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Experimental And Analytical Study Of Plate Formed Box Column Subjected To Axial Compression And Biaxial Bending

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ABSTRACT

Experimental and analytical investigation of plate welded box column is the objective of this study. Manual design of rectangular or square welded box columns are worked out according to IS 800:2007. The box column is subjected to axial compression and biaxial bending are checked for the combined loading and the interaction ratio is designed to be less than one. Different loading conditions, support conditions and cross sections are tried out for the manual design. 3mm and 6mm thick steel plates are welded together along with a base plate of side 130mm to form box columns of size 100mmx100mm is subjected to axial compression. Four models were made with different specifications. These specimens were tested in an Universal Testing machine which could test up to a maximum load of 100 tonnes. The

maximum load carrying capacity of the box columns comes out to be 55 tonnes (maximum) according to IS 800:2007.
The Experimental results are tabulated and compared with the manual design. The variations in results are studied and a conclusion is made to propose to the Bureau of Indian Standards regarding the increase in strength of welded box columns. This idea is further supported by FEM analysis in STAAD.Pro which also yields similar results. The box column is plate modelled in STAAD.Pro, the plates are subjected to uniform pressure at the top which is equivalent to the yield point load from the experimental investigation which in turn produced stress greater than the characteristics compressive strength of the box column thereby supplementing the experimental investigation.

Keywords: welded box column, universal testing machine, IS 800:2007, STAAD.Pro, FEM analysis, Plate modelling

INTRODUCTION
General
Built up box columns are more efficient than wide flange columns in areas of high seismic risk because of two principal advantages large bending, torsional stiffness and strength. The Complexities involved when a Box column is subjected to axial compression and biaxial bending will be dealt in this project. Individual plate buckling and the buckling class of box columns are major areas of concern.

Objective
To study the design of welded box columns when subjected to axial compression and biaxial bending. For the study three different methods are used. Manual design, Experimental Investigations and FEM analysis by STAAD Pro software. The results from these investigations are compared and a conclusion is to be drawn. To propose the conclusion drawn to the Bureau of Indian Standards(BIS) for further investigation.

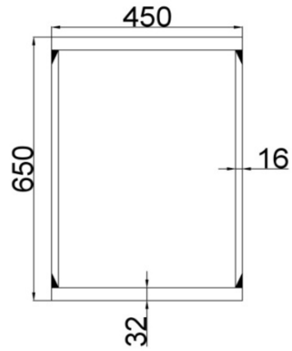
Scope
Hollow, square or rectangular sections, due to the even distribution of the material around their longitudinal axis, represent an efficient structural form. They are often used as compression members in industrial and off shore jacket deck structures. The load-carrying capacity of these sections can be significantly reduced by the interaction between the local buckling of the component plates and the overall buckling of the compressed columns.
The study aims at giving an idea about the advantages of using box columns over other sections of steel. Various researchers have done different methods to determine the strength of the column, but in this study the justification of buckling class of the box columns will be carried out. If in any way we can prove the buckling class of the column to be better than “C”, then the usage of box columns can be promoted to masses. Though the connections in the box columns are difficult, the need of the study is to bring out the advantages of box columns and to formulate a easy and efficient

method to determine the strength of the box column considering the non- linear behaviour of the system.

MANUAL DESIGN

1) Find the design strength in axial compression of plate formed welded Box-Section using steel IS : 262 for which f_y = 250 Mpa having its ends fixed and the unsupported length is 8.5m.

Effective Length ($l_{ez} = l_{ey}$) =
0.65 x 8.5 = 5.525 (Table 11 of IS:800)
□ = √250/250 = 1
b/t_f = 450 – (2x16) /32 = 13.06
This an internal element as per clause 3.7.3



[NOTE : ALL DIMENSIONS ARE IN 'MM' FOR THE FIGURES]

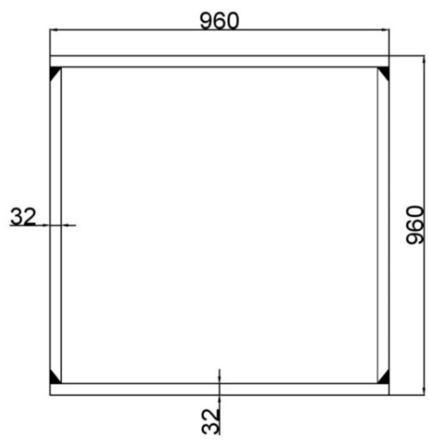
of IS:800. The value of b/t_f = 42□
(table 2 of IS:800). Hence the section is semi compact with regect to web of box section,
d/t_w= 650 – (2x32) / 16 = 36.625 < 42□ for axial compression
Hence based on the width thickness ratio of web also the section fails in semi compact category.

<u>Design Compressive Strength :</u>

$P_d = A_e \times f_{cd}$ (cl. 7. 1. 2 of IS : 800)
A_e = 450 x 650 – 418 x586 = 47552 mm^2
$f_{cd}= (f_y/ \gamma_{mo}) /$ □ + [□ - $\lambda^2]^{0.5}$=
$\chi f_y/ \gamma_{mo} < f_y/$ □$_{mo}$(cl.7.1.2.1 IS:800)
$\lambda = \sqrt{f_y/ f_{cc}}$,
[f_{cc} – Euler's Buckling Stress = $\pi^2 E / (kL/\gamma)^2$]
i.e Radius of gyration about major axis
Z-Z
$r_{zz} = \sqrt{I_{zz}/ A}$
I_{zz}= [(450x650^3 / 12) – (415x586^3 / 12)] = 7009.5x10^6 mm^4

I_{yy} = [(650x450^3 / 12) –(586x418^3 / 12)] = 4638.73x10^6 mm^4
r_{zz}= √7009.5x10^6 / 47552 = 383.94 mm
r_{yy} = √4638.73x106 / 47552 = 312.33 mm
λ_{zz} = 250 x $(5525/383.94)^2$ / π^2 x 2x10^5 = 0.287
λ_{yy} = 250 x $(5525/312.33)^2$ / π^2 x 2x10^5 = 0.5
$\varphi = 0.5 [1+ \alpha (\lambda – 0.2) + \lambda^2]$
To find the imperfection factor α, Buckling class (Table 10 of IS:800)
b/t_f = 13.06 < 30 & d/t_w = 36.625 > 30
Hence for ZZ axis : Buckling class c
for YY axis : Bucling class b
α_{zz} = 0.49; α_{yy} = 0.34
φ_{zz} = 0.5[1+0.49(0.287-0.2) + 0.287^2] = 0.562
φ_{yy} = 0.5[1+0.34(0.5-0.2)+0.5^2] = 0.676
$\chi_{zz} = 1 / [\varphi_{zz}{}^2 + (\varphi_{zz}{}^2 - \lambda_{zz}{}^2)^{0.5}]$
= 1/[0.562 + $(0.562^2–0.287^2)^{0.5}$]= 0.95676
χ_{yy} = 1 / [0.676 +$(0.676^2-0.5^2)^{0.5}$] = 0.8842
f_{cdzz}= χ_{zz}x (f_y/γ_{mo})
= 0.95676 x 250/1.1 = 217.4 Mpa
f_{cdyy}= 0.8842 x 250/1.1 = 201 Mpa
Axial load carrying capacity = 201 x 47552/1000 = 9558 KN (factored)

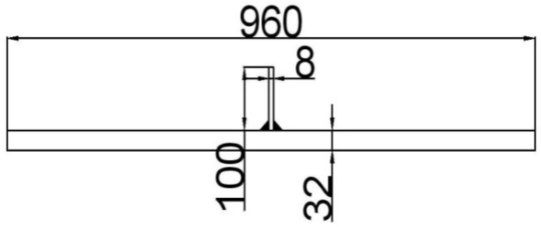
2) Check the capacity of a 960mm square box column fabricated using IS:2062 f_y= 330Mpa steel plates of 32mm thick to take up an axial compression due to dead and imposed loads of 20000 KN. Unsupported length is 15m. Both ends are held in position but without Restraint.
Since both ends are held in position but are without restraint,

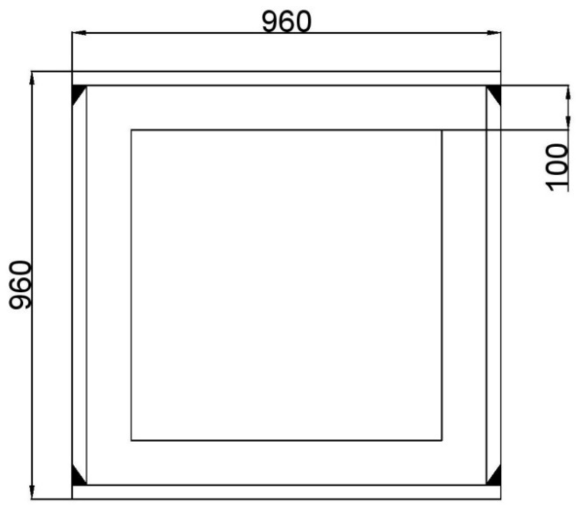


$l_{ez} = l_{ey}$ = KL = 1 x 15 = 15m =15000mm
Ae = 960^2 – $(960 – 2x32)^2$ = 960^2 - 896^2=118784mm^2
Section Clasification for Axial Compression
For Flange b / t_f = (960 – 2x32) / 32 = 28

For Web d/ t_w= 28
 = √250/330 = 0.87
Internal element of (axial compression) Compression flange, (Table 2 of IS:800)
b/t_f< 42 = 42 x 0.87 = 36.54 and also d/t_w. Hence section is semi compact
To find f_{cd}
$I_{zz} = I_{yy}$=[(960 x 960^3 / 12) – (896 x 896^3 / 12)] = 1.707 x 10^{10} mm^4
$r_{zz} = r_{yy}$ = √ I/A = √ 1.707 x10^{10} / 118784 = 379.08 mm
$KLz/r_z = KLy/r_y = l_{ez}/r_z = l_{ey}/r_y$=15000/379.08 = 39057
$\lambda_{zz} = \sqrt{[f_y (KLz/r_z)^2 / \pi^2 E]} = \sqrt{[330 \times 39.57^2 / \pi^2 \times 2\times10^5]}$ = 0.5118
Buckling Class
Box section (b/t_f) > 30 & (h/t_w) > 30
Hence class b (Table 7 of IS:800)
φ_{zz} = 0.5[1+0.34(0.5118-0.2) + 0.5118^2] = 0.684
$\chi_{zz} = 1 / [\varphi_{zz}^2 + (\varphi_{zz}^2 - \lambda_{zz}^2)^{0.5}] = 1 / [0.684 + (0.684^2 – 0.5118^2)^{0.5}]$ = 0.879
$f_{cdzz}= \chi_{zz} x (f_y/\gamma_{mo})$ = 0.879 x 330/1.1 = 263.67 Mpa
p = Ae x f_{cd} = 118784 x (263.67/1000) = 31320 KN
Unfactored load = 31320 / 1.5
= 20880 > 20000 KN
Hence the section is adequate
Note : Please note that the section strength is aqequate. However, the overall strength have to be checked with minimum eccentricity.

3) Check the adequacy of the above column with miminmum eccentricity.
Factored load = 1.5 X 20000 = 30000 KN
Minimum Eccentricity = 100mm from the face of the column (cl.7.3.3.1 of IS:800)
$e_z = e_y$ = 960/2 + 100 = 580mm
$M_z = M_y$ = 30000/2 x 0.58 = 8700KNm
Half the moment is considered since the same section is continued further with (I / l) ratio not varying more than 1.5 (cl.7.3.3.1 of IS:800)
Design Forces and Moments
Pu = 30000KN ;$M_z = M_y$ = 8700 KNm
Let the stiffener Spacing be c/d>= √2
C = √2 x d = √2 x (960 – 2x32) = 1267mm
Let us provide stiffeners @ 1000mm c/c
Is required = 0.75 x 896 x 8^3 = 0.344x10^6 mm^4
Keepwid (d) = 100mm
2x20x32 = 1280
But the stiffener spacing = 1000mm





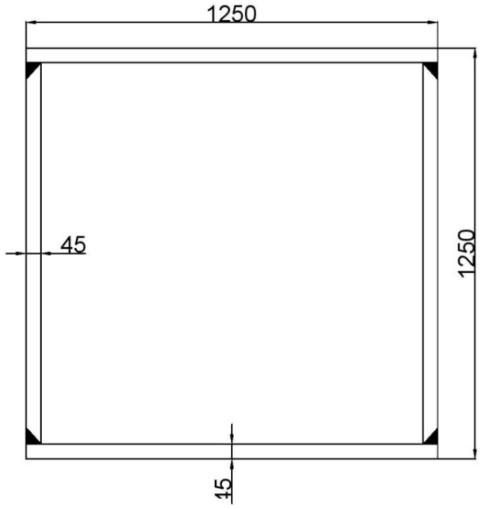
C.G=(1000x32x16+100x8x82) / (1000x32+100x8)
= 577600/32800 = 17.61mm
Is = $[8x100^3/12 + 100x8x64.39^2] + [1000x32^3/12 + 1000x32x1.61]$
= $6.8x10^6$ mm^4>Is required
$N_d = Ag \times f_y / \gamma_{mo}$
= 118784 x 330 / 1.1x1000 = 35635.2KN (cl.9.3 of IS:800)
$\eta = N / N_d$= 30000 / 35635.2 = 0.842 >0.2
<u>To find $M_{dz}$and M_{dy} :</u> (cl.8.2.1.2 of IS:800)
Section is Semi-compact
$\beta_p = Z_e / Z_p$
$Z_e = I / y = 1.0707x10^{10} / (960/2) = 35.5625x10^6$ mm^3
$Z_p = 4 \times [480x32x(480/2)] + 2 \times [896x32x464] = 41.353x10^6$
β_p= 35.5625 / 41.353 = 0.86
$M_{dz}= M_{dy}= \beta_p \times Z_p \times f_y / \gamma_{mo}$
= 0.86 x $41.353x10^6$ x 330 / 1.1 x 10^6 = 10669.07KNm
This shall be less than,
$1.2Z_e f_y / \gamma_{mo}$ = 1.2x35.5625x330 / 1.1 = 12802.5 KNm
$N/N_d + M_y/M_{dy} + M_z/M_{dz} < 1.0$
0.842 + 8700/10669.07 + 8700/10669.07= 2.47 >1.0
Hence the section is inadequate

4) Design a suitable Box section to carry an axial load of 16000 KN with moment on each axis = 6000KNm. Material is IS:2062&f_y = 330Mpa. The loads and moments are due to dead , imposed and wind loads. The unsupported length of the compression member is 12 and its ends are fully restrained.

Assuming stress reduction factor = 0.5

f_{cd} = 0.5x330 = 165 Mpa

<u>Area required for axial load alone</u>



Factored axial load = 1.2x16000 = 19200 KN

[Load factor 1.2, Table 4 of IS:800)

Since moment is also there, we have to provide more area

Let us try this section,

A = 1250^2 - 1160^2 = 216900 mm^2

Effective length l_{ez} = l_{ey} = 0.65 x 12 = 7.8m

ε = √(250/330) = 0.87

b/t_f = (1250 - 2x45) / 45 = 25.77< (42ε = 36.54)

d/t_w = (1250 - 2x45) / 45 = 25.77< (42ε = 36.54)

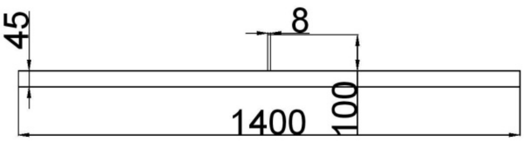
Hence the section is semi compact

Let us provide diaphragm plates (stiffeners) @ c/d>= √2

C = √2 x 1160 =1640.48mm

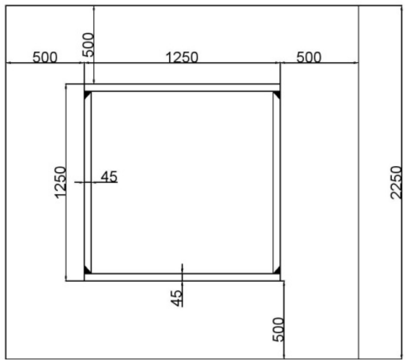
Provide diaphragm plates @ 1400 c/c and the thickness shall be 8mm

Is required = $0.75dt_w^3$ = $0.75 \times 1160 \times 8^3$ = 0.45×10^6 mm^4

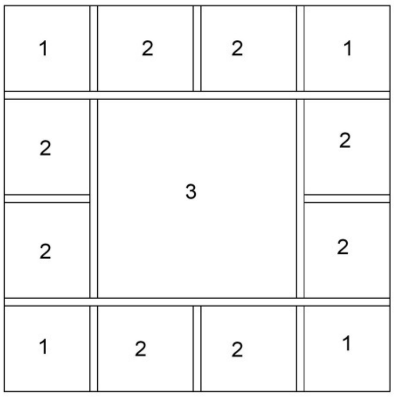


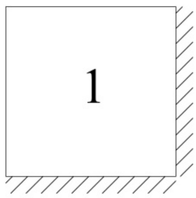
To find C.G:
= [1400x45x22.5 + 100x8x95] / [1400x45 + 100x8]
= 23.409mm
Is = $[(8x100^3/12) + 800x71.59^2] + [(1400x40^3/12) + 1400x40x0.909^2]$
= $15.443x10^6$ mm^4>Is required
$N_d = Ag \times f_y / \gamma_{mo}$ (cl.9.3 of IS:800)
= $216900x330 /1.1x10^3$ = 65070 KN
$\eta = N / N_d$= 19200 / 65070 = 0.295> 0.2
To find $M_{dz}$and M_{dy} : (cl.8.2.1.2 of IS:800)
Section is plastic,
$b/t_f = d/t_w$= 23 < 25.49
$M_{dz}= M_{dy}= \beta_p \times Z_p \times f_y / \gamma_{mo}$
Z_e = I /y
= $[(1250x1250^3/12) – (1160x1160^3/12)]$
/ (1250/2) = $84.102x10^6mm^3$
Z_p = 4 x [625x45x(625/2)] + 2 x [1160x45x580] = $95.71x10^6mm^3$
$M_{dz}= M_{dy}= 95.71x10^6x330 / 1.1x10^6$ = 28712.48KNm
$N/N_d + M_y/M_{dy} + M_z/M_{dz}< 1.0$
= 0.295 + 1.2x6000/28712.48 + 1.2x6000/28712.48
= 0.796> 1.0
Hence the section is adequate.

5) Design the slab base and stiffened base for the above column.
Let us have a square base plate od side ‘a’ mm.
Maximum pressure = $P/A + M_z/Z_z+ M_y/Z_y$
This should be less than the bearing stress on concrete pedestral = 0.6 f_{ck} (cl.7.4.1 of IS:800)
Let us provide concrete grade M_{30}
The bearing strength of concrete pedestral = 0.6x30 = 18 Mpa
$P/A + M_z/Z_z+ M_y/Z_y$
$P/a^2 + M_z/(a^3/6)+ M_y/(a^3/6)$
$P/a^2 + 2(6M/a^3)$
$18000 = 19200/a^2 + 2(6x7200 / a^3)$
$18000a^3$ -19200a – 86400 = 0
Solving we get a= 1.9m
Providing 2250x2250 base plate,
Z of base plate = $2250x2250^2 / 6 = 1898.43x10^6$ mm^3
$q_{max} = P/A + M_z/Z_z+ M_y/Z_y$
=$(19200x10^3/2250^2)$+ $(7200x10^6/1898.43x10^6)$+ $7200x10^6/1898.43x10^6)$
= 3.79 + 3.79 + 3.79 = 11.37<18Mpa
Slab base (cl.7.4.3.1 of IS:800)



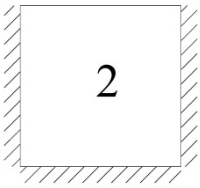
$t = \sqrt{[2.5w(a^2 - 0.3b^2) / f_y/\gamma_{mo}]}$
Whr, w = 11.37Mpa ; a = b = 500mm
$t = \sqrt{[2.5x11.37 (500^2 - 0.3x500^2) / 330/1.1]} = 128.768$ mm
Thickness of base plate required = 140 mm
Stiffened Base:





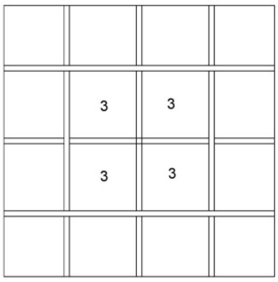
Part nos 1 to 3 are marked based on size and edge condition.

b/a = 1.0; $\rho = 0.0678$
$M = 0.0678 \times 11.37 \times 500^2$
= 209841 Nmm



b/a = 1.0; ρ = 0.06
M = 0.06 x 11.37 x 625^2
= 290156.25 Nmm

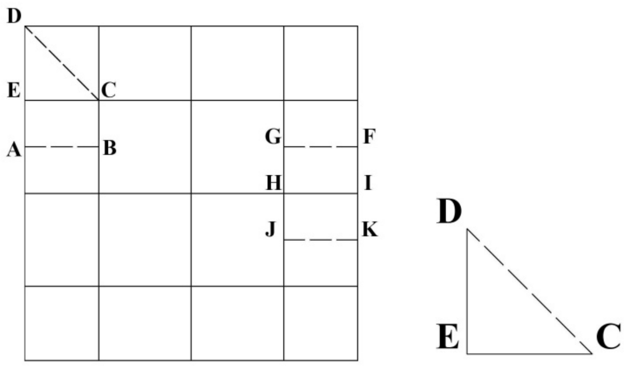
b/a = 1.0; ρ = 0.0513
M = 0.0513 x 11.37 x 1170^2
= 869380.17 Nmm
Maximum moment = 869380.17 Nmm
Which governs the design
$Z_z = bxt^2 /6$
$M = 1.2xZ_exf_y / \gamma_{mo}$
$869380 = 1.2 \text{ x } 1xt^2/6 \text{ x } 330/1.1$
$869380 = 60t^2$
t = 120.37mm
Note that the moment in part 1 and 2 are far less than part 3. Let us provide stiffeners for part 3 as shown.



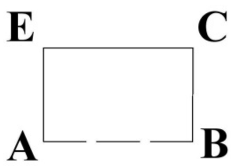
b/a = 1.0; ρ = 0.0513 (3)
M = 0.0513 x 11.37 x 585^2 = 217345.04 Nmm
Design moment = 290156.25 Nmm
Design thickness = 70mm

6) Design the stiffener for the the above base plate

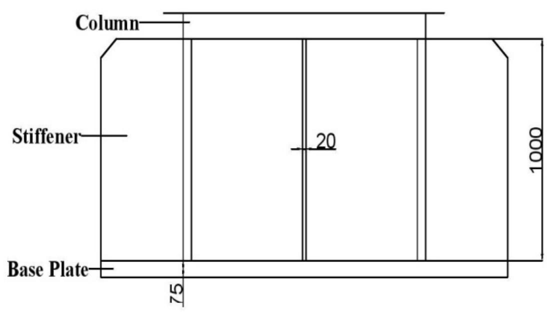
The stiffener EC takes the load (pressure) in area ABCD



$M = 12.38x(1/2x500x500)\ x\ (2/3x500) = 515.83x10^6$



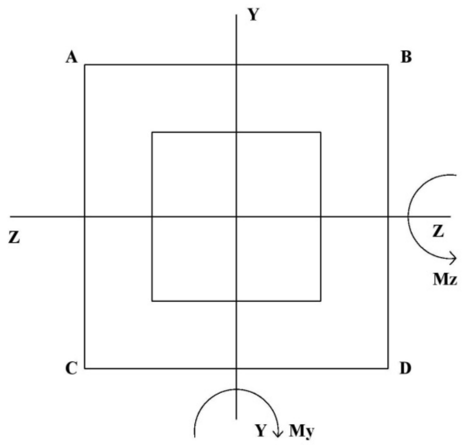
$M = 12.38x585/2x585^2/2$
$= 619.62x10^6 Nmm$
Total moment = $1135.45x10^6$
Let us keep the height of stifferner as 1000mm
$Z = tx1000^2/6$
$M = 1.2x\ (tx1000^2/6)x(330/1.1)$
$1135.45x10^6 = 1.2x\ (tx1000^2/6)x(330/1.1)$
$t = 1135.45x10^6\ x\ 6.6\ /\ 1.2x1000^2x330$
t = 18.92mm
provide 20mmx1000mm stiffner.



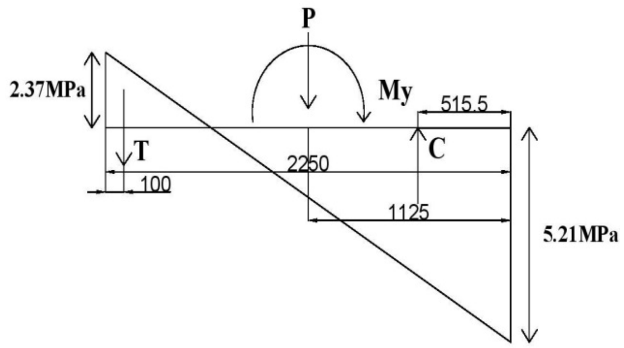
7) Design the anchor bolts for the above if the imposed loas is 10000 KN

Total axial load = 16000KN
Due to imposedload = 10000KN
Due to dead load = 6000KN

<u>To find tension:</u>

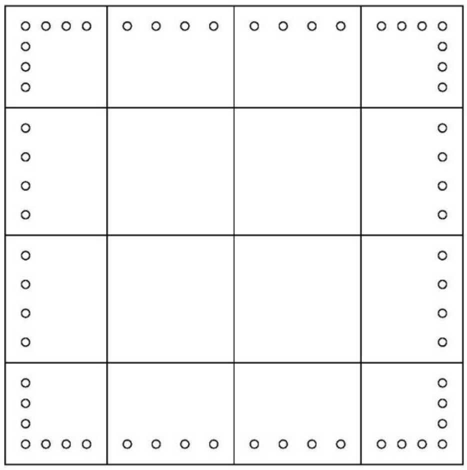


Mz& My will create tension @ corner A. Mz will create tension along AB and My will create tension along AC



Maximum Pressure when considering P &Mz.
P/A + Mz/Zz
$=(1.2x6000x1000/2250^2)+(7200x10^6/1898.43x10^6)$
= 1.42 + 3.79
= 5.21MPa
Minimum pressure when considering P &Mz = 1.42 – 3.79 = -2.37 MPa
C = 2250x5.21 / (2.37+5.21)
= 1546.50mm / 3
= 515.5mm
Let us provide bolts @ 100 mm from edge.
Lever arm = 2250-515.5-100 = 1634.5mm
Taking moment about c
$(-Tx1634.5) – (1.2x6000x10^3) x (1125-515.5) + (1.2x6000x10^3) = 0$
$T = 4.3884x10^9 + 7.2x10^6 / 1634.5$
= 2680.5 KN
Providing 16 bolts on each side,
T/bolt = 2680.5 / 16 = 167.53KN

$Tdn = 0.9Anfu / \gamma_{mb}$
$167.53x10^3 = 0.9 \times An \times 400 / 1.25$
$An = 167.53x10^3 x1.25 / 0.9x400$
$= 581.7mm^2$
Gross area required = 581.7 / 0.78
$= 745.77\ mm^2$
$\Pi \times d^2 / 4 = 745.77$
$d = \sqrt{(4x745.77 / \pi)} = 30.81$ mm
Provide M_{36} thread in 40mm dia rod.



PLAN OF BASE PLATE

EXPERIMENT INVESTIGATION

In the ex perimental investigation the welded box column was subjected to axial compression with the help of the Universal testing machine(UTM). The cross section of the box column was (100x100)mm in all cases. Four models where made with different specifications.

Model No . 1: 6mm plates were welded together with a unsupported length of 500mm. It is covered with a base plate of (130x130x10) mm on top and bottom. 3.5mm fillet weld was used to keep the plates intact.

Model No . 2: It is the replica of first model except there are four vertical stiffeners plates of 3mm thick and 10mm projection inside the face of the box column.

Model No . 3: The third model was made with 3mm plates by increasing the unsupported length to 700mm. This was done aimed at reducing the failure load in order to get the results in an extensive manner.

Model No . 4: The fourth one is same as third model with 1mm thick vertical stiffener with 10mm projection on all sides along the height of the box column.

The following observations where recorded.
a)Load vs Displacement curves b)Stress vs Strain curves
c)Stress vs Displacement curves
d)Displacement modulus Data
The stress vs strain curves of the four models is under:

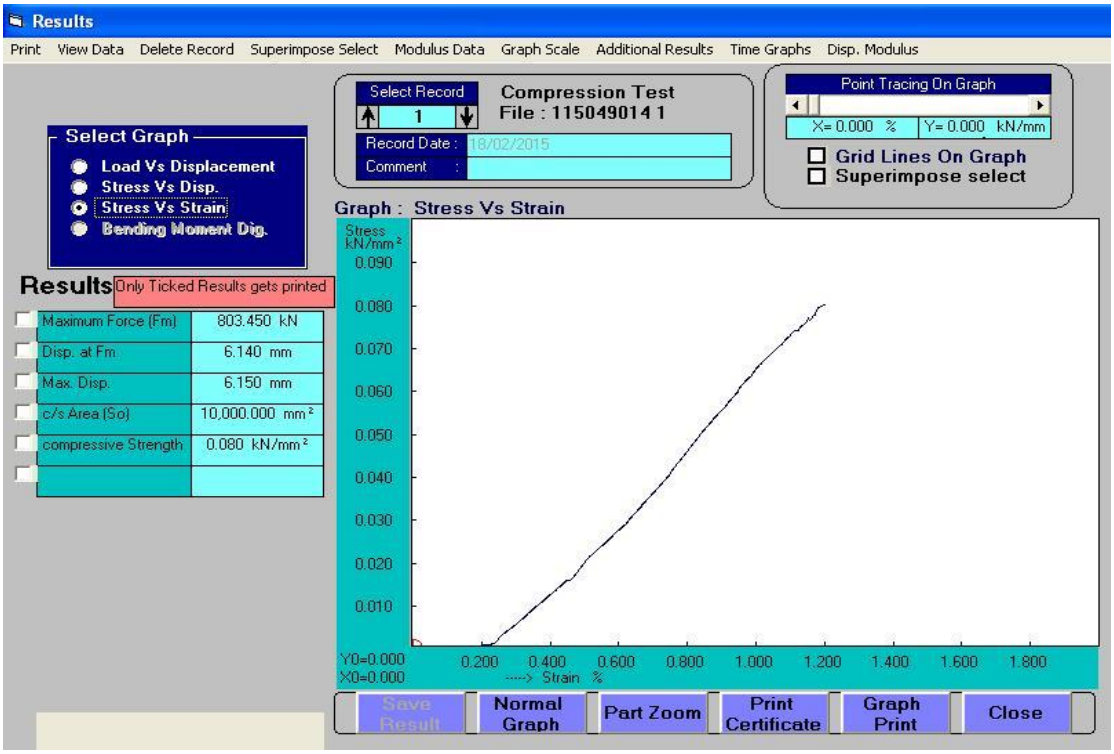


Fig : 1First model (6mm without stifferners)

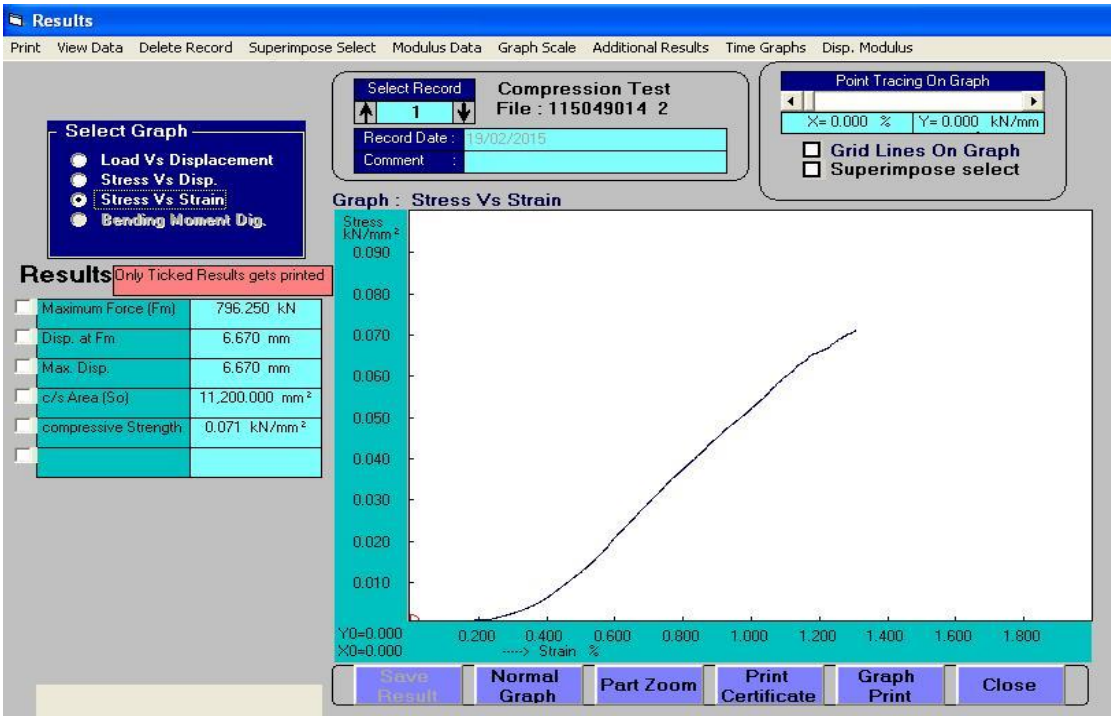


Fig : 2Second model (6mm with stiffeners)

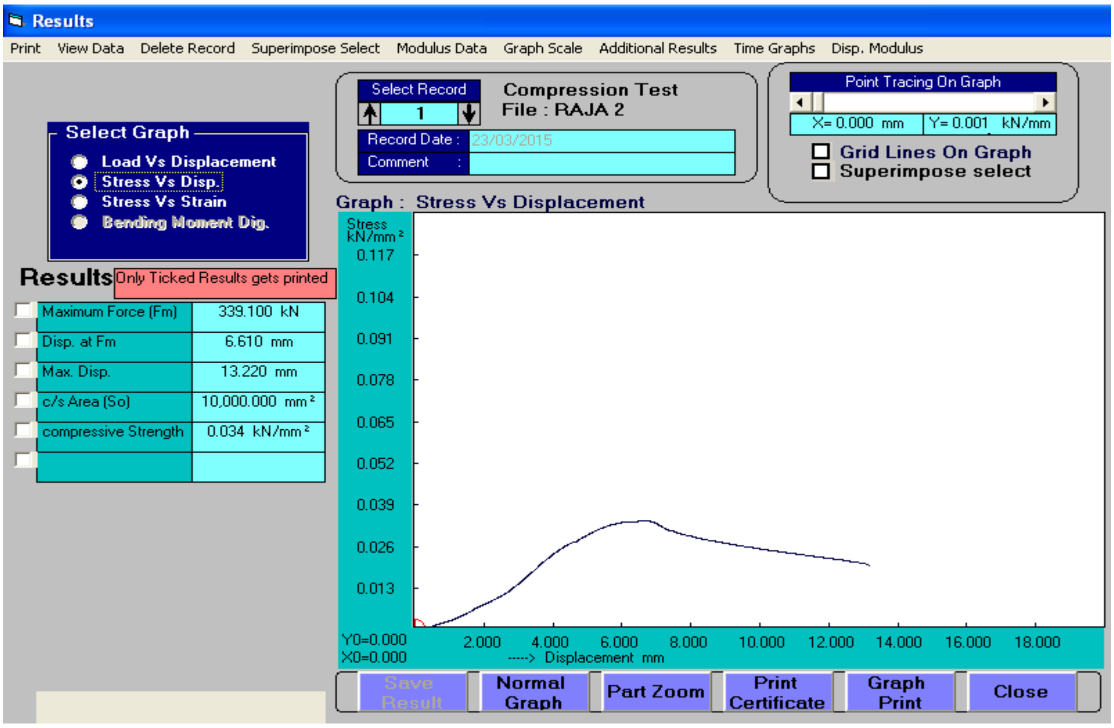


Fig : 3Third model (3mm without stiffeners)

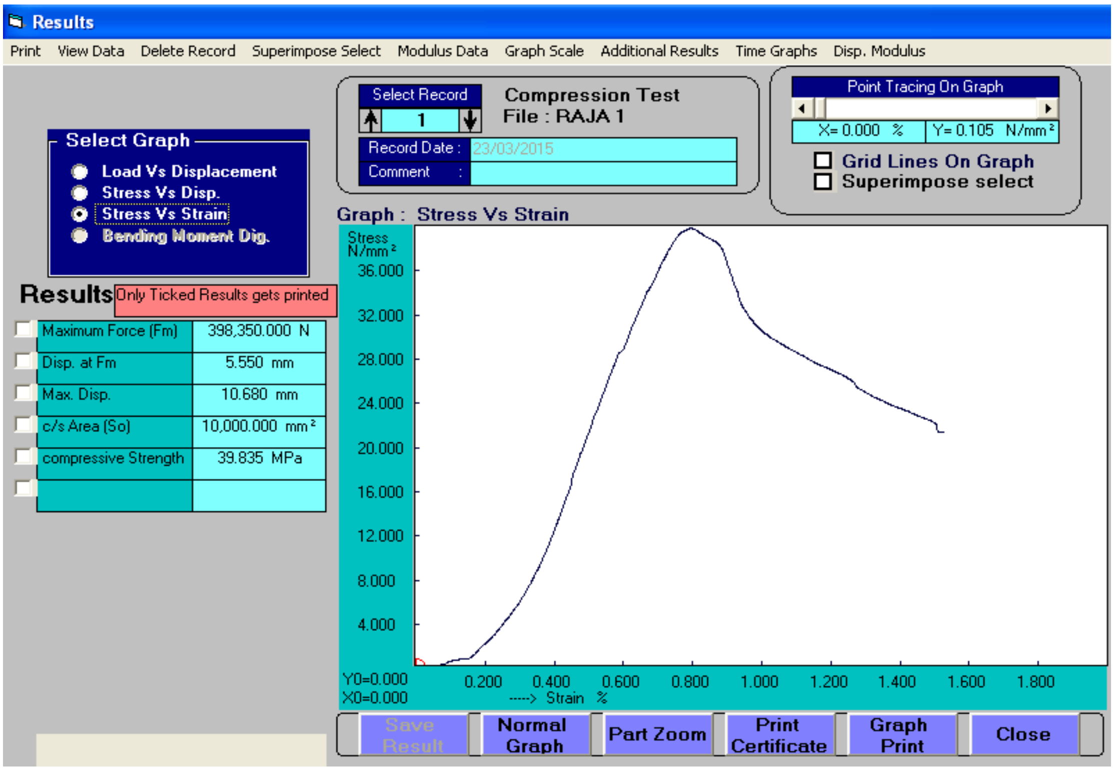


Fig : 4Fourth model (3mm with stiffeners)

It is observed that the maximum displacement was greater in the with stiffener case, this also resulted in the decrease of the compressive strength.
This was due to the loss in the strength of the member caused by residual stresses. This was not accounted in the manual design. If this experiment was done for bigger box columns with bigger equipment then the results would have been better. It is not correct to extrapolate these values to bigger columns since there are various parameters involved. The characteristic compressive strength of the box column was found out using IS 800:2007 and the values seemed to match the experimental investigations with minor variations. The
FEM analysis in STAAD.Pro was carried out and it was compatible with the manual calculations.

Model No : 1 (6mm Plate without Stiffener)

P_d = Ae x f_{cd} (Table 10 IS:800)
Ae = 2[100x6 + 88x6]
= 2256mm^2
r = √I/A
I = 2 [(6x100^3/12 + (88x6^3/12 + 88x6x47^2]
= 3335872 mm^4
r = √ 3335872 / 2256 = = 38.45
KL / r = 0.65x500 / 38.45 = 8.45
KL / r <10 take it as 10
f_{cd} = 227 N/mm^2
P_d= 2256x 227
= 512112 N = 51.112 KN = 52.2 tons

Model No : 2 (6mm with Stiffener)

P_d = Ae x f_{cd} (Table 10 IS:800)
Ae = 2[100x6 + 88x6] + 4[10x3]
= 2376mm^2

r = √I/A
I = 3335872 + 2 [(10x3^3/12) + (3x10^3/12) + 10x3x39^2]
= 3427677 mm^4
r = √ 3427677 / 2376
= 37.98
KL / r = 0.65x500 / 37.98 = 8.55
KL / r <10 take it as 10

f_{cd} = 227 N/mm^2 (Table 9c IS:800)
P_d= 2376x 227
= 539352 N = 539.352 KN = 54.97 tons

Model No : 3 (3mm without Stiffener)

P_d = Ae x f_{cd} (Table 10 IS:800)
Ae = 2[100x3 + 94x3]
= 1164mm^2
r = √I/A
I=2[(3x100^3/12)+(94x3^3/12 + 94x3x48.5^2)]
= 1827092 mm^4
r = √ 1827092 / 1164
= 39.61
KL / r = 0.65x700 / 39.61 = 11.48
So by Table 9c of IS:800
f_{cd} = (224-227/10) x 1.48 + 227
= 226.556 N/mm^2
P_d= 2376x 226.556
= 263711.18 N = 263.72 KN
= 26.88 tons

Model No : 4 (3mm with Stiffener)

P_d = Ae x f_{cd} (Table 10 IS:800)
Ae = 2[100x6 + 88x6] + 4[10x1]
= 1204mm^2
r = √I/A
I = 1827092 + 2 [(10x1^3/12) + (1x10^3/12 + 10x1x43.5^2)]
= 1865105.33 mm^4
r = √ 1865105.33 / 1204
= 39.35
KL / r = 0.65x700 / 39.61 = 11.48
So by Table 9c of IS:800
f_{cd} = (224-227/10) x 1.56 + 227
= 226.532 N/mm^2
P_d= 1204 x 226.532
= 272747.52 N = 272.75 KN
= 27.8 tons

STAAD Analysis

The four models were plate modelled in STAAD. The maximum absolute stresses Corresponding to the characteristic Compressive strength of the respective Box columns formulated in the manual Calculations are shown below and a Consensus is made.

When the box column is subjected toP_d the maximum stress move towards

the yield stress in the stress contour shown
above. This proves that the manual calculation goes in hand with FEM analysis. The variation in stress can be observed in the stress contours. The maximum stress is said to be concentrated at the ends. Experimental models after the application of load can be seen below.
From the experimental observation we couldn't get a definite yield point for the system but it reaches a ultimate point before failure. In the 6mm plate we couldn't reach the ultimate load since the machine capacity was only 80tons. Whereas in the 3mm plate model the ultimate load can be seen. In model no.3 we observed a slight kink at around 280KN which we may consider as the yield point but the actual yield point load is 263.72 KN which is slightly less. Our motive of this project is to show whether the box columns are having more strength than it is stated but the current evidences doesn't lead to that conclusion. Many number of models with varying cross section, height and plate thickness should be tested for satisfactory results. If we could prove that the box column could come under buckling class B, considering the imperfections in plate, loss due to residual stresses and interaction between local and overall buckling then it could be more extensively by replacing conventionally build structures. The advantage of using box column on structural aspect can be clarified by this research.

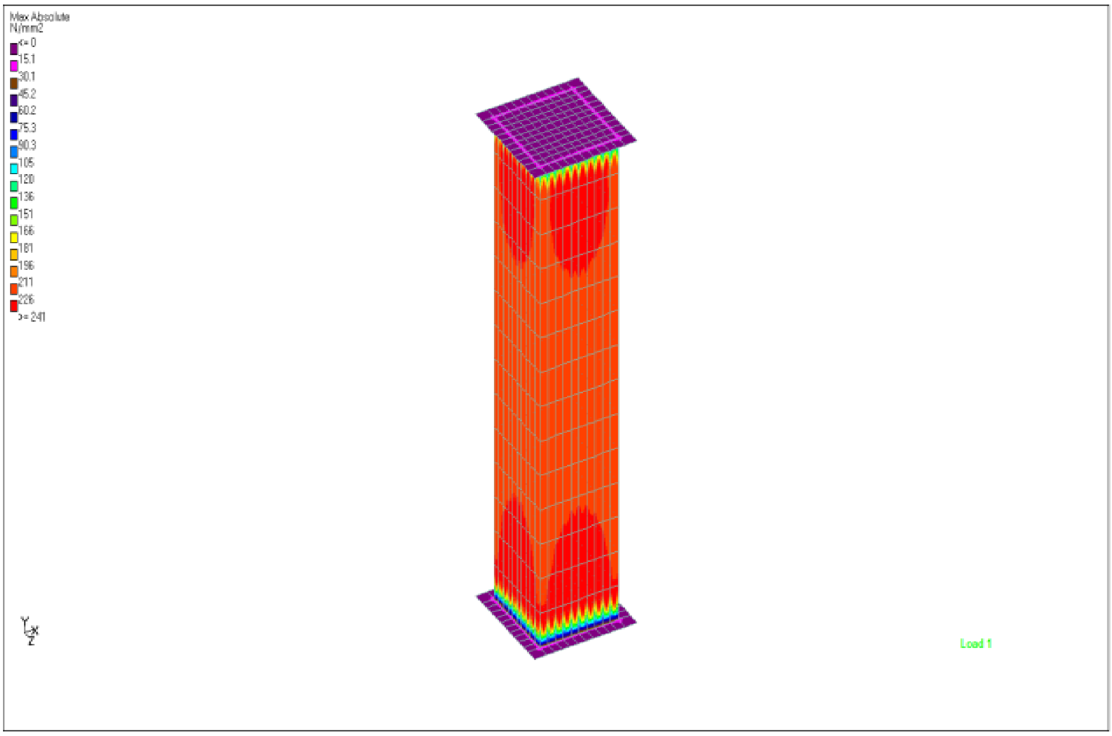


Fig : 5MODEL NO.1

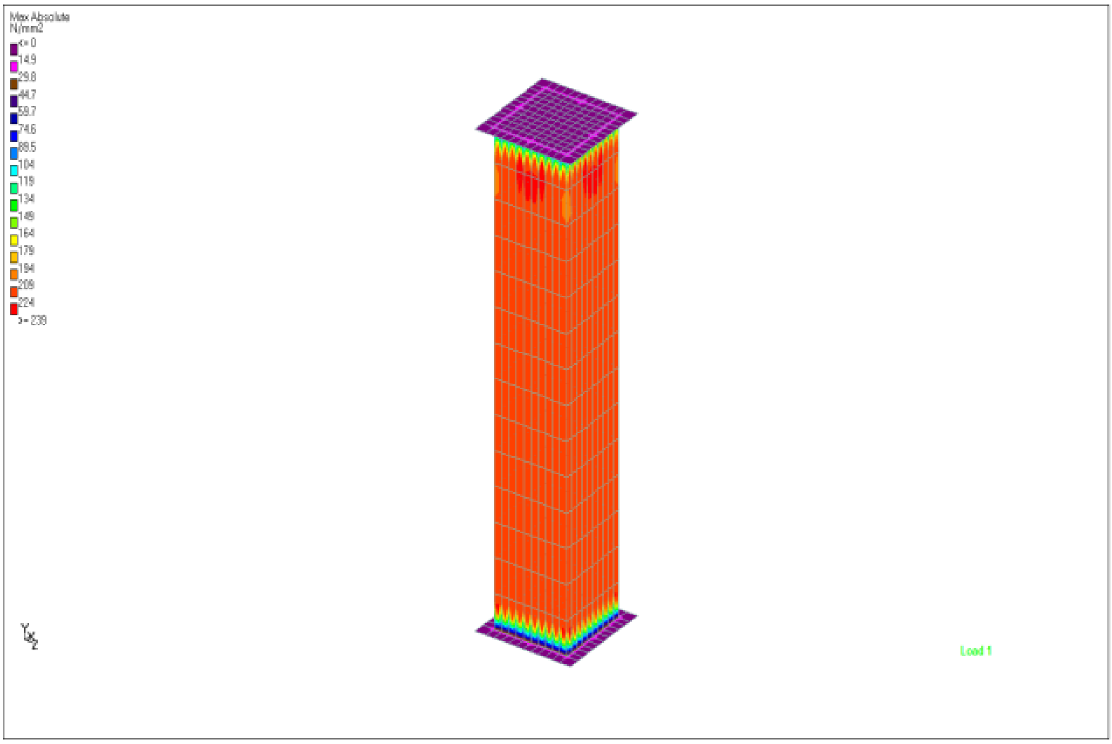


Fig : 6MODEL NO.2

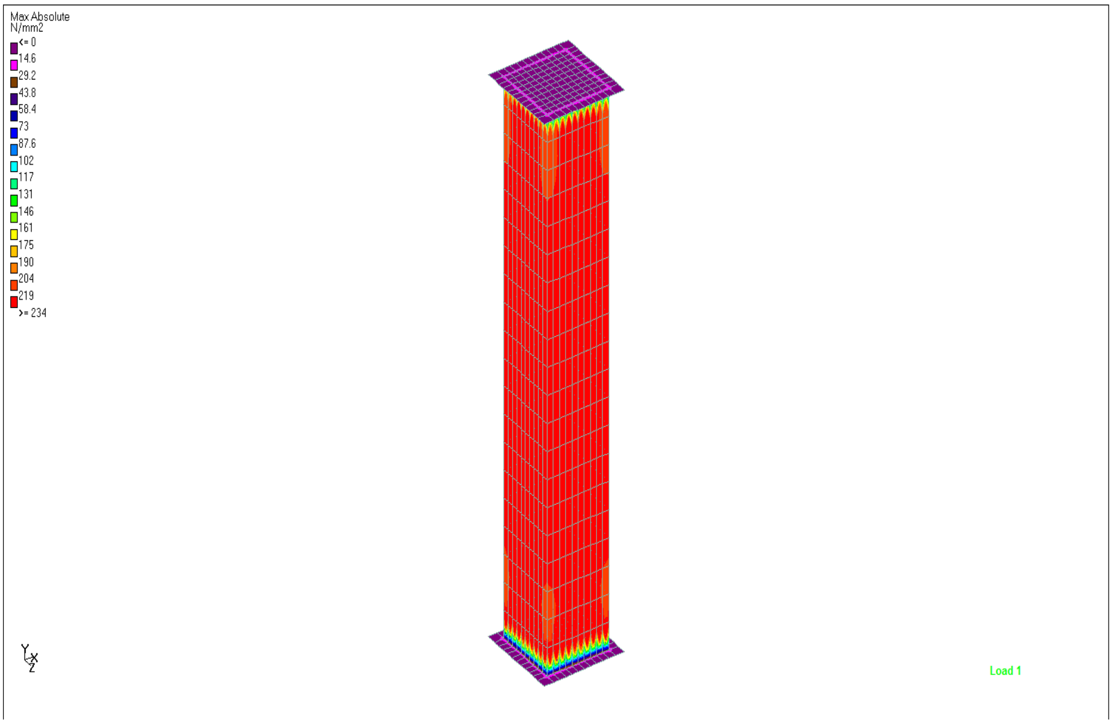


Fig : 7MODEL NO.3

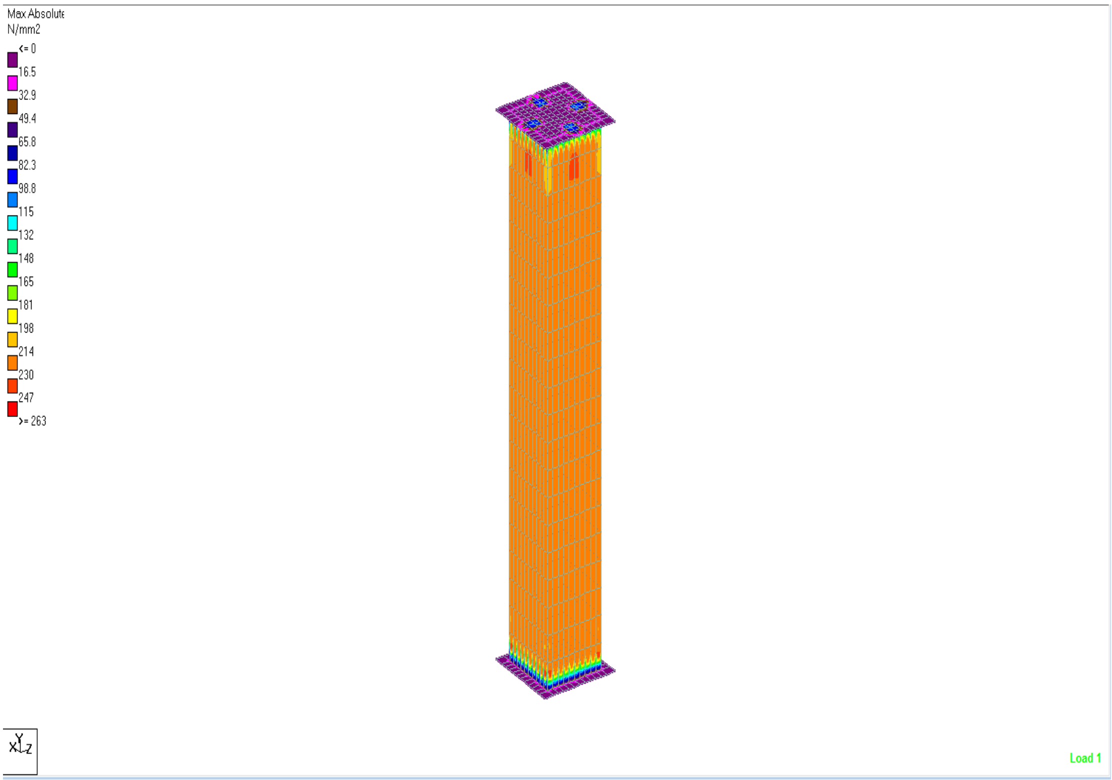


Fig : 8MODEL NO.4

CONCLUSION:

This paper aimed at giving an idea towards the practical design of welded box columns subjected to axial compression and biaxial bending. IS800:2007 was used as the reference for designing the box columns. Several examples have been dealt by varying the magnitude and direction of forces. Appropriate cross section and plate thickness have been designed by following the codal provisions. Later small scale experimental investigations were compared with FEM analysis in STAAD and the consensuses have been discussed. There by I conclude by saying there is yet be an extensive experimental study possible in this area.

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