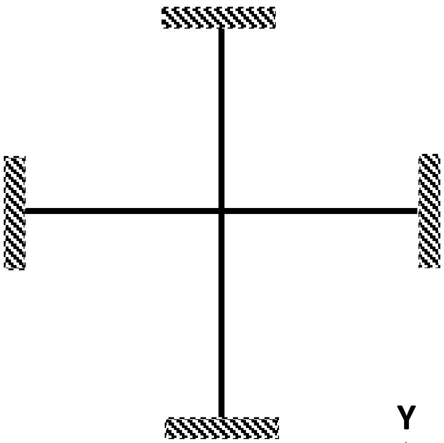
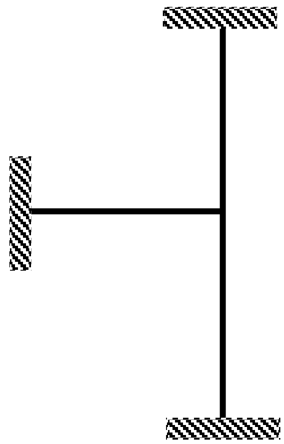
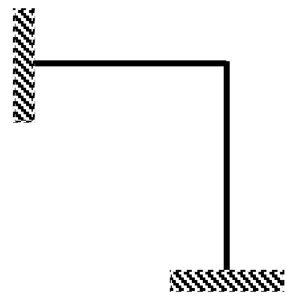
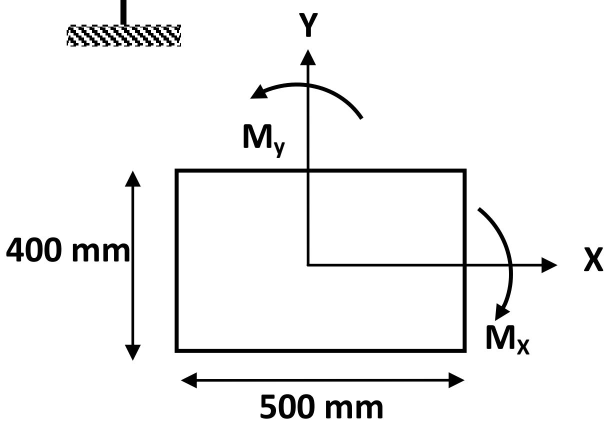








- **<u>Moment of inertia about x- axis</u>**

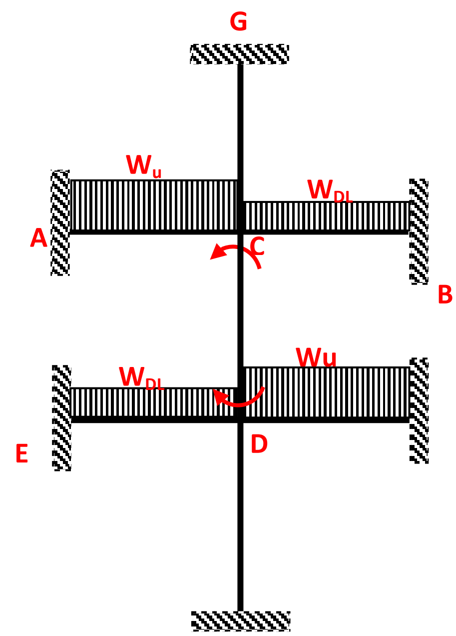
$$I_X = \frac{b * h^3}{12} = \frac{500 * 400^3}{12} = 2.667 * 10^9 \ \ mm^4$$

- **<u>Moment of inertia about Y- axis</u>**

$$I_Y = \frac{b * h^3}{12} = \frac{400 * 500^3}{12} = 4.167 * 10^9 \ \ mm^4$$

Example 1)

from previous example 2 in one way solid slab. Determine the bending moment acting on interior column CD in the second floor



$L_{CA} = 9m \;, L_{CB} = 9m \;, hight\ of\ columns = 3.6\ m$

$W_u = 88.24\ KN/m \;, W_{DL} = 65.8\ KN/m$

$beam\ dimenssion\ (400{,}700) \quad , \quad column\ dimenssion\ (500{,}400)$

$$(FEM)_{CA} = +\frac{W_u * L_{CA}^2}{12} = \frac{88.24 * 9^2}{12} = 595.62\ KN.m$$

$$(FEM)_{CB} = +\frac{W_u * L_{CB}^2}{12} = \frac{65.8 * 9^2}{12} = 444.15\ KN.m$$

The unbalanced moment at the column joint C is:

$$M_C = (FEM)_{CA} + (FEM)_{CB} = 595.62 - 444.15 = 151.47\ KN.m$$

The moment in the top of column is given by:

$$M_{CD} = -M_C * \frac{\left[\frac{I}{L}\right]_{CD}}{\left[\frac{I}{L}\right]_{CG} + \left[\frac{I}{L}\right]_{CD} + \left[\frac{I}{L}\right]_{CA} + \left[\frac{I}{L}\right]_{CB}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{500 * 400^3}{12}}{3600}\right] = 740.74 * 10^3\ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{CB} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{I}{L}\right]_{DF} = \left[\frac{\frac{400 * 700^3}{12}}{9000}\right] = 1270.37 * 10^3\ \frac{mm^4}{mm}$$

$$M_{CD} = -151.47 * \frac{740.74 * 10^3}{740.74 * 10^3 \ + 740.74 * 10^3 \ + 1270.37 * 10^3 + 1270.37 * 10^3}$$

$$M_{CD} = -151.47 * 0.184 = -27.9\ KN.m$$

$$(FEM)_{DF} = -\frac{W_u * L_{DF}^2}{12} = \frac{88.24 * 9^2}{12} = -595.62\ KN.m$$

$$(FEM)_{DE} = +\frac{W_{DL} * L_{DE}^2}{12} = \frac{65.8 * 9^2}{12} = +\ 444.15\ KN.m$$

The unbalanced moment at the column joint D is:

$$M_D = (FEM)_{DF} + (FEM)_{DE} = -595.62 + 444.15 = -151.47\ KN.m$$

<u>The moment in the bottom of column is given by:</u>

$$M_{DC} = -M_C * \frac{\left[\frac{I}{L}\right]_{DC}}{\left[\frac{I}{L}\right]_{DC} + \left[\frac{I}{L}\right]_{DH} + \left[\frac{I}{L}\right]_{DE} + \left[\frac{I}{L}\right]_{DF}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{500*400^3}{12}}{3600}\right] = 740.74*10^3\ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{CB} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{I}{L}\right]_{DF} = \left[\frac{\frac{400*700^3}{12}}{9000}\right] = 1270.37*10^3\ \frac{mm^4}{mm}$$

$$M_{CD} = -(-151.47) * \frac{740.74*10^3}{740.74*10^3 + 740.74*10^3 + 1270.37*10^3 + 1270.37*10^3}$$

$$M_{CD} = 151.47 * 0.184 = 27.9\ KN.m$$

$L_{CA} = 4.4m$, $L_{CB} = 4.4m$, $hight\ of\ columns = 3.6\ m$

$W_u = 24.22\ KN/m$, $W_{DL} = 24.22\ KN/m$

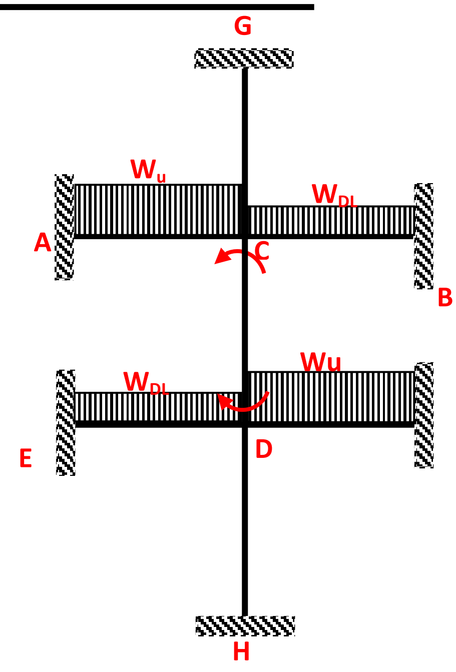
$beam\ dimenssion\ (400,700)$, $column\ dimenssion\ (500,400)$

$$(FEM)_{CA} = +\frac{W_u * L_{CA}^2}{12} = \frac{24.22*4.4^2}{12} = 39.075\ KN.m$$

$$(FEM)_{CB} = +\frac{W_u * L_{CB}^2}{12} = \frac{24.22*4.4^2}{12} = 39.075\ KN.m$$

The unbalanced moment at the column joint C is:

$$M_C = (FEM)_{CA} + (FEM)_{CB} = 39.075 - 39.075 = 0\ KN.m$$

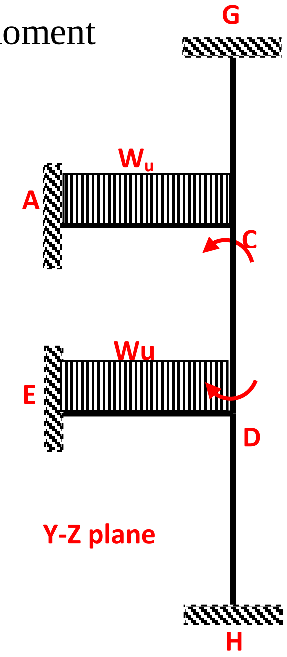


X-Z plane

Due to equality of loading (dead load and ultimate load) over the beams the unbalance moment at the column in joint C in X-Z plane is zero that means there is no moment acting in the column CD in the top and the bottom because is similar geometry to the top

Example 2)

from previous **example 2** in one way solid slab. Determine the bending moment acting on the corner column CD in the second floor .



$L_{CA} = 9m \ , L_{CB} = 9m \ , hight\ of\ columns = 3.6\ m$

$W_u = 88.24\ KN/m\ , W_{DL} = 65.8\ KN/m$

$beam\ dimenssion\ (400{,}700) \quad , \quad column\ dimenssion\ (500{,}400)$

$$(FEM)_{CA} = +\frac{W_u * L_{CA}^2}{12} = \frac{88.24 * 4.4^2}{12} = 595.62\ KN.m$$

The unbalanced moment at the column joint C is:

$$M_C = (FEM)_{CA} = 595.62\ KN.m$$

The moment in the top of column is given by:

$$M_{CD} = -M_C * \frac{\left[\frac{I}{L}\right]_{CD}}{\left[\frac{I}{L}\right]_{CG} + \left[\frac{I}{L}\right]_{CD} + \left[\frac{I}{L}\right]_{CA}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{500 * 400^3}{12}}{3600}\right] = 740.74 * 10^3\ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{\frac{400 * 700^3}{12}}{9000}\right] = 1270.37 * 10^3\ \frac{mm^4}{mm}$$

$$M_{CD} = -595.62 * \frac{740.74 * 10^3}{740.74 * 10^3 + 740.74 * 10^3 + 1270.37 * 10^3}$$

$$M_{CD} = -595.62 * 0.2692 = -160.341\ KN.m$$

$$(FEM)_{DE} = +\frac{W_{DL} * L_{DE}^2}{12} = \frac{88.24 * 4.4^2}{12} = 595.62\ KN.m$$

The unbalanced moment at the column joint D is:

$$M_D = (FEM)_{DE} = 595.62\ KN.m$$

<u>The moment in the bottom of column is given by:</u>

$$M_{DC} = -M_C * \frac{\left[\frac{I}{L}\right]_{DC}}{\left[\frac{I}{L}\right]_{DC} + \left[\frac{I}{L}\right]_{DH} + \left[\frac{I}{L}\right]_{DE}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{500 * 400^3}{12}}{3600}\right] = 740.74 * 10^3 \ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{CB} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{\frac{400 * 700^3}{12}}{9000}\right] = 1270.37 * 10^3 \ \frac{mm^4}{mm}$$

$$M_{CD} = -595.62 * \frac{740.74 * 10^3}{740.74 * 10^3 + 740.74 * 10^3 + 1270.37 * 10^3}$$

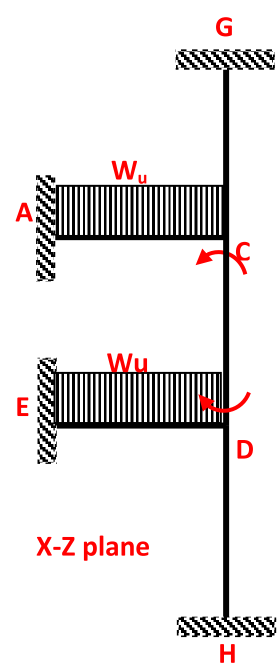
$$M_{CD} = -595.62 * 0.2692 = -160.341 \ KN.m$$

$L_{CA} = 4.4m$, $L_{CB} = 4.4m$, $hight\ of\ columns = 3.6\ m$

$W_u = 24.22\ KN/m$, $W_{DL} = 24.22\ KN/m$

$beam\ dimenssion\ (400,700)$, $column\ dimenssion\ (500,400)$

$$(FEM)_{CA} = +\frac{W_u * L_{CA}^2}{12} = \frac{24.22 * 4.4^2}{12} = 39.075\ KN.m$$



The unbalanced moment at the column joint C is:

$$M_C = (FEM)_{CA} = 39.075 = 39.075\ KN.m$$

<u>The moment in the top of column is given by:</u>

$$M_{CD} = -M_C * \frac{\left[\frac{I}{L}\right]_{CD}}{\left[\frac{I}{L}\right]_{CG} + \left[\frac{I}{L}\right]_{CD} + \left[\frac{I}{L}\right]_{CA}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{400 * 500^3}{12}}{3600}\right] = 1157.407 * 10^3 \ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{\frac{400*700^3}{12}}{9000}\right] = 1270.37*10^3\ \frac{mm^4}{mm}$$

$$M_{CD} = -39.075 * \frac{1157.407*10^3}{1157.407*10^3 + 1157.407*10^3 + 1270.37*10^3}$$

$$M_{CD} = -39.075 * 0.3228 = -12.613\ KN.m$$

$$(FEM)_{DE} = +\frac{W_{DL}*L_{DE}^2}{12} = \frac{88.24*4.4^2}{12} = 595.62\ KN.m$$

The unbalanced moment at the column joint D is:

$$M_D = (FEM)_{DE} = 595.62\ KN.m$$

<u>The moment in the bottom of column is given by:</u>

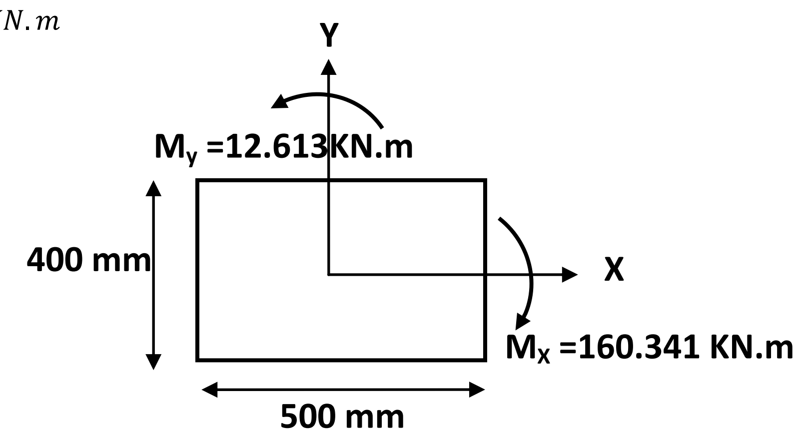
$$M_{DC} = -M_C * \frac{\left[\frac{I}{L}\right]_{DC}}{\left[\frac{I}{L}\right]_{DC} + \left[\frac{I}{L}\right]_{DH} + \left[\frac{I}{L}\right]_{DE}}$$

$$\left[\frac{I}{L}\right]_{CD} = \left[\frac{I}{L}\right]_{DC} = \left[\frac{I}{L}\right]_{CG} = \left[\frac{I}{L}\right]_{DH} = \left[\frac{\frac{400*500^3}{12}}{3600}\right] = 1157.407*10^3\ \frac{mm^4}{mm}$$

$$\left[\frac{I}{L}\right]_{CA} = \left[\frac{I}{L}\right]_{CB} = \left[\frac{I}{L}\right]_{DE} = \left[\frac{\frac{400*700^3}{12}}{9000}\right] = 1270.37*10^3\ \frac{mm^4}{mm}$$

$$M_{CD} = -39.075 * \frac{1157.407*10^3}{1157.407*10^3 + 1157.407*10^3 + 1270.37*10^3}$$

$$M_{CD} = -39.075 * 0.3228 = -12.613\ KN.m$$



Type of column:-

1- Ties column :

$S = \min[16\, d_b\, , 48\, d_s\, , \min(b, h)]$

d_b = main bar diameter

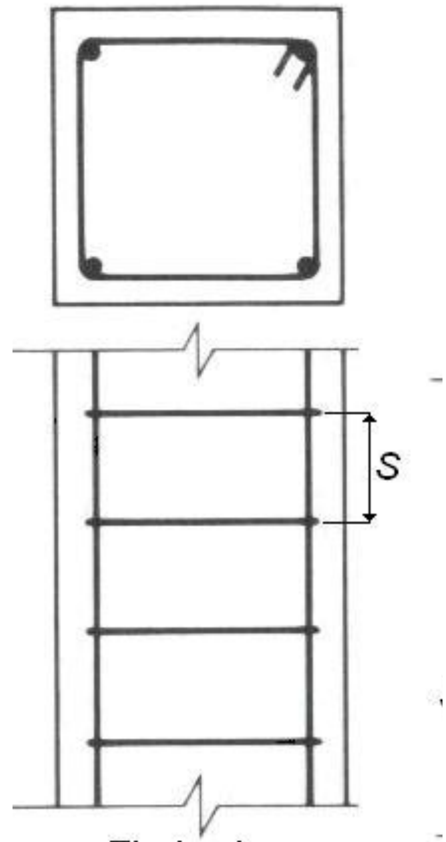
d_s = tie (stirrup) diameter

(b, h) = section dimensions

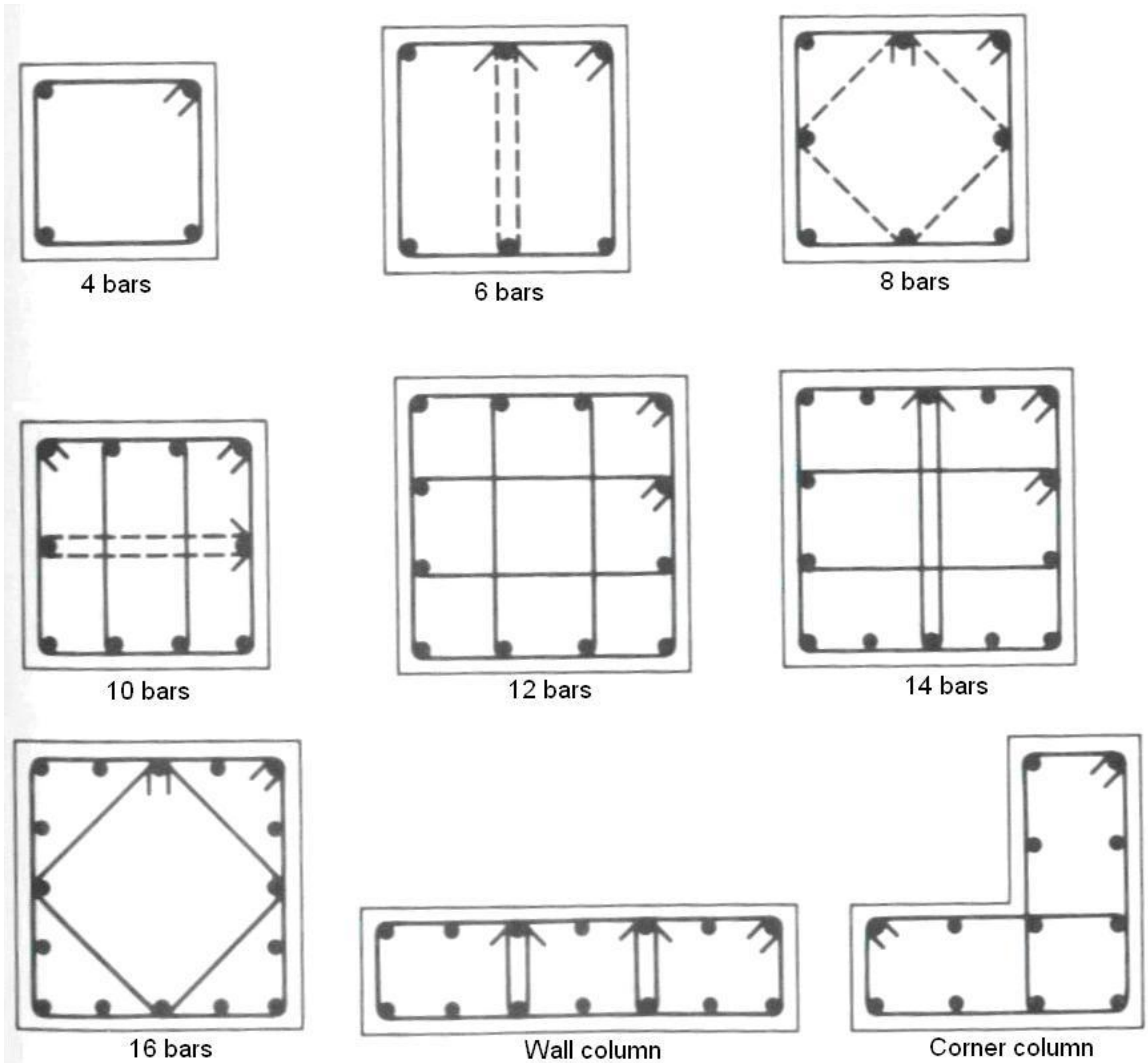
$if\ d_b = 32\ or\ less \Rightarrow\ d_s \nless 10\ mm$

$if\ d_b > 32 \Rightarrow\ d_s \nless 12\ mm$

- Maximum distance between untied bar and tied one is 150 mm
- Minimum number of bars are 4 for rectangular or circular ties
- The maximum angle in a tie is 135º
- First tie at a distance of half spacing above slab and above footing
- Last tie at a distance of half spacing below lowest reinforcement bar of slab



Tied column


4 bars
6 bars
8 bars
10 bars
12 bars
14 bars
16 bars
Wall column
Corner column

2-spiral column:

$S = \min[16\, d_b\, , 48\, d_s\, , \min(b, h)]$

d_b = main bar diameter

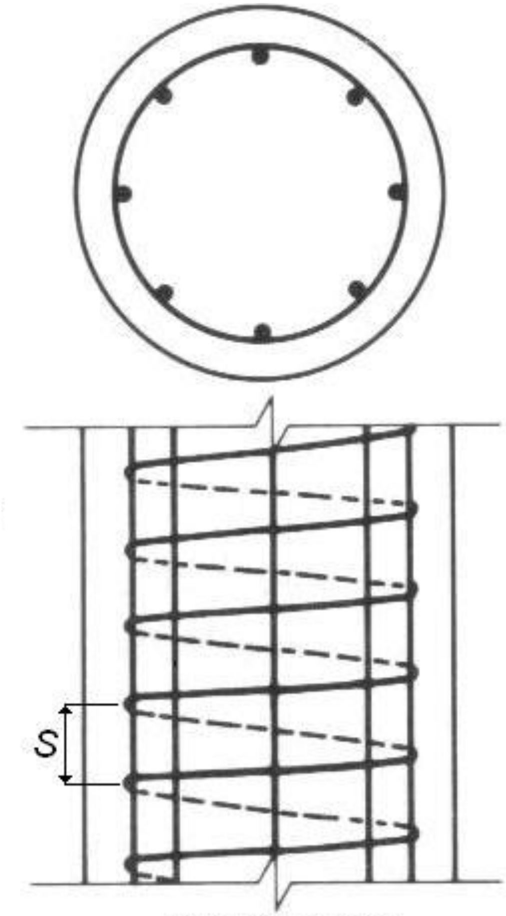
d_s = tie (stirrup) diameter

(b, h) = section dimensions

$if\ d_b = 32\ or\ less\ \Rightarrow\ d_s \nless 10\, mm$

$if\ d_b > 32\ \Rightarrow\ d_s \nless 12\, mm$

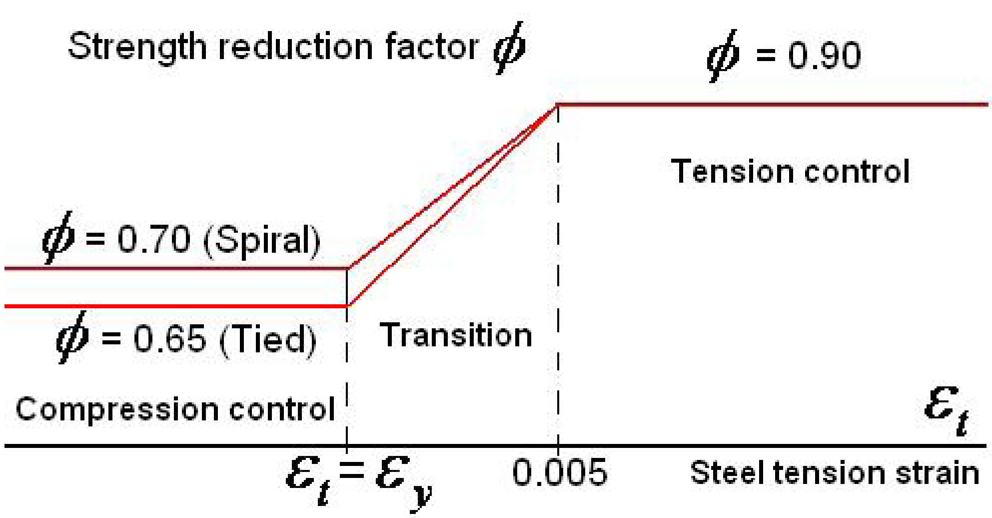
- Minimum number of bars are 6 for spirals



Spiral column

- **Strength reduction factor for columns:-**

$$\emptyset \boldsymbol{P_n} \geq \boldsymbol{P_u} \qquad , \qquad \emptyset \boldsymbol{M_n} \geq \boldsymbol{M_u}$$



- **Longitudinal reinforcement:-**

$$\rho = \frac{A_s}{A_g}$$

$$\rho = (1-8)\% \;\; of\ gross\ area\ \ A_g\ of\ section$$

$$the\ most\ economical\ tied\ column\ section\ generaly\ involves\ \rho\ (1-2\ )\ percent$$

- **Strength of columns in axial compression:-**

$$\emptyset P_{n,max} \ = 0.85 * 0.65 * \left[0.85\ f_c`\left(A_g - A_{st}\right) + \ f_y * A_{st}\ \right] \Rightarrow for\ Spiral\ column$$

$$\emptyset P_{n,max} \ = 0.80 * 0.65 * \left[0.85\ f_c`\left(A_g - A_{st}\right) + \ f_y * A_{st}\ \right] \Rightarrow for\ Ties\ column$$

- **Column tension strength:-**

$$\emptyset P_{n,Tension} \ = -\ 0.9 * f_y * A_g$$

- **Concrete shear strength for columns**

$$\phi V_c = 0.75 * \left[1 + \frac{P_u}{14 * A_g}\right] \frac{\sqrt{f_c}}{6} * b_w * d * 10^{-3}$$

if $0.5\phi V_c < V_u$ shear reinforcement is required

- **Design of concrete section:-**

  The minimum gross section of column is

  1- Tied column:-

$$A_g \geq \frac{P_u}{0.4(f_c` + \rho * f_y)}$$

  2- Spiral column:-

$$A_g \geq \frac{P_u}{0.5(f_c` + \rho * f_y)}$$

Example 3)

From previous example 1 .Design a tied interior column CD in the second floor.

$Pu = 900.728\,kn$, $Lu = 3.6\,m$, $f_c^{'} = 35MPa$, $f_y = 420\,MPa$, $\rho = 1\%$

Estimate of own weight of column =(2 - 5)% of reaction of beams

$$Pu = (2 * 900.728) + \left(\frac{2 * 900.728}{100}\right) = 1819.471\,kn$$

$$A_g \geq \frac{P_u}{0.4(f_c` + \rho * f_y)} = \frac{1819.471 * 10^3}{0.4(30 + 0.01 * 420)} = 133002.27\,mm^2$$

- For square section b=h $\geq \sqrt{133002.2} = 364.7\,mm$

  Use (b , h) =(370mm , 370mm)

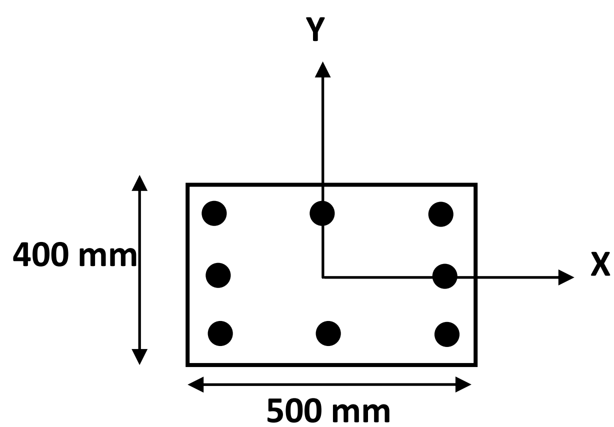
- For rectangluar section b= 400 mm then h $\geq \frac{133002.27}{400} = 332.5\,mm$
  Column size (400,500) is adequate because actual area is greater than required area .

Select reinforcement:-

$$\rho = \frac{A_s}{A_g}$$

$$A_s = \rho * A_g = 0.01 * (400 * 500) = 2000\,mm^2$$

$use\ d_b = 20\,mm \Rightarrow n = \frac{2000}{314} = 6.37\ use\ even\ number = 8$



Check maximum compression capacity:

$$\emptyset P_{n,max} = 0.80 * 0.65 * [0.85 * 30 * (200000 - 2512) + 420 * 2512] * 10^{-3} = Pu = 1$$

$$\emptyset P_{n,max} = 3167.272\, KN > Pu = 1819.471\, kn$$