Columns

- Columns-
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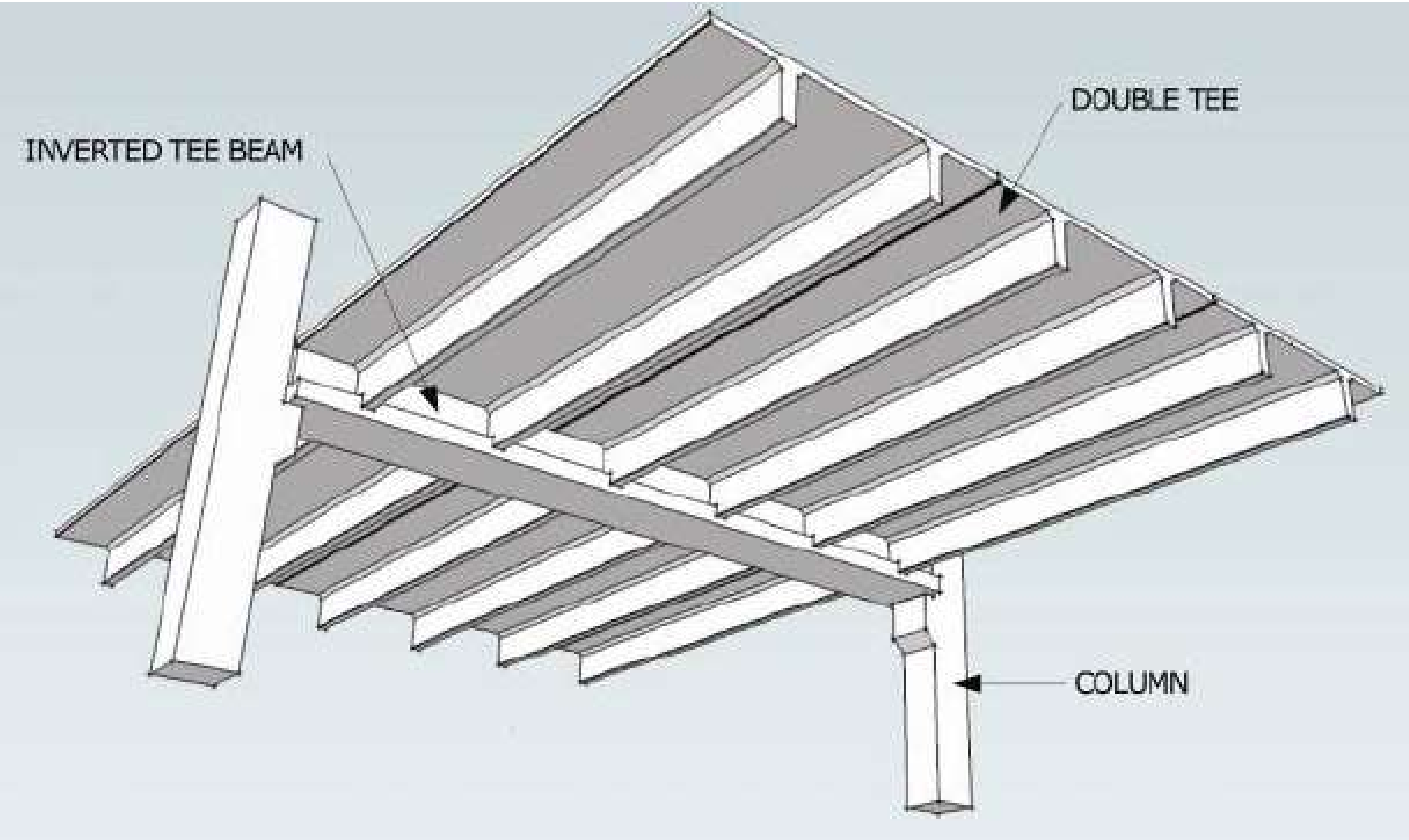
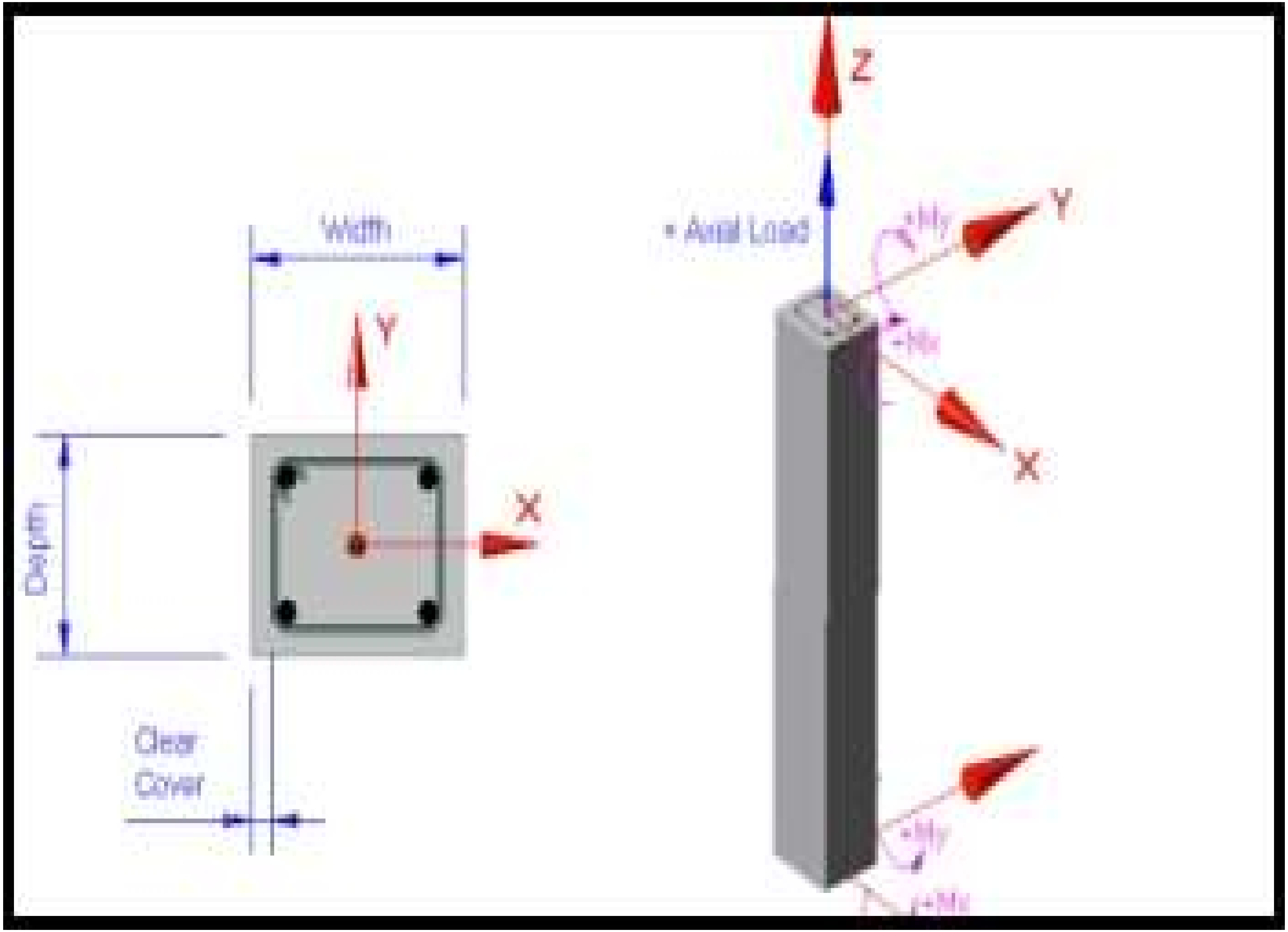
  - effective length-
  - short column-long column-
  - reinforcement-
  - IS specifications regarding columns-
  - limit state of collapse: compression –
  - design of axially loaded short columns-
  - design examples with rectangular ties and helical reinforcement



COLUMN - BEAM FLOOR SYSTEM



Width
Y
X
Depth
Clear
Cover
Z
+ Axial Load
Y
+My
+Mx
X
+My
+Mx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Column is a vertical compression member whose unsupported length as per *Cl. 25.3(pg.42)*

- *shall not exceed sixty times of b (least lateral dimension), if restrained at the two ends.*

- *For a cantilever column shall not exceed $100b^2/D$, where D is the larger lateral dimension which is also restricted up to four times of b*

Effective length: *Clause 25.2 &pg94*

The vertical distance between the points of inflection of the compression member in the buckled configuration in a plane is termed as effective length L_e *in that plane. It depends on the type of end restraints.*

$$L_e = k\,L$$

where *k is the ratio of effective to the unsupported lengths.*

Based on Types of Lateral Reinforcement

Tied Column

Helical reinforcement

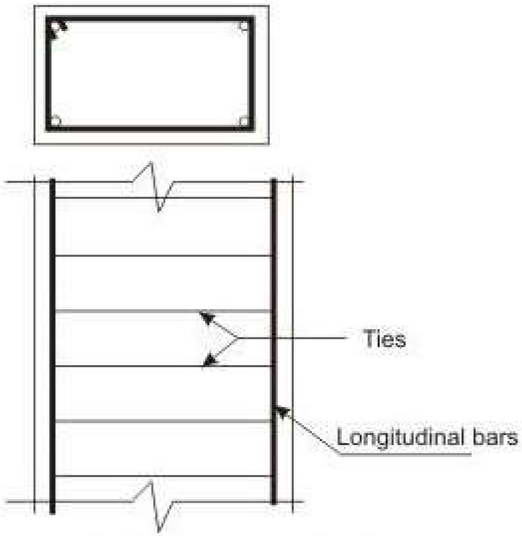


Fig. 10.21.2(a): Tied column

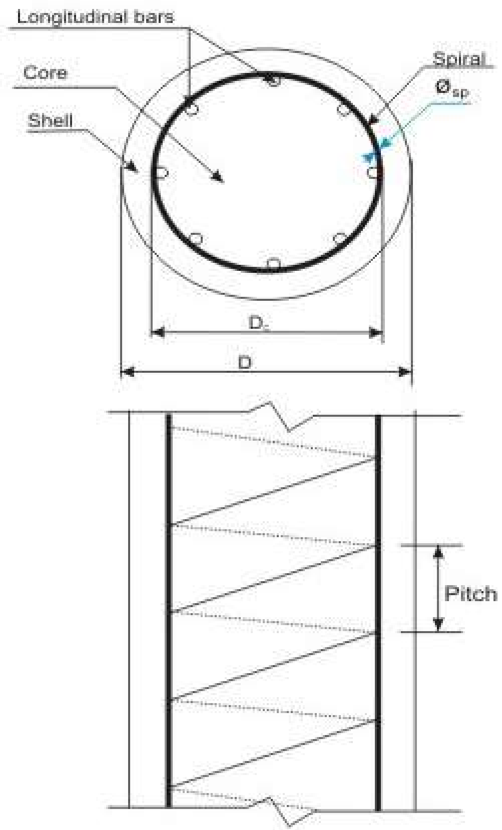


Fig. 10.21.2b: Column with helical reinforcement

Based on Loadings

- Axial loads only (concentric)
  – Interior Columns
- Axial load and uniaxial bending
  – Peripheral Columns
- Axial load and bi-axial bending
  – Corner Columns

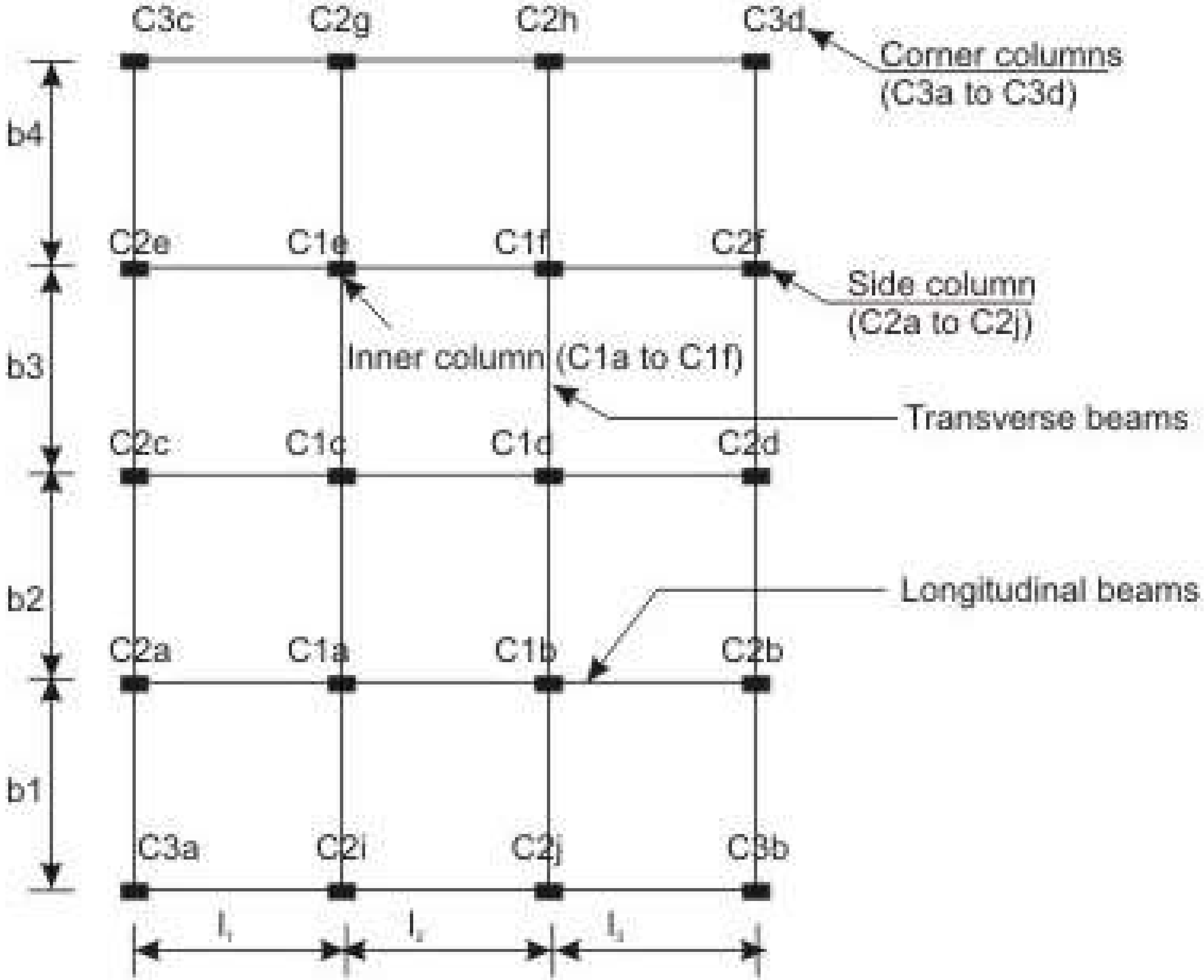


Fig. 10.21.4: Grid of beams and columns

Minimum Eccentricity pg.42; CL. 25.4

A column loaded axially will have accidental eccentricity due to inaccuracy in construction or variation of materials etc. Hence should be designed considering the minimum eccentricity

$$e_{x\ min} \geq (L/500 + D/30) \text{ or } 20 \text{ mm}$$

$$e_{y\ min} \geq (L/500 + b/30) \text{ or } 20 \text{ mm}$$

L = unsupported length, D = larger lateral dimension
b = least lateral dimension

Slenderness Ratio Cl. 25.1.2

Column is considered as *short* when both the slenderness ratios

L_{ex} / D and L_{ey} / b are less than 12

Otherwise considered as *slender*

Failure Modes

Three modes of failure with different slenderness ratios.

Column does not undergo any lateral deformation and collapses due to material failure. This is known as *compression failure*.

Due to the combined effects of axial load and moment a short column may have material failure. Such failure is called *combined compression and bending failure*.

Failure by elastic instability of very long column even under small load much before the material reaches the yield stresses. This type of failure is known as *elastic buckling. Should be avoided*

Longitudinal Reinforcement (pg.48) CL 26.5.3.1

The longitudinal reinforcing bars carry the compressive loads along with the concrete and tensile forces if any.

1. Ast, min = 0.8 % of the gross cross-sectional area required if the provided area is more than the required area.

2. Ast,max = 4 % of the gross cross-sectional area of the column so that it does not exceed 6 % when bars have to be lapped

3. Minimum Four and six for rectangular and circular columns

4. Dia > = 12 mm.
5. The bars shall be spaced not exceeding 300 mm along the periphery of the column.

Transverse Reinforcement CL 26.5.3.2

The transverse reinforcement, provided in form of lateral ties or spirals, for

(a) preventing premature / local buckling of the longitudinal bars,

(b) improving ductility and strength by the effect of confinement of the core concrete,

(c) holding the longitudinal bars in position during construction, and

(d) providing resistance against shear and torsion, if present.

Pg.49,cl.26.5.3.3.c

1) *Pitch*—The pitch of transverse reinforcement shall be not more than the least of the following distances:

   i) The least lateral dimension of the compression members;

   ii) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied; and

   iii) 300 mm.

2) *Diameter*—The diameter of the polygonal links or lateral ties shall be not less than one-fourth of the diameter of the largest longitudinal bar, and in no case less than 6 mm.

Transverse reinforcement shall only go round corner and alternate bars if the longitudinal bars are not spaced more than 75 mm on either side .

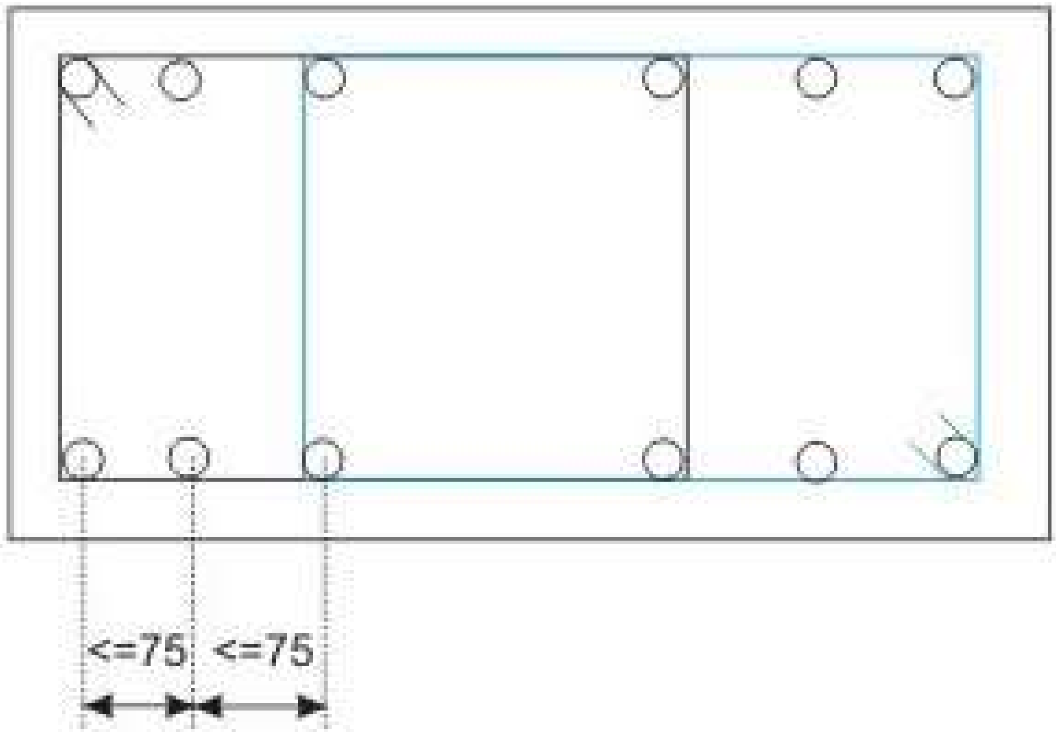


Fig. 10.21.7: Lateral tie (Scheme 1)

Longitudinal bars spaced at a maximum distance of 48 times the diameter of the tie shall be tied by single tie and additional open ties for in between longitudinal bars

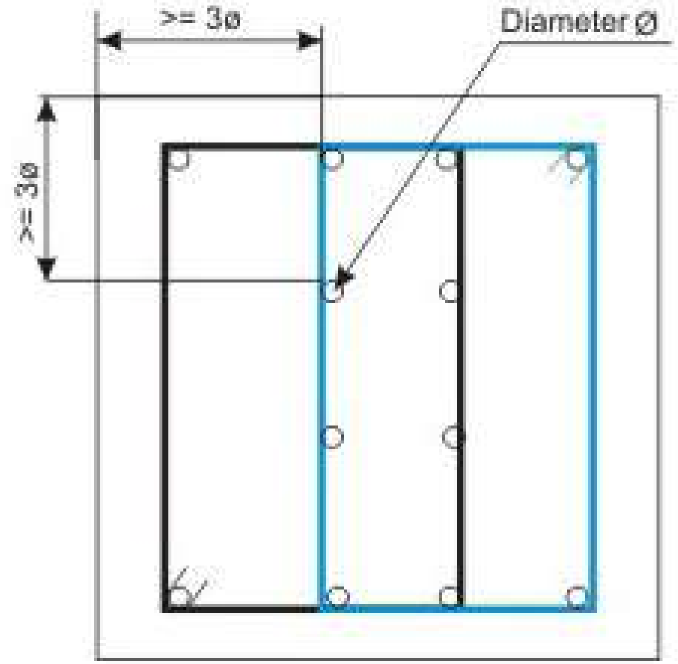


Fig. 10.21.9: Lateral tie (Scheme 3)

Design of Short Axially Loaded Compression members

CL 39.3. pg.71

39.3 Short Axially Loaded Members in Compression

The member shall be designed by considering the assumptions given in 39.1 and the minimum eccentricity. When the minimum eccentricity as per 25.4 does not exceed 0.05 times the lateral dimension, the members may be designed by the following equation:

$$P_u = 0.4 f_{ck} . A_c + 0.67 f_y . A_{sc}$$

where

P_u = axial load on the member,

f_{ck} = characteristic compressive strength of the concrete,

A_c = Area of concrete, $A_c = A_g - A_{sc}$

f_y = characteristic strength of the compression reinforcement, and

A_{sc} = area of longitudinal reinforcement for columns.

Example 1: To find Asc for given column dimensions

Design the reinforcement in a column of size 400 mm x 600 mm subjected to an axial load of 2000 kN under service dead load and live load. The column has an unsupported length of 4.0 m and effectively held in position and restrained against rotation in both ends (Both ends Fixed). Use M 25 concrete and Fe 415 steel.

Step 1: check if the column is short or slender

L= 4000 mm, b = 400 mm and D = 600 mm.

From Table 28 of IS 456

$Lex = Ley = 0.65(L) = 2600$ mm.

$Lex/D = 2600/600 = 4.33 < 12$

$Ley/b = 2600/400 = 6.5 < 12$

Hence, it is a short column. As per CL 25.1.2

Step 2: Minimum eccentricity **CL 25.4**

$e_{x\,min}$ = Max (l_{ex}/500 + D/30 ; 20 mm) = 25.2 mm

$e_{y\,min}$ = Max (l_{ey} /500 + b/30 ; 20 mm) = 20 mm

0.05 D = 0.05(600) = 30 mm > 25.2 mm ($e_{x\,min}$)

0.05 b = 0.05(400) = 20 mm = 20 mm ($e_{y\,min}$)

Hence, CL.39.3 is applicable.

Step 3: Area of steel

$$P_u = 0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$$

$$3000\text{x}10^3 = 0.4(25)\{(400)(600) - A_{sc}\} + 0.67(415)\, A_{sc}$$

$$A_{sc} = 2238.39 \text{ mm}^2$$

Provide 6- #20 + 2-#16 ; Asc provided = 2287 mm^2

Step 4: Check for pt

p_t = 2287x100 /(400x600) = 0.953

> 0.8% and < 4%   OK

Step 4: Lateral ties

Dia : cl.26.5.3.2 C-2

not less than

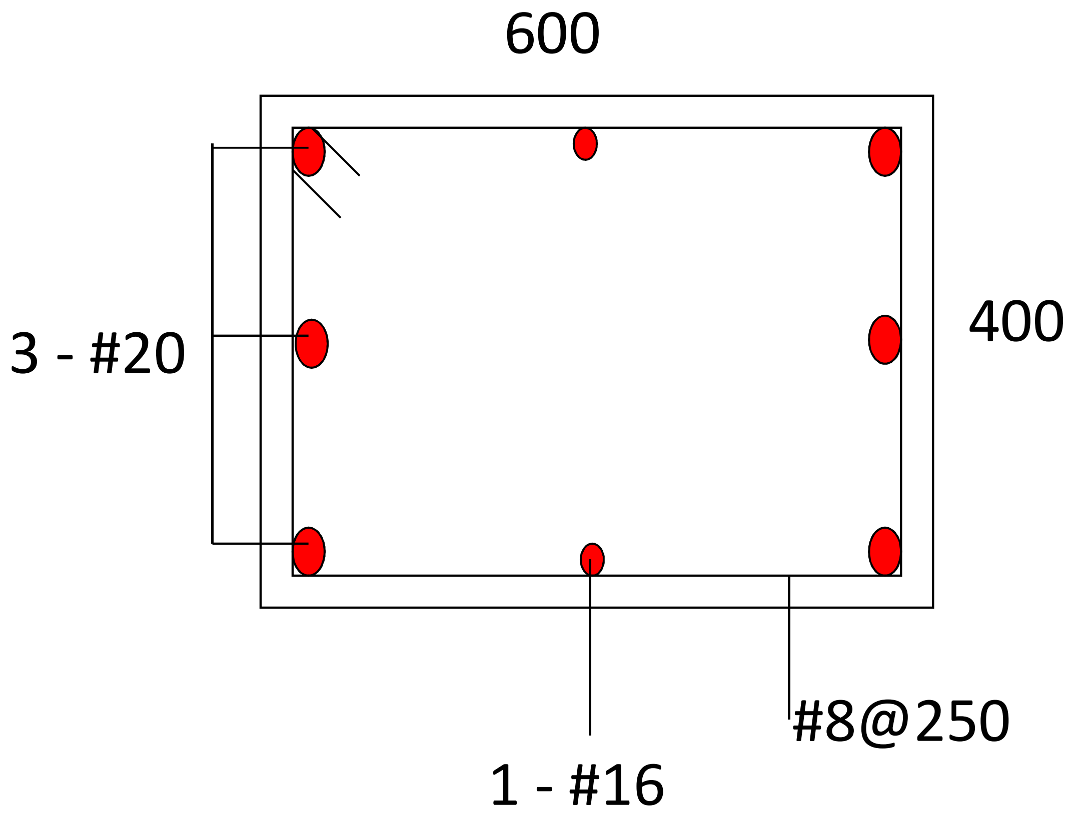
(i) $\phi/4$ = 20/4 = 5 mm and (ii) 6 mm. *Adopt #8*

pitch of lateral ties, as per cl.26.5.3.2 C-1

not more than the least of

(i) 400 mm; (ii) 16(16) = 256 mm; (iii) 300 mm

Adopt # 8 @ 250mm c/c



Clear Cover = 40 mm

Spacing of longitudinal bars in both directions < 300 mm OK

Example 2 : To find Asc if Column size is more than required

Data as in Example 1 but Pu = 1500 kN

Step 3: Area of steel

$$P_u = 0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$$

$$1500 \times 10^3 = 0.4 \times 25\{400 \times 600 - A_{sc}\} + 0.67 \times 415 \times A_{sc}$$

$$A_{sc} = \text{-ve value}$$

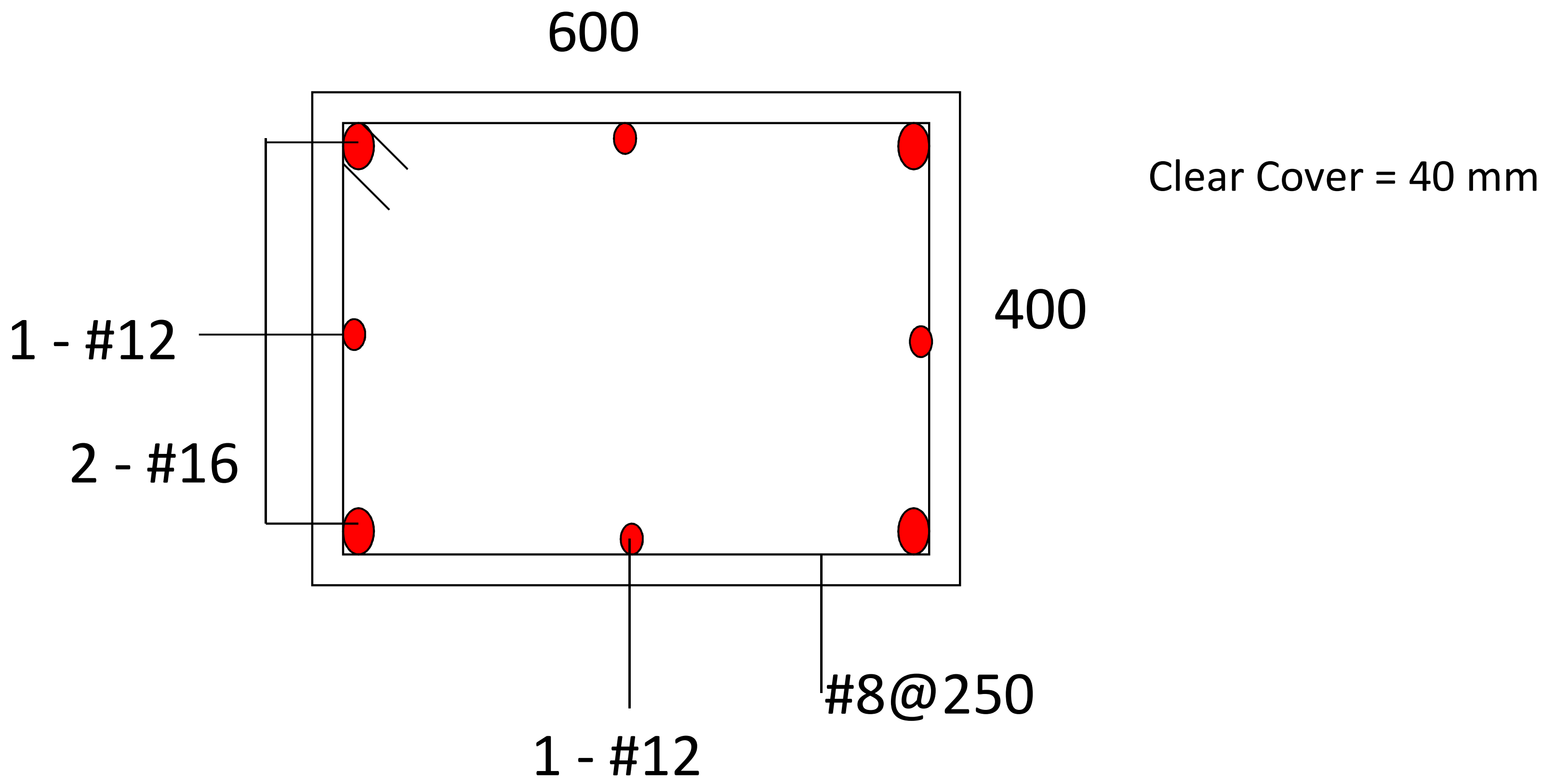
In such cases, column size is more than what is required.

Ac, required = $1500 \times 10^3/(0.4 \times 25)$ = 150000 mm^2

As per 26.5.3.1 (b)

Ast = (0.8 x 150000/100) = 1200 mm^2

Provide 4- #16 + 4 - #12 , Ast = 1256 mm^2



Spacing of longitudinal bars in both directions < 300 mm OK

Example 3: To find dimension of column for given Asc

Design a short rectangular tied column of b = 300 mm having the maximum amount of longitudinal reinforcement, to carry an axial load of 2000 kN under service dead load and live load using M 25 and Fe 415. The column is effectively held in position at both ends and restrained against rotation at one end (One end Fixed and other Hinged). Determine the unsupported length of the column.

Step 1: Dimension D and area of steel Asc

$P_u = 1.5(2000) = 3000$ kN

$A_{sc} = 0.04x300xD = 12D$ (4% maximum)

$A_c = (300D – A_{sc})$
$= 300D(1- 0.04) =288D$

$3000x10^3 = 0.4x25[288D] + 0.67x415[12D]$

D = 482.57 mm.

Adopt 300 mm x 500 mm column.

Asc = 0.04(300)(500) = 6000 mm^2

Step 2: Rebar details

i) Longitudinal Reinforcement:

Provide 4 - #32 + 6 - #25

A_{sc} Provided

3217 + 2945 = 6162 mm^2 > 6000 mm^2.

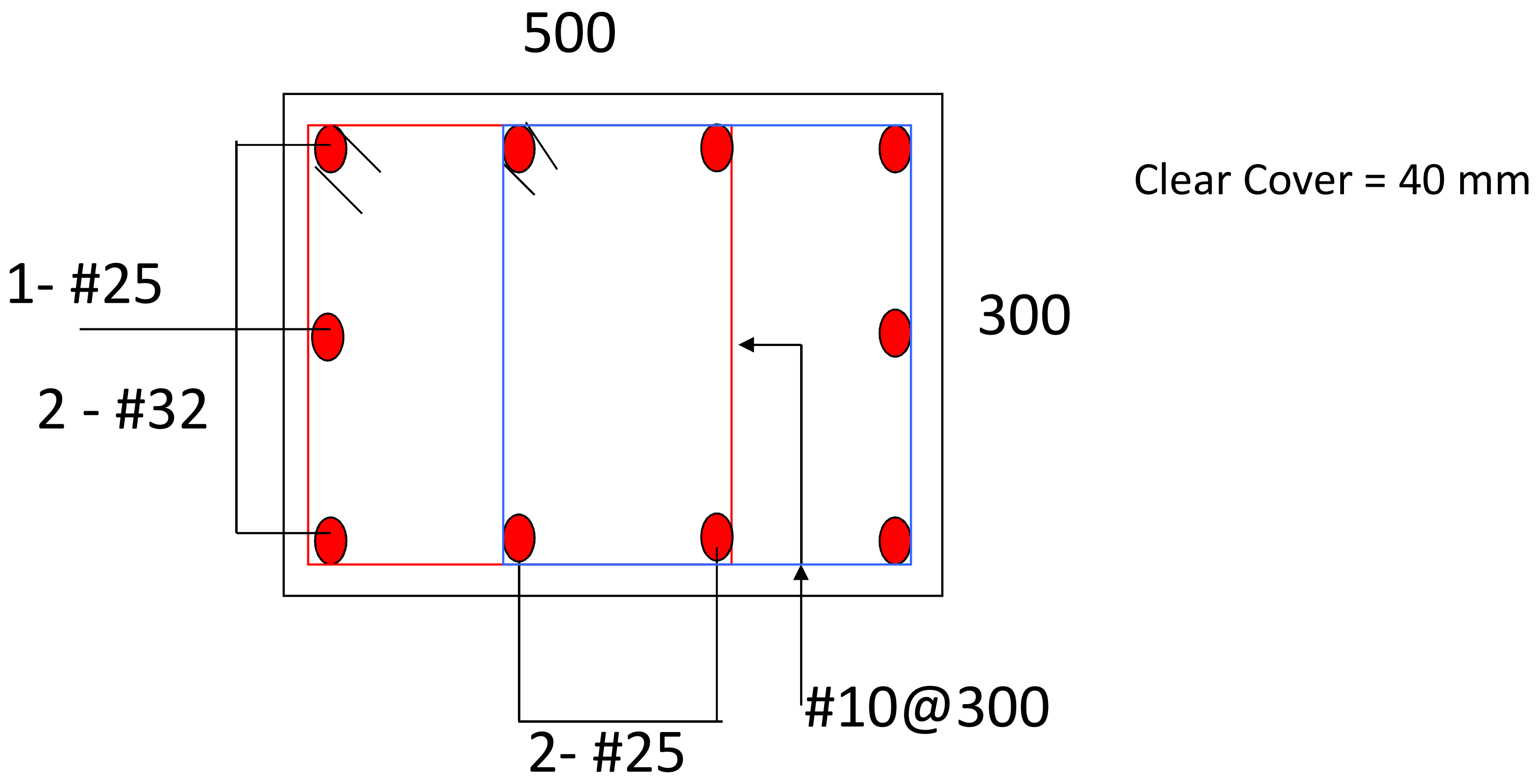
ii) Lateral ties

Diameter not less than the larger of

(i) 32/4 = 8 mm and (ii) 6 mm. *Adopt #10*

Pitch of the lateral ties shall not be more than the least of (i) 300 mm, (ii) 16(25) = 400 mm (iii) 300 mm.

Adopt #10@300 mm c/c.



Spacing of longitudinal bars in both directions < 300 mm OK

Step 3: Unsupported length

Based on Slenderness Ratio = 12

L_{ex} = 12(*500*) = 6000 mm
L_{ey} = 12(300) = 3600 mm.

From Table 28; k = 0.8

(i) L = L_{ex} / 0.8 = 6000/0.8 = 7500 mm
(ii) L = L_{ey} / 0.8 = 3600/0.8 = 4500 mm.

<u>Based on minimum eccentricity</u>

(i) $0.05 \times 500 = (L / 500) + (500/30)$

$L = 4167$ mm

(ii) $0.05 \times 300 = (L / 500) + (300/30)$

$L = 2500$ mm

Maximum Unsupported Length = 2500mm

Example 4: To find Column Size and Asc

Design a short rectangular tied column, to carry a factored axial load of 3000 kN using M 20 and Fe 415. The column is effectively held in position at both ends and not restrained against rotation at both ends (Both ends hinged). The unsupported length of the column = 3m.

Step 1: Column Size (b x D) and area of steel Asc

Assume Short column and CL 39.3 is applicable
(e_{min} < 0.05 D or b)

i) Column Size

P_u = 3000 kN ; Assume Asc =2% of Ag ; A_{sc} = 0.02bD

$A_c = (bD – A_{sc}) = bD(1- 0.02) =0.98bD$

$3000x10^3 = 0.4x20[0.98bD] + 0.67x415[0.02bD]$

bD = 223864 mm^2.

Assume D=1.5b; b = 386mm, D = 579 mm
Adopt 400 mm x 600 mm column.

ii)Find Asc

Ac = 400x600 - Asc = 240000 - Asc

$3000 \times 10^3 = 0.4 \times 20\ [240000 - Asc] + 0.67 \times 415[Asc]$

Asc = 4000 mm^2

Step 2: Rebar details

i) ***Longitudinal Reinforcement:***

Provide 6 - #25 + 4 - #20 mm

A_{sc} Provided

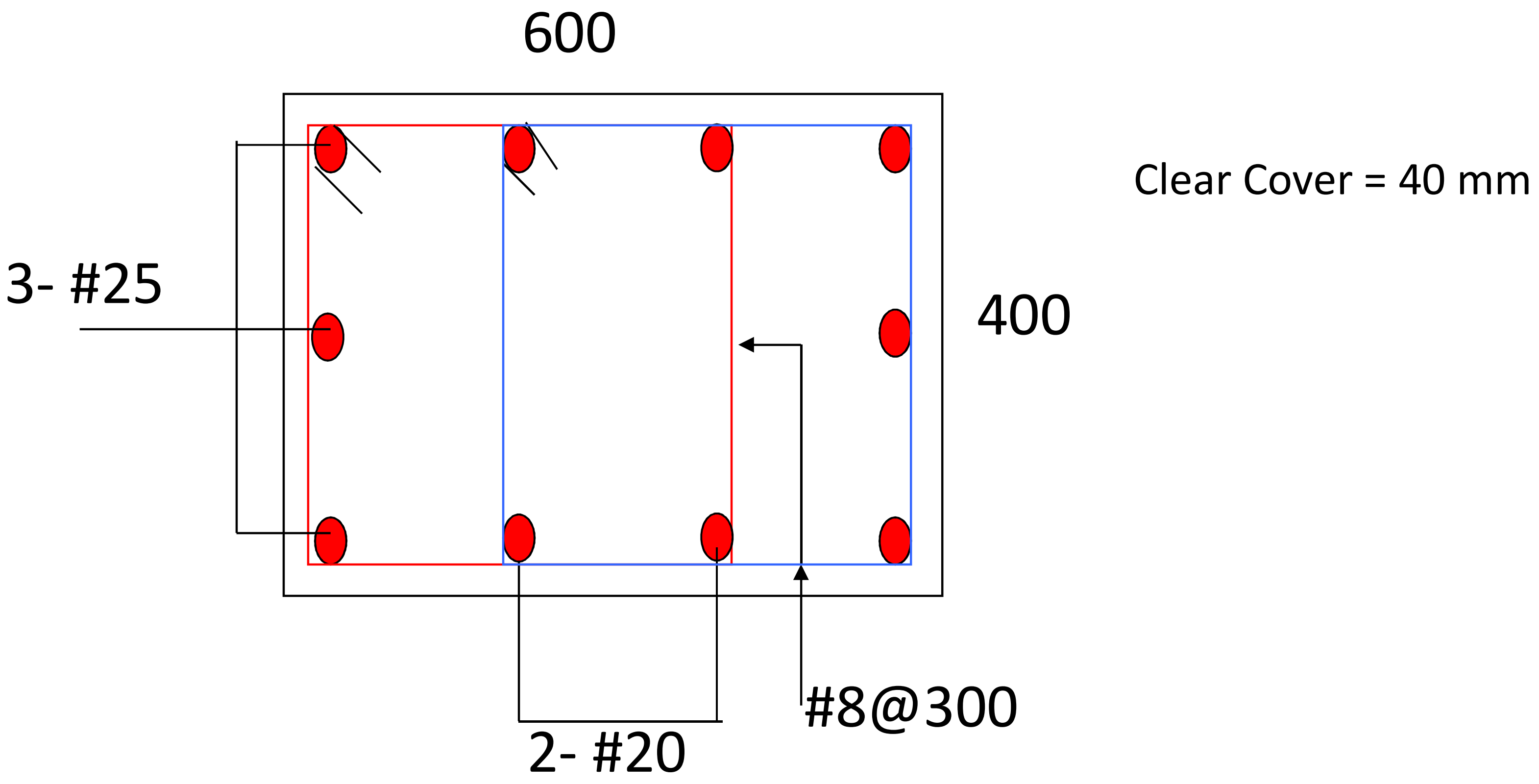
$2945 + 1257 = 4202 \text{ mm}^2 > 4000 \text{ mm}^2$.

ii) ***Lateral ties***

Diameter not less than the larger of
(i) 25/4 = 6.25 mm and (ii) 6 mm. ***Adopt #8***

Pitch of the lateral ties shall not be more than the least of (i) 400 mm, (ii) 16(20) = 320 mm (iii) 300 mm.

Adopt #8@300 mm c/c.



Spacing of longitudinal bars in both directions < 300 mm OK

Step 3: Check for Slenderness

From Table 28, k = 1, Lex = Ley = kL = 3000

$L_{ex} / D = 3000/600 = 5 < 12$

$L_{ey} = 3000/400 = 7.5 < 12$

Hence the Column is short   Assumption is Ok

Step 4:

__Check minimum eccentricity < 0.05 lateral dimension__

(i) (3000 / 500) + (600/30) = 26 mm

0.05D = 0.05x600 = 30 mm > 26 OK

(ii) (3000 / 500) + (400/30) =19.33 mm < 20 = 20 mm

0.05b = 0.05x400 = 20 mm = 20 OK

CL 39.3 is applcable

Circular Column with Helical Reinforcement (CL 39.4)

39.4 Compression Members with Helical Reinforcement

The strength of compression members with helical reinforcement satisfying the requirement of **39.4.1** shall be taken as 1.05 times the strength of similar member with lateral ties.

39.4.1 The ratio of the volume of helical reinforcement to the volume of the core shall not be less than $0.36\,(A_g/A_c-1)\,f_{ck}/f_y$

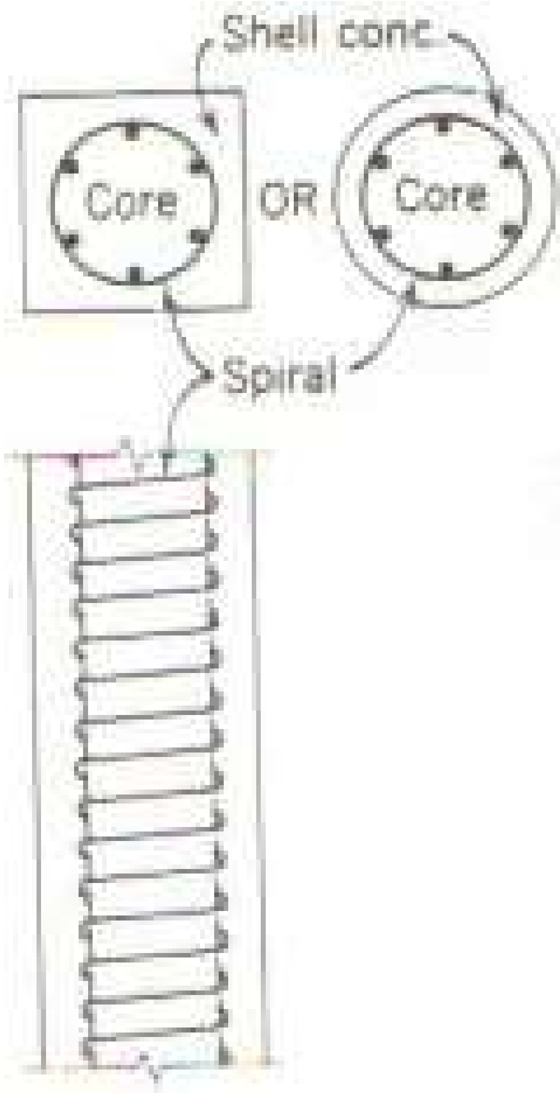
where

A_g = gross area of the section,

A_c = area of the core of the helically reinforced column measured to the outside diameter of the helix,

f_{ck} = characteristic compressive strength of the concrete, and

f_y = characteristic strength of the helical reinforcement but not exceeding 415 N/mm².



Example 5 :

Design a short, helically reinforced circular column with minimum amount of longitudinal steel to carry a total factored axial load of 3000 kN with both ends pinned. Use M 25 and Fe 415. Also determine its unsupported length.

Step 1:
Diameter of helically reinforced circular column (D)

As per cl. 39.4

$P_u = 1.05(0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc})$

$P_u = 3000\ kN\ ,\ A_g = (\pi/4)\ D^2$

$A_{sc} = 0.008(\pi/4)\ D^2$ *(assuming minimum Ast = 0.8% Ag)*

$A_c = A_g - A_{sc} = (\pi/4)\ (D^2)\ (1 - 0.008)$

$A_c = 0.992\ (\pi/4)\ (D^2)\ ,\ f_{ck} = 25MPa,\ f_y = 415MPa$

$P_u = 1.05(0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc})$
$\quad = 1.05 x \pi/4\ (D^2)\ (9.92 + 2.22)$

$3000 \times 10^3 = 1.05 \times 12.14 \times (\pi/4)D^2$

$D = 547.4$ mm <u>*Provide diameter of 550 mm.*</u>

Step 2: Area of longitudinal steel

$A_{sc} = 0.008\,A_g = 0.008(237583) = 1900\ \text{mm}^2$.

<u>*Provide 10- #16*</u>

Step 3: Pitch of Helix (p)

Assume #8 bars, clear cover to Helix = 40 mm

i) <u>Volume ratio</u>

$Dc = 550 – 40x2 = 470\ mm;\ \varphi_{sp} = 8\ ;\ a_{sp} = (\pi/ 4)\ x8^2 = 50\ mm^2$

Volume of helix in one loop = $\pi\ (D_c - \varphi_{sp})\ a_{sp}$

$= \pi\ x(470\text{-}8)50 = 7257\ 0\ mm^3$

Volume of core = $(\pi/4)\ x\ p\ x\ Dc^2 = 173494.5\ p\ mm^3$

Ratio = $72570\ /\ 173494.5\ p = 0.418/p$

ii) $0.36(A_g / A_c - 1)(f_{ck} / f_y)$

$$A_g = (\pi / 4) \times 550^2 = 237583 \text{ mm}^2$$

$$A_{c} = (\pi / 4) \times 470^2 = 173494.45 \text{ mm}^2$$

$$0.36\,(1.37-1)(25/415) = 0.008024$$

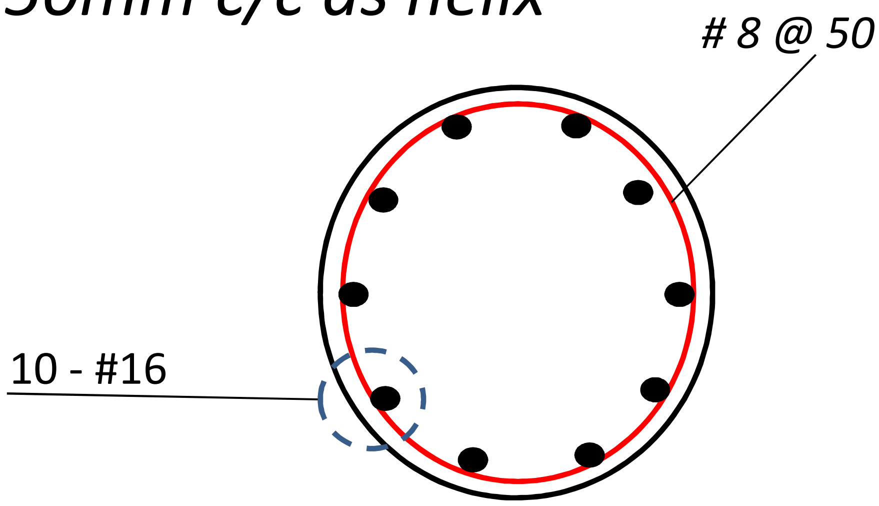
iii) Pitch, p

$$0.418 / p = 0.008024$$
$$p = 52 \text{ mm}$$

iv) Check for Spacing as per CL 26.5.3.2 (d)

$$p \quad < 75mm$$
$$< (1/6) \times 470 = 78\ mm$$
$$>= 25\ mm$$
$$>= 3 \times 8 = 24\ mm$$

Adopt # 8 @ 50mm c/c as helix



Step 4: Unsupported length

Short column requirement

$$L = L_e = 12\,D = 12(550) = 6600\ mm$$

minimum eccentricity requirement

$Le/500 + 550/30 = 0.05 \times 550$

$Le = 4583.3\ mm$

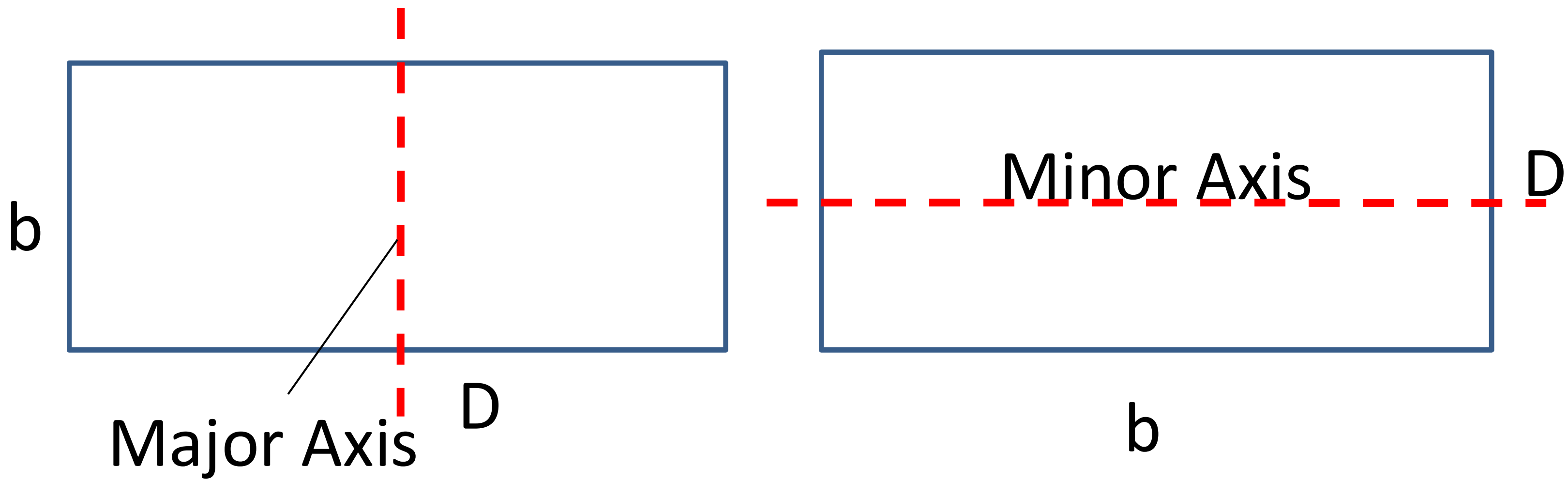
Pinned at both ends, $k=1$; $Le = kL$; $L = 4.58\ m$

Columns subjected to

Axial Load + Uni-axial Bending

Uni axial moments are generated either due to eccentricity of load or due to moments transferred form beam

Major and Minor Axis of a Rectangular Section



Major Axis:

Centroidal Axis about which Moment of Inertia is maximum

Minor Axis:

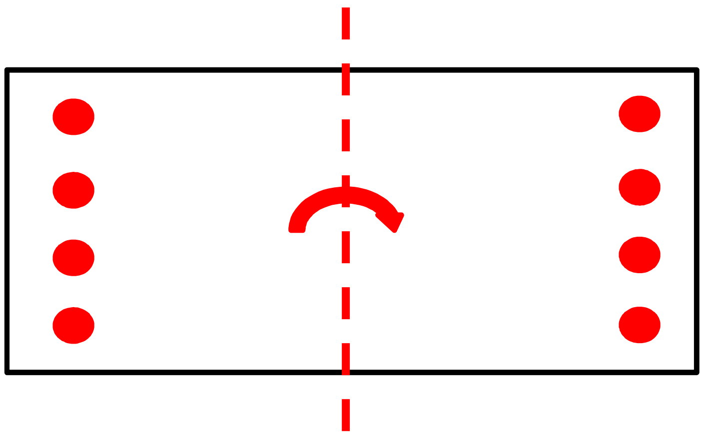
Centroidal Axis about which Moment of Inertia is minimum

Dimension parallel to axis is 'b' and perpendicular to axis is 'D'

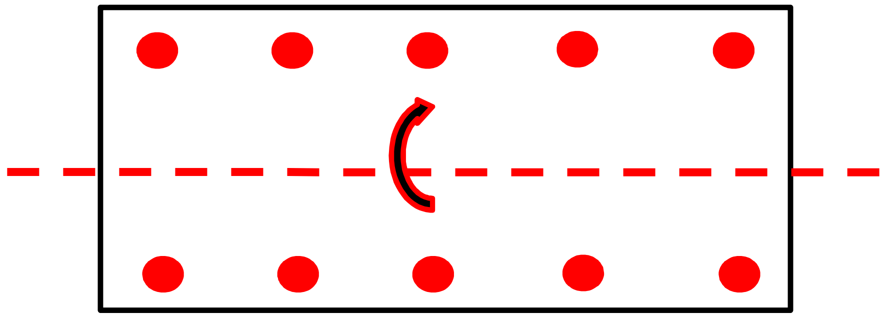
Rebar Arrangement in Uni Axial Bending

1. Two Face Arrangement (Distributed equally)

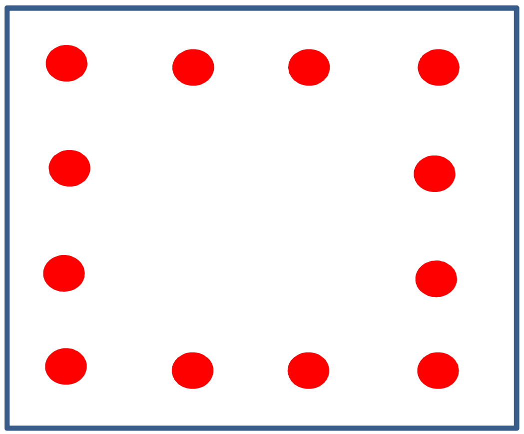
Case A: Bending About Major Axis



Case B: About Minor Axis



2. Four Face Arrangement (Distributed Equally)



Not efficient for uni axial bending

Adopted for Axial loads and Biaxial bending cases

Column Section should be square

Column Orientation

In the case of rectangular columns, orientation should be such that larger dimension is available to resist moments.

<u>Design Charts: (SP 16)</u>

<u>Charts 31– 38</u>

Rectangular Section with Two face Reinforcement

<u>Charts 39 – 50</u>

Rectangular Section with Four face Reinforcement

<u>Charts 51 – 62</u>

Circular sections

Use of Charts

Charts are given for

Non dimensional parameters :($P_u/f_{ck}bD$, $M_u/f_{ck}bD^2$)

Three Grades of Steel; fy = 250,415,500MPa

Four Values of d'/D = 0.05,0.1,0.15,0.2

Ref appropriate chart and get p/fck

Asc = pbD/100