



CHANDRASHEKAR & ASSOCIATES

Consulting Structural Engineer, Valuer and Quantity Surveyors

Title of Project: Proposed Residential Villas, Frangipanni Phase II -Aldeia De Goa Bambolim

Promoters & Developers: M/s. Goan Real Estate & Construction Pvt.ltd.

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Structural Engineer Registration no. TCP/SE/0052/2010

The following IS codes & method of Design have been followed

As per IS 13920:1993 Clause 7.1.2,

The minimum dimension of column shall not be less than 200 mm.

For the columns with unsupported length exceeding 4 m ,
the shortest dimension of the column shall not be less than 300 mm.

As per IS 13920:1993 Clause 7.1.3 of IS 13920:1993,

The ratio of the shortest cross sectional dimensions to the
perpendicular dimension shall preferably not be less than 0.4.

Terms Used In Calculation :

In biaxial column design,

$$P_u = P_{uc} + P_{us(Total)}$$

where,

P_u = external axial compressive load,

P_{uc} = axial compressive resistance offered by concrete,

$P_{us(Total)}$ = total axial compressive resistance offered by steel
at different levels in the section.

$$M_u = M_{uc} + M_{us(Total)}$$

where,

M_u = external moment about centroidal axis,

M_{uc} = moment of resistance offered by concrete in compression,

$M_{us(Total)}$ = total moment of resistance offered by steel
at different levels in the section.

i = serial no. of row of reinforcement,

A_{si} = cross-sectional area of reinforcement in the i th row,

f_{si} = stress in the reinforcement in the i th row (compression + ve, tension - ve),

f_{ci} = compressive stress in concrete at the level of i th row of reinforcement,

e_i = strain at the i th row of reinforcement from the stress-strain curve of steel and
concrete,

x_i = distance of the bars in the i th row from the centroid of the section

Values of stress in steel :

Design strength in bending compression (f_{yd}) = $0.87 f_y$

(a) Fe 250,

For $e_i \geq f_{yd} / E_s$, $f_{si} = f_{yd}$

For $e_i < f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

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(b) Fe 415,

For $e_i \geq 0.8 \times f_{yd} / E_s$, f_{si} = value obtained from stress-strain curve

For $e_i < 0.8 \times f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

Stress	Stress level (N/mm ²)	Strain
0.800 f_{yd}	288.7	0.00144
0.850 f_{yd}	306.7	0.00163
0.900 f_{yd}	324.8	0.00192
0.950 f_{yd}	342.8	0.00241
0.975 f_{yd}	351.8	0.00276
1.000 f_{yd}	360.9	0.00380

Values of stress in concrete :

For $e_i \geq 0.002$, $f_{ci} = 0.446 f_{ck}$

For $e_i < 0.002$, $f_{ci} = (446.e_i \times (1 - 250.e_i)) \times f_{ck}$

Values of strain :

Values of strain at different levels are obtained by taking maximum strain as 0.0035 as the reference value at the highly compressed edge.

Transverse Reinforcement:

As per IS 13920 : 1993, the design shear force for columns shall be the maximum of

i) calculated factored shear force as per analysis, and

ii) a factored shear force given by

$V_u = 1.4 \times (M_{ubL,lim} + M_{ubR,lim}) / \text{storey height}$

where $M_{ubL,lim}$, $M_{ubR,lim}$ are moments of resistance, of oppsite sign framing into the column from oppsite faces (to be calculated as per IS 456 : 1978)

General Design Parameters :

Below Floor4 - Default Level at 12.000 m

Column size, (B x D) = 200 x 475 mm

Column height, (L) = 2450 mm

From analysis results, loads on column

Axial load, (P) = 378.00 kN

MomentX, (M_x) = 52.97 kN-m

MomentY, (M_y) = 22.69 kN-m

$f_{ck} = 20.00$ N/mm²

$f_y = 415.00$ N/mm²

Load combination = 1.50 DL + 1.50 EQL X +

Column Name	Orientation Angle
C4	0.00

Check For Slenderness :

Slenderness RatioX = ($L_o \times \text{Effective Length FactorX}$) / Depth
= (2450 x 1.000) / 475 = 5.16 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = ($L_o \times \text{Effective Length FactorY}$) / Width
= (2450 x 1.000) / 200 = 12.25 > 12.00,



column is slender in this direction.

$$P_{uz} = (0.45 \times f_{ck} \times A_g) + (0.75 \times f_y - 0.45 \times f_{ck}) \times A_{st} \\ = 0.45 \times 20.00 \times 95000.01 + (0.75 \times 415.00 - 0.45 \times 20.00) \times 1709.03 = \\ 1371553.345 \text{ N}$$

$$k_1 = 0.174400$$

$$k_2 = -0.076400 \text{ MPa}$$

$$P_{ub} = (k_1 \times f_{ck} \times A_g) + (k_2 \times A_{st} \times 100) \\ = (0.174400 \times 20.00 \times 95000.01) + (-0.08 \times 1709.03 \times 100) = 318303.070 \text{ N}$$

$$k = (P_{uz} - P_u) / (P_{uz} - P_{ub}) \leq 1 \\ = (1371553.345 - 377995.605) / (1371553.345 - 318303.070) = 0.943325$$

$$M_{uaddY} = P_u \times (\text{Width}/2000) \times (L_o/\text{Width}) \times (L_o/\text{Width}) \times k \\ = 377995.61 \times (200.00/2000) \times (2450.00/200.00) \times (2450.00/200.00) \times \\ 0.943325 \\ = 5350821.495 \text{ N.mm} = 5.351 \text{ kN.m}$$

$$M_{x_MinEccen} = 7.837 \text{ kN.m}$$

$$M_{y_MinEccen} = 7.560 \text{ kN.m}$$

$$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX} = 52.967 \text{ kN.m}$$

$$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY} = 28.038 \text{ kN.m}$$

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4, all columns shall be designed for minimum eccentricity equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

$$\text{Actual eccen}_x = M_x / P = 52.97/378.00 = 140 \text{ mm}$$

$$\text{Actual eccen}_y = M_y / P = 22.69/378.00 = 60 \text{ mm}$$

$$\text{eccen}_{xMin} = (L/500) + (D/30) = (2450/500) + (475/30) = 21 \text{ mm}$$

$$\text{eccen}_{xMin} = \max(21, 20) = 21 \text{ mm}$$

$$\text{eccen}_{yMin} = (L/500) + (B/30) = (2450/500) + (200/30) = 12 \text{ mm}$$

$$\text{eccen}_{yMin} = \max(12, 20) = 21 \text{ mm}$$

$$\text{eccen}_x = \max(\text{Actual eccen}_x, \text{eccen}_{xMin}) = \max(140, 21) = 140 \text{ mm}$$

$$\text{eccen}_y = \max(\text{Actual eccen}_y, \text{eccen}_{yMin}) = \max(60, 20) = 60 \text{ mm}$$

As per IS 456: 2000 Clause 39.3, when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.

$$\text{eccen}_x(140 \text{ mm}) > 0.05 \times 200(10 \text{ mm})$$

$$\text{and eccen}_y(60 \text{ mm}) > 0.05 \times 475(24 \text{ mm})$$

Hence the column will be designed as a column with biaxial moments.

Provided steel :

Provide #12 - 8 nos. + #16 - 4 nos. (1709 mm²)

Biaxial Check Calculations :



For X axis :

Row No. (i)	A _{si} (mm ²)	e _i	f _{si} (N/mm ²)	f _{ci} (N/mm ²)	(f _{si} - f _{ci}) (N/mm ²)	P _{usi} (kN)	x _i (mm)	M _{usi} (kN-m)
1	402	0.00283	352.38	8.92	343.46	138.12	190	26.17
2	226	0.00175	314.00	8.78	305.22	69.04	113	7.77
3	226	0.00070	139.01	5.12	133.89	30.29	38	1.14
4	226	-0.00036	-71.36	0.00	-71.36	-16.14	-38	0.61
5	226	-0.00141	-281.73	0.00	-281.73	-63.72	-113	7.17
6	402	-0.00249	-344.82	0.00	-344.82	-138.66	-190	26.28
Total						18.91		69.13

$$x_{ux} = 250 \text{ mm}$$

$$P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 k_u = 0.36 \times 250/475 = 0.189$$

$$P_{uc} = 0.189 \times 20.00 \times 200 \times 475$$

$$= 359.37 \text{ kN}$$

$$P_{ux1} = P_{uc} + P_{us(\text{Total})}$$

$$= 359.37 + (18.91)$$

$$= 378.28 \text{ kN} > P, \text{hence O.K.}$$

$$M_{uc} = P_{uc} \cdot (0.5 \cdot D - C_2 \cdot D)$$

$$C_2 = 0.416 k_u = 0.416 \times 250/475$$

$$= 0.219$$

$$M_{uc} = 359.37 \times (0.5 \times 475 - 0.219 \times 475) = 48.04 \text{ kN-m}$$

$$M_{ux1} = M_{uc} + M_{us(\text{Total})}$$

$$= 48.04 + (69.13)$$

$$= 117.17 \text{ kN-m}$$

For Y axis :

Row No. (i)	A _{si} (mm ²)	e _i	f _{si} (N/mm ²)	f _{ci} (N/mm ²)	(f _{si} - f _{ci}) (N/mm ²)	P _{usi} (kN)	x _i (mm)	M _{usi} (kN-m)
1	855	0.00192	324.73	8.91	315.82	269.87	52	14.03
2	855	-0.00151	-295.05	0.00	-295.05	-252.13	-52	13.11
Total						17.75		27.14

$$x_{uy} = 106 \text{ mm}$$

$$P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 k_u = 0.36 \times 106/200 = 0.191$$

$$P_{uc} = 0.191 \times 20.00 \times 475 \times 200$$

$$= 363.38 \text{ kN}$$

$$P_{uy1} = P_{uc} + P_{us(\text{Total})}$$

$$= 363.38 + (17.75)$$

$$= 381.12 \text{ kN} > P, \text{hence O.K.}$$

$$M_{uc} = P_{uc} \cdot (0.5 \cdot B - C_2 \cdot B)$$

$$C_2 = 0.416 k_u = 0.416 \times 106/200$$



$$= 0.221$$

$$M_{uc} = 363.38 \times (0.5 \times 200 - 0.221 \times 200) = 20.28 \text{ kN-m}$$

$$M_{uy1} = M_{uc} + M_{us(\text{Total})}$$

$$= 20.28 + (27.14)$$

$$= 47.42 \text{ kN-m}$$

$$P_u/P_{uz} = 0.276$$

$$\text{For } P_u/P_{uz} \leq 0.2, \alpha_n = 1.0$$

$$\text{For } P_u/P_{uz} \geq 0.8, \alpha_n = 2.0$$

$$\text{For } P_u/P_{uz} = 0.276, \alpha_n = 1.126$$

$$((M_{ux}/M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$$

$$= ((52.97/117.17)^{1.126}) + ((28.04/47.42)^{1.126})$$

$$= 0.962 < 1.0, \text{hence O.K.}$$

Ties Details :

Load combination for ties design = 0.90 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the diameter of the polygonal links or lateral ties shall be not less than the following

i) one-fourth the diameter of the largest longitudinal bar = $16/4 = 4 \text{ mm}$

ii) in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 8 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

$$V_u = 1.4 \times (M_{ubL, \text{lim}} + M_{ubR, \text{lim}}) / \text{storey height}$$

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

$$\tau_{auc} = 0.760 \text{ MPa for 1.80\% steel}$$

$$\Delta = 1 + (3.P_u/A_g.f_{ck}) \text{ but not exceeding } 1.5 = 1.35$$

$$\tau_{auc} =$$

$$\tau_{auc} \times \Delta = 1.029 \text{ MPa}$$

For X - axis :

As per IS 456: 2000, the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 200 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to

be tied

$$= 16 \times 12 = 192 \text{ mm}$$

iii) 300 mm

Required spacing = minimum of (200, 192, 300) = 192 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 100 mm

Required spacing = minimum of (192, 100) = 100 mm

V_u for columns in the group :

Column.No	$M_{ubL, \text{lim}}(\text{kN-m})$	$M_{ubR, \text{lim}}(\text{kN-m})$	$V_u(\text{kN})$
C4	146.37	146.37	136.61

$$V_u = 136.61 \text{ kN}$$

$$\text{Design shear force, } (V_{ud}) = 1.80 \text{ kN (from analysis)}$$

$$V_{ud} = \max(V_{ud}, V_u) = 136.61 \text{ kN}$$

$$\text{Capacity of concrete, } (V_{uc}) = \tau_{auc} \times \text{Area} = 97.76 \text{ kN}$$

$$V_{us} = V_{ud} - V_{uc} = 38.85 \text{ kN}$$



Stirrup legs assumed

$$= 4$$

V_{umin}

$$= 0.87 \times f_{ys} \times A_{sv} \times d / s_{vmin}$$

$$= 0.87 \times 415 \times 201 \times 156 / 100 = 113245.74 \text{ N}$$

$$= 113.25 \text{ kN}$$

$V_{us} < V_{usmin}$

/

Spacing from min.shear R/F requirement = $(0.87 \times f_{ys} \times a_{sv}) / (0.4 \times \text{width})$

$$= ((0.87 \times 415.000 \times 201.062) / (0.4 \times 475.000))$$

$$= 382 \text{ mm}$$

Required spacing

$$= \min(382, 100) = 100 \text{ mm}$$

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 200 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to

be tied

$$= 16 \times 12 = 192 \text{ mm}$$

iii) 300 mm

Required spacing = minimum of (200, 192, 300) = 192 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 100 mm

Required spacing = minimum of (192, 100) = 100 mm

V_u for columns in the group :



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