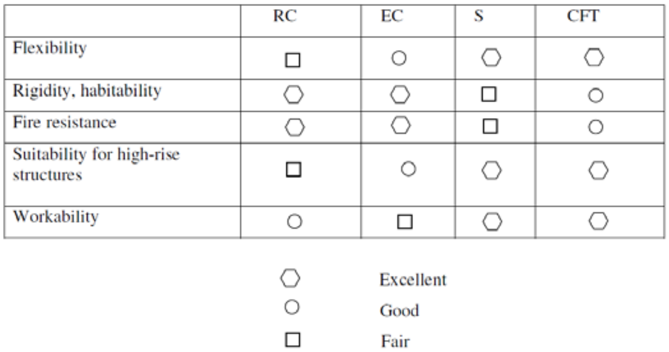
Table- 1: Comparison of Column Cost

Type of Column	RCC	Concrete with steel Erection Column	Concrete Encased Steel Strut	Tube Filled with Reinforced Concrete	Steel Tube Filled with Concrete	Full Steel Column
Relative Cost 10 levels	1	1.22	1.53	1.14	1.1	2.27
Relative cost 30 levels	1	1.13	1.85	1.11	1.02	2.61

Source: Webb J. and Peyton J.J., in the Institutions of Engineers Australia, Structural Engineering Conference 1990

1.1 Advantages of CFT

Table- 2: Comparisons between CFT, RC, EC& S



	RC	EC	S	CFT
Flexibility	□	○	⬡	⬡
Rigidity, habitability	⬡	⬡	□	○
Fire resistance	⬡	⬡	□	○
Suitability for high-rise structures	□	○	⬡	⬡
Workability	○	□	⬡	⬡

⬡	Excellent
○	Good
□	Fair

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2 COMPOSITE COLUMN DESIGN (AXIAL LOADING)

2.1 Method of Design

At present, there is no Indian Standard covering Composite Columns. The method of design suggested in this seminar largely follows EC4, which incorporates the latest research on composite construction. Isolated symmetric columns having uniform cross sections in non-sway frames may be designed by the Simplified design method described in this seminar. This method also adopts the European buckling curves for steel columns as the basis of column design. It is formulated in such a way that only hand calculation is required in practical design. This method cannot be applied to sway columns [8].

Resistance of cross-section to compression: - The plastic resistance of an encased steel section or concrete filled rectangular or square section is given by the sum of the resistances of the components as follows: [7].

$$\left.\begin{aligned} &Pp = Aa.fy / \gamma\ a + \alpha c.A\ c.\ (fck)cy / \gamma\ c + A\ s\ .f\ sk / \gamma\ s\text{.......(for rect./sq. CFT)} \\ &Pp = Aa.fy / \gamma\ a + \alpha c.A\ c.\ [0.80^*\ (fck)cu] / \gamma\ c + A\ s\ .f\ sk / \gamma\ \square s\cdots\text{(encased column)} \end{aligned}\right\} (1)$$

where,

Aa, A c and A s = the areas of the steel section, the concrete and the reinforcing steel respectively.

fy , (fck)cy and f sk= the yield strength of the steel section, the characteristic compressive strength (cylinder) of the concrete, and the yield strength of the reinforcing steel respectively.

(fck)cu = the characteristic compressive strength (cube) of the concrete

αc = strength coefficient for concrete, which is 1.0 for concrete filled tubular sections, and 0.85 for fully or partially concrete encased steel sections.

Therefore, Pp = Aa py + A c pck + A s psk

At this stage it should be pointed out that the Indian Standards for composite construction (IS: 11384-1985) does not make any specific reference to composite columns. The provisions contained in IS: 456 - 2000 are often invoked for design of composite structures. Extension of IS: 456 - 2000 to composite columns will result in the following equation:

Pp = Aa py + A c pck + A s psk

where, py = 0.87fy pck=0.4(fck)cu psk=0.67fy

Concrete filled circular tubular sections: Special Provisions: - The method described above is valid for rectangular and square tubular sections. For composite columns using circular tubular sections, there is an increased resistance of concrete due to the confining effect of the circular tubular section. However, this effect on the resistance enhancement of concrete is significant only in stocky columns. For composite columns with a non-dimensional slenderness of □λ *< 0.5*, or where the eccentricity, *e* of the applied load does not exceed the value *d*/*10*, (where *d* is the outer dimension of the circular tubular section) this effect has to be considered. [5]

The eccentricity *e*, is defined as follows:

$$e = \frac{M}{P} \leq \frac{d}{10} \qquad (2)$$

where,

e is the eccentricity

M is the maximum applied design moment

P is the applied design load

The plastic compression resistance of concrete filled circular tubular sections is calculated by using two coefficients *ηa* and *ηc* as given below.

$$\left.\begin{aligned} &N_{pl.Rd} = A_a.\eta_a.\frac{f_y}{\gamma_{Ma}} + A_c.\frac{f_{ck}}{\gamma_c}\left[1+\eta_c.\frac{t}{d}\frac{f_y}{f_{ck}}\right] + A_s.\frac{f_{sk}}{\gamma_s} \\ &\qquad\qquad (1) \\ &\eta_a = 0{,}25\left(3+3\overline{\lambda}\right) \\ &\eta_c = 4{,}9 - 18{,}5\overline{\lambda} + 17\overline{\lambda}^2 \end{aligned}\right\} (3)$$

Table- 3: Coefficient values of *ηa* & *ηc* from EC4

λ	0	0.1	0.2	0.3	0.4	>/=0.5
η_a	4.9	3.22	1.88	0.88	0.22	0
η_c	0.75	0.8	0.85	0.9	0.95	1

2.2 Non-dimensional slenderness

The plastic resistance to compression of a composite cross-section *Pp*, represents the maximum load that can be applied to a short column. For slender columns with low elastic critical load, overall buckling may be critical. In a typical buckling curve for an ideal column as shown in Fig. (*a*), the horizontal line represents *Pp*, while the curve represents *Pcr*, which is a function of the column slenderness. These two curves limit the compressive resistance of ideal column. For convenience, column strength curves are plotted in non dimensionalised form as shown in Fig. (*b*) the buckling resistance of a column may be expressed as a proportion χ of the plastic resistance to compression, *Pp* thereby non-dimensioning the vertical axis of Fig. (*a*), where χ □s called the reduction factor. The horizontal axis may be non dimensionalised similarly by *Pcr* as shown in Fig. 1 [8].

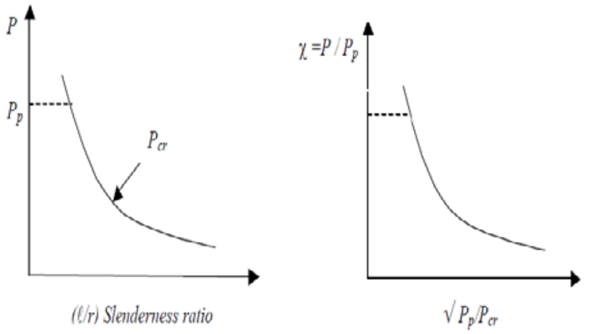


Fig.-1: Idealized column buckling curve and Non-dimensionalised column curve

For using the European buckling curves, the non-dimensional slenderness of the column should be first evaluated as follows:

$$\overline{\lambda} = \sqrt{\frac{P_{pu}}{P_{cr}}} \qquad (4)$$

where,

Ppu= plastic resistance of the cross-section to compression with $\gamma a = \gamma\, c = \gamma s = 1.0$

Pcr = the elastic buckling load of the column.

Local buckling of steel sections: - To prevent premature local buckling, the width to thickness ratio of the steel sections in compression must satisfy the following limits:

$$\left.\begin{array}{ll} \bullet\ \dfrac{d}{t} \le 85\,\epsilon^2 & \text{for concrete filled circular tubular sections} \\[2ex] \bullet\ \dfrac{h}{t} \le 50\,\epsilon & \text{for concrete filled rectangular tubular sections} \\[2ex] \bullet\ \dfrac{b}{t_f} \le 43\,\epsilon & \text{for partially encased I sections} \end{array}\right\} \qquad (5)$$

where,

$$\epsilon = \sqrt{\frac{250}{f_y}}$$

fy is the yield strength of the steel section in *N/mm²(MPa)*.

For fully encased steel sections, no verification for local buckling is necessary as the concrete surrounding effectively prevents local buckling. Local buckling may be critical in some concrete filled rectangular tubular sections with large *h/t* ratios.

Effective elastic flexural stiffness: - Composite columns may fail in buckling and one important parameter for the buckling design of composite columns is its elastic critical buckling load (Euler Load), *P*cr, which is defined as follows:

$$P_{cr} = \frac{\pi^2 (EI)_e}{\ell^2} \qquad (6)$$

where,

(EI) e is the effective elastic flexural stiffness of the composite column *l* is the effective length of the column, which may be conservatively taken as system length *L* for an isolated non-sway composite column.

However, the value of the flexural stiffness may decrease with time due to creep and shrinkage of concrete. Two design rules for the evaluation of the effective elastic flexural stiffness of composite columns are given below.

Short term loading: - The effective elastic flexural stiffness, (EI)e , is obtained by adding up the flexural stiffness of the individual components of the cross-section:

(EI) e = Ea Ia + 0.8 Ecd Ic + Es Is

where,

Ia , Ic and Is = the moments of inertia of the steel section, the concrete (assumed uncracked) about the axis of bending considered respectively.

Ea and Es = the moduli of elasticity of the steel section

0.8 Ecd Ic = the effective stiffness of the concrete; the factor 0.8 is an Empirical multiplier (determined by a calibration exercise to give good agreement with test results). Note Ic is the moment of inertia about the centroid of the uncracked column section.

Ecd = Ecm / γ c*

Ecm = the secant modulus of the concrete

γ c* = reduced to 1.35 for the determination of the effective stiffness of concrete according to Euro code 2.

Resistance of members to axial compression: - For each of the principal axes of the column, the designer should check that

$$P \le \chi P_p \qquad (7)$$

where,

Pp = the plastic resistance to compression of the cross-section

χ = the reduction factor due to column buckling and is a function of the non dimensional slenderness of the composite column.

The European buckling curves illustrated in Fig.2 are proposed to be used for composite columns. They are selected according to the types of the steel sections and the axis of bending [7].

curve a: for concrete filled tubular sections

curve b: for fully or partially concrete encased I-sections buckling about the strong axis of the steel sections (x-x axis).

curve c: for fully and partially concrete encased I-sections buckling about the weak axis of the steel sections (y-y axis).

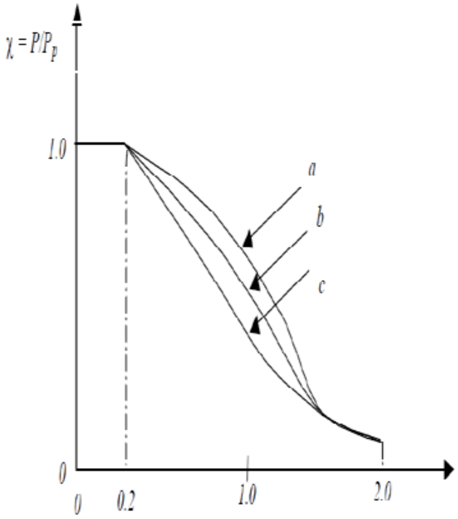


Fig. - 2: European buckling curves

Table- 4: Imperfection factor α for the buckling curves

European Buckling Curve	a	b	c
Imperfection Factor (α)	0.21	0.34	0.49

The isolated non-sway composite columns need not be checked for buckling, if anyone of the following conditions is satisfied:

1. The axial force in the column is less than 0.1 Pcr where Pcr is the elastic buckling load of the column
2. The non-dimensional slenderness is less than 0.2.

OR, Reduction factor (χ) can be calculated from following equation:

$$\phi_x = 0.5\left[1+\alpha_x\left(\bar{\lambda}_x - 0.2\right)+\bar{\lambda}_x^{\,2}\right] \quad (8)$$

$$\phi_y = 0.5\left[1+\alpha_y\left(\bar{\lambda}_y - 0.2\right)+\bar{\lambda}_y^{\,2}\right] \quad (9)$$

$$\chi_y = \frac{1}{\left(\phi_y + \left\{\phi_y^{\,2} - \bar{\lambda}_y^{\,2}\right\}^{\frac{1}{2}}\right)} \quad (10)$$

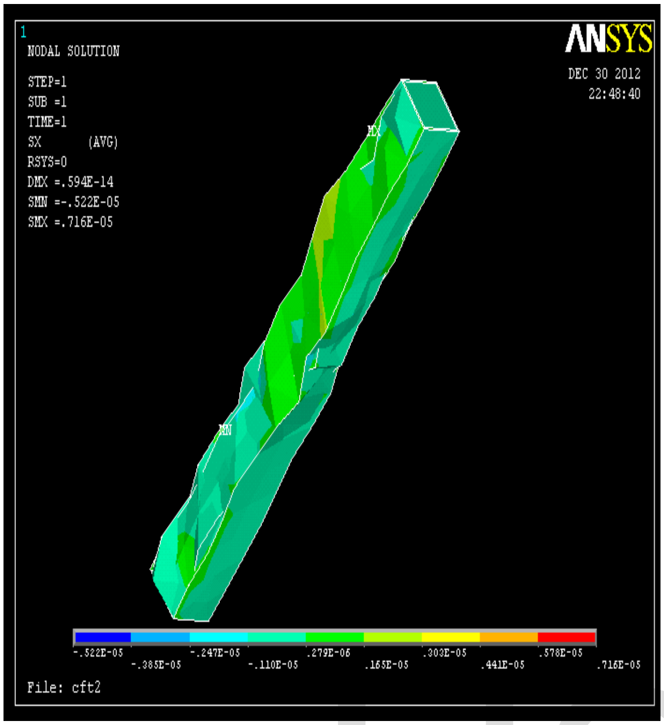
From the above design steps, the following designs results are carried out by me, which are tabulated in Table 5.

Table- 5: Comparison of buckling load for different CFT cases

SR. NO.	**SHAPE OF COLUMN & DESCRIPTION**	**WT/M**	**CROSS-SECTION AREA (mm²)**	**BUCKLING LOAD (KN)**
		kg/m		
1	Square concrete filled steel tube (350X350)	380.39	122500	4567.06
2	Double skin concrete filled steel tube (350X350) & (250X250)	317.16	71052.16	6254.2
3	Concrete filled circular steel tube (with same perimeter as square tube) dia.450mm	470.32	159043.12	5174.59
4	Concrete filled circular steel tube (with same area as square tube)dia. 395 mm	369.34	122541.74	4528.74

3 Ansys'11 Result

3.1 Square concrete filled steel tube (350X350)



```
1
NODAL SOLUTION

STEP=1
SUB =1
TIME=1
SX       (AVG)
RSYS=0
DMX =.594E-14
SMN =-.522E-05
SMX =.716E-05

File: cft2
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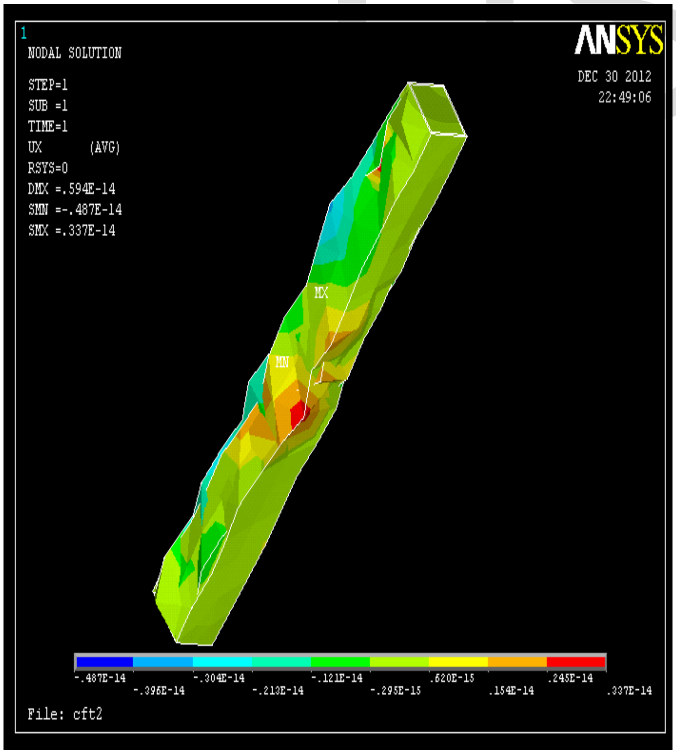
Fig.-3a: Stress in X- direction



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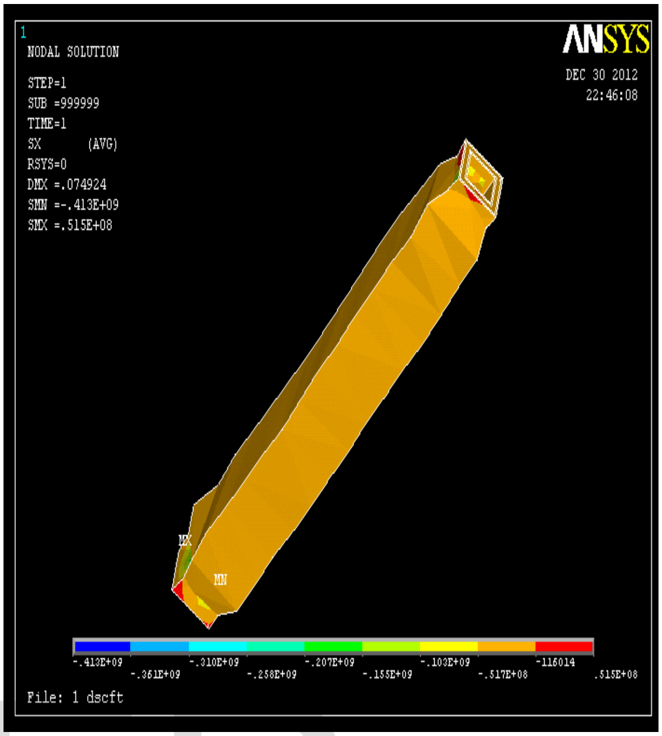
1
NODAL SOLUTION

STEP=1
SUB =1
TIME=1
UX       (AVG)
RSYS=0
DMX =.594E-14
SMN =-.487E-14
SMX =.337E-14

File: cft2
```

Fig.-3b: Displacement in X- direction

3.2 Double skin concrete filled steel tube (350X350) & (250X250)



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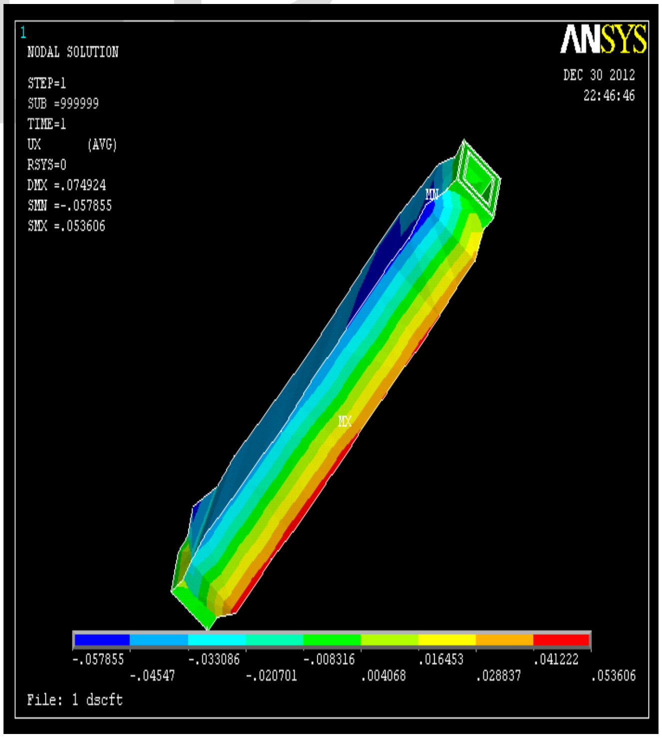
1
NODAL SOLUTION

STEP=1
SUB =999999
TIME=1
SX       (AVG)
RSYS=0
DMX =.074924
SMN =-.413E+09
SMX =.515E+08

File: 1 dscft
```

Fig. - 4a: Stress in X- direction



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1
NODAL SOLUTION

STEP=1
SUB =999999
TIME=1
UX       (AVG)
RSYS=0
DMX =.074924
SMN =-.057855
SMX =.053606

File: 1 dscft
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Fig. -4b: Displacement in X- direction

3.3 Concrete filled circular steel tube (with same perimeter as square tube) dia.450mm

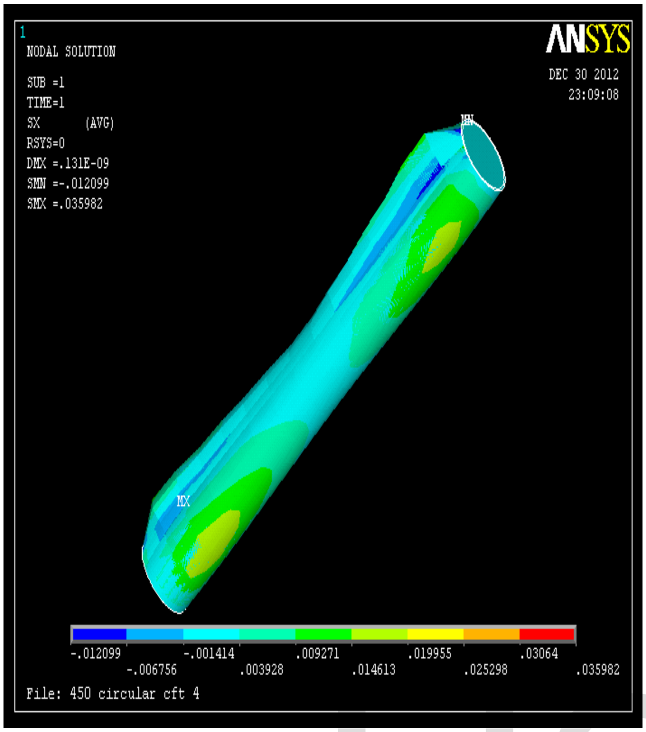

Fig. - 5a: Stress in X- direction

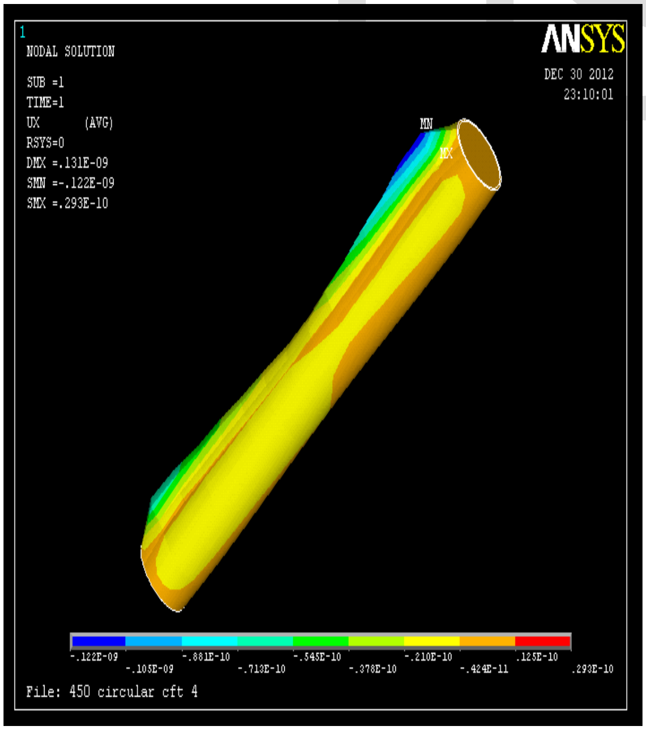

Fig.- 5b: Displacement in X- direction

3.4 Concrete filled circular steel tube (with same area as square tube)dia. 395 mm

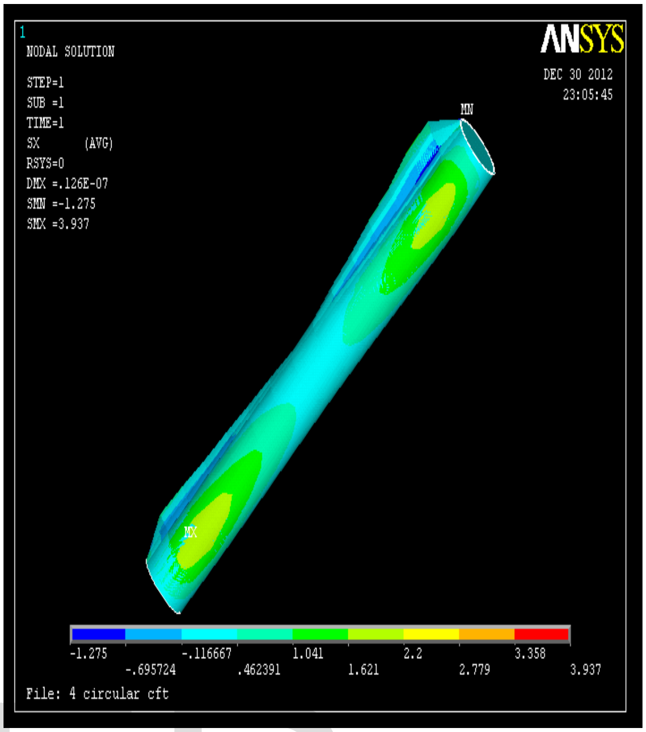

Fig. - 6a: Stress in X- direction

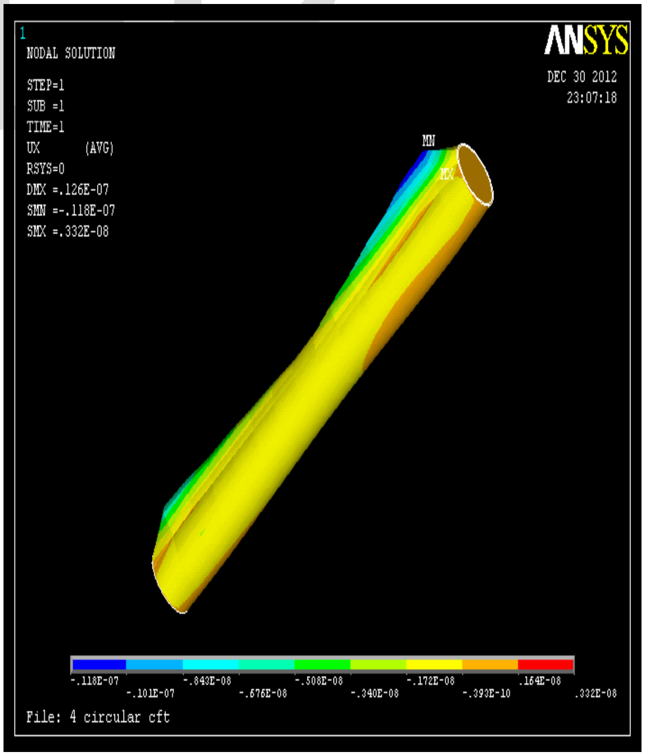

Fig.- 6b: Displacement in X- direction

3. RESULTS

I. From the comparison of circular and square CFST columns, we can see that the advantages of circular CFST columns are far exceed than that of square one. This is got from the comparisons of the behaviors, structures, manufactures and fire-proofing:
   - The load carrying capacity under axial compression is more than that of square cross-section. From this the steel and concrete can be saved.
   - The axial and bending rigidities are more.
   - The behavior of anti-seismic is well. Manufacture is more convenient.
   - The fireproofing coat can be saved.

I. The buckling failure can be avoided and the load carrying capacity can be increased by lowering the slenderness ratio for hollow columns and RC columns.

II. From my design & calculation, I conclude that, load carrying capacity of DSCFT column is maximum & also wt./m and c/s area is less than other 3 cases so, design case 2 is efficient one.

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