COLUMNS

BY

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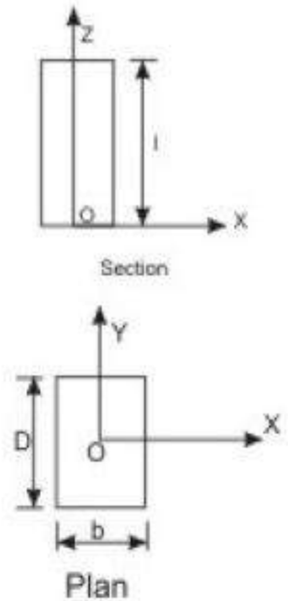
OBJECTIVE

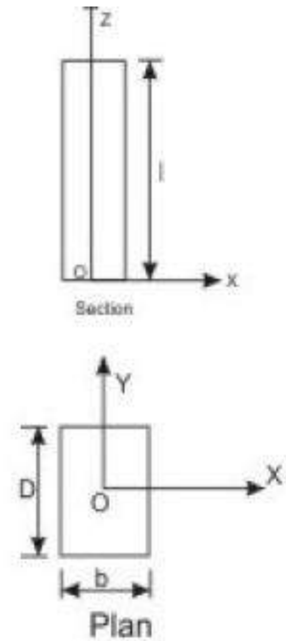
- After completion of the course ,the students will be able
- Classify the different types of materials.
- Design the columns with different types of loading with codal provisions.
- Differentiate braced and non braced columns.

FIGURE FOR PEDESTRAL /COLUMN



l_e <= 3b
D <= 4b
b < D

Pedestal



l <= 60b (restrained at both ends)
l <= $100b^2$ /D (cantilever)
D <= 4b
b < D

Columns

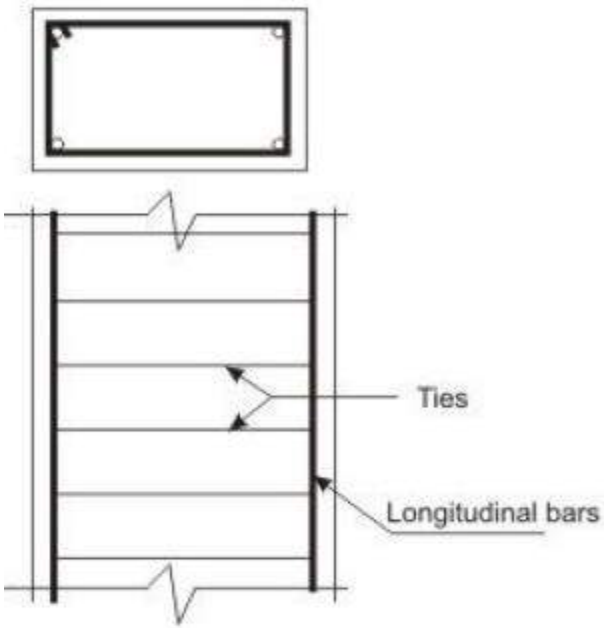
Leff/b $\leq$ 3 pedestal

3 <Leff/b $\leq$12 , Short Columns

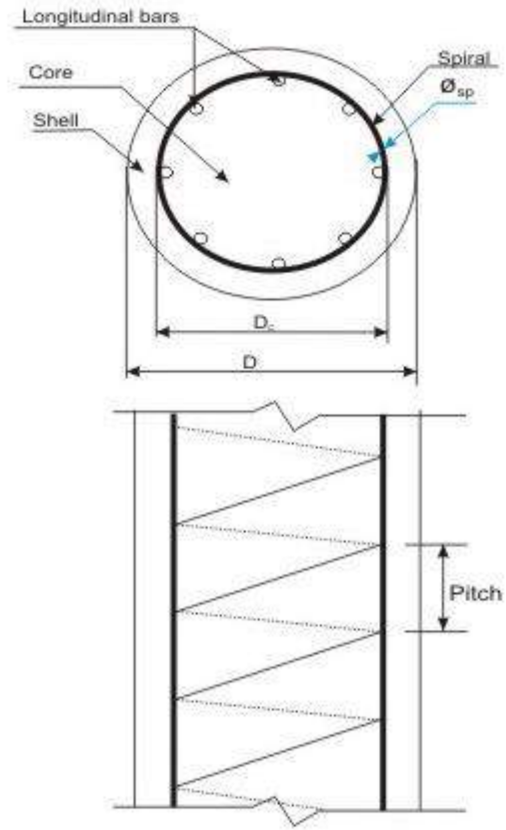
Leff/b>12 , Long Column

CLASSIFICATION ON SLENDERNESS RATIO

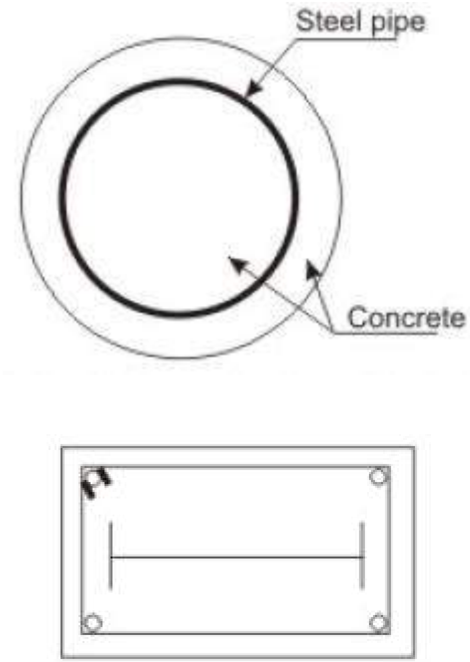
According to Reinforcement



Tied Column



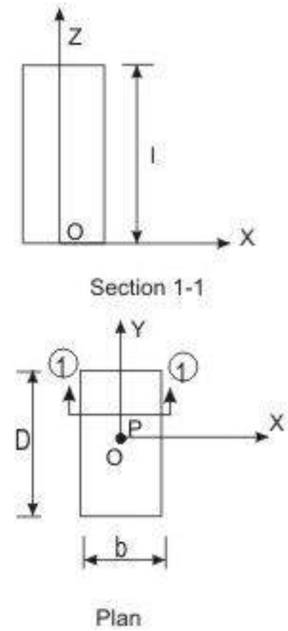
Helical reinforcement



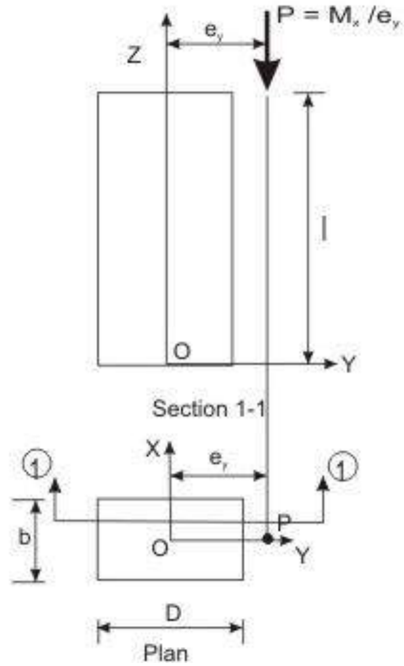
Composite Section

Composite section

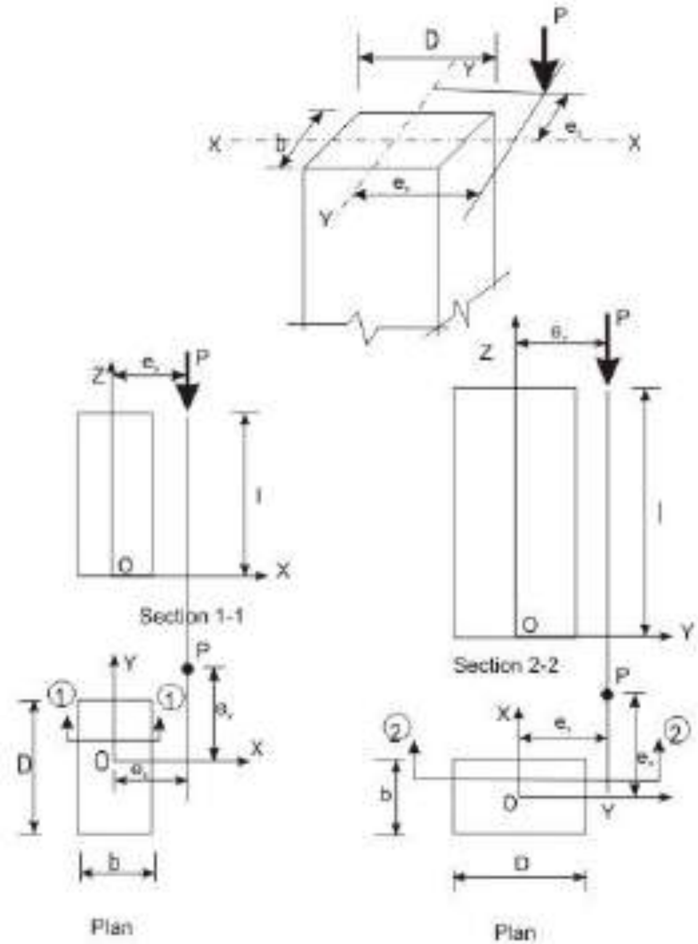
- Tied columns: The main longitudinal reinforcement bars are enclosed within closely spaced lateral ties .
- Columns with helical reinforcement: The main longitudinal reinforcement bars are enclosed within closely spaced and continuously wound spiral reinforcement. Circular and octagonal columns are mostly of this type .
- Composite columns: The main longitudinal reinforcement of the composite columns consists of structural steel sections or pipes with or without longitudinal bars.



Axial Loading



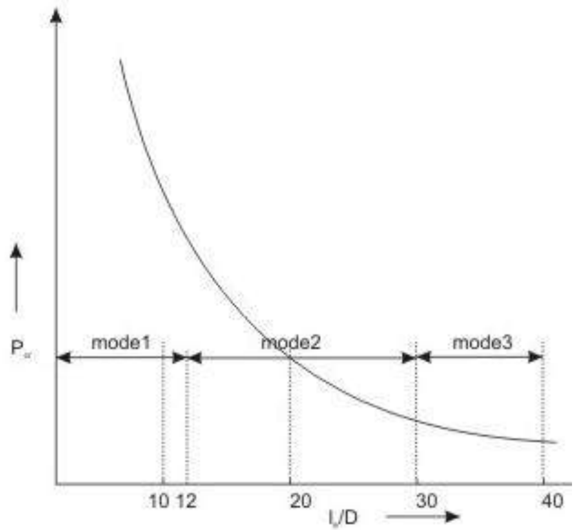
Eccentrically Loading



According to Loading condition

Columns are classified into the following two types based on the slenderness ratios:

- (i) Short columns (ii) Slender or long columns



The failure diagram

- presents the three modes of failure of columns with different slenderness ratios
- when loaded axially. In the mode 1, 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 of mode 2. On the other hand, a slender column subjected to axial load only undergoes deflection due to beam-column effect and may have material failure under the combined action of direct load and bending moment. Such failure is called combined compression and bending failure of mode 2.

- Mode 3 failure is 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.
- The slenderness ratio of steel column is the ratio of its effective length l^e *to its least radius of gyration r. In case of reinforced concrete column, however, IS 456 stipulates the slenderness ratio as the ratio of its effective length* l^e *to its least lateral dimension. As mentioned earlier, the effective length* l_e *is different from the unsupported length, the rectangular reinforced concrete column of cross-sectional dimensions b and D shall have two effective lengths in the two directions of b and D. Accordingly, the column may have the possibility of buckling depending on the two values of slenderness ratios as given below:*
- Slenderness ratio about the major axis = l_{ex}/D
- Slenderness ratio about the minor axis = l_{ey}/b

- If, slenderness ratios l_{ex}/D *and* l_{ey}/b *are less than 12, it is known as short column.*

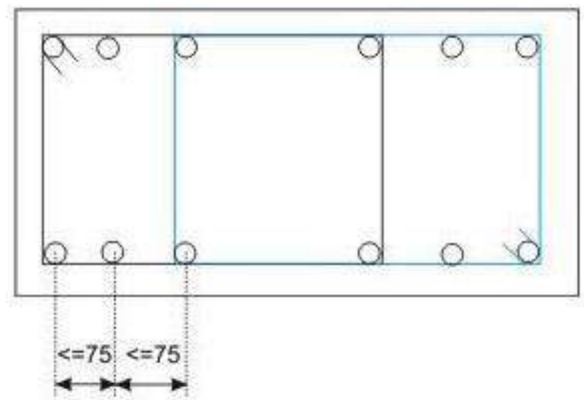
where l_{ex} *= effective length in respect of the major axis, D = depth in respect of the major axis,* l_{ey} *= effective length in respect of the minor axis, and b = width of the member. Else known as a slender compression member.*

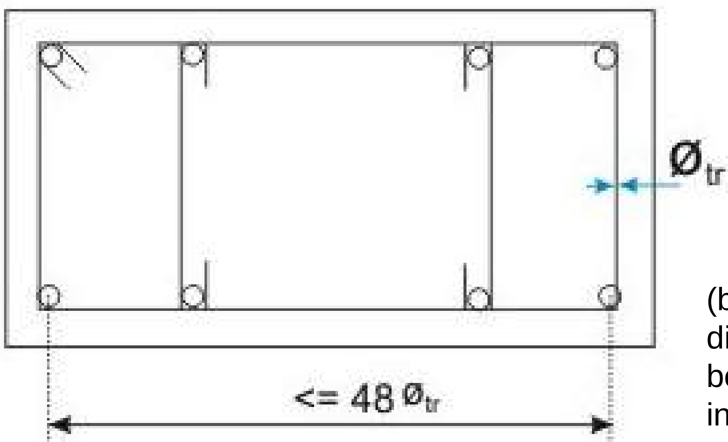
- Further, it is essential to avoid the mode 3 type of failure of columns so that all columns should have material failure (modes 1 and 2) only. Accordingly, cl. 25.3.1 of IS 456 stipulates the maximum unsupported length between two restraints of a column to sixty times its least lateral dimension. For cantilever columns, when one end of the column is unrestrained, the unsupported length is restricted to $100b^2/D$ *where b and D are as defined earlier.*
- **Longitudinal Reinforcement**
- Clause 26.5.3.1
- (a) The minimum amount of steel should be at least 0.8 per cent of the gross cross-sectional area of the column required if for any reason the provided area is more than the required area.
- (b) Maximum percentage =6%, but generally restricted to 4% of gross cross sectional area to avoid congestion and for vibrating facility.
- (c) minimum Four and six number of longitudinal bars in rectangular and circular columns to be provided , respectively.
- (d) The diameter of the longitudinal bars should be at least 12 mm.
- (e) Columns having helical reinforcement shall have at least six longitudinal bars within and in contact with the helical reinforcement. The bars shall be placed equidistant around its inner circumference.
- (f) The bars shall be spaced not exceeding 300 mm along the periphery of the column.
- (g) The amount of reinforcement for pedestal shall be at least 0.15 per cent of the cross-sectional area provided.
- (h) Nominal cover 40 mm or largest bar dia which ever is higher

TRANSVERSE REINFORCEMENT

- Transverse reinforcing bars are provided in forms of circular rings, polygonal links (lateral ties) with internal angles not exceeding 135^o or helical reinforcement. The transverse reinforcing bars are provided to ensure that every longitudinal bar nearest to the compression face has effective lateral support against buckling. Clause 26.5.3.2 stipulates the guidelines of the arrangement of transverse reinforcement. The salient points are:

(a) Transverse reinforcement shall only go round corner and alternate bars if the longitudinal bars are not spaced more than 75 mm on either side lateral tie-1.





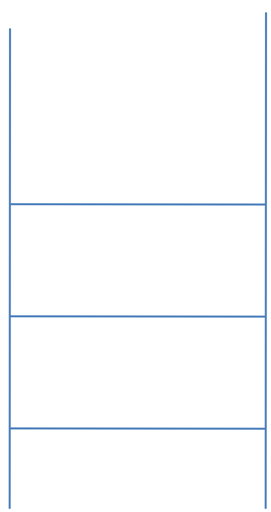
(b) Longitudinal bars spaced at a maximum distance of 48 times the diameter of the tie shall be tied by single tie and additional open ties for in between longitudinal bars as in Lateral tie-2

PITCH AND DIAMETER OF LATERAL TIES

- (a) Pitch: The maximum pitch of transverse reinforcement shall be the least of the following:
- (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.
- (b) 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.

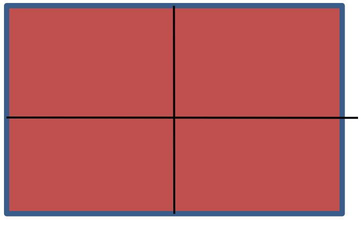
PITCH AND DIAMETER OF HELICAL TIES

Pitch-Helical reinforcement shall be of regular formation with the turns of the helix spaced evenly and its ends shall be anchored properly by providing one and a half extra turns of the spiral bar. Where an increased load on the column on the strength of the helical reinforcement is allowed for, the pitch of helical turns shall be
not more than 75 mm,
nor more than one-sixth of the core diameter of the column,
nor less than 25 mm,
nor less than three times the diameter of the steel bar forming the helix. In other cases, the requirements of 26.5.3.2 shall be complied with.
The diameter of the helical reinforcement shall be in accordance with IS-456-26.5.3.2 (c) (2)

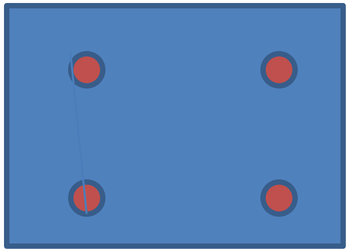


DESIGN OF COMPRESSION MEMBERS

- **Assumptions in the Design of Compression Members by Limit State of Collapse**
- The following are the assumptions in addition to given in 38.1 (a) to (e) for flexure for the design of compression members (cl. 39.1 of IS 456).
- (i) The maximum compressive strain in concrete in axial compression is taken as 0.002.
- (ii) The maximum compressive strain at the highly compressed extreme fiber in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fiber.
- **Minimum Eccentricity**
- It is difficult to design axial compression members due to construction and loading. So,, all axially loaded columns should be designed considering the minimum eccentricity as stipulated in cl. 25.4 of IS 456 and given below (Fig.3.2c)
- $e_x\ min \geq$ greater of ($l/500 + D/30$) or 20 mm
- $e_y\ min \geq$ greater of ($l/500 + b/30$) or 20 mm
- where *l, D* and *b* are the unsupported length, larger lateral dimension and least lateral dimension, respectively.



- **Governing Equation for Short Axially Loaded Tied Columns**
- Factored concentric load applied on short tied columns is resisted by concrete of area *Ac* and longitudinal steel of areas A_{sc} effectively held by lateral ties at intervals. Assuming the design strengths of concrete and steel are $0.4f_{ck}$ and $0.67f_y$, respectively, we can write



- $$P_u = 0.4 * f_{ck} * A_c + 0.67 * f_y * A_{sc}$$
- $A_g = A_c + A_{sc}$
- Where P_u = factored axial load on the member,
- f_{ck} = characteristic compressive strength of the concrete,
- A_c = area of concrete,
- f_y = characteristic strength of the compression reinforcement, and
- A_{sc} = area of longitudinal reinforcement for columns.
- The above equation, given in cl. 39.3 of IS 456, has two unknowns *Ac* and A_{sc} to be determined from one equation. The equation is recast in terms of A_g, the gross area of concrete and *p*, the percentage of compression reinforcement employing
- $A_{sc} = pA_g/100 \quad p=(A_{sc}/A_g)*100$
- $A_c = A_g - A_{sc} = (A_g - pA_g/100) = A_g(1 - p/100)$

Columns with helical reinforcement take more load than that of tied columns due to additional strength of spirals in contributing to the strength of columns. Accordingly, cl. 39.4 recommends a multiplying factor of 1.05 regarding the strength of such columns. The code further recommends that the ratio of volume of helical reinforcement to the volume of core shall not be less than 0.36 *(A_g/A_c – 1) (f_{ck}/f_y), in order to apply the additional strength factor of 1.05 (cl. 39.4.1). Accordingly, the governing equation of the spiral columns may be written*

$$P_u = 1.05 * [0.4 * f_{ck} * A_c + 0.67 * f_y * A_{sc}]$$

Dia. of core= outside to outside of helix

$$Volume\ of\ one\ helical\ loop\ per\ pitch = \pi(D_c - \varphi_{sp})a_{sp}$$

$$Volume\ of\ core\ \ per\ pitch = (\pi/4)(D_c^2)p$$

where D_c *= diameter of the core ()*

φ_{sp} *= diameter of the spiral reinforcement*

a_{sp} *= area of cross-section of spiral reinforcement*

p = pitch of spiral reinforcement

$$\frac{Volume\ of\ helical\ reinforcement}{Volume\ of\ core} \geq 0.36 \left[\frac{A_g}{A_c} - 1\right] \frac{f_{ck}}{f_y}$$

$$\{\pi(D_c - \varphi_{sp})a_{sp}\}/\{\left(\frac{\pi}{4}\right)(D_c^2)p\} \geq 0.36 \left[\frac{A_g}{A_c} - 1\right] \frac{f_{ck}}{f_y}$$

$$p \leq 11.1\{(D_c - \varphi_{sp})a_{sp} * f_y\}/\{(D^2 - D_c^2)f_{ck}\}$$

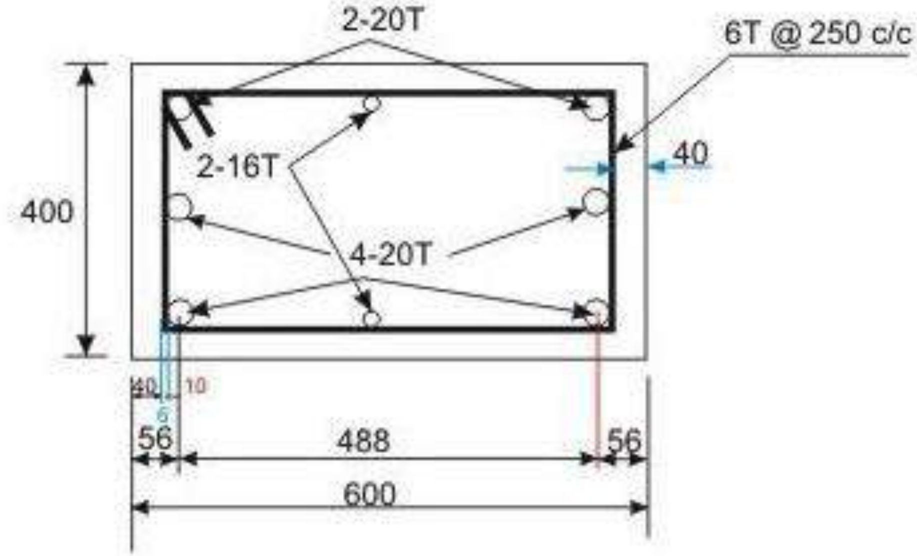
Design the reinforcement in a column of size 400 mm x 600 mm subjected to an axial load of 2000 kN under service dead load and live load. The column has an unsupported length of 3.5m and effectively held in position and restrained against rotation in both ends. Use M 25 concrete and Fe 415 steel.

- **Step 1: To check if the column is short or slender**
- Given l = 3500 mm, b = 400 mm and D = 600 mm. Table 28 of IS 456 = $l_{ex} = l_{ey} = 0.65(l)$ = 2275 mm. So, we have
- $l_{ex}/D = 2275/600 = 3.79 < 12$
- $l_{ey}/b = 2275/400 = 5.68 < 12$
- Hence, it is a short column.
- **Step 2: Minimum eccentricity**
- $e_x\ min$ = Greater of ($l_{un}/500 + D/30$) and 20 mm = 27 mm
- $e_y\ min$ = Greater of ($l_u/500 + b/30$) and 20 mm = 20 mm
- 0.05 D = 0.05(600) = 30 mm > 27mm (= *ex min*)
- 0.05 b = 0.05(400) = 20 mm = 20 mm (= *ey min*)
- Hence, the equation given in cl.39.3 of IS 456 (Eq.(1)) is applicable for the design here.
- **Step 3: Area of steel**
- $P_u = 0.4 * f_{ck} * A_c + 0.67 * f_y * A_{sc}$

$$3000 * 10^3 = 0.4 * 25 * (400 * 600 - A_{sc}) + 0.67 * 415 * A_{sc}$$
$$A_{sc} = 2238.39\ mm^2$$

- Provide 6-20 mm diameter and 2-16 mm diameter rods giving 2287 mm^2 (> 2238.39 mm2) and *p* = 0.953 per cent, which is more than minimum percentage of 0.8 and less than maximum percentage of 4.0.

- **Step 4: Lateral ties**
- The diameter of transverse reinforcement (lateral ties) is determined from cl.26.5.3.2 C-2 of IS 456 as not less than
- (i) φ/4 and (ii) 6 mm. Here, φ = largest bar diameter used as longitudinal reinforcement = 20 mm. So, the diameter of bars used as lateral ties = 6 mm.
- The pitch of lateral ties, as per cl.26.5.3.2 C-1 of IS 456, should be not more than the least of
- (i) the least lateral dimension of the column = 400 mm
- (ii) sixteen times the smallest diameter of longitudinal reinforcement bar to be tied = 16(16) = 256 mm
- (iii) 300 mm



Reinforcement Detailing

Let us use p = pitch of lateral ties = 250 mm

- Design the a column mm subjected to an axial load of factored load 2000 kN. The column has an unsupported length of 3 m and effectively held in position and restrained against rotation in both ends. Use M 20 concrete and Fe 415 steel.
- Solution: Assuming a short column

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

- Assuming 1% steel
- $P=1/100=.01=A_{sc}/A_g$
- $A_{sc}=0.01A_g$
- $A_c=A_g-A_{sc}=A_g-0.01A_g$
- $A_c=0.99A_g$
- $2000*1000=0.4*20*0.99A_g+0.67*415*0.01A_g$
- A_g=186907.2 If it is a square section, D= 432.32 mm
- b-=D=450 mm

- Checking short or long column:

L_{eff}=.65*3=1.95 m

l_{eff}/(b=D)=1.95/.45=4.33<12 short column

$e_{min}=l_{un}/500+D/30=(3000/500)+450/30=21$

e_{min} can not be less than 20 mm.

$e_{min}=21$

0.05*450= 22.5 mm

- So, it is short column and axial loading.

Area provided=Ag=450*450=202500 mm2

Asc=0.01*Ag=1869.17 mm^2

Take 4,20 and 4,16 mm dia bars

Area=2060mm^2,

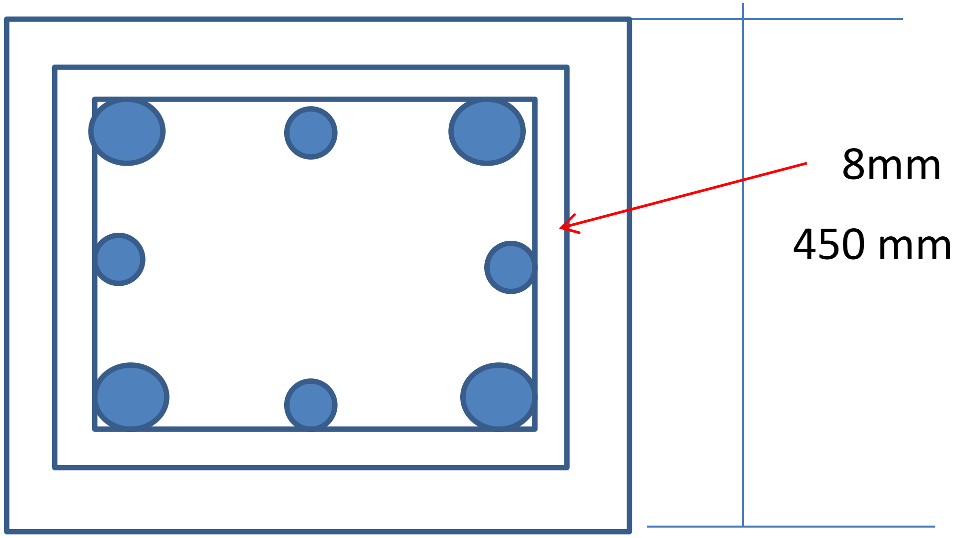
Transverse steel=1/4 of 20 mm=5 mm can not be less than 6 mm

Spacing: can not be more than 450 mm

16*16=256 mm

300 mm

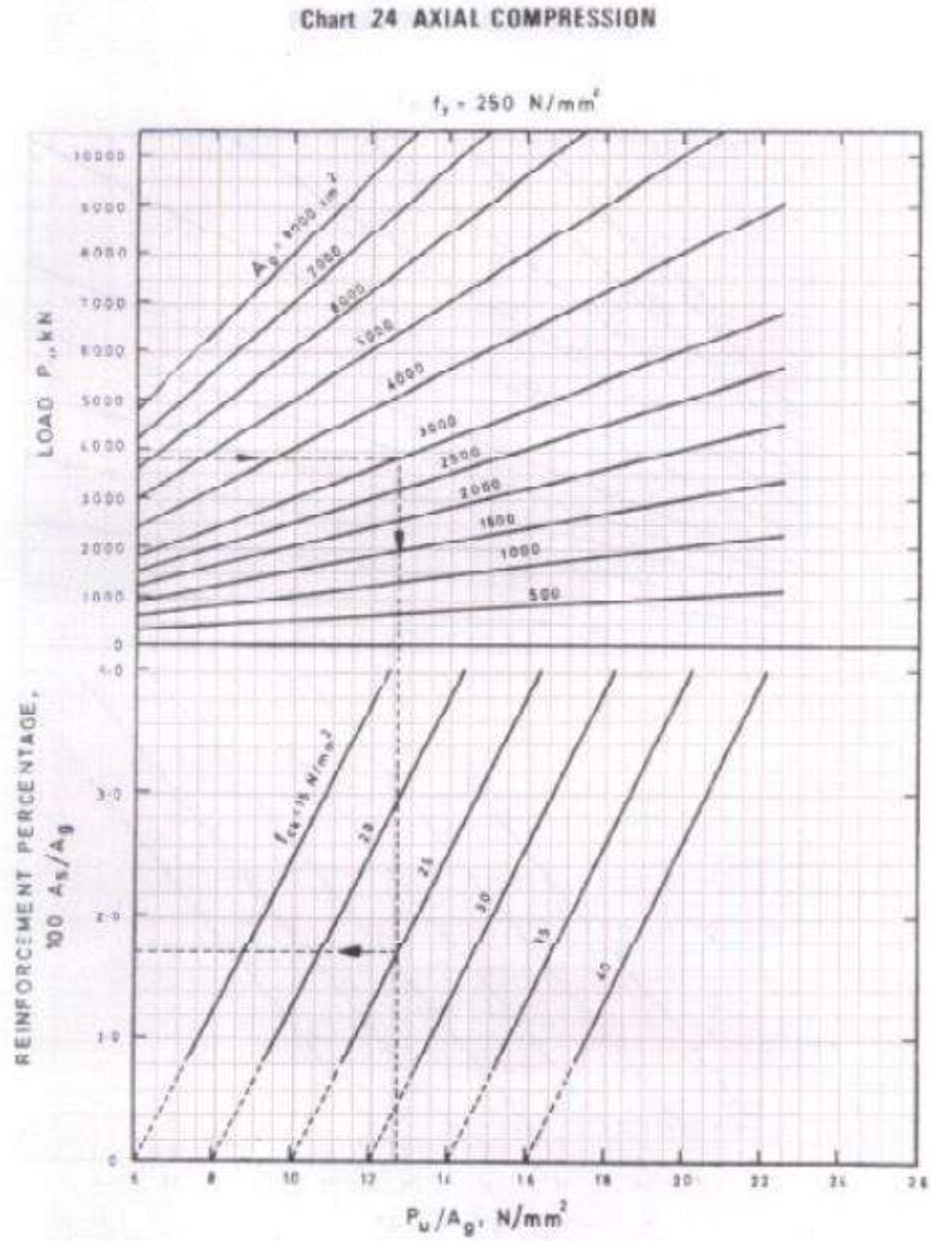
8 mm dia. 250 mm c/c.



- For circular section=487.37 mm
- D=500 mm
- For a rectangular section: Ag= 186907.2 =b*D
- b=2/3 of D
- (2/3)D*D=186907.2

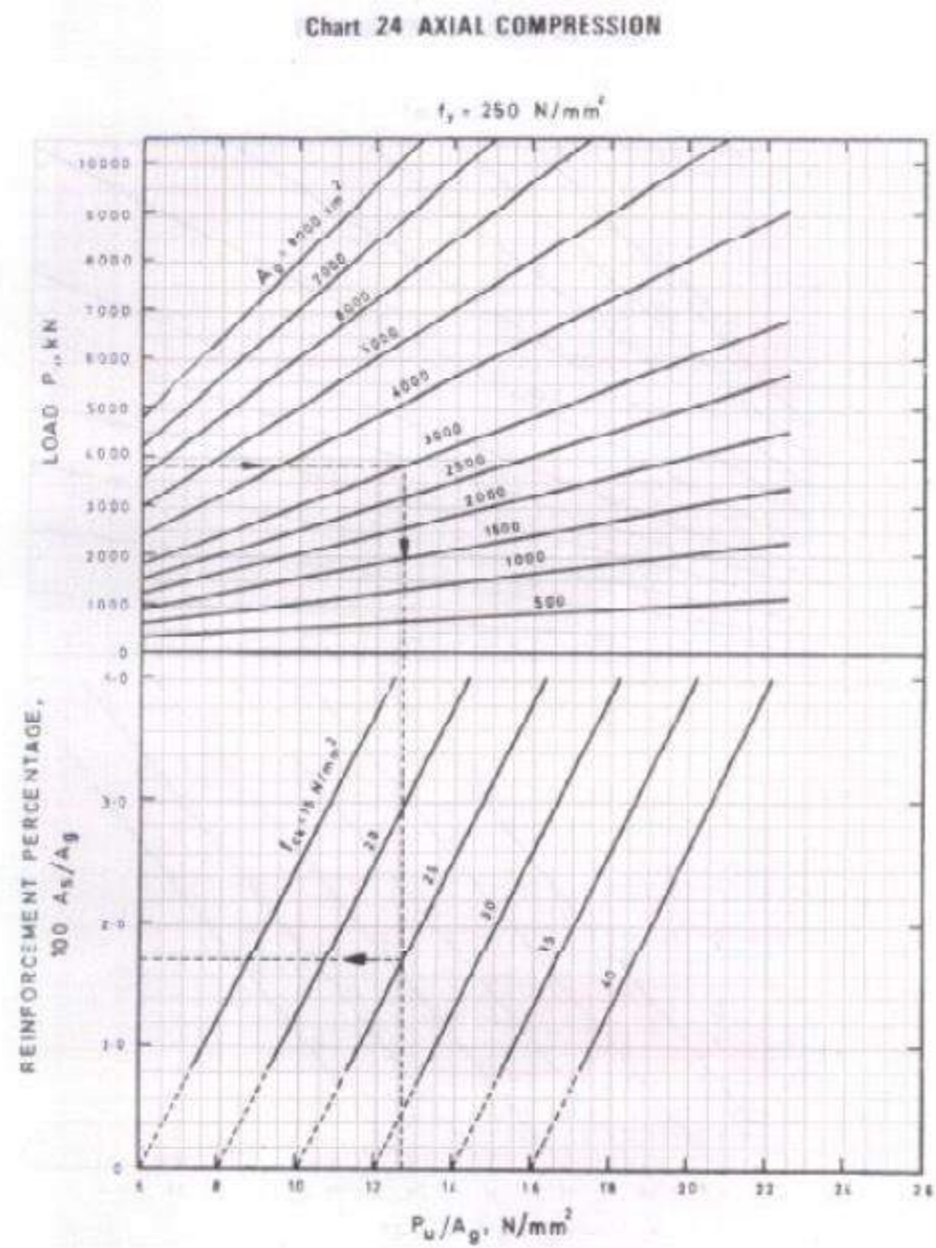
USE OF SP-16

Charts 24 to 26 can be used for designing short columns in accordance with the above equations. In the lower section of these charts, P_u/A_g, has been plotted against reinforcement percentage p for different grades of concrete. If the cross section of the column is known, P_U/Ag can be calculated and the reinforcement percentage read from the chart.



f_{ck}
15
20
25
30
35
40

In the upper section of the charts, Pu/Ag is plotted against P_U for various values of Ag. The combined use of the upper and lower sections would eliminate the need for any calculation. This is particularly useful as an aid for deciding the sizes of the columns at preliminary design stage of multi-storeyed buildings.



f_{ck}
15
20
25
30
35
40

Design a circular column of 400 mm diameter with helical reinforcement subjected to an axial load of 1500 kN under service load and live load. The column has an unsupported length of 3 m effectively held in position at both ends but not restrained against rotation. Use M 25 concrete and Fe 415 steel.

Step 1: To check the slenderness ratio

Given data are: unsupported length l = 3000 mm, D = 400 mm. Table 28 of Annex E of IS 456 gives effective length $l_e = l$ = 3000 mm. Therefore, $l_e/D = 7.5 < 12$ confirms that it is a short column.

Step 2: Minimum eccentricity

$$e_{min} = \text{Greater of } (l/500 + D/30) \text{ or } 20 \text{ mm} = 20 \text{ mm}$$

$$0.05\,D = 0.05(400) = 20 \text{ mm}$$

As per cl.39.3 of IS 456, e_{min} should not exceed $0.05D$ to employ the equation given in that clause for the design. Here, both the eccentricities are the same. So, we can use the equation given in that clause of IS 456 i.e., Eq.10.8 for the design.

$$\frac{Volume\ of\ helical\ reinforcement}{Volume\ of\ core} \geq 0.36\left[\frac{A_g}{A_c} - 1\right]\frac{f_{ck}}{f_y}$$

$$\{\pi(D_c - \varphi_{sp})a_{sp}\}/\{\left(\frac{\pi}{4}\right)(D_c^2)p\} \geq 0.36\left[\frac{A_g}{A_c} - 1\right]\frac{f_{ck}}{f_y}$$

$$p \leq 11.1\{(D_c - \varphi_{sp})a_{sp} * f_y\}/\{(D^2 - D_c^2)f_{ck}\}$$

- Calcula(tion of Steel:

$1.5(1500)(10^3) = 1.05\{0.4(25)(125714.29 – A_{sc}) + 0.67(415)\ A_{sc}\}$

we get the value of $A_{sc} = 3304.29\ mm^2$. *Provide 11 nos. of 20 mm diameter bars (= 3455* mm^2*) as longitudinal reinforcement giving p = 2.75%. This p is between 0.8 (minimum) and 4 (maximum) per cents. Hence o.k.*

Step 4: Lateral ties

Diameter of helical reinforcement (cl.26.5.3.2 d-2) shall be not less than greater of (i) one-fourth of the diameter of largest longitudinal bar, and (ii) 6 mm.

Therefore, with 20 mm diameter bars as longitudinal reinforcement, the diameter of helical reinforcement = 6 mm.

$$p \leq 11.1\{(D_c - \varphi_{sp})a_{sp} * f_y\}/\{(D^2 - D_c^2)f_{ck}\}$$

where $D_c = 400 – (40\text{-}6) – (40\text{-}6) = 332$ mm, $\varphi_{sp} = 6$ mm, $a_{sp} = 28\ mm^2$, $f_y = 415\ N/mm^2$, $D = 400$ mm and $f_{ck} = 25\ N/mm^2$.

$$p \leq 11.1(332 – 6)(28)(415)/(400^2 – 332^2)(25) \leq 33.78\ mm$$

- As per cl.26.5.3.2 d-1, the maximum pitch is the lesser of 75 mm and 332/6 = 55.33 mm and the minimum pitch is lesser of 25 mm and 3(6) = 18 mm. We adopt pitch = 25 mm which is within the range of 18 mm and 53.34 mm. So, provide 6 mm bars @ 25 mm pitch forming the helix.
- **Checking of cl. 39.4.1 of IS 456**

Substituting the values of D_c *=332 mm* φ_{sp}*=6 mm asp=28 mm*^{2} *and pitch p=25 mm*

A_g=125714.29 mm^2

A_c=A_g- Ac= 125714.29 -*3455 mm*^{2} 5714.29 mm^2

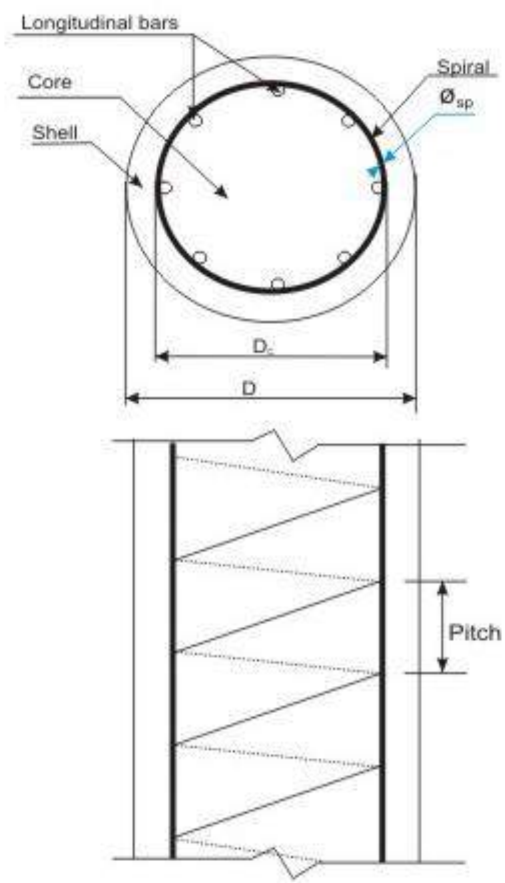
D_c*= 332 mm* a_{sp}*=28 mm*^{2}*,*

$$Volume\ of\ one\ helical\ loop\ per\ pitch = \pi(D_c - \varphi_{sp})a_{sp}$$

- Volume of helical reinforcement in one loop = 28688 mm^3 and

$$Volume\ of\ core\ \ per\ pitch = (\pi/4)(D_c^2)p$$

- Volume of core in one loop = 2165114.29 mm^3.
- Their ratio = 28688 / 2165114.29 = 0.0137375
- 0.36(A_g/A_c – 1) (f_{ck}/f_y) = 0.009794
- It is, thus, seen that the above ratio (0.0137375) is not less than 0.36(A_g/A_c – 1) (f_{ck}/f_y)=0.009794.



Longitudinal bars
Core
Shell
Spiral
$\O_{sp}$
D_c
D
Pitch

Volume of helical reinforcement in one loop = $\pi(D_c - \phi_{sp})\, a_{sp}$

(10.9)

Volume of core = $(\pi / 4)\, D_c^2\; p$

(10.10)

where D_c = diameter of the core (Fig.10.21.2b)

ϕ_{sp} = diameter of the spiral reinforcement (Fig.10.21.2b)

a_{sp} = area of cross-section of spiral reinforcement

p = pitch of spiral reinforcement (Fig.10.21.2b)

$$\{\pi(D_c - \phi_{sp})\, a_{sp}\} / \{(\pi / 4)\, D_c^2\; p\} \geq 0.36(A_g / A_c - 1)\,(f_{ck} / f_y)$$

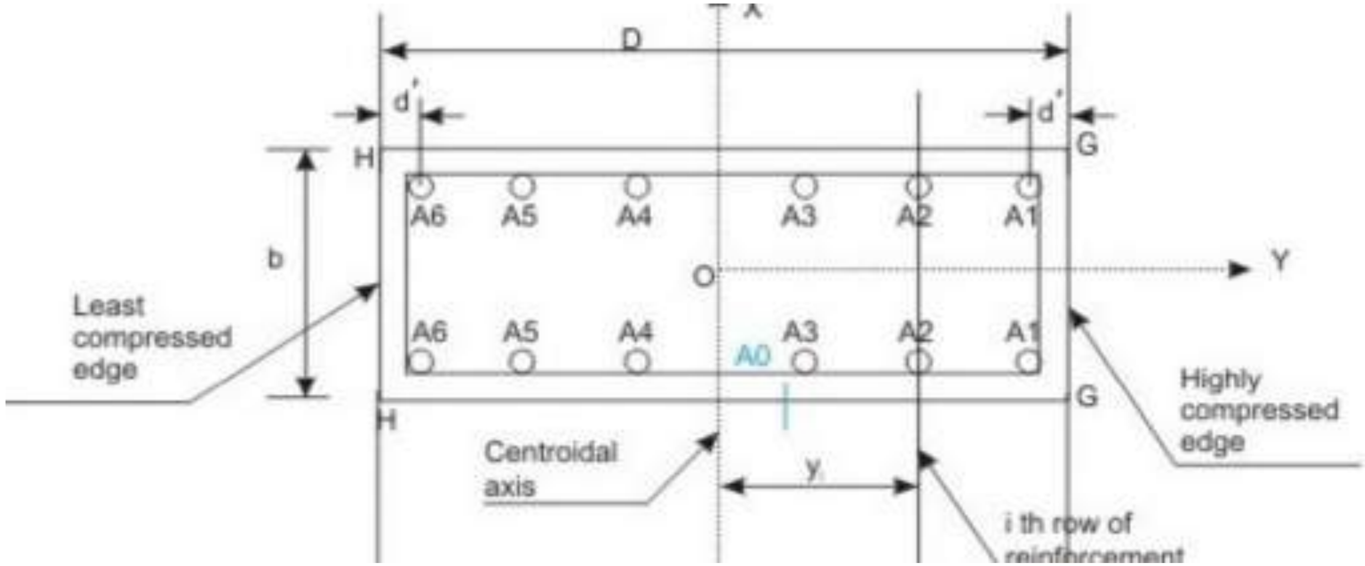
which finally gives

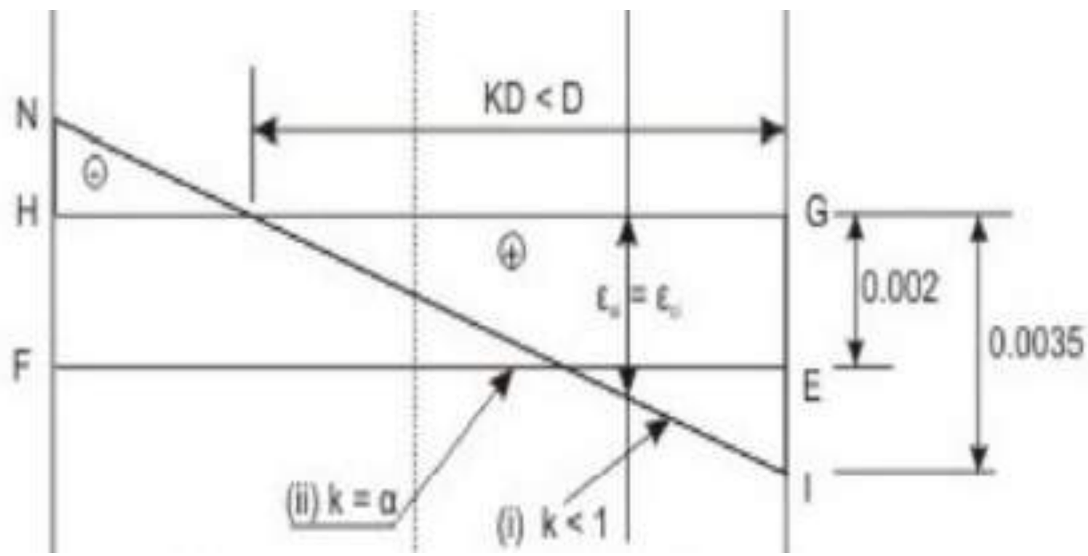
$$p \leq 11.1(D_c - \phi_{sp})\, a_{sp}\; f_y / (D^2 - D_c^2)\, f_{ck}$$

(10.11)

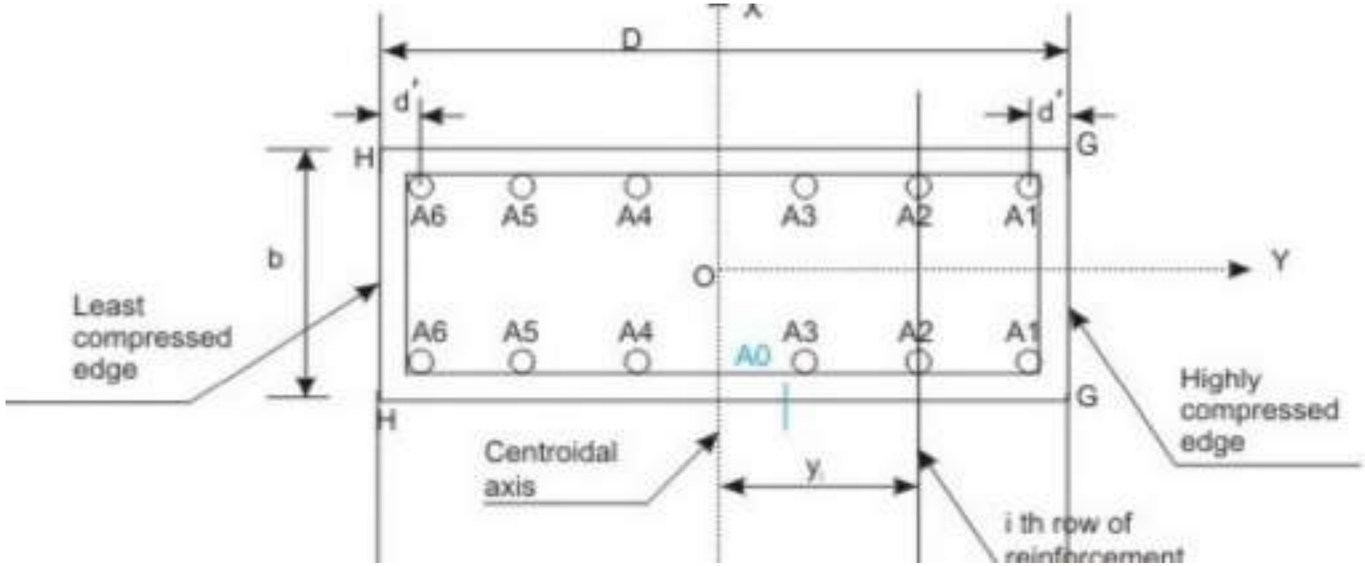
SHORT COLUMN WITH UNIAXIAL BENDING

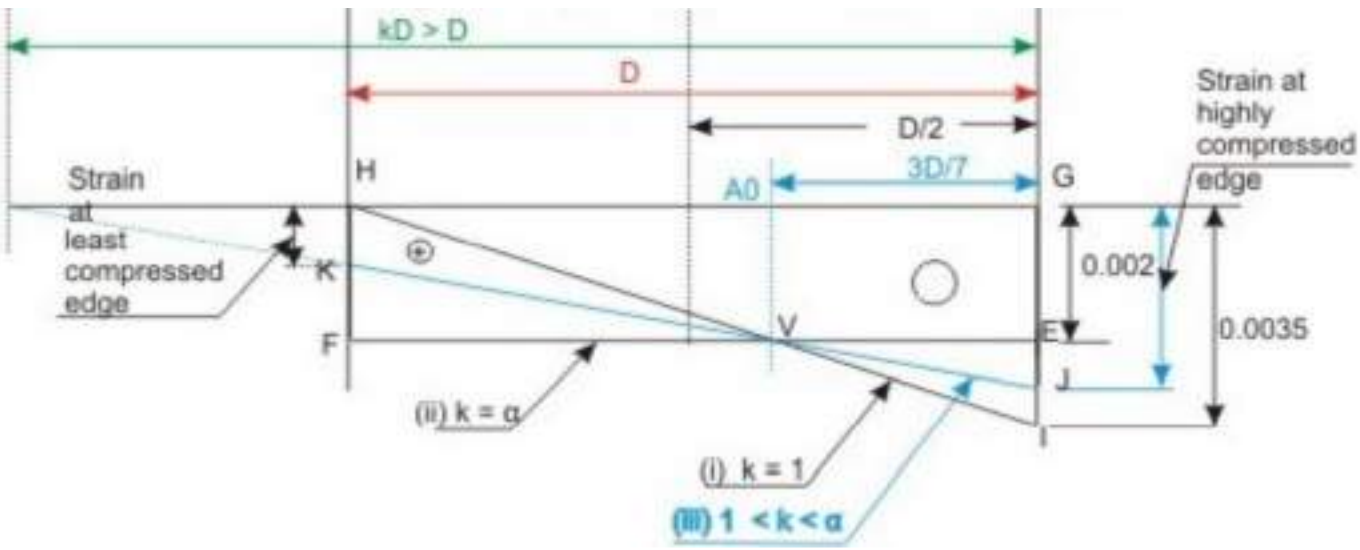
- Normally, the side columns of a grid of beams and columns are subjected to axial load P and uniaxial moment Mx causing bending about the major axis xx, hereafter will be written as M. The moment M can be made equivalent to the axial load P acting at an eccentricity of e (= M/P). Let us consider a symmetrically reinforced short rectangular column subjected to axial load Pu at an eccentricity of e to have Mu causing failure of the column.





Strain profiles when (i) k<1 and (ii) k = α





Strain profiles when (i) k = 1, (ii) k = α and (iii) 1 < k < α

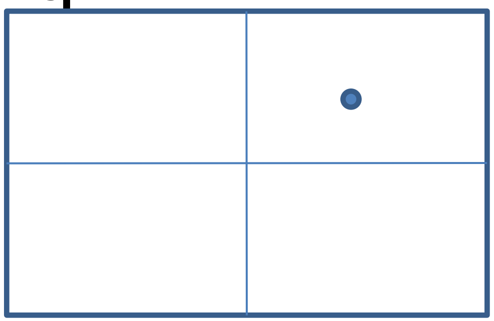
DESIGN OF FOOTING

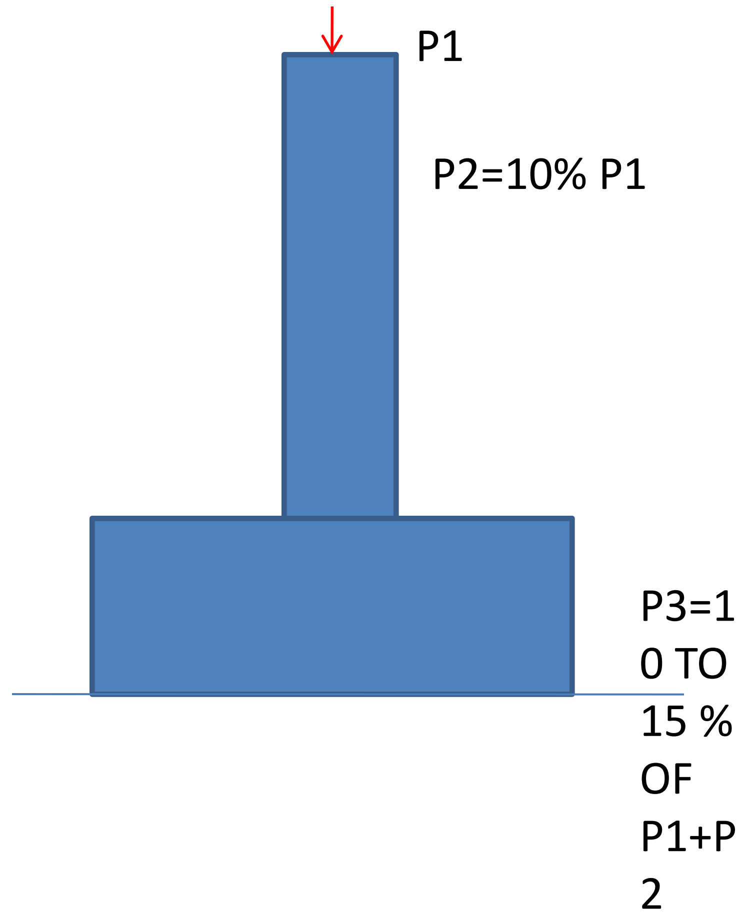
G.C. BEHERA

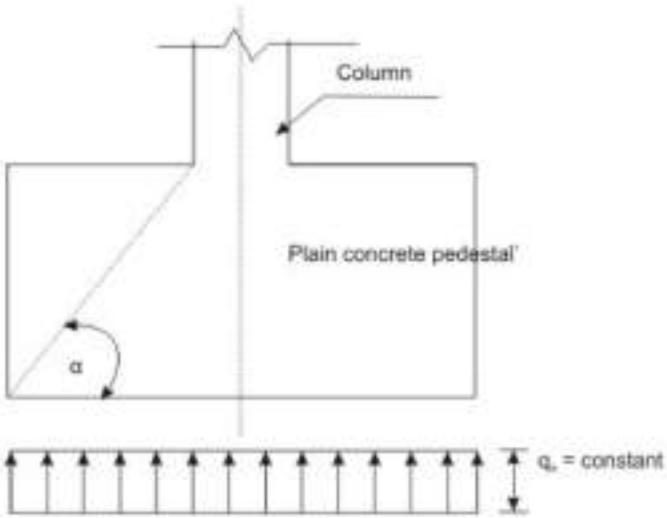
1. Foundation structures should be able to sustain the applied loads, moments, forces and induced reactions without exceeding the safe bearing capacity of the soil.
2. 2. The settlement of the structure should be as uniform as possible and it should be within the tolerable limits. It is well known from the structural analysis that differential settlement of supports causes additional moments in statically indeterminate structures. Therefore, avoiding the differential settlement is considered as more important than maintaining uniform overall settlement of the structure.

- LOAD IS P
- SBC=q

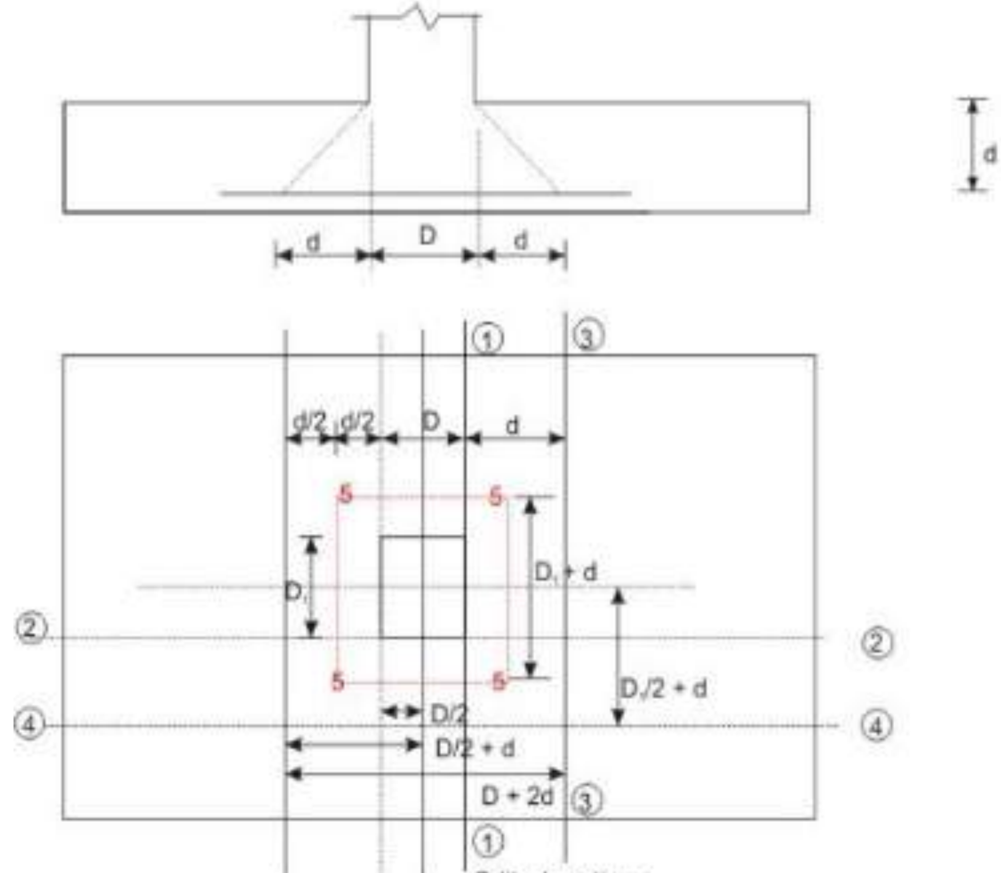
P=P1+P2+P3







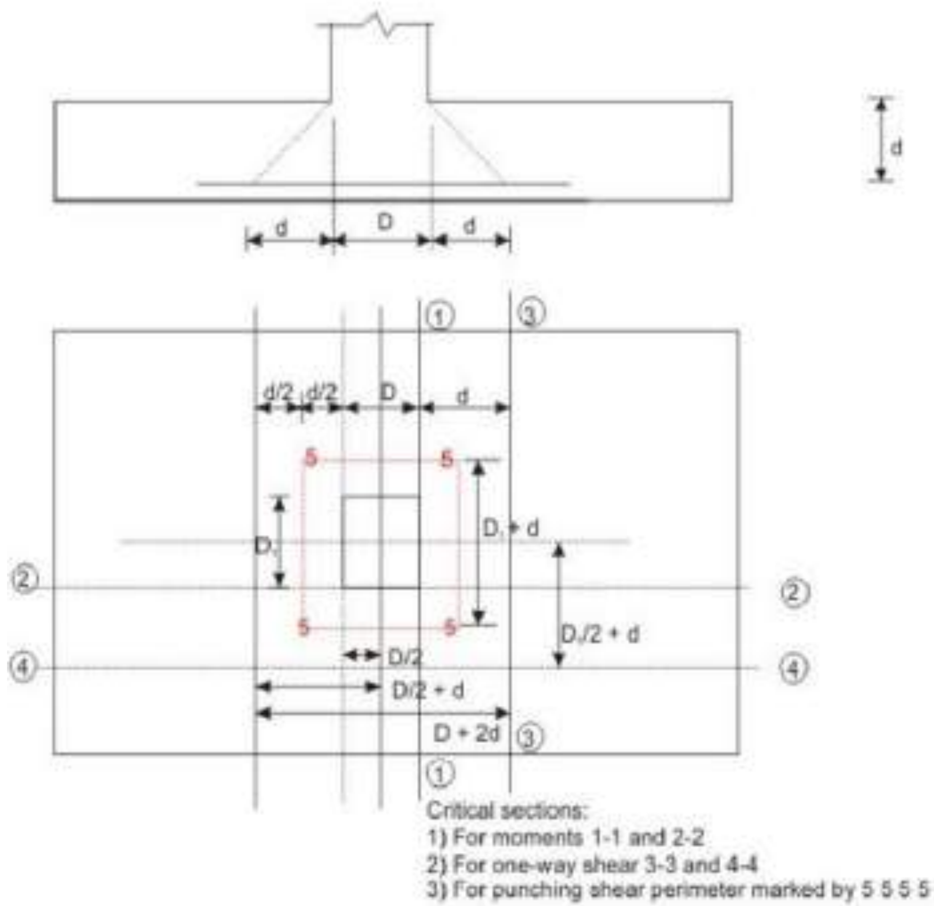
PLAIN CONC.
FOUNDATION



Critical sections:
1) For moments 1-1 and 2-2
2) For one-way shear 3-3 and 4-4
3) For punching shear perimeter marked by 5 5 5 5

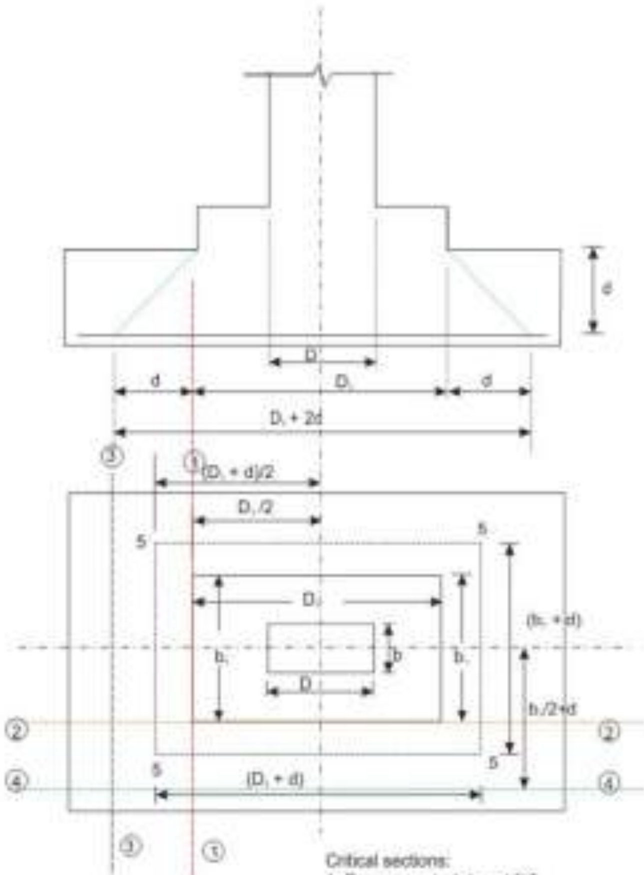
ISOLATED RC

ISOLATED FOOTING

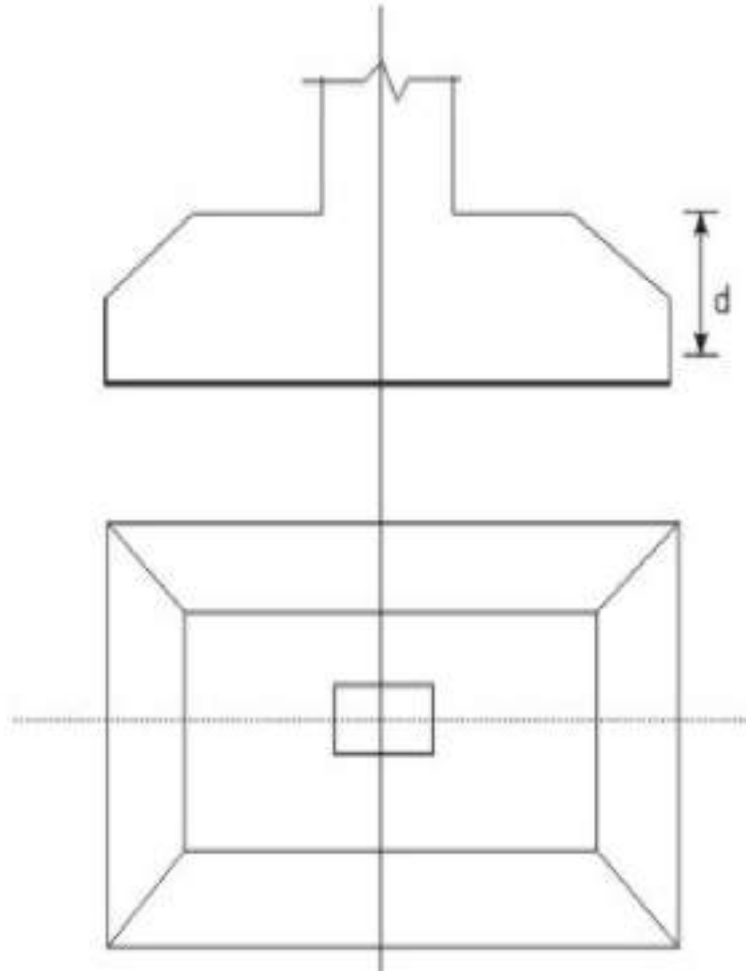


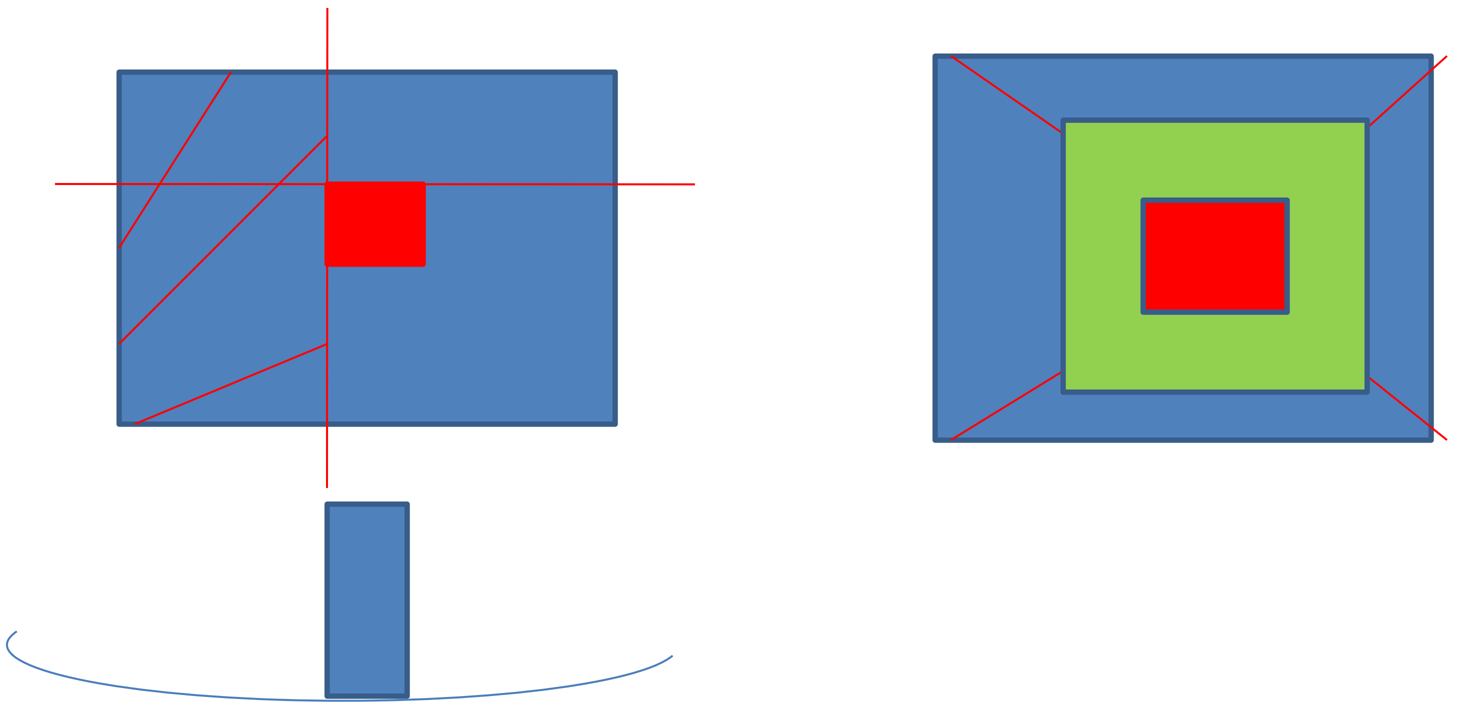
Uniform and rectangular footing

STEPPED AND SLOPED



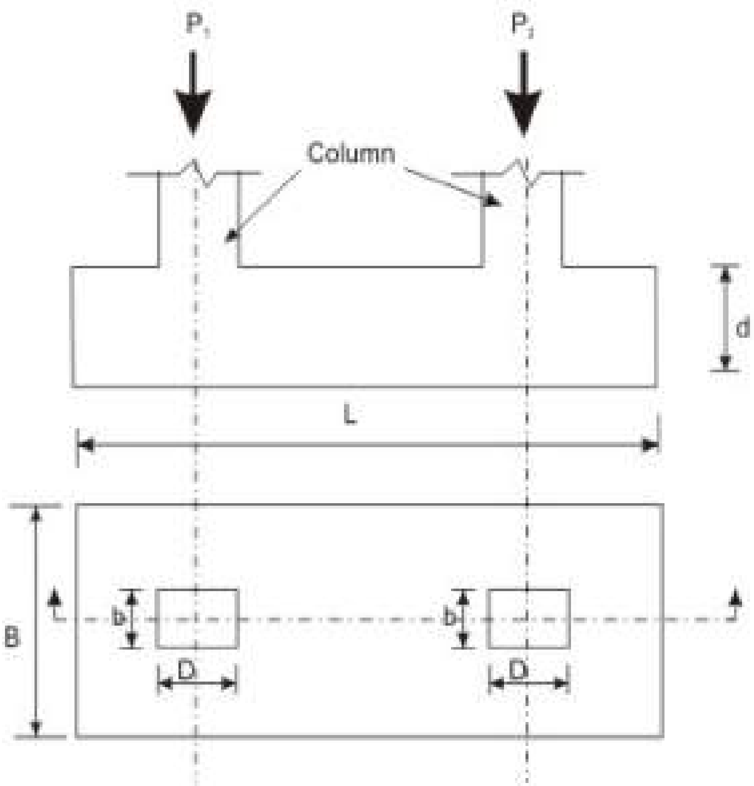
Critical sections:
1. For moments 1-1 and 2-2
2. One way shear 3-3 and 4-4
3. Two-way punching shear marked by 5555



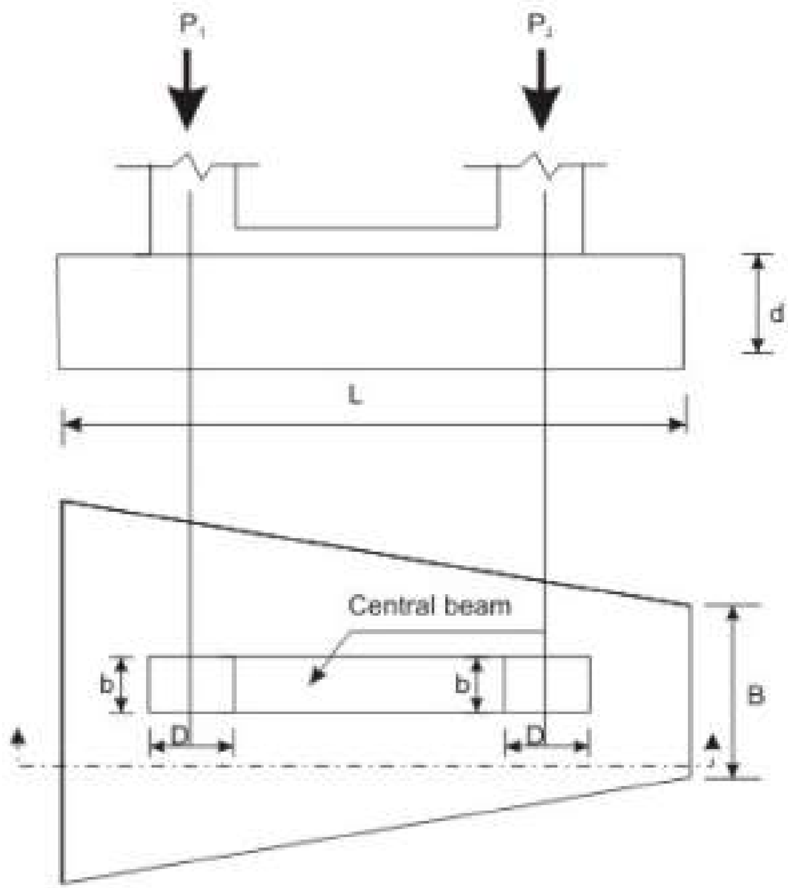


COMBINED FOOTING

- FIG.

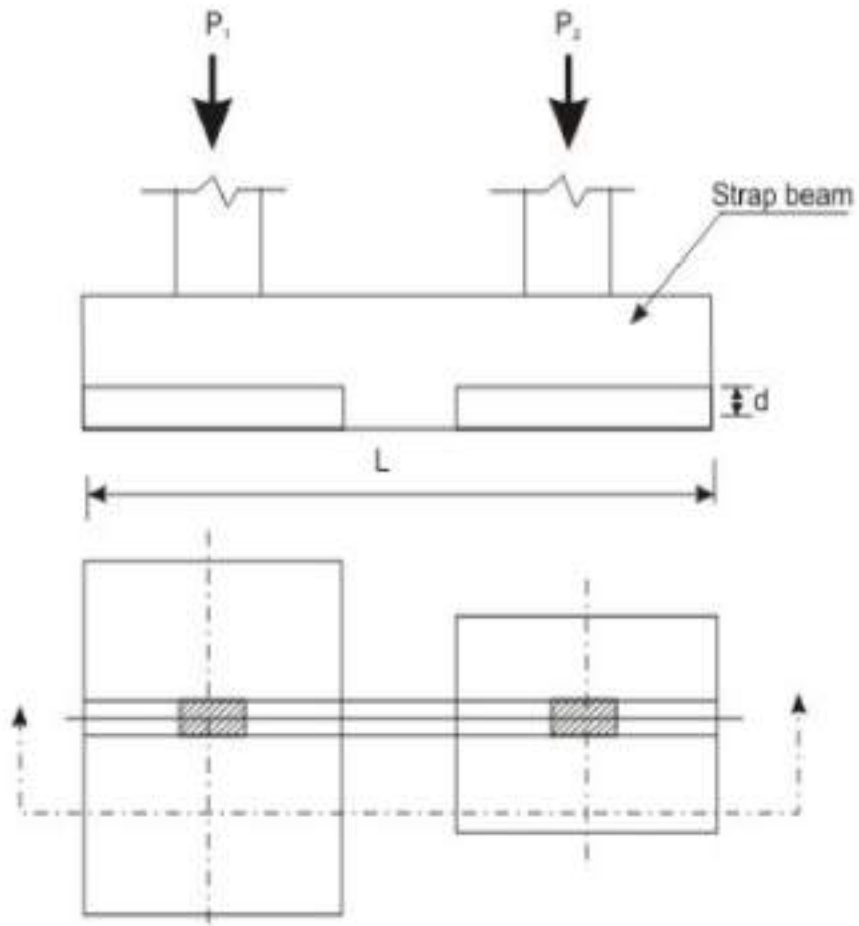


Combined footing without central beam



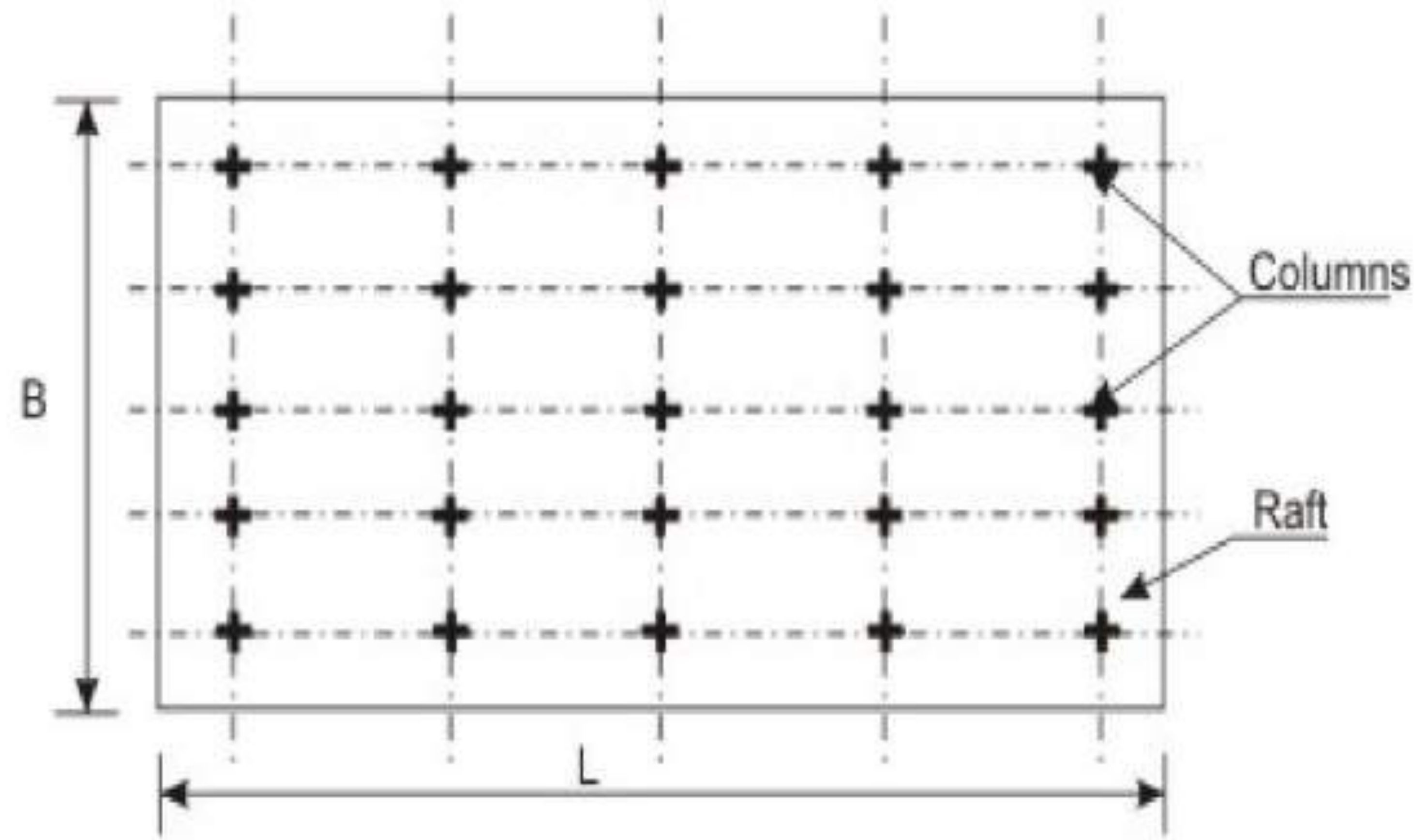
Combined footing with central beam

STRAP FOOTING

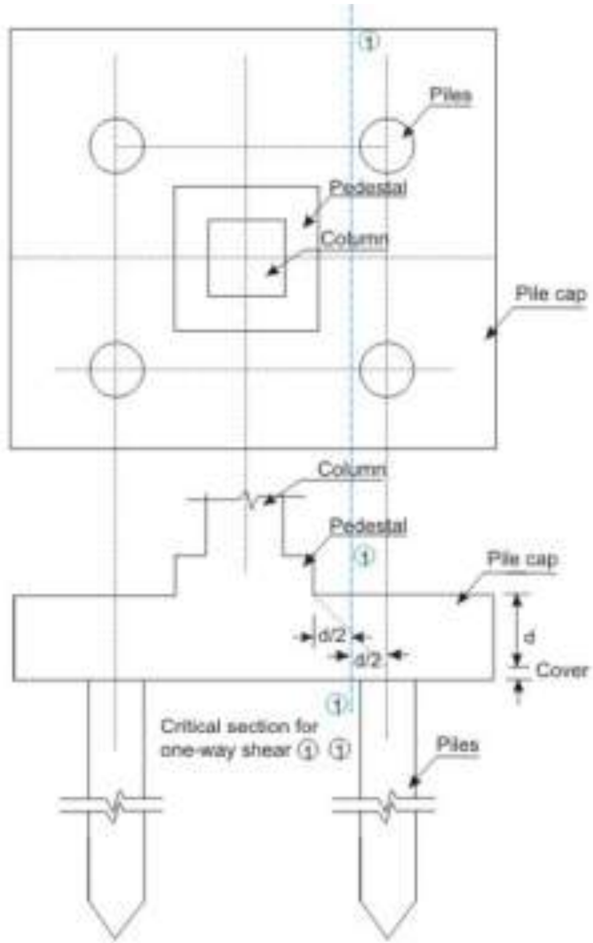


When two isolated footings are combined by a beam with a view to sharing the loads of both the columns by the footings, the footing is known as strap footing. The connecting beam is designated as strap beam. These footings are required if the loads are heavy on columns and the areas of foundation are not overlapping with each other.

RAFT FOUNDATION



DEEP FOUNDATION



PILE FOUNDATION

DESIGN REQUIREMENTS

- **(a) Minimum nominal cover (cl. 26.4.2.2 of IS 456)**
- The minimum nominal cover for the footings should be more than that of other structural elements of the superstructure as the footings are in direct contact with the soil. Clause 26.4.2.2 of IS 456 prescribes a minimum cover of 50 mm for footings. However, the actual cover may be even more depending on the presence of harmful chemicals or minerals, water table etc.
- **(b) Thickness at the edge of footings (cls. 34.1.2 and 34.1.3 of IS 456)**
- The minimum thickness at the edge of reinforced and plain concrete footings shall be at least 150 mm for footings on soils and at least 300 mm above the top of piles for footings on piles, as per the stipulation in cl.34.1.2 of IS 456.
- For plain concrete pedestals, the angle α between the plane passing through the bottom edge of the pedestal and the corresponding junction edge of the column with pedestal and the horizontal plane shall be determined from the following expression (cl.34.1.3 of IS 456

Depth of Foundation:

$$Depth\ of\ Foundation = d = \frac{q_0}{\gamma}\left(\frac{1-\sin\varphi}{1+\sin\varphi}\right)^2$$

- where d = *minimum depth of foundation*
- q_0= *bearing capacity of soil*
- γ= density of soil
- ϕ = angle of repose of soil
- This is from Rankine's formulae

(C) Bending moments (cl. 34.2 of IS 456)

- 1. It may be necessary to compute the bending moment at several sections of the footing depending on the type of footing, nature of loads and the distribution of pressure at the base of the footing. However, bending moment at any section shall be determined taking all forces acting over the entire area on one side of the section of the footing, which is obtained by passing a vertical plane at that section extending across the footing (cl.34.2.3.1 of IS 456).
- 2. The critical section of maximum bending moment for the purpose of designing an isolated concrete footing which supports a column, pedestal or wall shall be:
- (i) at the face of the column, pedestal or wall for footing supporting a concrete column, pedestal or reinforced concrete wall, Fig.1
- (ii) halfway between the centre-line and the edge of the wall, for footing under masonry wall . This is stipulated in cl.34.2.3.2 of IS 456. Fig.2
- For round or octagonal concrete column or pedestal, the face of the column or pedestal shall be taken as the side of a square inscribed within the perimeter of the round or octagonal column or pedestal . Fig.3

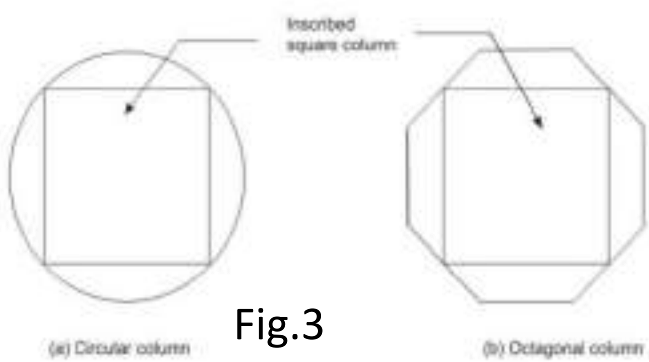


Fig.3

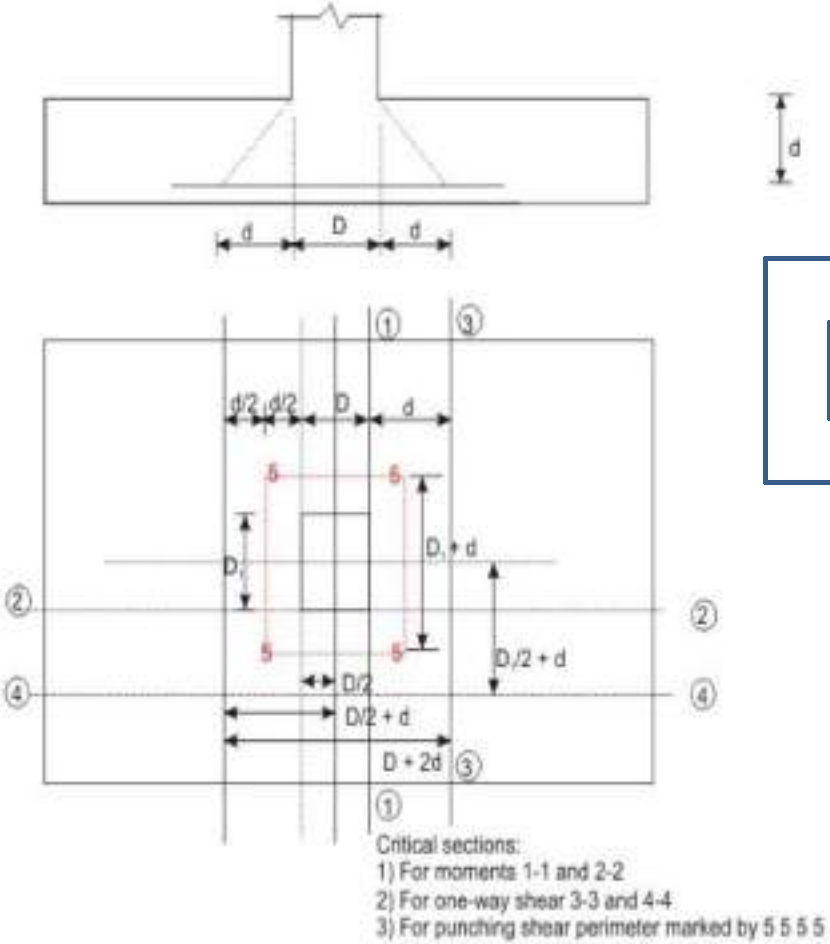
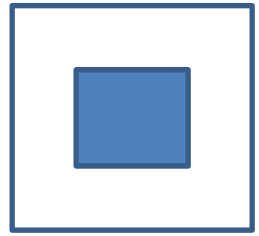

d
D
d
d/2 d/2
D
d
5
D₁ + d
D/2 + d
D/2
D/2 + d
D + 2d
Critical sections:
1) For moments 1-1 and 2-2
2) For one-way shear 3-3 and 4-4
3) For punching shear perimeter marked by 5 5 5 5

Fig-1



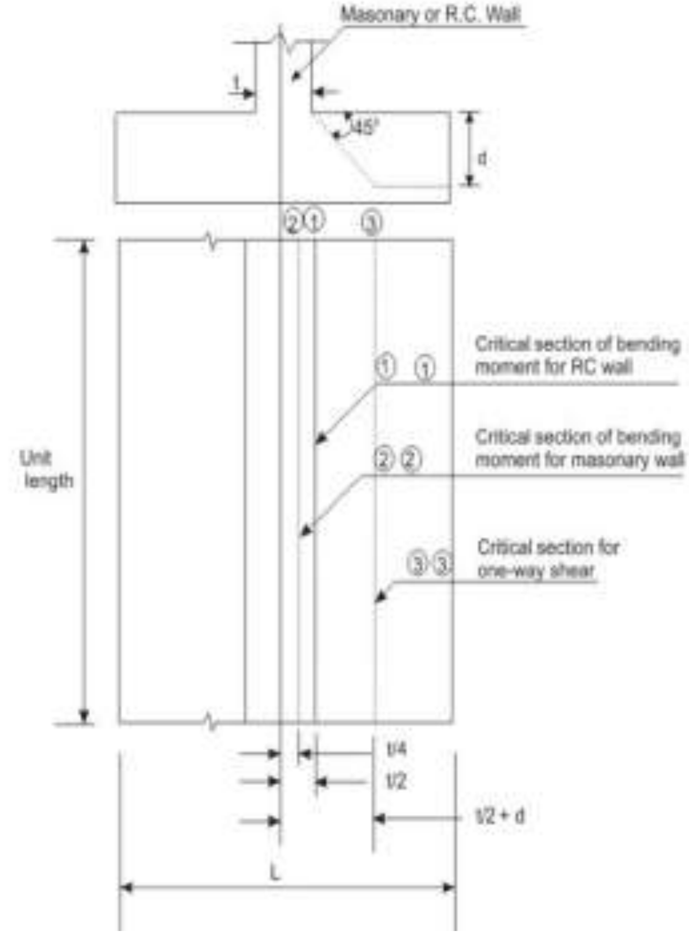

Masonary or R.C. Wall
t
45°
d
Unit length
Critical section of bending moment for RC wall
Critical section of bending moment for masonary wall
Critical section for one-way shear
t/4
t/2
t/2 + d
L

Fig-2

- **(d) Shear force (cl. 31.6 and 34.2.4 of IS 456)**
- Footing slabs shall be checked in one-way or two-way shears depending on the nature of bending. If the slab bends primarily in one-way, the footing slab shall be checked in one-way vertical shear. On the other hand, when the bending is primarily two-way, the footing slab shall be checked in two-way shear or punching shear. The respective critical sections and design shear strengths are given below:
- **1. One-way shear (cl. 34.2.4 of IS 456)**

One-way shear has to be checked across the full width of the base slab on a vertical section located from the face of the column, pedestal or wall at a distance equal to

(i) effective depth of the footing slab in case of footing slab on soil, and

(ii) half the effective depth of the footing slab if the footing slab is on piles.

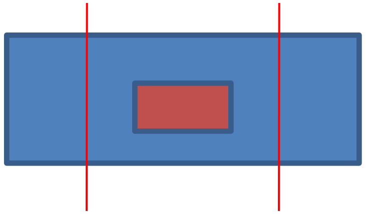
The design shear strength of concrete without shear reinforcement is given in Table 19 of cl.40.2 of IS 456.

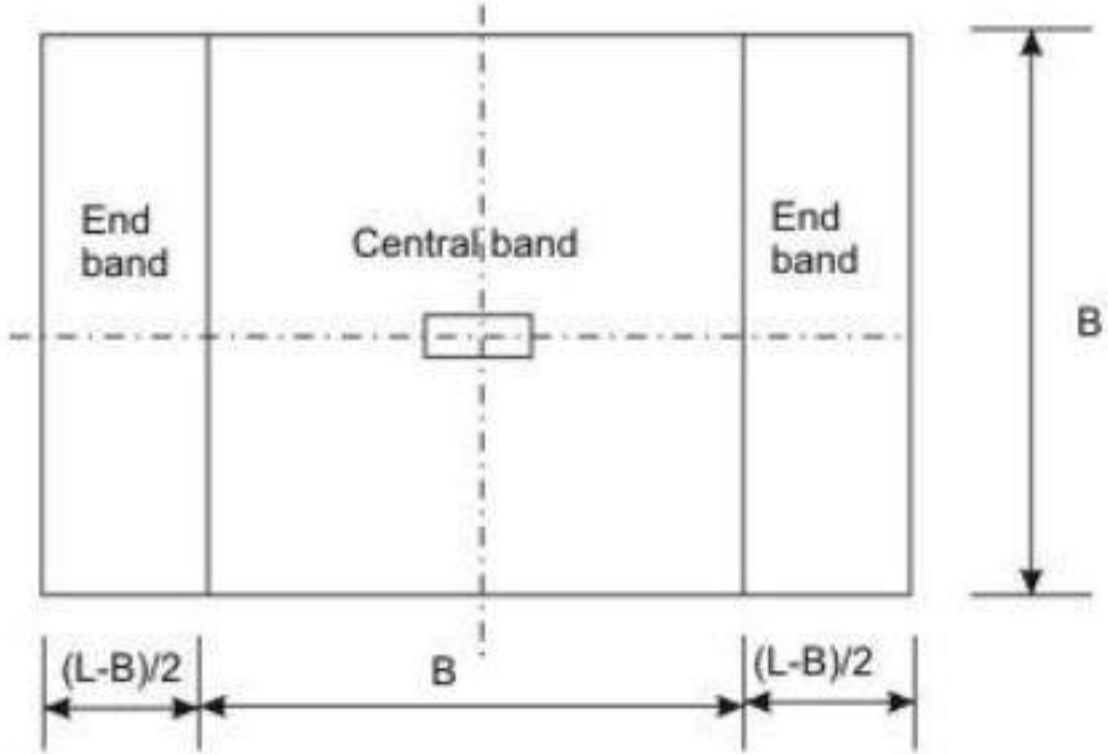
- **2. Two-way or punching shear (cls.31.6 and 34.2.4)**

Two-way or punching shear shall be checked around the column on a perimeter half the effective depth of the footing slab away from the face of the column or pedestal .

- The permissible shear stress, when shear reinforcement is not provided, shall not exceed *$k_s * \tau_s$, where $k_s = (0.5 + \beta_c)$, but not greater than one, β_c being the ratio of short side to long side of the column, and $\tau_c = 0.25(f_{ck})^{1/2}$ in limit state method of design, as stipulated in cl.31.6.3 of IS 456.*
- Normally, the thickness of the base slab is governed by shear. Hence, the necessary thickness of the slab has to be provided to avoid shear reinforcement.

- **(e) Bond (cl.34.2.4.3 of IS 456)**
- The critical section for checking the development length in a footing slab shall be the same planes as those of bending moments in part (c) of this section. Moreover, development length shall be checked at all other sections where they change abruptly. The critical sections for checking the development length are given in cl.34.2.4.3 of IS 456, which further recommends to check the anchorage requirements if the reinforcement is curtailed, which shall be done in accordance with cl.26.2.3 of IS 456.
- **(f) Tensile reinforcement (cl.34.3 of IS 456)**
- The distribution of the total tensile reinforcement, calculated in accordance with the moment at critical sections, as specified of this section, shall be done as given below for one-way and two-way footing slabs separately.
- (i) In one-way reinforced footing slabs like wall footings, the reinforcement shall be distributed uniformly across the full width of the footing i.e., perpendicular to the direction of wall. Nominal distribution reinforcement shall be provided as per cl. 34.5 of IS 456 along the length of the wall to take care of the secondary moment, differential settlement, shrinkage and temperature effects.
- (ii) In two-way reinforced square footing slabs, the reinforcement extending in each direction shall be distributed uniformly across the full width/length of the footing.
- (iii) In two-way reinforced rectangular footing slabs, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing slab. In the short direction, a central band equal to the width of the footing shall be marked along the length of the footing, where the portion of the reinforcement shall be determined as given in the equation below. This portion of the reinforcement shall be distributed across the central band:



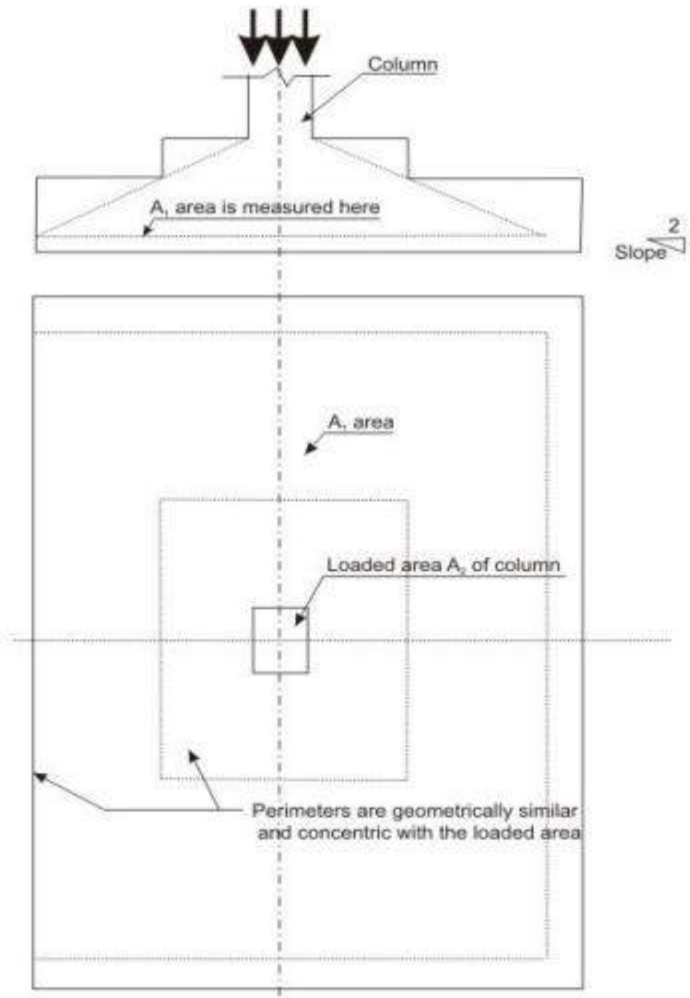


Reinforcement in the central band = {2/(β+1)} (Total reinforcement in the short direction)

Where β is the ratio of longer dimension to shorter dimension of the footing slab

Each of the two end bands shall be provided with half of the remaining reinforcement, distributed uniformly across the respective end band.

- (g) Transfer of load at the base of column (cl.34.4 of IS 456)



: Bearing area in sloped or stepped footing

All forces and moments acting at the base of the column must be transferred to the pedestal, if any, and then from the base of the pedestal to the footing, (or directly from the base of the column to the footing if there is no pedestal) by compression in concrete and steel and tension in steel. Compression forces are transferred through direct bearing while tension forces are transferred through developed reinforcement. The permissible bearing stresses on full area of concrete shall be taken as given below from cl.34.4 of IS 456:

$\sigma_{br} = 0.45f_{ck}$, in limit state method

It has been mentioned that the stress of concrete is taken as $0.45f_{ck}$ while designing the column. Since the area of footing is much larger, this bearing stress of concrete in column may be increased considering the dispersion of the concentrated load of column to footing. Accordingly, the permissible bearing stress of concrete in footing is given by (cl.34.4 of IS 456):

$\sigma_{br} = 0.45f_{ck}\,(A_1/A_2)^{1/2}$

with a condition that $(A_1/A_2)^{1/2} \leq 2.0$ (11.8)

where A_1 = maximum supporting area of footing for bearing which is geometrically similar to and concentric with the loaded area A_2, as shown in Fig.

A_2 = loaded area at the base of the column.

The above clause further stipulates that in sloped or stepped footings, A1 may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal, as shown in Fig.

If the permissible bearing stress on concrete in column or in footing is exceeded, reinforcement shall be provided for developing the excess force (cl.34.4.1 of IS 456), either by extending the longitudinal bars of columns into the footing (cl.34.4.2 of IS 456) or by providing dowels as stipulated in cl.34.4.3 of IS 456 and given below:

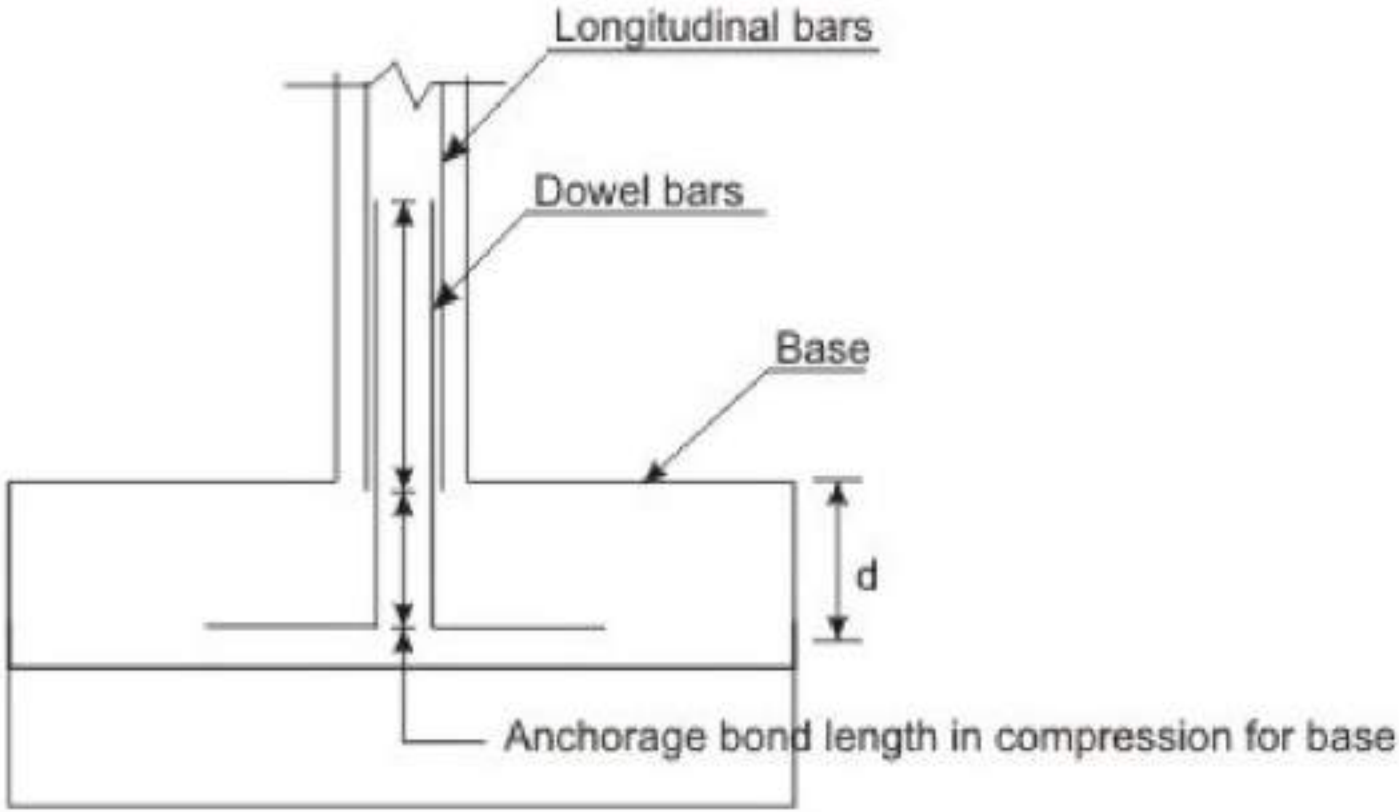
(i) Sufficient development length of the reinforcement shall be provided to transfer the compression or tension to the supporting member in accordance with cl.26.2 of IS 456, when transfer of force is accomplished by reinforcement of column (cl.34.4.2 of IS 456).

(ii) Minimum area of extended longitudinal bars or dowels shall be 0.5 per cent of the cross-sectional area of the supported column or pedestal (cl.34.4.3 of IS 456).

(iii) A minimum of four bars shall be provided (cl.34.4.3 of IS 456).

(iv) The diameter of dowels shall not exceed the diameter of column bars by more than 3 mm.

(v) Column bars of diameter larger than 36 mm, in compression only can be doweled at the footings with bars of smaller size of the necessary area. The dowel shall extend into the column, a distance equal to the development length of the column bar and into the footing, a distance equal to the development length of the dowel, as stipulated in cl.34.4.4 of IS 456.


Longitudinal bars
Dowel bars
Base
d
Anchorage bond length in compression for base

(h) Nominal reinforcement (cl. 34.5 of IS 456)

1. Clause 34.5.1 of IS 456 stipulates the minimum reinforcement and spacing of the bars in footing slabs as per the requirements of solid slab (cls.26.5.2.1 and 26.3.3b(2) of IS 456, respectively).

2. The nominal reinforcement for concrete sections of thickness greater than 1 m shall be 360 mm2 per metre length in each direction on each face, as stipulated in cl.34.5.2 of IS 456. The clause further specifies that this provision does not supersede the requirement of minimum tensile reinforcement based on the depth of section.

Problem 2:

Design an isolated footing for a square column, 400 mm x 400 mm with 12-20 mm diameter longitudinal bars carrying service loads of 1500 kN with M 20 and Fe 415. The safe bearing capacity of soil is 250 kN/m^2 at a depth of 1 m below the ground level. Use M 20 and Fe 415.

Solution 2:

Step 1: Size of the footing

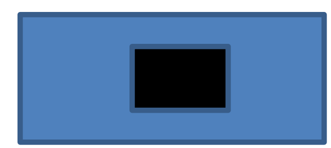
Given *P = 1500 kN (Including self wt. of Column),* q_c *= 250 kN/m*^{2} *at a depth of 1 m below the ground level. Assuming the weight of the footing and backfill as 10 per cent of the load, the base area required = 1500(1.1)/250 = 6.6 m*^{2}*. Provide 2.6 m x 2.6 m, area = 6.76 m*^{2}*.*

Step 2: Thickness of footing slab based on one-way shear

Factored soil pressure = 1500(1.5)/(2.6)(2.6) = 0.3328 N/mm^2, say, 0.333 N/mm^2.

Assuming *p = 0.25% in the footing slab, for M 20 concrete* τ_c *= 0.36 N/mm*^{2} *(Table 19 of IS 456).* V_c *= 0.36(2600)d and*

V_u *(actual) Nominal shear force= 0.333(2600)(1100 - d).*



From the condition that V_c *should be more than or equal to the actual* V_c*,*

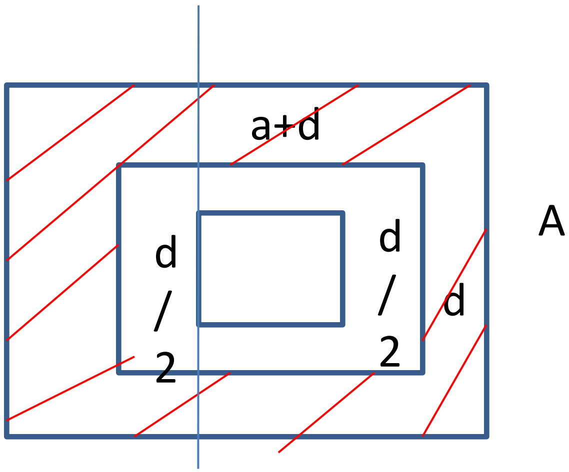
0.36(2600)*d ≥ 0.333(2600) (1100 – d)*

So, *d . ≥ 528.57 mm*

Provide *d = 536 mm. The total depth becomes 536 + 50 + 16 + 8 (with 50 mm cover and diameter of reinforcing bars = 16 mm) = 610 mm.*

- **Step 3: Checking for two-way shear**
- The critical section is at a distance of *d/2 from the periphery of the column. The factored shear force =* $0.333\{(2600)^2 - (400 + d)^2\}(10)^{-3} = 1959.34$ *kN.*

  Here d=536 mm
- *Shear resistance is calculated with the shear strength =* $k_s \tau_c = k_s(0.25)\,(f^{ck})^{1/2}$*; where* $k_s = 0.5 + \beta_c$ *(cl. 31.6.3 of IS 456). Here* $\beta_c = 1.0$*,* $k_s = 1.5 > 1$*; so* $k_s = 1.0$*. This gives shear strength of concrete =* $0.25\,(f_{ck})^{1/2} = 1.118$ *N/mm*^{2}*. So, the shear resistance = (1.118)* 4* (936)(536) = 2243.58 kN > 1959.34 kN. Hence, ok.*
- Thus, the depth of the footing is governed by one-way shear.
- a=400 mm
- d=536 mm



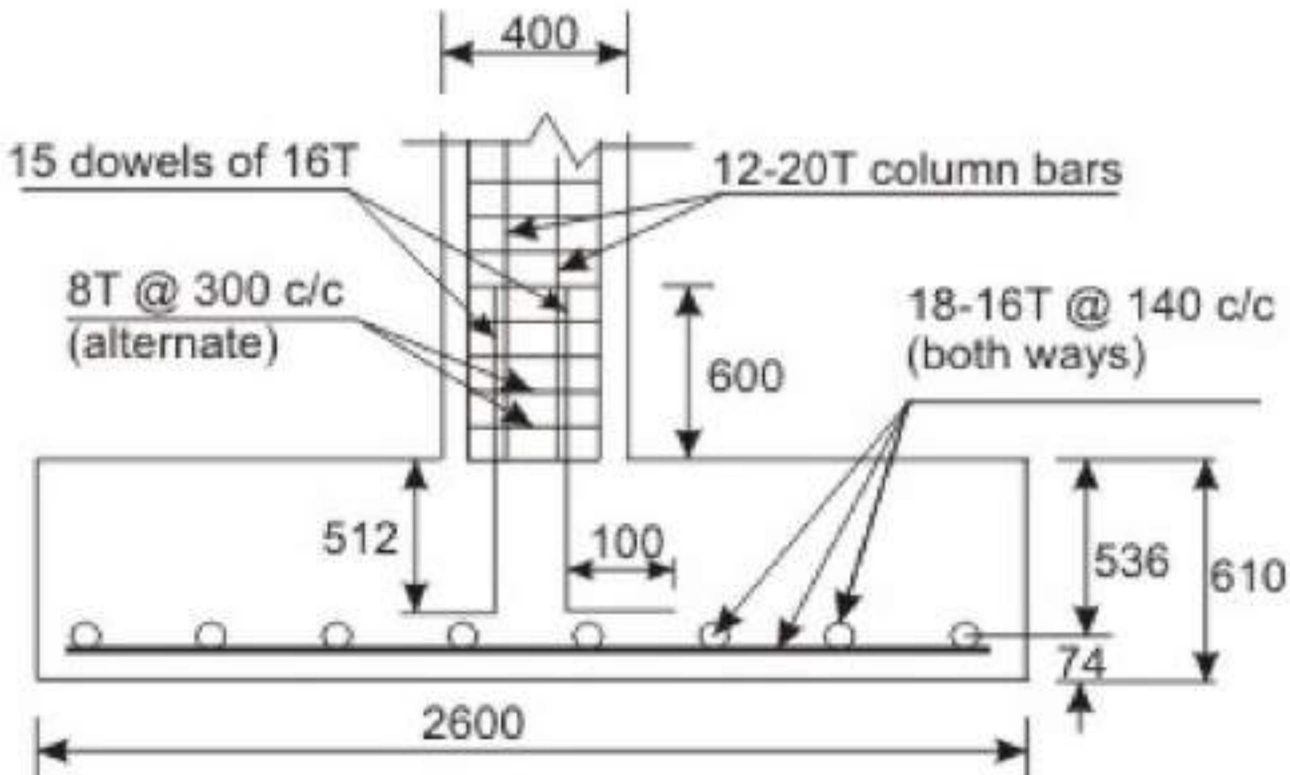
Area of shaded portion=A^2-$(a+d)^2$

- Step 4: Gross bearing capacity
- Assuming unit weights of concrete and soil as 24 kN/m3 and 20 kN/m^3 , respectively: Given, the service load = 1500 kN Weight of the footing = 2.6(2.6)(0.61)(24) = 98.967 kN Weight of soil = 2.6(2.6)(1.0-0.61)(20) = 52.728 kN (Assuming the depth of the footing as 1.0 m). Total = 1635.2 kN
- Gross bearing pressure = 1635.2/(2.6)(2.6) = 241.893 $kN/m^2 < 250\ kN/m^2$. Hence, ok.

- **Step 5: Bending moment**
- The critical section, is at the face of the column.
- $M_u = 0.333(2600)(1100)(550)$ Nmm = 523.809 kNm
- Moment of resistance of the footing = $0.138*f_{ck}*bd^2$ *where* $R = 0.138*20 = 2.76$
- Moment of resistance = 2.76(2600)(536)(536) = 2061.636 kNm > 523.809 kNm.
- Area of steel shall be determined :
- $M_u = 0.87\, f_y A_{st}\, d\, \{1 - (A_{st} f_y / f_{ck}\, bd)\}$ ....
- Substituting $M_u = 523.809$ kNm, $f_y = 415$ N/mm^2, $f_{ck} = 20$ N/mm^2, $d = 536$ mm, $b = 2600$ mm,
- Solving, we get $A_{st} = 2825.5805$ mm^2.
- Alternatively, we can use Table 2 of SP-16 to get the A_{st} *as explained below:*
- $M_u/bd^2 = 523.809(10^6)/(2600)(536)(536) = 0.7013$ N/mm^2. *Table 2 of SP-16 gives* $p = 0.2034$.
- $A_{st} = 0.2034(2600)(536)/100 = 2834.58$ mm^2.
- However, one-way shear has been checked assuming $p = 0.25\%$. *So, use* $p = 0.25\%$. *Accordingly,* $A_{st} = 0.0025(2600)(536) =$ 3484 mm^2.
- Provide 18 bars of 16 mm diameter (= 3616 mm^2) both ways. The spacing of bars = {2600 – 2(50) – 16}/17 = 146.117 mm. The spacing is 140 mm c/c

- **Step 6: Development length**
- $L_d = f_s \varphi/4(\tau_{bd}) = 0.87(415)(16)/4(1.6)(1.2) = 47(16) = 752\ mm$
- Length available = 1100 – 50 = 1050 mm > 752 mm.
- **Step 7: Transfer of force at the base of the column**
- $P_u = 1500(1.5) = 2250\ kN$
- Compressive bearing resistance = 0.45 fck(A1/A2) $^{1/2}$. For the column face A1/A2 = 1 and for the other face A1/A2 > 2 but should be taken as 2. In any case, the column face governs.
- Force transferred to the base through column at the interface = 0.45(20)(400)(400) = 1440 kN < 2250 kN.
- The balance force 2250 – 1440 = 810 kN has to be transferred by the longitudinal reinforcements, dowels or mechanical connectors.
- As it is convenient, we propose to continue the longitudinal bars (12-20 mm diameter) into the footing.
- The required development length of 12-20 mm diameter bars, assuming a stress level of $0.87f_y(810/2250) = 129.978\ N/mm^2$,
- *129.978(20)/4(1.6)(1.2)(1.25) = 270.8 mm. Here* τ_{bd} *for M 20 = 1.2* N/mm^2*, increased factor of 1.6 is due to deformed bars and increased factor of 1.25 is for the compression.*
- Length available = 610 – 50 – 16 – 16 – 16 = 512 mm > 270.8 mm . Hence, o.k.

- Alternatively: Design of dowels
- For the balance force 810 kN, the area of dowels = 810000/0.67(415) = 2913.15 mm^2. Minimum area = 0.5(400)(400)/100 = 800 mm^2 < 2913.15 mm^2 (cl.34.4.3 of IS 456). Therefore, number of 16 mm dowels = 2913.15/201 = 15. The development length of 16 mm dowels in compression = 0.87(415)(16)/4(1.6)(1.2)(1.25) = 601.76 mm. Available vertical embedment length = 610 – 50 – 16 – 16 – 16 = 512 mm. So, the dowels will be extended by another 100 mm horizontally, as shown in Fig.



Numerical Problem

Design an isolated footing of uniform thickness of a RC column bearing a vertical load of 600 KN and having a base of size 500x500 mm. the safe bearing capacity of soil may be taken as 120 KN/m2. Use M20 concrete and Fe 415 steel.

- **Solution**
- Size of footing
- W=600 KN;
- Self weight of footing @ 10% =60 KN
- Total load =660 KN
- Size of footing = 660/120 = 5.5 m^2

Since square footing , B= $\sqrt{5.5}$ =2.345 m2

Provide a square footing = 2.4mx 2.4m

Net upward pressure , p_0= 600/(2.4x2.4) = 104.17 KN/m^2

<u>Design of section</u>

The maximum BM acts at the face of column

M= $p_o \frac{B}{8}(B-b)^2 = 112.8\ KN-m$

Mu = 1.5M =169.2 KN-m

Therefore d = 160 mm; D = 160+60 = 220mm

<u>Depth on the basis of one-way shear</u>

For a one way shear, critical section is located at a distance ‘d’ from the face of the column where shear force V is given by

$$V = p_o B\{0.5(B-b)-d\} = 104.17x2.4\{0.5x(2.4-0.5)-0.001d\}$$

Vu = 1.5V

$$\tau_c = \frac{V_u}{bd} = \frac{375012(0.95-0.001d)}{2400d}$$

From table B.5.2.1.1 of IS 456:2000 k=1.16 for D = 220mm.

Also for under-reinforced section with p_t = 0.3% for M20 concrete, τ_c=0.384 N/mm^2.

Hence design shear stress = k τ_c =0.445 N/mm^2

From which we get d= 246.7 $\simeq$ 250 mm

<u>Depth for two way shear</u>

Take d greater one of the two i.e. 250mm. for two-way shear, the section lies at d/2 from the column face all round. The width bo of the section = b+d = 750mm

Shear force around the section

$$F = p_o[B^2 - b_o^2] = 541.42\,KN$$

Fu=1.5F

$$\tau_v = \frac{F_u}{4b_o d} = \frac{812.13x10^6}{4x750x250} = 1.083\,N/mm^2$$

Permissible shear stress = $k_s \tau_c$

Where $k_s = (0.5 + \beta_c) = (0.5 + 1)$ with a maximum value 1. ks=1

$\tau_c = 0.25\sqrt{f_{ck}}$ =1.118 N/mm2

Permissible shear stress= 1.118 N/mm2

Hence safe.

Hence d=250 mm, using 60 mm as effective cover and keeping D=330 mm, effective depth = 330-60= 270 mm in one direction and other direction d=270- 12 =258 mm.

Calculation of reinforcement

A_{st}= 1944 mm^2

Using 12 mm bars, spacing required = 138.27 mm

Design a rectangular footing of uniform thickness for an axially loaded column of size 300 mm x 600 mm, load on column is 1150 kN. SBC=200kN/m^2. M20 concrete and Fe 415.

- Load on column=1150 kN
- Load of foundation= 10% of column=115 kN
- Total load= 1265 kN
- SBC=200 kn?m2
- Area of footing= 1265/200=6.325m2
- Taking length of footing two times of width(x*2x=6.325, x=B=1.78 m say 1.8 m
- Another side=3.6 m
- Soil pressure=1150/(1.8*3.6)=177.47kn/m2
- Factored Pressure=1.5*177.47=266.21kN/m2

CALCULATION OF DEPTH OF FOOTIG:

SF in longer direction: 266.21*3.6*[(1.8-.3)/2-d]=718.767-958.356d

SF in shorter direction: 266.21*1.8*[(3.6-.6)/2-d]=718.767-479.178d

SF in shorter direction is more

SF

Design a sloped footing for a square column of 400 mm x 400 mm with 16 longitudinal bars of 16 mm diameter carrying a service load of 1400 kN. Use M 20 and Fe 415 both for column and footing slab. The safe bearing capacity of soil is 150 kN/m2

Given

Step 1: Size of the footing

P = 1400 kN and qc = 150 kN/m2 . Assuming the weight of the footing and the back file as 10 per cent of the load, the required base area is: 1400(1.1)/150 = 10.27 m2 . Provide 3400 x 3400 mm giving 11.56 m

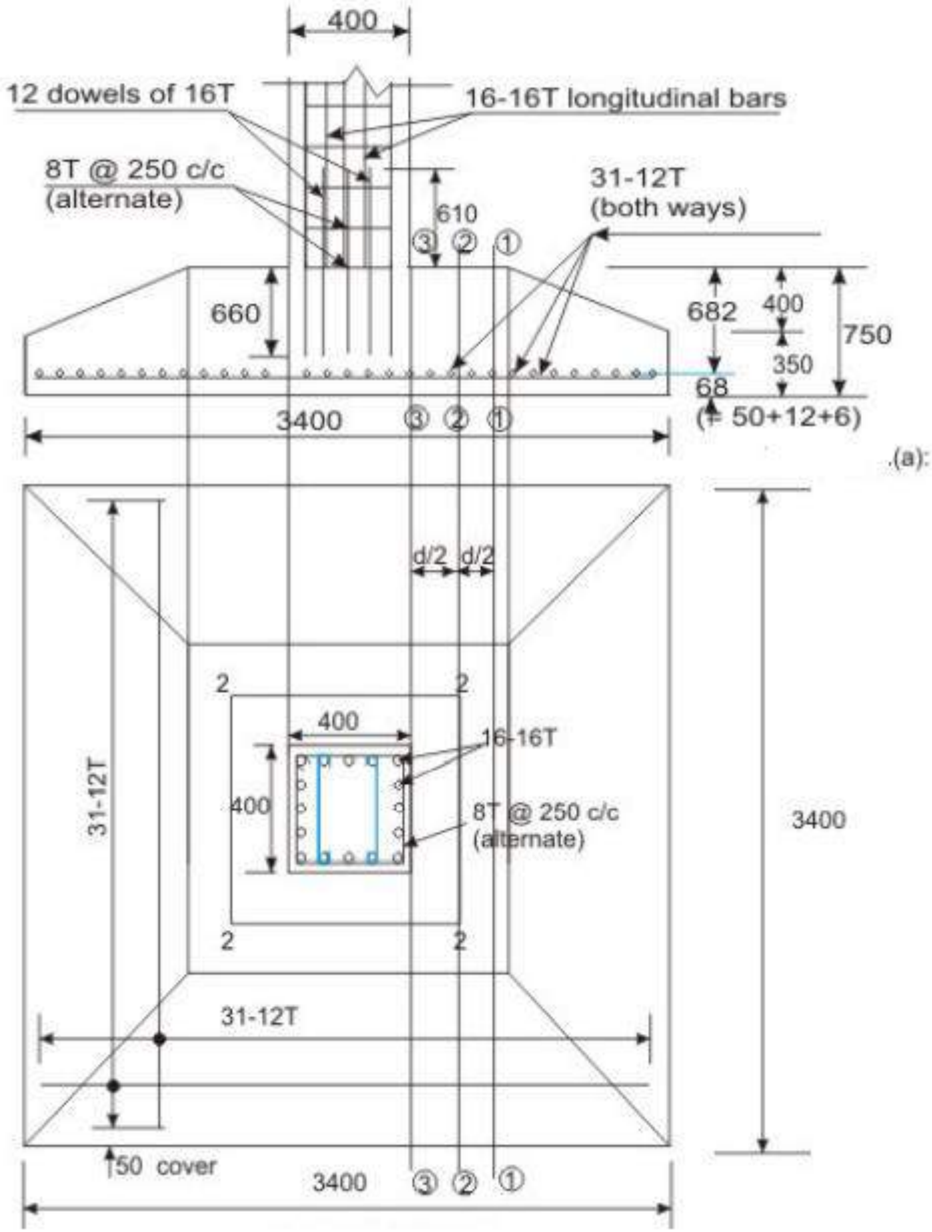
Step 2: Thickness of footing slab based on one-way shear

Factored bearing pressure = 1400(1.5)/(3.4)(3.4) = 181.66 kN/m2 = 0.18166 N/mm2 . Assuming 0.15 per cent reinforcement in the footing slab, Table 19 of IS 456 gives c τ for M 20 = 0.28 N/mm2 . From the condition that the one-way shear resistance one-way shear force, we have at a distance d from the face of the column (sec.1-1of Figs.a and b).

$0.28(3400)d \geq 0.18166(1500 – d)(3400)$

or $d \geq 590.24$ mm.

Provide total depth of footing as 670 mm, so that the effective depth = 670 – 50 – 16 – 8 = 596 mm. (The total depth is, however, increased to 750 mm in Step 7.)


400
12 dowels of 16T
16-16T longitudinal bars
8T @ 250 c/c
(alternate)
610
31-12T
(both ways)
660
682
400
350
750
68
(= 50+12+6)
3400
(a):
d/2 d/2
2
400
16-16T
400
8T @ 250 c/c
(alternate)
31-12T
3400
31-12T
50 cover
3400

b

Fig.a

Step 3: Checking for two-way shear

At the critical section 2222 (Figs. a and b),
the shear resistance = 4(400 + 596)(596)(0.25)$(fck)^{1/2}$ = 2654.73 kN.
The shear force = {(3.4)(3.4) – (0.996)(0.996)}0.18166 = 1919.78 kN < 2654.73 kN. Hence, o.k.

Step 4: Gross bearing capacity

Assuming unit weights of concrete and soil as 25 kN/m^3 and 18 kN/m^3 , respectively, we have: Load on footing = 1400.00 kN Weight of footing = (3.4)(3.4)(0.67)(25) = 193.63 kN Weight of soil = (3.4)(3.4)(1.25 - 0.67)(18) = 120.69 kN (Assuming the depth of the footing as 1.25 m). Total = 1714.32 kN
Pressure= 1714.32 / (3.4)(3.4) = 148.30 kN/m^2 < 150 kN/m^2 . Hence o.k.

Step 5: Bending moment

We have to determine the area of steel in one direction as it is a square footing. So, we consider the lower effective depth which is 596 mm. The critical section is sec.33 (Figs. a and b), where we have Mu = 3400(1500)(0.18166)(1500)/2 = 694.8495 kNm
Mu/bd^2 = 694.8495(106)/(3400)(596)(596) = 0.575 N/mm^2
Table 2 of SP-16 gives, p = 0.165%. Accordingly, area of steel = 0.165(3400)(596)/100 = 3343.56 mm^2 . Provide 30 bars of 12 mm diameter (= 3393 mm^2), both ways.

Step 6: Development length Development length of 12 mm diameter bars = 0.87(415)(12)/4(1.6)(1.2) = 564.14 mm. Hence, o.k.

Step 7: Providing slope in the footing slab

Since the three critical sections (.i.e., of bending moment, two-way shear and one-way shear) are within a distance of 596 mm from the face of the column, the full depth of the footing slab is provided up to a distance of 700 mm from the face of the column. However, by providing slope the available section now is a truncated rectangle giving some less area for the one-way shear. Accordingly, the depth of the footing is increased from 670 mm to 750 mm. With a cover of 50 mm and bar diameter of 12 mm in both directions, the revised effective depth = 750 – 50 – 12 – 6 = 682 mm. Providing the minimum depth of 350 mm at the edge, as shown in Figs. a and b, we check the one-way shear again, taking into account of the truncated rectangular cross-section at a distance of 682 mm from the face of the column.

One-way shear force = 0.18166(1500 – 682)(3400) = 505232.792 N

Area of truncated rectangle = (1800=400+700+700)(682) + (3400-1800=1600) (282) + 1600(682 – 282)/2 = 1998800 mm2

The shear stress = 505232.792/ 1998800 = 0.2527 N/mm^2 < 0.28 N/mm^2 . Hence, o.k.

Step 8: Revised area of steel

The bending moment in step 5 is 694.8495 kNm at the face of the column. With d = 682 mm now, we have Mu/bd2 = $694.8495(10^6)/(3400)(682)(682) = 0.4394$ N/mm2 Table 2 of SP-16 gives, p is less than 0.15 per cent. Provide p = 0.15 per cent due to the one-way shear.

So, Ast = 0.15(3400)(682)/100 = 3478.2 mm^2 . Provide 31 bars of 12 mm (Ast = 3506 mm2), both ways. Effectively, the number of bars has increased from 30 to 31 now.

Step 9: Transfer of force at the base of the column

Pu = 1400(1.5) = 2100 kN. Compressive bearing resistance = 0.45 fck = 0.45(20) = 9 N/mm2 .

Force transferred at the base through the column = $9(400)(400)(10^{-3})$

= 1440 kN < 2100 kN.

Provide dowels for the excess (2100 – 1440) = 660 kN.

The area of dowels = 660(103)/(0.67)(415) = 2373.67 mm2 .

Minimum area of dowels = 0.5(400)(400)/100 = 800 mm^2 .

Provide 12 dowels of 16 mm diameter (area = 2412 mm2).

The development length of 16 mm dowels = 0.87(415)(16)/4(1.6)(1.2)(1.25) = 601.76 mm. The vertical length available 750 – 50 – 12 – 12 – 16 = 660 mm > 601.76 mm. Hence, o.k. The arrangement of reinforcement and dowels is shown in Fig. a and b.

COLUMN WITH MOMENT

Design one isolated footing for a column 300 mm x 450 mm, having 20 bars of 20 mm diameter (Ast = 4021 mm^2) carrying Pu = 1620 kN and Mu = 170 kNm using M 25 and Fe 415. Assume that the moment is reversible. The safe bearing capacity of the soil is 200 kN/m^2 at a depth of 1 metre from ground level. Use M 25 and Fe 415 for the footing.

Step 1: Size of the footing

Given Pu = 1620 kN and Mu = 170 kNm.

The footing should be symmetric with respect to the column as the moment is reversible.

Assuming the weights of footing and backfill as 15 per cent of P_u,

the eccentricity of load P_u at the base is e = Mu/P(1.15) = $170(10^6)/1620(1.15)(10^3)$ = 91.25 mm. This eccentricity may be taken as < L/6 of the footing.

The factored bearing pressure is 200(1.5) = 300 kN/m^2 .

For the footing of length L and width B, we, therefore, have: $Pu/BL + 6M/BL^2 \le 300$ or, $1620(1.15)/BL + 6(170)/BL^2 \le 300$ or, $BL^2 – 6.21L – 3.4 \le 0$ (EQN-1)

For the economic proportion, let us keep equal projection beyond the face of the column in the two directions. This gives (L – 0.45)/2 = (B – 0.3)/2 or, B = L – 0.15 (EQN-2)

Using Eq.(2) in Eq.(1), we have (L – 0.15) L2 – 6.212 – 3.4 ≤ 0

or $L^3 – 0.15 L^2 – 6.21 L – 3.4 \le 0$ We have L = 2.8 m and B = 2.65 m.

Let us provide L = 2.85 m and B = 2.70 m (Fig.11.29.4b).

We get the maximum and minimum pressures as

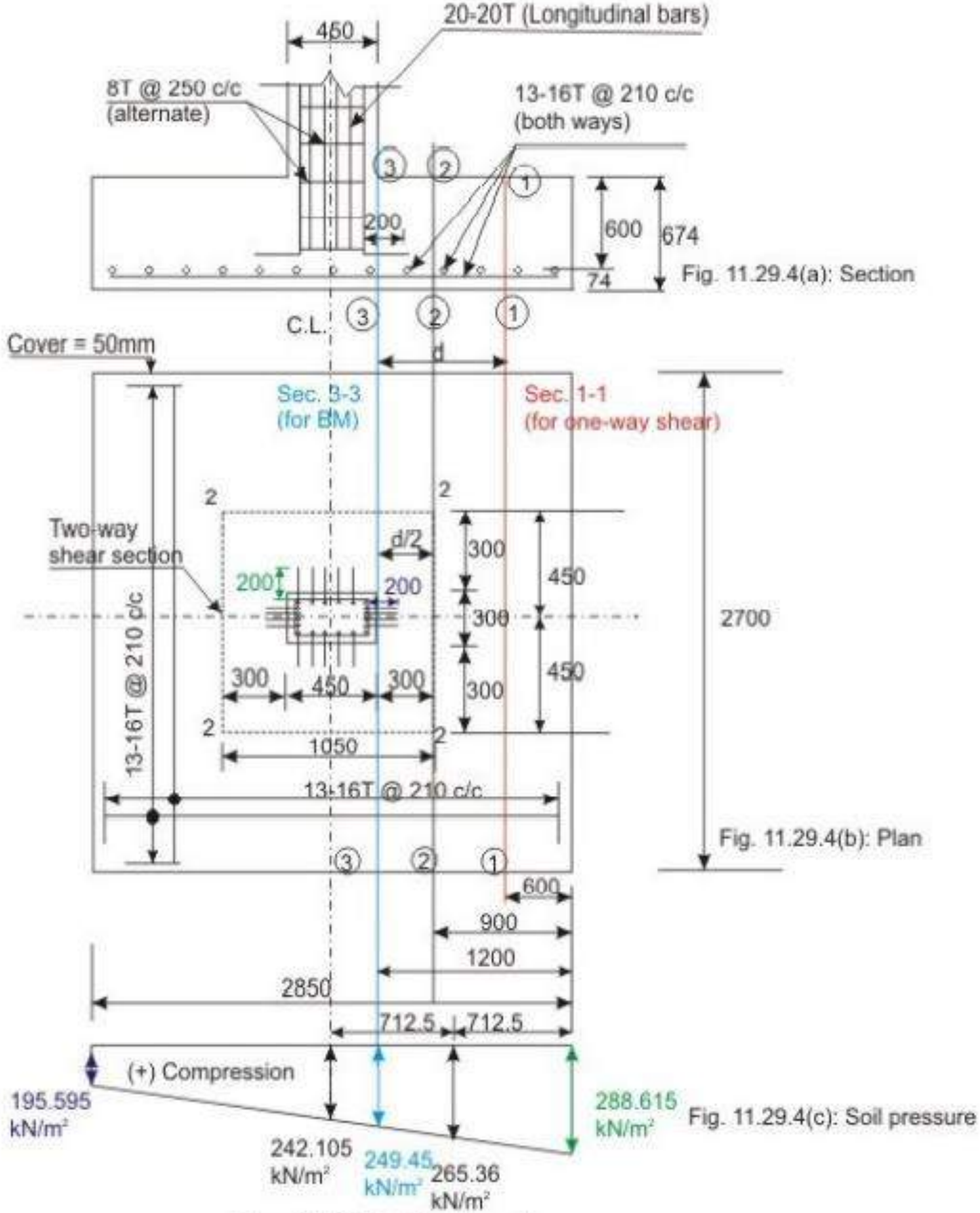


Fig. 11.29.4: Problem 4

P/A±M/Z=1620(1.15)/(2.85)(2.70) ± 170(6)/(2.7)(2.85)(2.85) = 242.105 ± 46.51 = 288.615 kN/m^2 and 195.595 kN/m^2 , respectively (Fig.11.29.4c).

Both the values are less than 300 kN/m^2 . Hence, o.k.

Step 2: Thickness of footing slab based on one-way shear

The critical section (sec.11 of Figs.11.29.4a and b) is at a distance d from the face of the column. The average soil pressure at sec.11 is {288.615 – (288.615 – 195.595)(1200 – d)/2850} = 249.449 + 0.0326d. The one-way shear force at sec.11 = (2.7)(1.2 – 0.001d)*(249.449 + 0.0326d) kN. Assuming 0.15 per cent reinforcement in the footing slab, the shear strength of M 25 concrete = 0.29 N/mm^2 . Hence, the shear strength of the section = 2700(d)(0.29)(10^{-3}) kN. From the condition that shear strength has to be ≥ shear force, we have

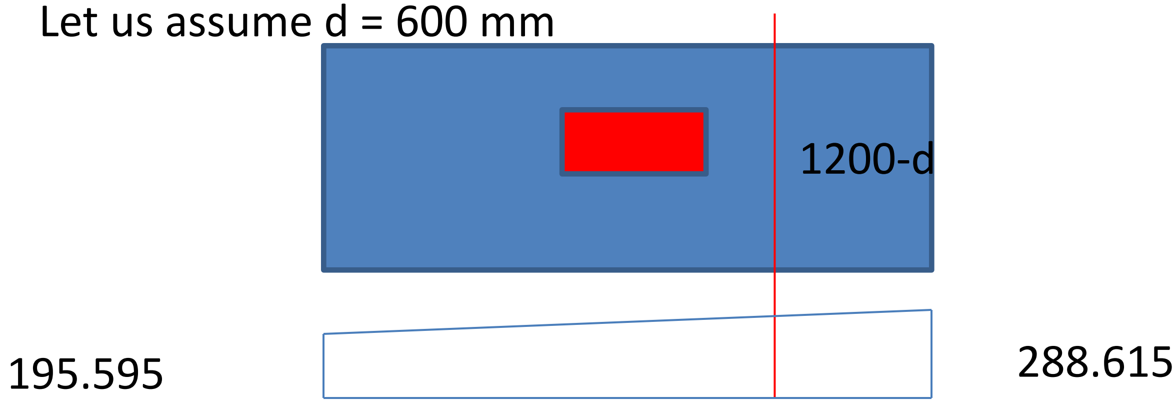
$2700(d)(0.29)(10^{-3}) = (2.7)(1.2 – 0.001d)(249.449 + 0.0326d)$

$2700(d)(0.29)(10^{-3}) = (2.7)(1.2 – 0.001d)(249.449 + 0.0326d)$

This gives,

$d^2 + 15347.51534d – 9182171.779 = 0$ Solving, we get d = 576.6198.

Let us assume d = 600 mm



Step 3: Checking for two-way shear

At the critical section 2222 (Figs.11.29.4a and b), the shear resistance is obtained cl.31.6.31 of IS 456, which gives $\tau_c = (0.5 + 450/300)(0.25)(25)^{1/2}$ but the multiplying factor (0.5 + 450/300) >/ 1.0. So, we have $\tau_c = 0.25(25)^{1/2} = 1.25$ N/mm^2 .

Hence, the shear resistance = (1.25)(2){(300 + 600) + (450 + 600)}(600) = 2925 kN. Actual shear force is determined on the basis of average soil pressure at the centre line of the cross-section which is (195.595 + 288.615)/2 = 242.105 kN/m^2 (Fig.11.29.4c). So, the actual shear force = Vu = (242.105){(2.7)(2.85) – (0.3 + 0.6)(0.45 + 0.6)} = 1634.209 kN < shear resistance (= 2925 kN).

Hence, the depth of the footing is governed by one-way shear. With effective depth = 600 mm, the total depth of footing = 600 + 50 (cover) + 16 (bar dia) + 8 (half bar dia) = 674 mm.

Step 4: Gross bearing capacity

Assuming the unit weights of concrete and soil as 25 kN/m^3 and 18 kN/m^3 , respectively,

we have the bearing pressure for (i) Pu = 1620 kN, (ii) Mu = 170 kNm and (iii) self weight of footing and backfill soil.

(i) Due to Pu = 1620 kN: pressure = 1620/(2.7)(2.85) = 210.53 kN/m2

(ii) Due to Mu = 170 kNm: pressure = ± 170(6)/(2.7)(2.85)(2.85) = ± 46.51 kN/m ^{2}

(iii) Self weight of footing of depth 674 mm and soil of (1000 – 674) = 326 mm:

pressure = 0.674(25) + 0.326(18) = 22.718 kN/m^2

Thus, the maximum and minimum pressures are = 210.53 + 22.718 ± 46.51 = 279.758 kN/m ^{2} and 186.738 kN/m^2 < 300 kN/m^2 . Hence, o.k.

Step 5: Bending moment

(i) In the long direction (along the length = 2850 mm)

Bending moment at the face of column (sec.33 of Figs.11.29.4a and b) is determined where the soil pressure = 288.615 – (288.615 – 195.595)(1200)/2850 = 249.45 kN/m^2 .

So, the bending moment = 249.45(2.7)(1.2)(0.6) +

(½)*(288.615 – 249.45) (2.7)(1.2)*[(2/3)*(1.2)] = 535.68 kNm.

$M/Bd^2 = 527.23(10^6)/(2700)(616)(616) = 0.522\ N/mm^2 < 3.45\ N/mm^2$ for M 25 concrete.

Table 3 of SP-16 gives p = 0.1462 < 0.15 per cent as required for one-way shear.

Thus, Ast = 0.15(2700)(616)/100 = 2494.8 mm^2 .

Provide 13 bars of 16 mm diameter (area = 2613 mm^2),

spacing = (2700 – 100 – 16)/12 = 215.33 mm, say 210 mm c/c.

(ii) In the short direction (B = 2700 mm)

The average pressure on soil between the edge and centre of the footing = (288.615 + 242.105)/2 = 265.36 kN/m^2 .

The bending moment is determined with this pressure as an approximation. Bending moment = (265.36)(1.2)(0.6)(2.85) kNm = 544.519 kNm M/Ld^2 = 544.519(106)/(2850)(600)(600) = 0.531 Table 3 of SP-16 gives p = 0.15068, which gives area of steel = 0.15068(2850)(600)/100 = 2576.628 mm2 . Provide 13 bars of 16 mm diameter (area = 2613 mm2) 227.83 mm say@ 210 mm c/c; i.e. the same arrangement in both directions.

Step 6: Development length Development length of 16 mm diameter bars (M 25 concrete) = 0.87(415)(16)/4(1.6)(1.4) = 644.73 mm.

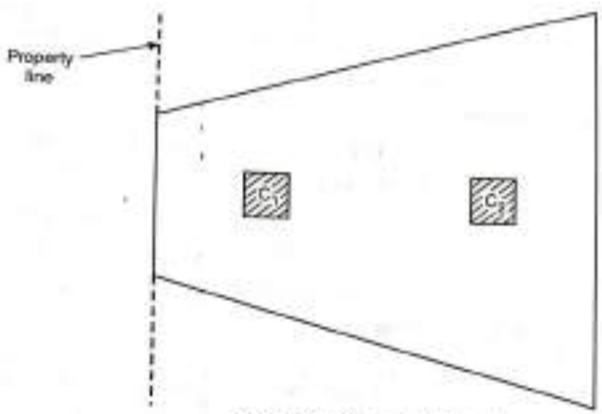
Length available = 1200 – 50 – 8 = 1142 mm > 644.73 mm.

Hence, o.k.

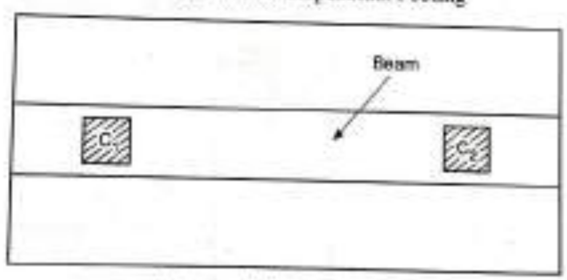
Step 7: Transfer of force at the base of the column

Since the column is having moment along with the axial force, some of the bars are in tension. The transfer of tensile force is not possible through the column-footing interface. So, the longitudinal bars of columns are to be extended to the footing. The required development length of 20 mm bars = 0.87(415)/4(1.4)(1.6) = 805.92 mm. Length available = 600 mm < 805.92 mm. The bars shall be given 90^o bend and then shall be extended by 200 mm horizontally to give a total length of 600 + 8(20) (bend value) + 200 = 960 mm > 805.92 mm (Figs.11.29.4 a and b). The arrangement of reinforcement is shown in Figs.11.29.4a and b.

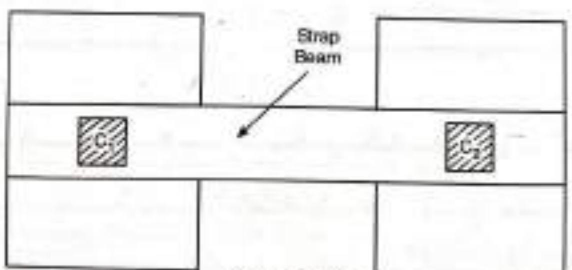
COMBINED FOOTING



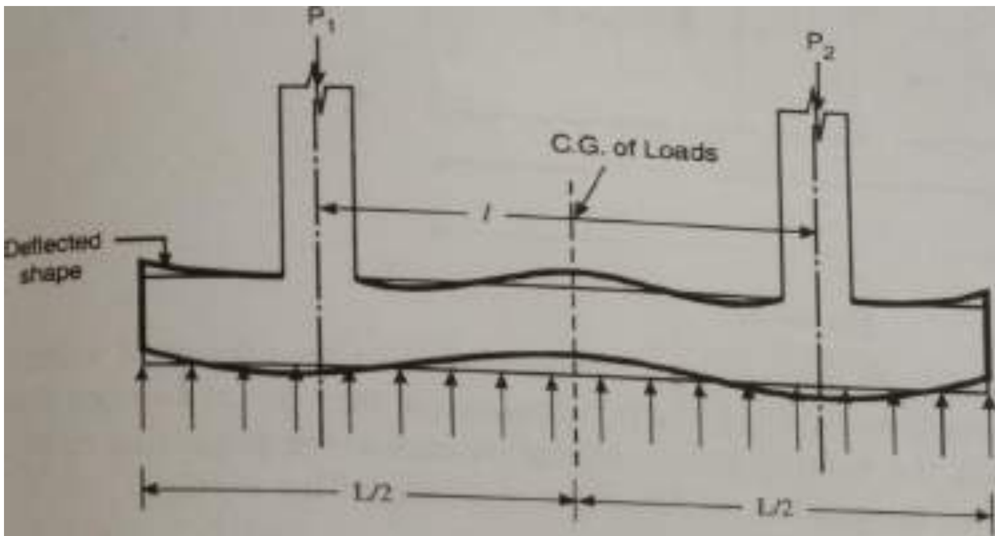
Combined Trapezoidal Footing



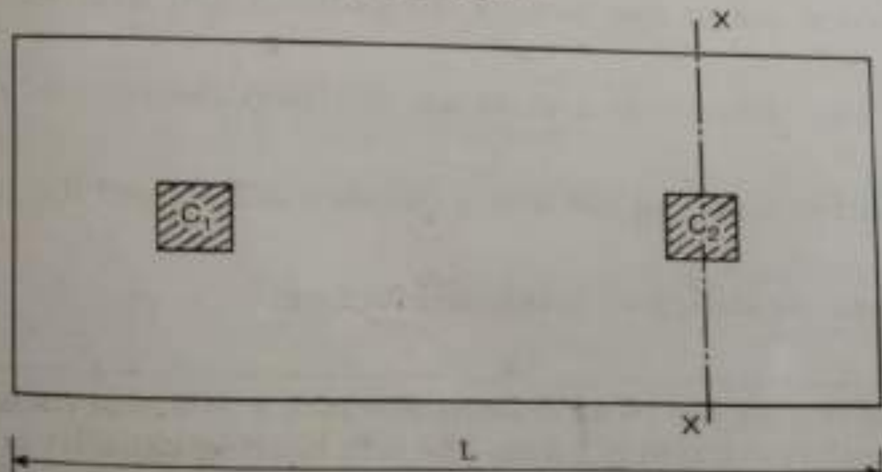
Beam and Slab Type Footing



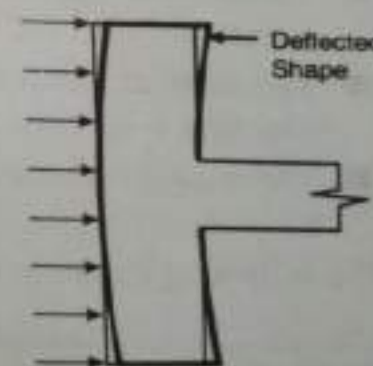
Strap Footing



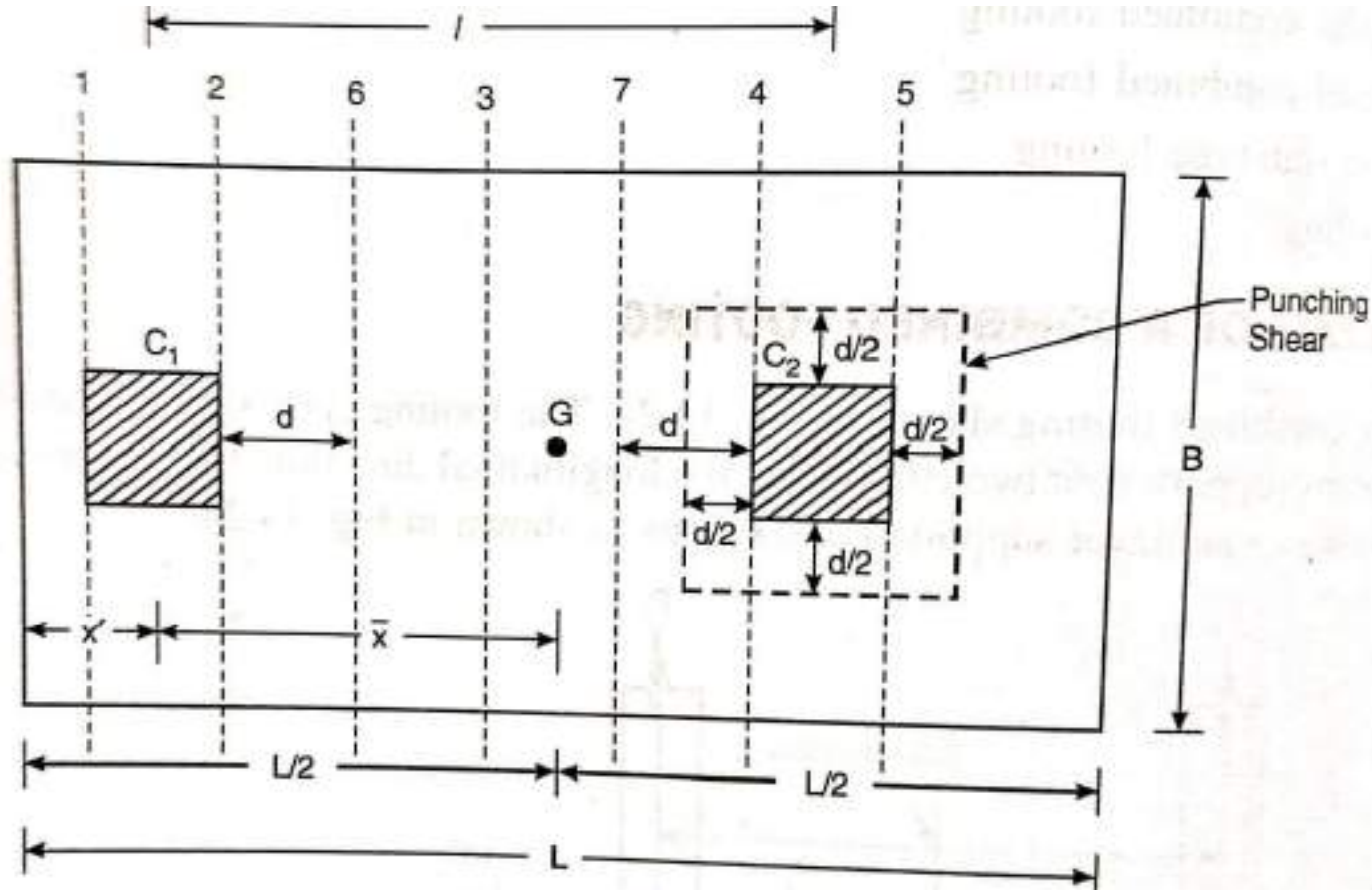
(*b*) L-section



(*a*) Plan



(*c*) Section (X-X)



l
1
2
6
3
7
4
5
C1
C2
d/2
d
G
d
d/2
d/2
d/2
Punching Shear
B
x′
x̄
L/2
L/2
L

Consider the combined footing. The footing carries upward base pressure and supported on two columns in the longitudinal direction. The distribution of load is like the below fig.

Assuming column C1 and C2 carrying P1 and P2 load respectively

The CG of combined load P_1+P_2 from centre of C_1 is

$$\bar{x} = \frac{P_2 l}{P_1 + P_2}$$

Point G should be should be the CG of the footing.

$$\bar{x} + x' = \frac{L}{2} \qquad L = 2(\bar{x} + x')$$

In longitudinal direction, footing can be designed as a beam with width B and max. BM and SF can be calculated from the loading diagram. In transverse direction, it can be designed as cantilever projecting from centre of column.

Salient Points.

The critical section for BM should be checked at face of the column and near the mid span,(1-1,2-2,3-3,4-4 and 5-5)

The critical section for one way shear is at a distance d from the face of the column(6-6, 7-7)

The heavier column should be checked for punching shear at a distance d/2 from the face of the column.

PROBLEM

- Design a combined footing for two columns 500 mm X 500 mm each, 5 m apart carrying a load of 1600 kN.SBC= 200 kN/m^2. Use M25 and Fe415 steel. Width is restricted to 2.4 m.
- Solution:
- Column size= 500 mm X 500 mm
- P_1=P_2=1600 kN
- l=5m
- q_0=200kN/m^2
- Area of footing:
- Total load on column=2*1600=3200kN
- Self wt. of the footing=10% of 3200=320kN
- Total Load W=3320 kN
- Area of footing required=Total load/SBC=3320/200=17.6 m^2
- Width available= 2.4 m, Length of footing=17.6/2.4=7.33 m

Taking a length of 7.5 m, Area provided=7.5*2.4=18 m2

Factored upward pressure=1.5*3200/(7.5*2.4)=266.7 kN/m^2

Design in longitudinal direction:

Upward soil pressure per unit length=266.7*2.4=640 kN

1.Calculation of maximum BM and SF:

SF DISTRIBUTION:

SF at C_1==-640*1.25=-800 kN

SF to the right of C_1=-800+1.5*1600=1600 kN

SF at C_2==640*1.25=800 kN

SF to the right of C_2=800-1.5*1600=-1600 kN

BM Distribution:

BM at C_1==-640*1.25*1.25/2=500 kNm

BM at mid span= (640*3.75*3.75/2)-2400*2.5=-1500kNm

For zero moment point=M_x=640*x*x/2-2400*(x-1.25)=0

x^2-7.5x+9.375=0

X=1.585 m

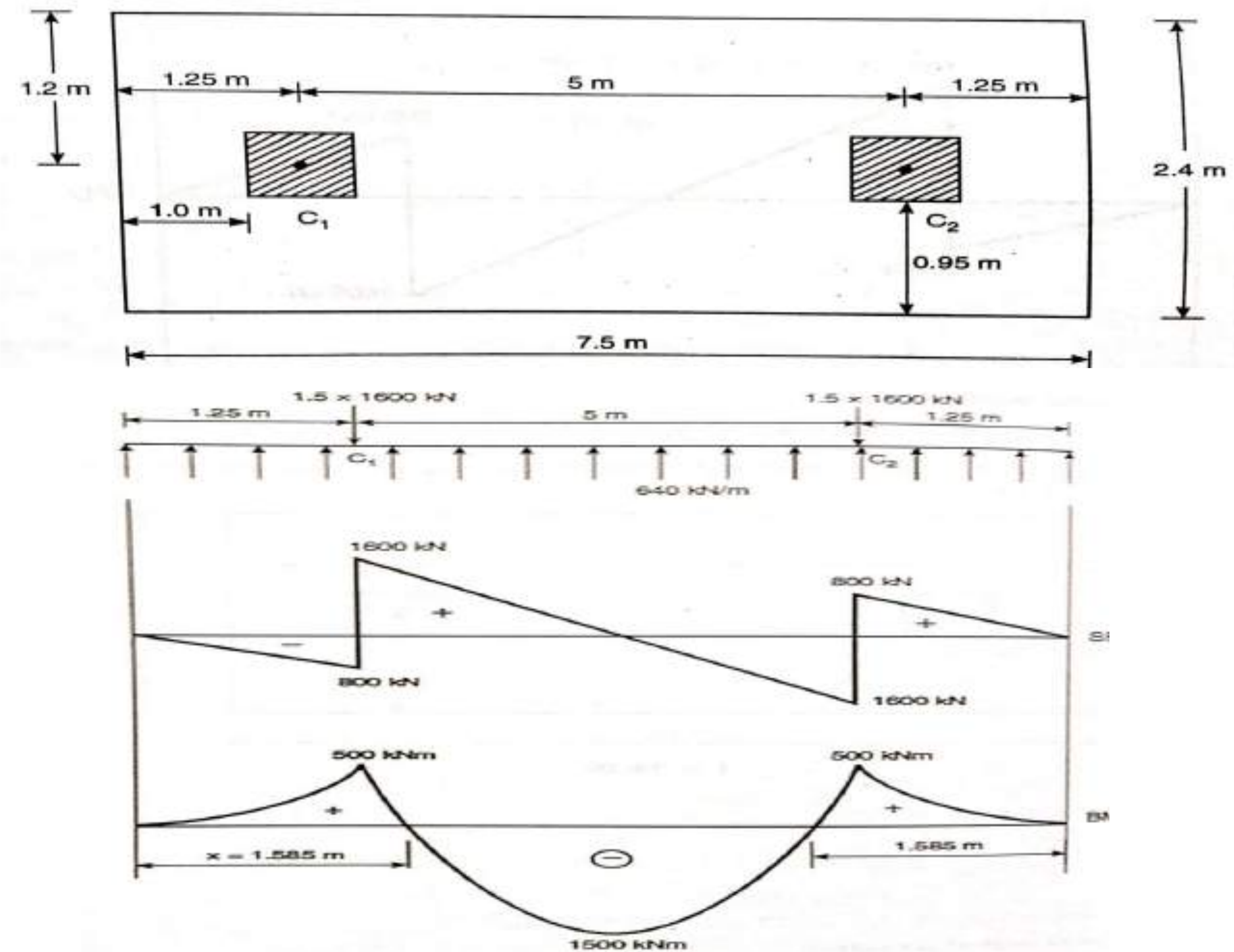

1.2 m
1.25 m
5 m
1.25 m
2.4 m
1.0 m
C1
C2
0.95 m
7.5 m
1.5 × 1600 kN
1.5 × 1600 kN
1.25 m
5 m
1.25 m
C1
C2
640 kN/m
1600 kN
800 kN
800 kN
1600 kN
500 kNm
500 kNm
x = 1.585 m
1.585 m
1500 kNm

Fig. 14.27

ii) Depth of footing:

1. From BM consideration: $M_u = 0.36 * f_{ck} * b * x_{umax}(d - 0.42 * x_{umax})$

$$M_u = 0.36 * 25 * 2400 * 0.48d(d - 0.42 * 0.48d)$$

$$1500 * 10^6 = 0.36 * 25 * 2400 * 0.48d_{req}^2(1 - 0.42 * 0.48)$$
$$d_{req} = 432.57\ mm$$

1. From one-way shear consideration:

$$V_u = \left\{1600 - 640\left[0.25 + \frac{d}{1000}\right]\right\} * 10^3$$

Assuming 0.2% steel, τ_c=0.32N/mm^2 $V_c = 0.32 * 2400 * d$

$$V_c = 0.32 * 2400 * d \geq V_u$$
$$= \left\{1600 - 640\left[0.25 + \frac{d}{1000}\right]\right\} * 10^3$$

d_{req}=1023 mm

Take overall depth D=1100 mm, d=1100-60=1040mm

3. From Two way shear: Critical section for two way shear d/2 from the face of the column.

Area of resistance for punching=4*(500+1040)*1040=6406400 mm2

Shear Force=V_u=2400-266.7*(.5+1.04)^2= 1768 kN

Nominal shear stress=τ_v =1768*1000/6406400=0.28 N/mm2

Shear resistance= $k_s.\ \tau_c$ =1*0.25*sqrt(f_{ck})=1.25 N/mm^2 > τ_v

Longitudinal Reinforcement: -ve moment

$$M_u = 1500 * 10^6 = 0.87 * f_y * A_{st} * d\left(1 - \frac{A_{st} * f_y}{f_{ck} bd}\right)$$

$$M_u = 1500 * 10^6$$

$$= 0.87 * 415 * A_{st} * 1040\left(1 - \frac{A_{st} * 415}{25 * 2400 * 1040}\right)$$

Ast = 4052 mm^2

Provide 21 number 16 mm dia. bars as top reinforcement,

Spacing=2400/21=110 mm c/c

For top transverse reinforcement=.12*2400*1100/100=3168 mm^2

No of 16 mm dia. Bars will be 16,

5 bars can be curtailed from point of contra flexure:

For +ve moment reinforcement:

bars can be curtailed from point of contra flexure:

$$M_u = 500 * 10^6 = 0.87 * f_y * A_{st} * d\left(1 - \frac{A_{st} * f_y}{f_{ck} bd}\right)$$

$$M_u = 500 * 10^6$$

$$= 0.87 * 415 * A_{st} * 1040\left(1 - \frac{A_{st} * 415}{25 * 2400 * 1040}\right)$$

A_{st}=1338mm^2< minimum

Provide the minimum 16 nos. 16 mm dia at bottom for a distance C1 to C2.

TRANSVERSE REINFORCEMENT:

In transverse direction , footing is designed as a cantilever supported on columns. Transverse reinforcement is to provided under each column within a band having a width equal to the width of the column plus two times effective depth of foundation.

Band width under $C_1$and C_2=0.5+1.04+1.0(on outer side this is length available)=2.54 m

Upward pressure =1.5*1600/(2.4)=1000 kN/m

BM on the face of the column=1000*0.95*0.95/2=451.25 kNm

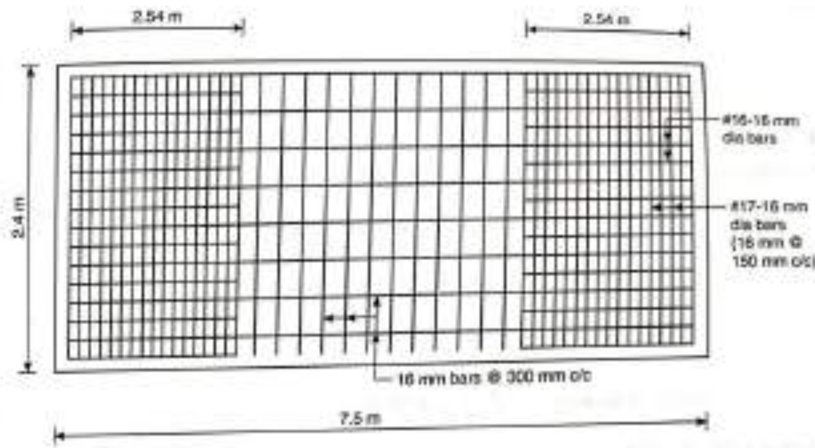
For this bending moment A_{st} will be less than the minimum.

A_{st} min=0.12*2540*1100/100=3353 mm^2

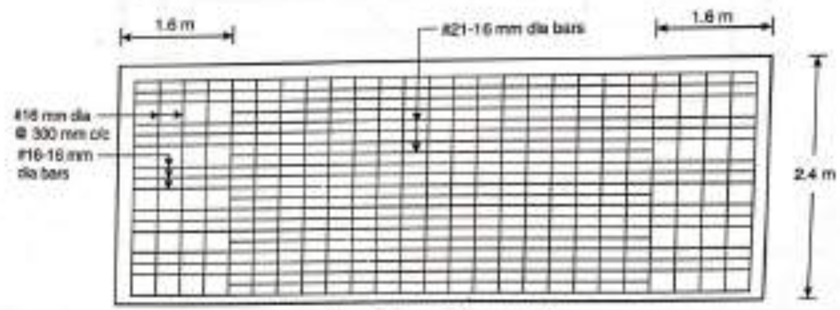
Spacing of 16 mm diameter bars=2540/(3353/201)=152 mm

Provide 16 mm dia. Bars 150 mm c/c for a distance of 2.54 m under each column and on rest 16 mm dia. Bars 300mm c/c.

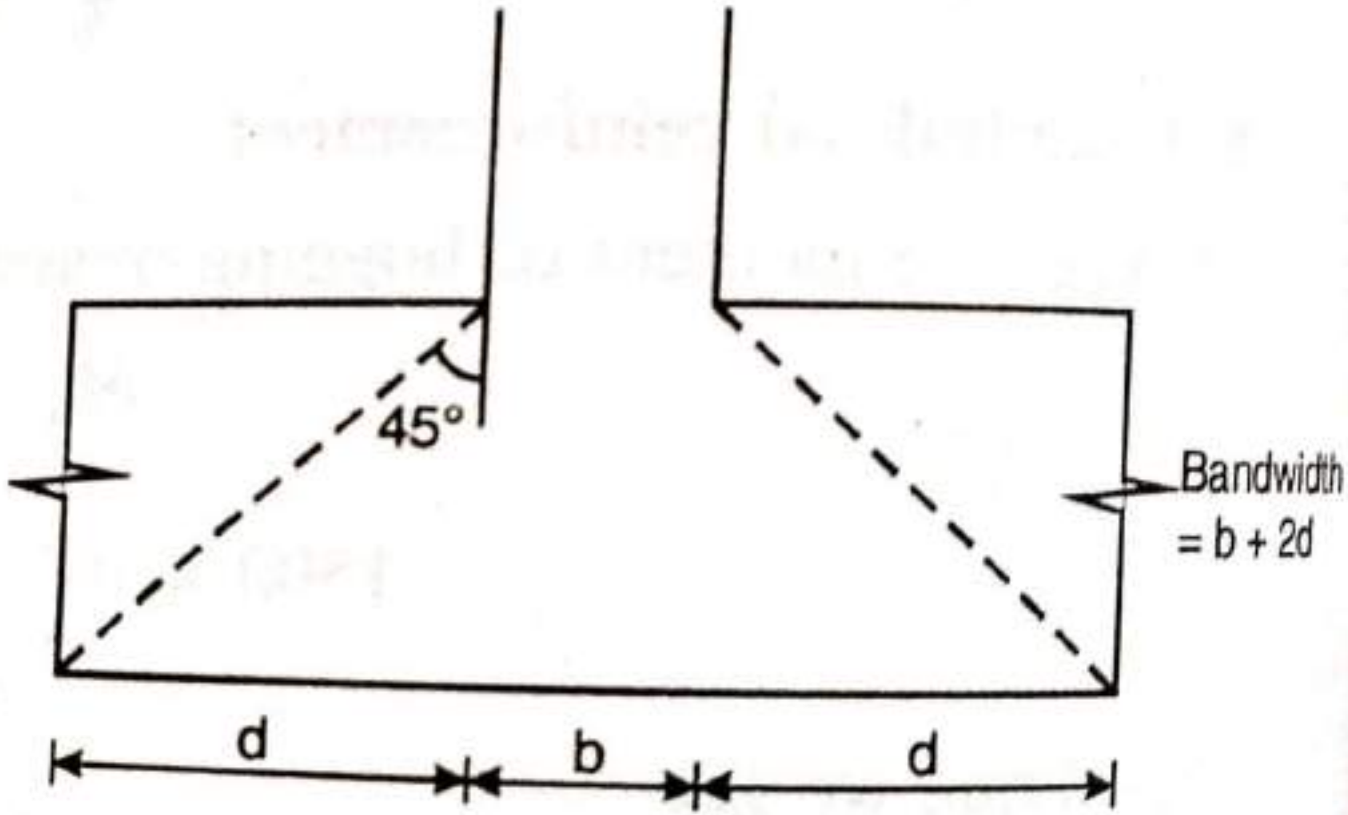
DETAILING



Plan of Bottom reinforcement



Plan of Top reinforcement


45°
Bandwidth
= b + 2d
d
b
d