

VISVESVARAYA TECHNOLOGICAL UNIVERSITY

BELGAUM



DESIGN OF RC STRUCTURAL ELEMENTS

(Subject Code: 21CV53)

LECTURE NOTES

(MODULE-5)

V-SEMESTER

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Module - 5

Design of Columns

Introduction:

A column is defined as a compression member, the effective length of which exceeds three times the least lateral dimension. Compression members, whose lengths do not exceed three times the least lateral dimension, may be made of plain concrete. A column forms a very important component of a structure. Columns support beams which in turn support walls and slabs. It should be realized that the failure of a column results in the collapse of the structure. The design of a column should therefore receive importance.

A column is a vertical structural member supporting axial compressive loads, with or without moments. The cross-sectional dimensions of a column are generally considerably less than its height. Columns support vertical loads from the floors and roof and transmit these loads to the foundations.

The more general terms compression members and members subjected to combined axial load and bending are sometimes used to refer to columns, walls, and members in concrete trusses or frames. These may be vertical, inclined, or horizontal. A column is a special case of a compression member that is vertical. Stability effects must be considered in the design of compression members.

5.1 Classification of columns

A column may be classified based on different criteria such as:

1. Based on shape

- Rectangle
- Square
- Circular
- L type
- T type
- + type

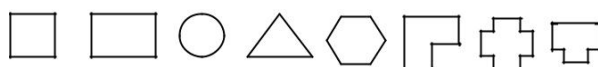


Fig 5.1 Types of Columns (Based on Shape)

2. Based on slenderness ratio or height

- Short column and Long column or Short and Slender Compression Members

A compression member may be considered as short when both the slenderness ratios namely l_{ex}/D and l_{ey}/b are less than 12: 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.

It shall otherwise be considered as a slender or long compression member

The great majority of concrete columns are sufficiently stocky (short) that slenderness can be ignored. Such columns are referred to as short columns. Short column generally fails by crushing of concrete due to axial force. If the moments induced by slenderness effects weaken a column appreciably, it is referred to as a slender column or a long column. Long columns generally fail by bending effect than due to axial effect. Long column carries less load compared to long column.

3. Based on pattern of lateral reinforcement

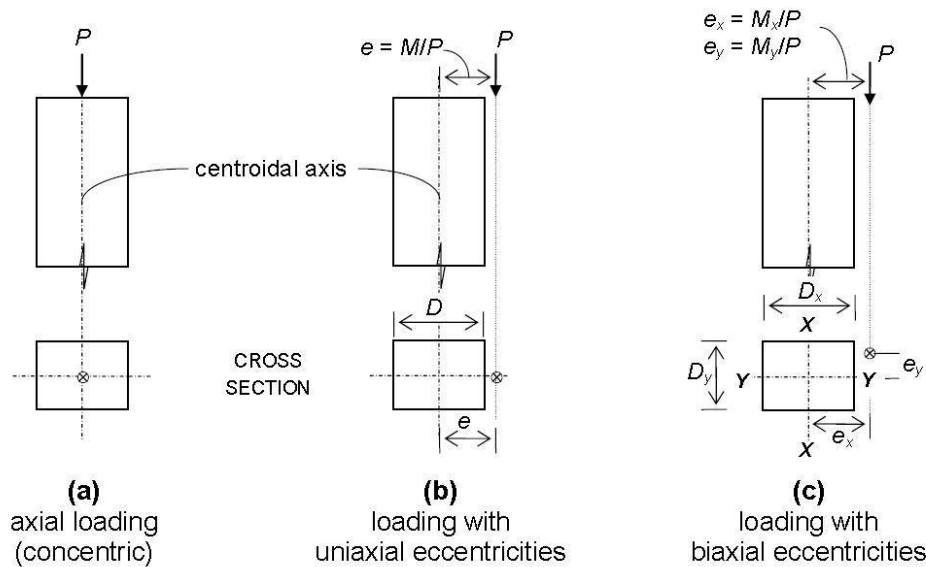
- Tied columns with ties as laterals
- columns with Spiral steel as laterals or spiral columns

Majority of columns in any buildings are tied columns. In a tied column the longitudinal bars are tied together with smaller bars at intervals up the column. Tied columns may be square, rectangular, L-shaped, circular, or any other required shape. Occasionally, when high strength and/or high ductility are required, the bars are placed in a circle, and the ties are replaced by a bar bent into a helix or spiral. Such a column, called a spiral column. Spiral columns are generally circular, although square or polygonal shapes are sometimes used. The spiral acts to restrain the lateral expansion of the column core under high axial loads and, in doing so, delays the failure of the core, making the column more ductile. Spiral columns are used more extensively in seismic regions. If properly designed, spiral column carries 5% extra load at failure compared to similar tied column.

4. Based on type of loading

- Axially loaded column or centrally or concentrically loaded column (P_u)

- A column subjected to axial load and uniaxial bending ($P_u + M_{ux}$) or ($P + M_{uy}$)
- A column subjected to axial load and biaxial bending ($P_u + M_{ux} + M_{uy}$)



Different loading situations in columns
 Fig 5.2 Types of Columns (Based on Loading)

5. Based on materials

Timber, stone, masonry, RCC, PSC, Steel, aluminium, composite column

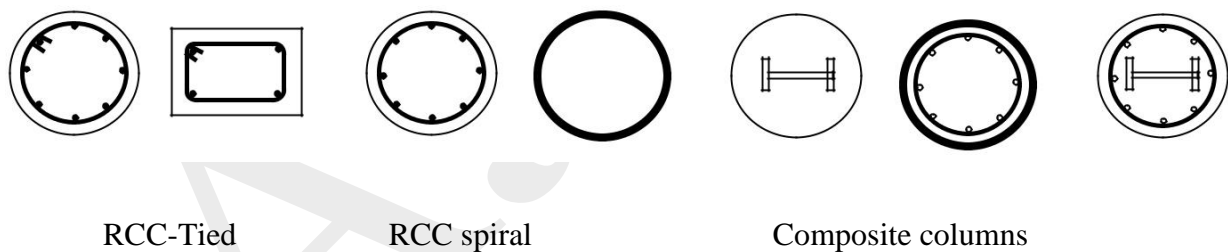


Fig 5.3 Types of Columns (Based on materials)

5.2 Behavior of Tied and Spiral Columns

Figure shows a portion of the core of a spiral column. Under a compressive load, the concrete in this column shortens longitudinally under the stress and so, to satisfy Poisson’s ratio, it expands laterally. In a spiral column, the lateral expansion of the concrete inside the spiral (referred to as the core) is restrained by the spiral. This stresses the spiral in tension. For equilibrium, the concrete is subjected to lateral compressive stresses. In a tied column in a non-seismic region, the ties are spaced roughly the width of the column apart and, as a result, provide relatively little lateral restraint to the core. Outward pressure on the sides of the ties

due to lateral expansion of the core merely bends them outward, developing an insignificant hoop-stress effect. Hence, normal ties have little effect on the strength of the core in a tied column. They do, however, act to reduce the unsupported length of the longitudinal bars, thus reducing the danger of buckling of those bars as the bar stress approaches yield. load-deflection diagrams for a tied column and a spiral column subjected to axial loads is shown in figure. The initial parts of these diagrams are similar. As the maximum load is reached, vertical cracks and crushing develop in the concrete shell outside the ties or spiral, and this concrete spalls off. When this occurs in a tied column, the capacity of the core that remains is less than the load on the column. The concrete core is crushed, and the reinforcement buckles outward between ties. This occurs suddenly, without warning, in a brittle manner. When the shell spalls off a spiral column, the column does not fail immediately because the strength of the core has been enhanced by the triaxial stresses resulting from the effect of the spiral reinforcement. As a result, the column can undergo large deformations, eventually reaching a second maximum load, when the spirals yield and the column finally collapses. Such a failure is much more ductile than that of a tied column and gives warning of the impending failure, along with possible load redistribution to other members. Due to this, spiral column carry little more load than the tied column to an extent of about 5%. Spiral columns are used when ductility is important or where high loads make it economical to utilize the extra strength. Both columns are in the same building and have undergone the same deformations. The tied column has failed completely, while the spiral column, although badly damaged, is still supporting a load. The very minimal ties were inadequate to confine the core concrete. Had the column ties been detailed according to ACI Code, the column will perform better as shown.

5.3 Specifications for covers and reinforcement in column

For a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used. For footings minimum cover shall be 50 mm.

Nominal Cover in mm to meet durability requirements based on exposure

Mild 20, Moderate 30, Severe 45, Very severe 50, Extreme 75

Nominal cover to meet specified period of fire resistance for all fire rating 0.5 to 4 hours is 40 mm for columns only

5.4 Effective length of compression member

Column or strut is a compression member, the effective length of which exceeds three times the least lateral dimension. For normal usage assuming idealized conditions, the effective length of in a given plane may be assessed on the basis of Table 28 of IS: 456-2000. Following terms are required.

Following are the end restraints:

- Effectively held in position and restrained against rotation in both ends
- Effectively held in position at both ends, restrained against rotation at one end
- Effectively held in position at both ends, but not restrained against rotation
- Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position
- Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position
- Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position
- Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end

5.5 Unsupported Length

The unsupported length, l , of a compression member shall be taken as the clear distance between end restraints (visible height of column). Exception to this is for flat slab construction, beam and slab construction, and columns restrained laterally by struts (Ref. IS:456-2000),

5.6 Slenderness Limits for Columns

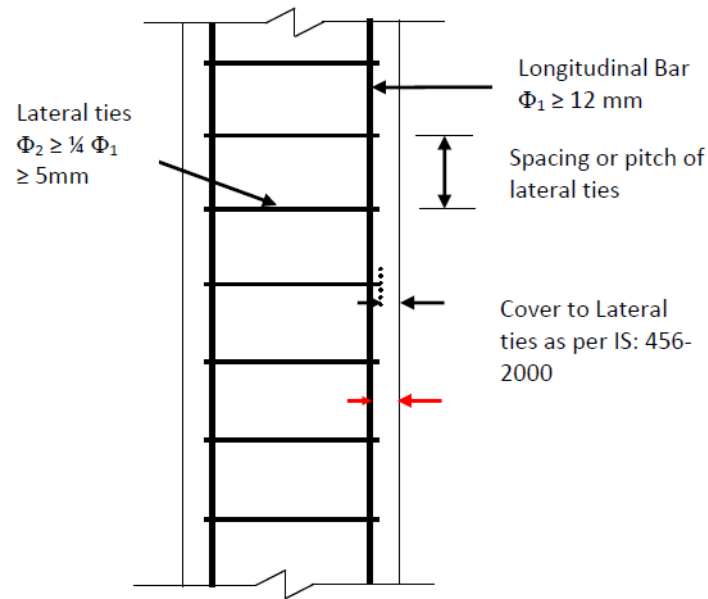
The unsupported length between end restraints shall not exceed 60 times the least lateral dimension of a column.

If in any given plane, one end of a column is unrestrained, its unsupported length, l , shall not exceed $100b^2/D$, where b = width of that cross-section, and D = depth of the cross-section measured in the plane under consideration.

5.7 Specifications as per IS: 456-2000

Longitudinal reinforcement

1. The cross-sectional area of longitudinal reinforcement, shall be not less than 0.8 percent nor more than 6 percent of the gross cross-sectional area of the column.
2. NOTE - The use of 6 percent reinforcement may involve practical difficulties in placing and compacting of concrete; hence lower percentage is recommended. Where bars from the columns below have to be lapped with those in the column under consideration, the percentage of steel shall usually not exceed 4 percent.
3. In any column that has a larger cross-sectional area than that required to support the load, the minimum percentage of steel shall be based upon the area of concrete required to resist the direct stress and not upon the actual area.
4. The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular columns.
5. The bars shall not be less than 12 mm in diameter
6. A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement.
7. In a helically reinforced column, the longitudinal bars shall be in contact with the helical reinforcement and equidistant around its inner circumference.
8. Spacing of longitudinal bars measured along the periphery of the column shall not exceed 300 mm.
9. In case of pedestals in which the longitudinal reinforcement is not taken in account in strength calculations, nominal longitudinal reinforcement not less than 0.15 percent of the cross-sectional area shall be provided.



5.8 Transverse reinforcement

A reinforced concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling.

The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135° . The ends of the transverse reinforcement shall be properly anchored.

5.9 Arrangement of transverse reinforcement

If the longitudinal bars are not spaced more than 75 mm on either side, transverse reinforcement need only to go round corner and alternate bars for the purpose of providing effective lateral supports (Ref. IS:456).

If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between these bars need to be tied in one direction by open ties (Ref. IS:456).

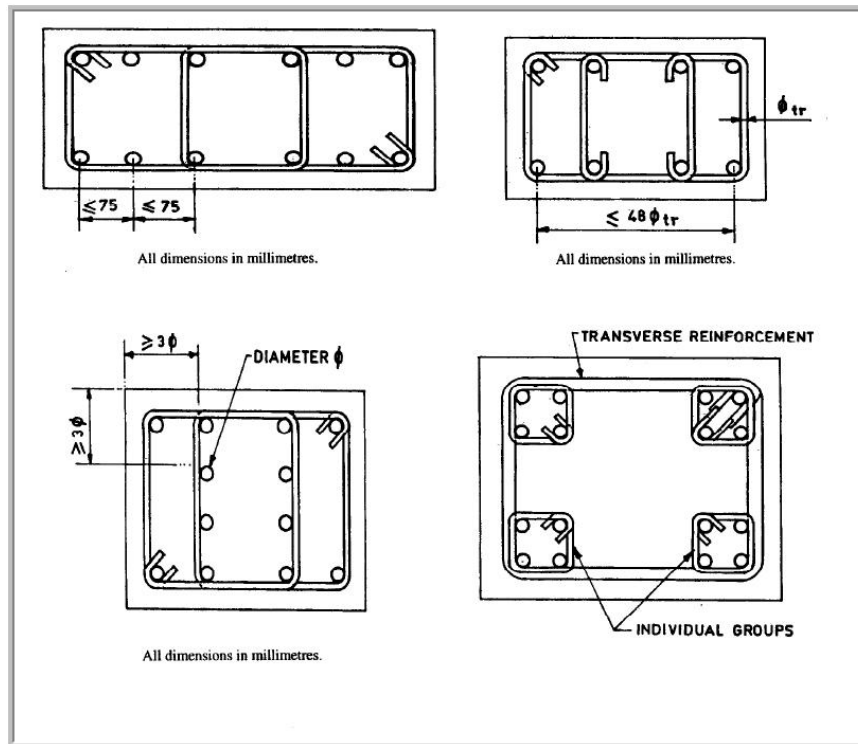


Fig 5.4 Arrangement of transverse reinforcement

5.10 Pitch and diameter of lateral ties

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.

5.11 Helical reinforcement

1) 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 7.5 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.

5.12 LIMIT STATE OF COLLAPSE: COMPRESSION

5.12.1 Assumptions

1. The maximum compressive strain in concrete in axial compression is taken as 0.002.
2. The maximum compressive strain at the highly compressed extreme fibre 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 fibre.

In addition, the following assumptions of flexure are also required

3. Plane sections normal to the axis remain plane after bending.
4. The maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.
5. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape which results in prediction of strength in substantial agreement with the results of test.
6. An acceptable stress strain curve is given in IS:456-2000. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γ of 1.5 shall be applied in addition to this.
7. The tensile strength of the concrete is ignored.
8. The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in IS:456-2000. For design purposes the partial safety factor equals to 1.15 shall be applied.

5.12.2 Minimum eccentricity

As per IS:456-2000, all columns shall be designed for minimum eccentricity, equal to the unsupported length of column/ 500 plus lateral dimensions/30, subject to a minimum of 20 mm, where bi-axial bending is considered, it is sufficient to ensure that eccentricity exceeds the minimum about one axis at a time.

5.13 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:

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

P_u = 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_s = area of longitudinal reinforcement for columns.

5.14 Compression Members with Helical Reinforcement

The strength of compression members with helical reinforcement satisfying the requirement of IS: 456 shall be taken as 1.05 times the strength of similar member with lateral ties.

The ratio of the volume of helical reinforcement to the volume of the core shall not be less than

$$V_{hs} / V_c > 0.36 (A_g/A_c - 1) f_{ck}/f_y$$

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.

5.15 Members Subjected to Combined Axial Load and Uni-axial Bending

Use of Non-dimensional Interaction Diagrams as Design Aids

Design Charts (for Uniaxial Eccentric Compression) in SP-16

The design Charts (non-dimensional interaction curves) given in the Design Handbook, SP: 16 cover the following three cases of symmetrically arranged reinforcement:

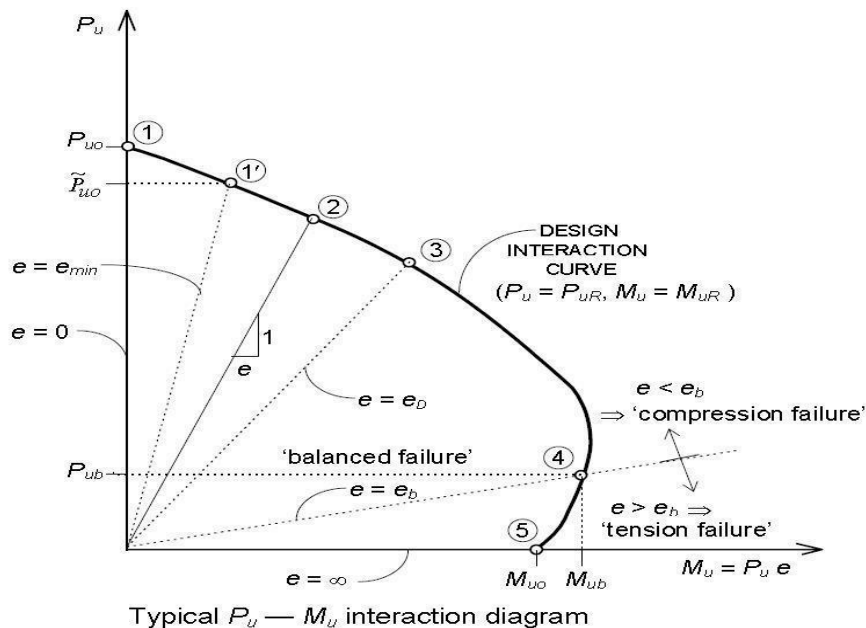
- (a) Rectangular sections with reinforcement distributed equally on two sides (Charts 27 – 38): the ‘two sides’ refer to the sides parallel to the axis of bending; there are no inner rows of bars, and each outer row has an area of $0.5A_s$ this includes the simple 4-bar configuration.

- (b) Rectangular sections with reinforcement distributed equally on four sides (Charts 39 – 50): two outer rows (with area $0.3A_s$ each) and four inner rows (with area $0.1A_s$ each) have been considered in the calculations; however, the use of these Charts can be extended, without significant error, to cases of not less than two inner rows (with a minimum area $0.3A_s$ in each outer row).
- (c) Circular column sections (Charts 51 – 62): The Charts are applicable for circular sections with at least six bars (of equal diameter) uniformly spaced circumferentially.

Corresponding to each of the above three cases, there are as many as 12 Charts available covering the 3 grades of steel (Fe 250, Fe 415, Fe 500), with 4 values of d'/D ratio for each grade (namely 0.05, .0.10, 0.15, 0.20). For intermediate values of d'/D , linear interpolation may be done. Each of the 12 Charts of SP-16 covers a family of non-dimensional design interaction curves with p/f_{ck} values ranging from 0.0 to 0.26.

From this, percentage of steel (p) can be found. Find the area of steel and provide the required number of bars with proper arrangement of steel as shown in the chart.

Typical interaction curve



Salient Points on the Interaction Curve

The salient points, marked 1 to 5 on the interaction curve correspond to the failure strain profiles, marked 1 to 5 in the above figure.

- The point 1 in figure corresponds to the condition of axial loading with $e = 0$. For this case of ‘pure’ axial compression.
- The point 1¹ in figure corresponds to the condition of axial loading with the mandatory minimum eccentricity e_{\min} prescribed by the Code.
- The point 3 in figure corresponds to the condition $x_u = D$, i.e., $e = e_D$. For $e < e_D$, the entire section is under compression and the neutral axis is located outside the section ($x_u > D$), with $0.002 < \varepsilon_{cu} < 0.0035$. For $e > e_D$, the NA is located within the section ($x_u < D$) and $\varepsilon_{cu} = 0.0035$ at the ‘highly compressed edge’.
- The point 4 in figure corresponds to the balanced failure condition, with $e = e_b$ and $x_u = x_{u,b}$. The design strength values for this ‘balanced failure’ condition are denoted as P_{ub} and M_{ub} .
- The point 5 in figure corresponds to a ‘pure’ bending condition ($e = \infty$, $P_u R = 0$); the resulting ultimate moment of resistance is denoted M_{u0} and the corresponding NA depth takes on a minimum value $x_{u, \min}$.

5.16 Procedure for using of Non-dimensional Interaction Diagrams as Design Aids to find steel

Given:

Size of column, Grade of concrete, Grade of steel (otherwise assume suitably) Factored load and Factored moment

Assume arrangement of reinforcement: On two sides or on four sides

Assume moment due to minimum eccentricity to be less than the actual moment

Assume suitable axis of bending based on the given moment (xx or yy)

Assuming suitable diameter of longitudinal bars and suitable nominal cover

1. Find d^1/D from effective cover d^1
2. Find non dimensional parameters $P_u/f_{ck}bD$ and $M_u/f_{ck}bD^2$
3. Referring to appropriate chart from S-16, find p/f_{ck} and hence the percentage of reinforcement, p
4. Find steel from, $A_s = p bD/100$
5. Provide proper number and arrangement for steel
6. Design suitable transverse steel
7. Provide neat sketch

5.17 Members Subjected to Combined Axial Load and Biaxial Bending

The resistance of a member subjected to axial force and biaxial bending shall be obtained on the basis of assumptions given in IS:456 with neutral axis so chosen as to satisfy the equilibrium of load and moments about two axes. Alternatively, such members may be designed by the following equation:

$$[M_{ux}/M_{ux1}]^{\alpha_n} + [M_{uy}/M_{uy1}]^{\alpha_n} \leq 1, \text{ where}$$

M_{ux} and M_y = moments about x and y axes due to design loads,

M_{ux1} and M_{y1} = maximum uni-axial moment capacity for an axial load of P_u bending about x and y axes respectively, and α_n is related to P_u/P_{uz} , where $P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}$

For values of $P_u/P_{uz} = 0.2$ to 0.8 , the values of α_n vary linearly from 1.0 to 2.0 . For values less than 0.2 and greater than 0.8 , it is taken as 1 and 2 respectively

NOTE -The design of member subject to combined axial load and uniaxial bending will involve lengthy calculation by trial and error. In order to overcome these difficulties interaction diagrams may be used. These have been prepared and published by BIS in SP:16 titled Design aids for reinforced concrete to IS 456-2000.

5.18 IS:456-2000 Code Procedure

1. Given P_u , M_{ux} , M_{uy} , grade of concrete and steel
2. Verify that the eccentricities $e_x = M_{ux}/P_u$ and $e_y = M_{uy}/P_u$ are not less than the corresponding minimum eccentricities as per IS:456-2000
3. Assume a trial section for the column (square, rectangle or circular).
4. Determine M_{ux1} and M_{uy1} , corresponding to the given P_u (using appropriate curve from SP-16 design aids)
5. Ensure that M_{ux1} and M_{uy1} are significantly greater than M_{ux} and M_{uy} respectively; otherwise, suitably redesign the section.
6. Determine P_{uz} and hence α_n
7. Check the adequacy of the section using interaction equation. If necessary, redesign the section and check again.

Slender Compression Members: The design of slender compression members shall be based on the forces and the moments determined from an analysis of the structure, including

the effect of deflections on moments and forces. When the effects of deflections are not taken into account in the analysis, additional moment given in 39.7.1 shall be taken into account in the appropriate direction.

Problems

- Determine the load carrying capacity of a column of size 300 x 400 mm reinforced with six rods of 20 mm diameter i.e, 6-#20. The grade of concrete and steel are M20 and Fe 415 respectively. Assume that the column is short.**

$$f_{ck} = 20 \text{ MPa}, f_y = 415 \text{ MPa}$$

$$\text{Area of steel } A_{SC} = 6 \times \pi \times 20^2/4 = 6 \times 314 = 1884 \text{ mm}^2$$

$$\text{Percentage of steel} = 100A_{sc}/bD = 100 \times 1884/300 \times 400 = 1.57 \%$$

$$\text{Area of concrete } A_c = A_g - A_{sc} = 300 \times 400 - 1884 = 118116 \text{ mm}^2$$

Ultimate load carried by the column

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

$$0.4 \times 20 \times 118116 + 0.67 \times 415 \times 1884 = 944928 + 523846 = 1468774 \text{ N} = 1468.8 \text{ kN}$$

$$\text{Therefore, the safe load on the column} = 1468.8 / 1.5 = 979.2 \text{ kN}$$

- Determine the steel required to carry a load of 980kN on a rectangular column of size 300 x 400 mm. The grade of concrete and steel are M20 and Fe 415 respectively. Assume that the column is short.**

$$f_{ck} = 20 \text{ MPa}, f_y = 415 \text{ MPa}, P = 980 \text{ kN}$$

$$\text{Area of steel } A_{SC} = ?$$

$$\text{Area of concrete } A_c = A_g - A_{sc} = (300 \times 400 - A_{sc})$$

Ultimate load carried by the column

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

$$1.5 \times 980 \times 1000 = 0.4 \times 20 \times (300 \times 400 - A_{sc}) + 0.67 \times 415$$

$$1470000 = 960000 - 8 A_{sc} + 278.06 A_{sc}$$

$$A_{sc} = 1888.5 \text{ mm}^2$$

Percentage of steel = $100A_{sc}/bD = 100 \times 1888.5 / 300 \times 400 = 1.57\%$ which is more than 0.8% and less than 6% and therefore ok.

Use 20 mm dia. bars, No. of bars = $1888.5/314 = 6.01$ say 6

3. Design a square or circular column to carry a working load of 980kN. The grade of concrete and steel are M20 and Fe 415 respectively. Assume that the column is short.

Let us assume 1.0% steel (1 to 2%)

Say $A_{sc} = 1.0\% A_g = 1/100 A_g = 0.01A_g$

$f_{ck} = 20$ MPa, $f_y = 415$ MPa, $P = 980$ kN

Area of concrete $A_c = A_g - A_{sc} = A_g - 0.01A_g = 0.99 A_g$

Ultimate load carried by the column

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

$980 \times 1.5 \times 1000 = 0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g$

$1470000 = 7.92 A_g + 2.78 A_g$

$A_g = 137383 \text{ mm}^2$

Let us design a square column:

$B = D = \sqrt{A_g} = 370.6 \text{ mm}$ say 375 x 375 mm

This is ok. However, this size cannot take the minimum eccentricity of 20 mm as $e_{min}/D = 20/375 = 0.053 > 0.05$. To restrict the eccentricity to 20 mm, the required size is 400x400 mm.

Area of steel required is $A_{sc} = 1373.8 \text{ mm}^2$. Provide 4 bar of 22 mm diameter. Steel provided is $380 \times 4 = 1520 \text{ mm}^2$

Actual percentage of steel = $100A_{sc}/bD = 100 \times 1520 / 400 \times 400 = 0.95\%$ which is more than 0.8% and less than 6% and therefore ok.

Design of Transverse steel:

Diameter of tie = $1/4$ diameter of main steel = $22/4 = 5.5$ mm or 6 mm, whichever is greater.

Provide 6 mm.

Spacing: $< 300 \text{ mm}$, $< 16 \times 22 = 352 \text{ mm}$, $< \text{LLD} = 400 \text{ mm}$. Say 300 mm c/c

Design of circular column:

Here $A_g = 137383 \text{ mm}^2$

$\pi \times D^2/4 = A_g$, $D = 418.2 \text{ mm}$ say 420 mm . This satisfies the minimum eccentricity of $20m$. Also provide 7 bars of 16 mm , $7 \times 201 = 1407 \text{ mm}^2$

Design of Transverse steel:

Dia of tie = $\frac{1}{4}$ dia of main steel = $16/4 = 4 \text{ mm}$ or 6 mm , whichever is greater. Provide 6 mm .

Spacing: $< 300 \text{ mm}$, $< 16 \times 16 = 256 \text{ mm}$, $< \text{LLD} = 420 \text{ mm}$. Say 250 mm c/c

4. Design a rectangular column to carry an ultimate load of 2500kN. The unsupported length of the column is 3m. The ends of the column are effectively held in position and also restrained against rotation. The grade of concrete and steel are M20 and Fe415 respectively.

Given:

$f_{ck} = 20 \text{ MPa}$, $f_y = 415 \text{ MPa}$, $P_u = 2500 \text{ kN}$

Area of concrete $A_c = A_g - A_{sc} = A_g - 0.01A_g = 0.99 A_g$

Ultimate load carried by the column

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

$2500 \times 1000 = 0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g$

$2500000 = 7.92 A_g + 2.78 A_g$

$A_g = 233645 \text{ mm}^2$

If it is a square column:

$B = D = \sqrt{A_g} = 483 \text{ mm}$. However, provide rectangular column of size $425 \times 550 \text{ mm}$.
The area provided = 233750 mm^2

Area of steel $A_{sc} = 2336 \text{ mm}^2$, also provide 8 bars of 20 mm , $6 \times 314 = 2512 \text{ mm}^2$

Check for shortness: Ends are fixed. $l_{ex} = l_{ey} = 0.65 l = 0.65 \times 3000 = 1950 \text{ mm}$

$l_{ex}/D = 1950/550 < 12$, and $l_{ey}/b = 1950/425 < 12$, Column is short

Check for minimum eccentricity:

In the direction of longer direction

$e_{\min, x} = l_{ux}/500 + D/30 = 3000/500 + 550/30 = 24.22\text{mm}$ or 20mm whichever is greater. $e_{\min, x} = 24.22 \text{ mm} < 0.05D = 0.05 \times 550 = 27.5 \text{ mm}$. O.K

In the direction of shorter direction

$e_{\min, y} = l_{uy}/500 + b/30 = 3000/500 + 425/30 = 20.17 \text{ mm}$ or 20mm whichever is greater.
 $e_{\min, y} = 20.17 \text{ mm} < 0.05b = 0.05 \times 425 = 21.25 \text{ mm}$. O.K

Design of Transverse steel:

Dia of tie = $\frac{1}{4}$ dia of main steel = $20/4 = 5 \text{ mm}$ or 6 mm, whichever is greater. Provide 6 mm or 8 mm.

Spacing: $< 300 \text{ mm}$, $< 16 \times 20 = 320 \text{ mm}$, $< \text{LLD} = 425\text{mm}$. Say 300 mm c/c

5. Design a circular column with ties to carry an ultimate load of 2500kN. The unsupported length of the column is 3m. The ends of the column are effectively held in position but not against rotation. The grade of concrete and steel are M20 and Fe 415 respectively.

Given:

$f_{ck} = 20 \text{ MPa}$, $f_y = 415 \text{ MPa}$, $P_u = 2500\text{kN}$

Let us assume 1.0% steel (1 to 2%)

Say $A_{sc} = 1.0\% A_g = 1/100 A_g = 0.01A_g$

Area of concrete $A_c = A_g - A_{sc} = A_g - 0.01A_g = 0.99 A_g$

Ultimate load carried by the column

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

$2500 \times 1000 = 0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g = 7.92 A_g + 2.78 A_g = 10.7 A_g$

$A_g = 233645 \text{ mm}^2$

$\pi \times D^2/4 = A_g$, $D = 55.4 \text{ mm}$ say 550 mm.

Area of steel $A_{sc} = 2336 \text{ mm}^2$, also provide 8 bars of 20 mm, $8 \times 314 = 2512 \text{ mm}^2$

Check for shortness: Ends are hinged $l_{ex} = l_{ey} = l = 3000 \text{ mm}$

$l_{ex}/D = 3000/550 < 12$, and $l_{ey}/b = 3000/425 < 12$, Column is short

Check for minimum eccentricity:

Here, $e_{min, x} = e_{min, y} = l_{ux}/500 + D/30 = 3000/500 + 550/30 = 24.22\text{mm}$ or 20mm whichever is greater.

$e_{min} = 24.22 \text{ mm} < 0.05D = 0.05 \times 550 = 27.5 \text{ mm}$. O.K

Design of Transverse steel:

Diameter of tie = $\frac{1}{4}$ dia of main steel = $20/4 = 5 \text{ mm}$ or 6 mm, whichever is greater. Provide 6 mm or 8 mm.

Spacing: $< 300 \text{ mm}$, $< 16 \times 20 = 320 \text{ mm}$, $< \text{LLD} = 550\text{mm}$. Say 300 mm c/c

Similarly, square column can be designed.

If the size of the column provided is less than that provided above, then the minimum eccentricity criteria are not satisfied. Then e_{min} is more and the column is to be designed as uni axial bending case or bi axial bending case as the case may be. This situation arises when more steel is provided (say 2% in this case).

Try to solve these problems by using SP 16 charts, though not mentioned in the syllabus.

6. Design the reinforcement in a column of size 450 mm × 600 mm, subject to an axial load of 2000 kN under service dead and live loads. The column has an unsupported length of 3.0m and its ends are held in position but not in direction. Use M 20 concrete and Fe 415 steel.

Solution:

Given: $l_u = 3000 \text{ mm}$, $b = 450 \text{ mm}$, $D = 600 \text{ mm}$, $P = 2000\text{kN}$, M20, Fe415

Check for shortness: Ends are fixed. $l_{ex} = l_{ey} = l = 3000 \text{ mm}$

$l_{ex}/D = 3000/600 < 12$, and $l_{ey}/b = 3000/450 < 12$, Column is short

Check for minimum eccentricity:

In the direction of longer direction

$e_{\min, x} = l_{ux}/500 + D/30 = 3000/500 + 600/30 = 26 \text{ mm}$ or 20mm whichever is greater. $e_{\min, x} = 26 \text{ mm} < 0.05D = 0.05 \times 600 = 30 \text{ mm}$. O.K

In the direction of shorter direction

$e_{\min, y} = l_{uy}/500 + b/30 = 3000/500 + 450/30 = 21 \text{ mm}$ or 20mm whichever is greater.
 $e_{\min, x} = 21 \text{ mm} < 0.05b = 0.05 \times 450 = 22.5 \text{ mm}$. O.K

Minimum eccentricities are within the limits and hence code formula for axially loaded short columns can be used.

Factored Load

$P_u = \text{service load} \times \text{partial load factor}$

$$= 2000 \times 1.5 = 3000 \text{ kN}$$

Design of Longitudinal Reinforcement

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \quad \text{or}$$

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

$$3000 \times 10^3 = 0.4 \times 20 \times (450 \times 600) + (0.67 \times 415 - 0.4 \times 20) A_{sc}$$

$$= 2160 \times 10^3 + 270.05 A_{sc}$$

$$\Rightarrow A_{sc} = (3000 - 2160) \times 10^3 / 270.05 = 3111 \text{ mm}^2$$

In view of the column dimensions (450 mm, 600 mm), it is necessary to place intermediate bars, in addition to the 4 corner bars:

Provide 4–25 ϕ at corners ie, $4 \times 491 = 1964 \text{ mm}^2$ and 4–20 ϕ additional ie, $4 \times 314 = 1256 \text{ mm}^2$

$$\Rightarrow A_{sc} = 3220 \text{ mm}^2 > 3111 \text{ mm}^2$$

$$\Rightarrow p = (100 \times 3220) / (450 \times 600) = 1.192 > 0.8 \text{ (minimum steel), OK.}$$

Design of transverse steel

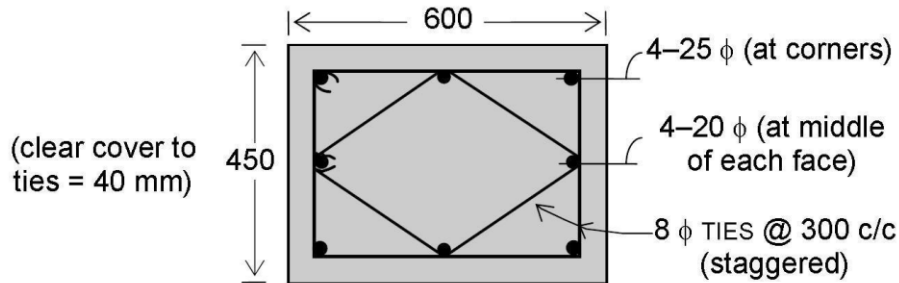
Diameter of tie = $\frac{1}{4}$ diameter of main steel = $25/4 = 6.25 \text{ mm}$ or 6 mm, whichever is greater.

Provide 6 mm.

Spacing: $< 300 \text{ mm}$, $< 16 \times 20 = 320 \text{ mm}$, $< \text{LLD} = 450 \text{ mm}$. Say 300 mm c/c

Thus provide ties $8 \text{ mm @ } 300 \text{ mm c/c}$

Sketch:



Example: Square Column with Uniaxial Bending

7. Determine the reinforcement to be provided in a square column subjected to uniaxial bending with the following data:

Size of column $450 \times 450 \text{ mm}$

Concrete mix M 25

Characteristic strength of steel 415 N/mm^2

Factored load 2500 kN

Factored moment 200 kN.m

Arrangement of reinforcement:

(a) On two sides

(b) On four sides

Assume moment due to minimum eccentricity to be less than the actual moment Assuming 25 mm bars with 40 mm cover,

$$d = 40 + 12.5 = 52.5 \text{ mm}$$

$$d^1/D = 52.5/450 = 0.12$$

Charts for $d^1/D = 0.15$ will be used

$$P_u/f_{ck}bD = (2500 \times 1000)/(25 \times 450 \times 450) = 0.494$$

$$M_u/f_{ck}bD^2 = 200 \times 10^6 / (25 \times 450 \times 450^2) = 0.088$$

a) Reinforcement on two sides,

Referring to Chart 33,

$$p/f_{ck} = 0.09$$

Percentage of reinforcement,

$$p = 0.09 \times 25 = 2.25 \%$$

$$A_s = p \, bD/100 = 2.25 \times 450 \times 450/100 = 4556 \text{ mm}^2$$

Reinforcement on four sides from Chart 45,

$$p/f_{ck} = 0.10$$

$$p = 0.10 \times 25 = 2.5 \%$$

$$A_s = 2.5 \times 450 \times 450/100 = 5063 \text{ mm}^2$$

8. Example: Circular Column with Uniaxial Bending

Determine the reinforcement to be provided in a circular column with the following data:

Diameter of column 500 mm

Grade of concrete M20

Characteristic strength 250 N/mm²

Factored load 1600 kN

Factored moment 125 kN.m

Lateral reinforcement:

(a) Hoop reinforcement

(b) Helical reinforcement

(Assume moment due to minimum eccentricity to be less than the actual moment).

Assuming 25 mm bars with 40 mm cover,

$$d^1 = 40 + 12.5 = 52.5 \text{ mm}$$

$$d^1/D = 52.5/50 = 0.105$$

Charts for $d^1/D = 0.10$ will be used.

(a) Column with hoop reinforcement

$$P_u/f_{ck} D D = (1600 \times 1000)/(20 \times 500 \times 500) = 0.32$$

$$M_u/f_{ck} D \times D^2 = 125 \times 10^6 / (20 \times 500 \times 500^2) = 0.05$$

Referring to Chart 52, for $f_y = 250 \text{ N/mm}^2$

$$p/f_{ck} = 0.87$$

Percentage of reinforcement,

$$p = 0.87 \times 20 = 1.74 \%$$

$$A_s = 1.74 \times (\pi \times 500^2/4)/100 = 3416 \text{ mm}^2$$

(b) *Column with Helical Reinforcement*

According to 38.4 of the Code, the strength of a compression member with helical reinforcement is 1.05 times the strength of a similar member with lateral ties. Therefore, the, given load and moment should be divided by 1.05 before referring to the chart.

$$P_u/f_{ck} D D = (1600/1.05 \times 1000)/(20 \times 500 \times 500) = 0.31$$

$$\begin{aligned} M_u/f_{ck} D \times D^2 &= 125/1.05 \times 10^6 / (20 \times 500 \times 500^2) \\ &= 0.048 \text{ Hence, From Chart 52, for } f_y = 250 \text{ N/mm}^2, p/f_{ck} \\ &= 0.078 \end{aligned}$$

$$p = 0.078 \times 20 = 1.56 \%$$

$$A_s = 1.56 \times (\pi \times 500 \times 500/4)/100 = 3063 \text{ cm}^2$$

According to 38.4.1 of the Code the ratio of the volume of helical reinforcement to the volume of the core shall not be less than

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

where A_g is the gross area of the section and A_c is the area of the core measured to the outside diameter of the helix. Assuming 8 mm dia bars for the helix,

$$\text{Core diameter} = 500 - 2(40 - 8) = 436 \text{ mm}$$

$$A_g/A_c = 500/436 = 1.315$$

$$0.36 (A_g/A_c - 1) \times f_{ck}/f_y = 0.36(0.315) 20/250 = 0.0091$$

Volume of helical reinforcement / Volume of core

$$= A_{sh} \pi \times 428 / (\pi/4 \times 436^2) s_h = 0.09 A_{sh} / s_h$$

where, A_{sh} is the area of the bar forming the helix and s_h is the pitch of the helix. In order to satisfy the codal requirement,

$$0.09 A_{sh} / s_h \geq 0.0091$$

For 8 mm dia bar,

$$s_h \leq 0.09 \times 50 / 0.0091 = 49.7 \text{ mm. Thus provide 48 mm pitch}$$

Example: Rectangular column with Biaxial Bending

9. Determine the reinforcement to be provided in a short column subjected to biaxial bending, with the following data: size of column = 400 x 600 mm

Concrete mix = M15

Characteristic strength of reinforcement = 415 N/mm²

Factored load, P_u = 1600 kN

Factored moment acting parallel to the larger dimension, M_{ux} = 120 kNm

Factored moment acting parallel to the shorter dimension, M_{uy} = 90 kNm

Moments due to minimum eccentricity are less than the values given above.

Reinforcement is distributed equally on four sides.

As a first trial assume the reinforcement percentage, $p = 1.2\%$

$$p/f_{ck} = 1.2/15 = 0.08$$

Uniaxial moment capacity of the section about xx-axis:

$$d^1/D = 52.5 / 600 = 0.088$$

Chart for $d^1/D = 0.1$ will be used.

$$P_u/f_{ck} b D = (1600 \times 1000) / (15 \times 400 \times 600) = 0.444$$

Referring to chart 44

$$M_u/f_{ck} b \times D^2 = 0.09$$

$$M_{ux1} = 0.09 \times 15 \times 400 \times 600^2) = 194.4 \text{ kN.m}$$

Uni-axial moment capacity of the section about yy axis:

$$d^1/D = 52.5 / 400 = 0.131$$

Chart for $d^1/D = 0.15$ will be used.

Referring to Chart 45,

$$M_u/f_{ck} b \times D^2 = 0.083$$

$$M_{ux1} = 0.083 \times 15 \times 600 \times 400^2) = 119.52 \text{ kN.m}$$

Calculation of P_{uz} :

Referring to Chart 63 corresponding to

$$p = 1.2, f_y = 415 \text{ and } f_{ck} = 15,$$

$$P_{uz}/A_g = 10.3$$

$$P_{uz} = 10.3 \times 400 \times 600 = 2472 \text{ kN}$$

$$M_{ux}/M_{ux1} = 120/194.4 = 0.62$$

$$M_{uy}/M_{uy1} = 90/119.52 = 0.75$$

$$P_u/P_{uz} = 1600/2472 = 0.65$$

Referring to Chart 64, the permissible value of M_{ux}/M_{ux1} corresponding to M_{uy}/M_{uy1} and P_u/P_{uz} is equal to 0.58

The actual value of 0.62 is only slightly higher than the value read from the Chart.

This can be made up by slight increase in reinforcement.

Using Boris load contour equation as per IS:456-2000

$$P_u/P_{uz} = 0.65 \text{ thus, } \alpha_n = 1 + [(2-1) / (0.8 - 0.2)] (0.65-0.2) = 1.75$$

$[0.62]^{1.75} + [0.75]^{1.75} = 1.04$ slightly greater than 1 and slightly unsafe. This can be made up by slight increase in reinforcement say 1.3%

$$\text{Thus, provide } A_s = 1.3 \times 400 \times 600 / 100 = 3120 \text{ mm}^2$$

Provide 1.3 % of steel

$$p/f_{ck} = 1.3/15 = 0.086$$

$$d^1/D = 52.5 / 600 = 0.088 = 0.1$$

From chart 44

$$M_u/f_{ck} b \times D^2 = 0.095$$

$$M_{ux1} = 0.095 \times 15 \times 400 \times 600^2 = 205.2 \text{ kN.m}$$

Referring to Chart 45,

$$M_u/f_{ck} b \times D^2 = 0.085$$

$$M_{ux1} = 0.085 \times 15 \times 600 \times 400^2 = 122.4 \text{ kN.m}$$

Chart 63: $P_{uz}/A_g = 10.4$

$$P_{uz} = 10.4 \times 400 \times 600 = 2496 \text{ kN}$$

$$M_{ux}/M_{ux1} = 120/205.2 = 0.585$$

$$M_{uy}/M_{uy1} = 90/122.4 = 0.735$$

$$P_u / P_{uz} = 1600 / 2496 = 0.641$$

Referring to Chart 64, the permissible value of M_{ux}/M_{ux1} corresponding to M_{uy}/M_{uy1} and P_u / P_{uz} is equal to 0.60

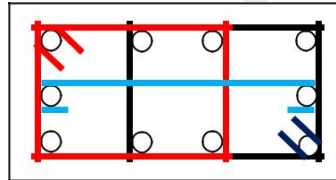
Hence the section is O.K.

Using Boris load contour equation as per IS:456-2000

$$P_u / P_{uz} = 0.641 \text{ thus, } \alpha_n = 1 + [(2-1) / (0.8 - 0.2)] (0.641 - 0.2) = 1.735$$

$$[120/205.2]^{1.735} + [90/122.4]^{1.735} = 0.981 \leq 1 \text{ Thus OK}$$

$A_s = 3120 \text{ mm}^2$. Provide 10 bars of 20 mm dia. Steel provided is $314 \times 10 = 3140 \text{ mm}^2$
Design of transverse steel: Provide 8 mm dia stirrups at 300 mm c/c as shown satisfying the requirements of IS: 456-2000



10. Verify the adequacy of the short column section 500 mm x 300 mm under the following load conditions: $P_u = 1400 \text{ kN}$, $M_{ux} = 125 \text{ kNm}$, $M_{uy} = 75 \text{ kNm}$. The design interaction curves of SP 16 should be used. Assume that the column is a ‘short column’ and the eccentricity due to moments is greater than the minimum eccentricity.

Solution:

Given: $D_x = 500 \text{ mm}$, $b = 300 \text{ mm}$, $A_s = 2946 \text{ mm}^2$, $M_{ux} = 125 \text{ kNm}$, $M_{uy} = 75 \text{ kNm}$, $f_{ck} = 25 \text{ MPa}$, $f_y = 415 \text{ MPa}$

Applied eccentricities

$$e_x = M_{ux} / P_u = 125 \times 10^3 / 1400 = 89.3 \text{ mm} \Rightarrow e_x / D_x = 0.179$$

$$e_y = M_{uy} / P_u = 75 \times 10^3 / 1400 = 53.6 \text{ mm} \Rightarrow e_y / D_y = 0.179$$

These eccentricities for the short column are clearly not less than the minimum eccentricities specified by the Code.

Uniaxial moment capacities: M_{ux1} , M_{uy1}

As determined in the earlier example, corresponding to $P_u = 1400 \text{ kN}$,

$$M_{ux1} = 187 \text{ kNm}$$

$$M_{uy1} = 110 \text{ kNm}$$

Values of P_{uz} and α_n

$$P_{uz} = 0.45f_{ck} A_g + (0.75f_y - 0.45f_{ck}) A_{sc}$$

$$= (0.45 \times 25 \times 300 \times 500) + (0.75 \times 415 - 0.45 \times 25) \times 2946$$

$$= (1687500 + 883800) \text{ N} = 2571 \text{ kN}$$

$$\Rightarrow P_u/P_{uz} = 1400/2571 = 0.545 \text{ (which lies between 0.2 and 0.8)}$$

$$\Rightarrow \alpha_n = 1.575$$

Check safety under biaxial bending

$$[125/187]^{1.575} + [75/110]^1$$

$$= 0.530 + 0.547$$

$$= 1.077 > 1.0$$

Hence, almost ok.

Design of Isolated R.C. Footings

1. General

Most of the structures built by us are made of reinforced concrete. Here, the part of the structure above ground level is called as the superstructure, where the part of the structure below the ground level is called as the substructure. Footings are located below the ground level and are also referred as foundations. Foundation is that part of the structure which is in direct contact with soil. The R.C. structures consist of various structural components which act together to resist the applied loads and transfer them safely to soil. In general, the loads applied on slabs in buildings are transferred to soil through beams, columns and footings. Footings are that part of the structure which are generally located below ground Level. They are also referred as foundations. Footings transfer the vertical loads, Horizontal loads, Moments, and other forces to the soil.

The important purpose of foundation are as follows;

1. To transfer forces from superstructure to firm soil below.
2. To distribute stresses evenly on foundation soil such that foundation soil neither fails nor experiences excessive settlement.
3. To develop an anchor for stability against overturning.
4. To provide an even surface for smooth construction of superstructure.

Due to the loads and soil pressure, footings develop Bending moments and Shear forces.

Calculations are made as per the guidelines suggested in IS 456 2000 to resist the internal forces.

2. Types of Foundations

Based on the position with respect to ground level, Footings are classified into two types;

1. Shallow Foundations

2. Deep Foundations

Shallow Foundations are provided when adequate SBC is available at relatively short depth below ground level. Here, the ratio of $D_f / B < 1$, where D_f is the depth of footing and B is the width of footing. Deep Foundations are provided when adequate SBC is available at large depth below ground level. Here the ratio of $D_f / B \geq 1$.

2.1 Types of Shallow Foundations

The different types of shallow foundations are as follows:

1. Isolated Footing
2. Combined footing
3. Strap Footing
4. Strip Footing
5. Mat/Raft Foundation
6. Wall footing

Some of the popular types of shallow foundations are briefly discussed below.

1. Isolated Column Footing

These are independent footings which are provided for each column. This type of footing is chosen when

1. SBC is generally high
2. Columns are far apart
3. Loads on footings are less

The isolated footings can have different shapes in plan. Generally, it depends on the shape of column cross section. Some of the popular shapes of footings are;

- Square
- Rectangular
- Circular

The isolated footing essentially consists of bottom slab. These bottom Slabs can be either flat, stepped or sloping in nature. The bottom of the slab is reinforced with steel mesh to resist the two internal forces namely bending moment and shear force.

The sketch of a typical isolated footing is shown in Fig. 1.

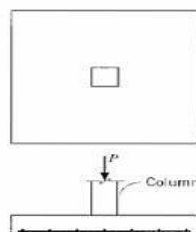


Fig. 5.5 Plan and section of typical isolated footing

2. Combined Column Footing

These are common footings which support the loads from 2 or more columns. Combined footings are provided when

1. SBC is generally less
2. Columns are closely spaced
3. Footings are heavily loaded

In the above situations, the area required to provide isolated footings for the columns generally overlap. Hence, it is advantageous to provide single combined footing. In some cases, the columns are located on or close to property line. In such cases footings cannot be extended on one side. Here, the footings of exterior and interior columns are connected by the combined footing.

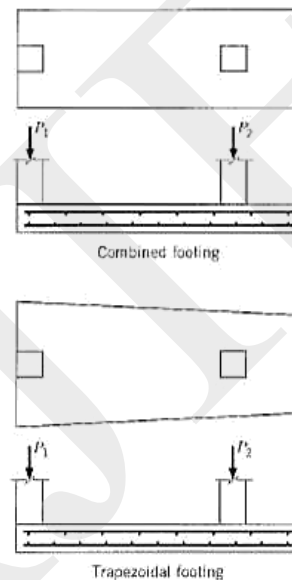


Fig. 5.6 Plan and section of typical combined footing

Combined footings essentially consist of a common slab for the columns it is supporting. These slabs are generally rectangular in plan. Sometimes they can also be trapezoidal in plan (refer Fig. 2). Combined footings can also have a connecting beam and a slab arrangement, which is similar to an inverted T – beam slab.

3. Strap Footing

An alternate way of providing combined footing located close to property line is the strap footing. In strap footing, independent slabs below columns are provided which are then connected by a strap beam. The strap beam does not remain in contact with the soil and does

not transfer any pressure to the soil. Generally, it is used to combine the footing of the outer column to the adjacent one so that the footing does not extend in the adjoining property. A typical strap footing is shown in Fig. 3.

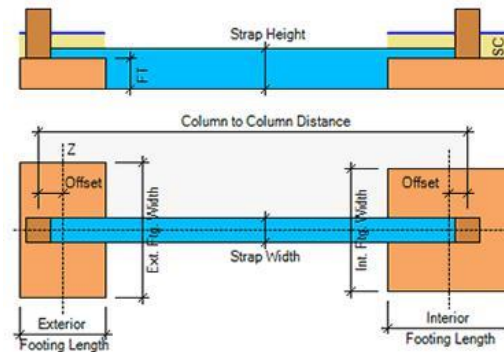


Fig. 5.8 Plan and section of typical strap footing

4. Strip Footing

Strip footing is a continuous footing provided under columns or walls. A typical strip footing for columns is shown in Fig. 4.

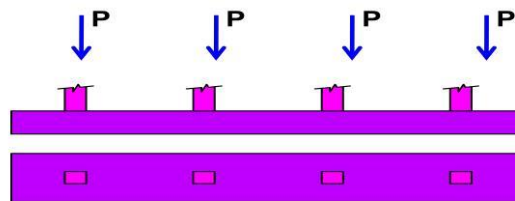


Fig. 5.9 Plan and section of typical strip footing

5. Mat Foundation

Mat foundation covers the whole plan area of structure. The detailing is similar to two way reinforced solid floor slabs or flat slabs. It is a combined footing that covers the entire area beneath structure and supports all the walls and columns. It is normally provided when

Soil pressure is low

Loads are very heavy

Spread footings cover > 50% area

A typical mat foundation is shown in Fig. 5.

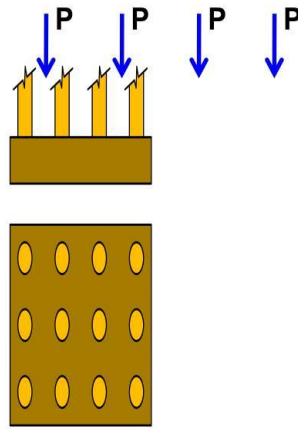


Fig. 5.10 Plan and section of typical strip footing

2.2 Types of Deep Foundations

Deep foundations are provided when adequate SBC is available at large depth below GL. There are different types of deep foundations. Some of the common types of deep foundations are listed below.

1. Pile Foundation
2. Pier Foundation
3. Well Foundation

3. Bearing Capacity of Soil

The safe bearing capacity of soil is the safe extra load soil can withstand without experiencing shear failure. The Safe Bearing Capacity (SBC) is considered unique at a particular site. But it also depends on the following factors:

- (d) Size of footing
- (e) Shape of footing
- (f) Inclination of footing
- (g) Inclination of ground
- (h) Type of load
- (i) Depth of footing etc.

SBC alone is not sufficient for design. The allowable bearing capacity is taken as the smaller of the following two criteria

- Limit states of shear failure criteria (SBC)
- Limit states of settlement criteria

Based on ultimate capacity, i.e., shear failure criteria, the SBC is calculated as

$$\text{SBC} = \text{Total load} / \text{Area of footing}$$

Usually the Allowable Bearing Pressure (ABP) varies in the range of 100 kN/m² to 400 kN/m². The area of the footing should be so arrived that the pressure distribution below the footing should be less than the allowable bearing pressure of the soil. Even for symmetrical Loading, the pressure distribution below the footing may not be uniform. It depends on the Rigidity of footing, Soil type and Conditions of soil. In case of Cohesive Soil and Cohesion less Soil the pressure distribution varies in a nonlinear way. However, while designing the footings a linear variation of pressure distribution from one edge of the footing to the other edge is assumed. Once the pressure distribution is known, the bending moment and shear force can be determined and the footing can be designed to safely resist these forces.

4. Design of Isolated Column Footing

The objective of design is to determine

- Area of footing
- Thickness of footing
- Reinforcement details of footing (satisfying moment and shear considerations)
- Check for bearing stresses and development length

This is carried out considering the loads of footing, SBC of soil, Grade of concrete and Grade of steel. The method of design is similar to the design of beams and slabs. Since footings are buried, deflection control is not important. However, crack widths should be less than 0.3 mm.

The steps followed in the design of footings are generally iterative. The important steps in the design of footings are;

- Find the area of footing (due to service loads)
- Assume a suitable thickness of footing
- Identify critical sections for flexure and shear

- Find the bending moment and shear forces at these critical sections (due to factored loads)
- Check the adequacy of the assumed thickness
- Find the reinforcement details
- Check for development length
- Check for bearing stresses

Limit state of collapse is adopted in the design of isolated column footings. The various design steps considered are;

- Design for flexure
- Design for shear (one-way shear and two way shear)
- Design for bearing
- Design for development length

The materials used in RC footings are concrete and steel. The minimum grade of concrete to be used for footings is M20, which can be increased when the footings are placed in aggressive environment, or to resist higher stresses.

Cover: The minimum thickness of cover to main reinforcement shall not be less than 50 mm for surfaces in contact with earth face and not less than 40 mm for external exposed face. However, where the concrete is in direct contact with the soil the cover should be 75 mm. In case of raft foundation, the cover for reinforcement shall not be less than 75 mm.

Minimum reinforcement and bar diameter: The minimum reinforcement according to slab and beam elements as appropriate should be followed, unless otherwise specified. The diameter of main reinforcing bars shall not be less 10 mm. The grade of steel used is either Fe 415 or Fe 500.

5. Specifications for Design of footings as per IS 456: 2000

The important guidelines given in IS 456: 2000 for the design of isolated footings are as follows:

5.1 General

Footings shall be designed to sustain the applied loads, moments and forces and the induced reactions and to ensure that any settlement which may occur shall be as nearly uniform as possible, and the safe bearing capacity of the soil is not exceeded (see IS 1904).

5.1.1 In sloped or stepped footings the effective cross-section in compression shall be limited by the area above the neutral plane, and the angle of slope or depth and location of steps shall be such that the design requirements are satisfied at every section. Sloped and stepped footings that are designed as a unit shall be constructed to assure action as a unit.

5.1.2 Thickness at the Edge of Footing

In reinforced and plain concrete footings, the thickness at the edge shall be not less than 150 mm for footings on soils, nor less than 300 mm above the tops of piles for footings on piles.

5.1.3 In the case of plain concrete pedestals, the angle 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 (see Fig. 20) shall be governed by the expression:

$$\tan \alpha \leq 0.9 * \sqrt{(100q_0/f_{ck}) + 1}$$

where

q_0 calculated maximum bearing pressure at the base of the pedestal in N/mm^2

- f_{ck} characteristic strength of concrete at 28 days in N/mm^2 .

5.2 Moments and Forces

5.2.1 In the case of footings on piles, computation for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the centre of the pile.

5.2.2 For the purpose of computing stresses in footings which support a 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.

5.2.3 Bending Moment

5.2.3.1 The bending moment at any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of the said plane.

5.2.3.2 The greatest bending moment to be used in the design of an isolated concrete footing which supports a column, pedestal or wall, shall be the moment computed in the manner prescribed in 5.2.3.1 at sections located as follows:

- At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall;
- Halfway between the center-line and the edge of the wall, for footings under masonry walls; and
- Halfway between the face of the column or pedestal and the edge of the gusseted base, for footings under gusseted bases.

5.2.4 Shear and Bond

5.2.4.1 The shear strength of footings is governed by the more severe of the following two conditions:

- The footing acting essentially as a wide beam, with a potential diagonal crack extending in a plane across the entire width; the critical section for this condition shall be assumed as a vertical section located from the face of the column, pedestal or wall at a distance equal to the effective depth of footing for footings on piles.
- Two-way action of the footing, with potential diagonal cracking along the surface of truncated cone or pyramid around the concentrated load; in this case, the footing shall be designed for shear in accordance with appropriate provisions specified in 31.6.
- 5.2.4.2 In computing the external shear or any section through a footing supported on piles, the entire reaction from any pile of diameter D_p whose centre is located $DP/2$ or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose centre is located $DP/2$ or more inside the section shall be assumed as producing no shear on the section, For intermediate positions of the pile centre, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line

interpolation between full value at DP/2 outside the section and zero value at DP/2 inside the section.

- 5.2.4.3 The critical section for checking the development length in a footing shall be assumed at the same planes as those described for bending moment in 5.2.3 and also at all other vertical planes where abrupt changes of section occur. If reinforcement is curtailed, the anchorage requirements shall be checked in accordance with 26.2.3.

5.3 Tensile Reinforcement

The total tensile reinforcement at any section shall provide a moment of resistance at least equal to the bending moment on the section calculated in accordance with 5.2.3.

5.3.1 Total tensile reinforcement shall be distributed across the corresponding resisting section as given below:

- In one-way reinforced footing, the-reinforcement extending in each direction shall be distributed uniformly across the full width of the footing;
- In two-way reinforced square footing, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing; and
- In two-way reinforced rectangular footing, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. For reinforcement in the short direction, a central band equal to the width of the footing shall be marked along the length of the footing and portion of the reinforcement determined in accordance with the equation given below shall be uniformly distributed across the central band:

$$\frac{\text{Reinforcement in central band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1}$$

where β is the ratio of the long side to the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

5.4 Transfer of Load at the Base of Column

The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting Pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to but not greater than 2, where A_1 = supporting area for bearing of footing, which in sloped or stepped footing 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; and A_2 = loaded area at the column base.

5.4.1 Where the permissible bearing stress on the concrete in the supporting or supported member would be exceeded, reinforcement shall be provided for developing the excess force, either by extending the longitudinal bars into the supporting member, or by dowels (see 5.4.3).

5.4.2 Where transfer of force is accomplished by, reinforcement, the development length of the reinforcement shall be sufficient to transfer the compression or tension to the supporting member in accordance with 26.2.

5.4.3 Extended longitudinal reinforcement or dowels of at least 0.5 percent of the cross-sectional area of the supported column or pedestal and a minimum of four bars shall be provided. Where dowels are used, their diameter shall not exceed the diameter of the column bars by more than 3 mm.

5.4.4 Column bars of diameters larger than 36 mm, in compression only can be dowelled 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.

5.5 Nominal Reinforcement

5.5.1 Minimum reinforcement and spacing shall be as per the requirements of solid slab.

5.5.2 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. This provision does not supersede the requirement of minimum tensile reinforcement based on the depth of the section.

Numerical Problems

Example 1

Design an isolated footing for an R.C. column of size 230 mm x 230 mm which carries a vertical load of 600 kN. The safe bearing capacity of soil is 200 kN/m². Use M20 concrete and Fe 415 steel.

Solution

Step 1: Size of footing

Load on column = 600 kN

Extra load at 10% of load due to self-weight of soil = 60 kN

Hence, total load, $P = 660$ kN

Required area of footing,

$$A = \frac{P}{SBC} = \frac{660}{200} = 3.3 \text{ m}^2$$

Assuming a square footing, the side of footing is

$$L = B = \sqrt{3.3} = 1.82 \text{ m}$$

Hence, provide a footing of size 1.85 m x 1.85 m

Net upward pressure in soil,

$$p = \frac{600}{1.85 \times 1.85} = 175.3 \text{ kN/m}^2 < 200 \text{ kN/m}^2$$

Hence O.K.

Hence, factored upward pressure of soil, $p_u = 263 \text{ kN/m}^2$ and, factored load, $P_u = 900$ kN.

Step 2: Two-way shear

Assume a uniform overall thickness of footing, $D = 450$ mm.

Assuming 12 mm diameter bars for main steel, effective thickness of footing 'd' is $d = 450 - 50 - 12 - 6 = 382$ mm

The critical section for the two-way shear or punching shear occurs at a distance of $d/2$ from the face of the column (See Fig. 6), where a and b are the sides of the column.

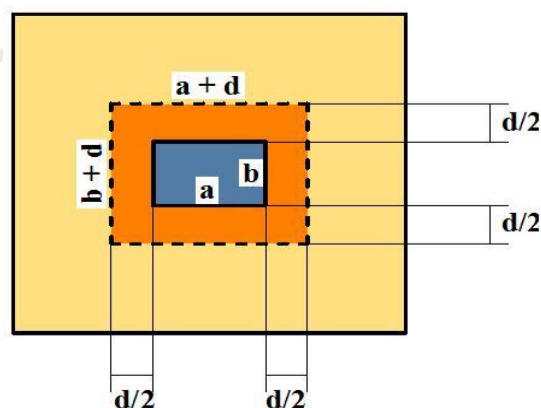


Fig. 6 Critical section in two-way shear

Hence, punching area of footing = $(a + d)^2 = (0.23 + 0.382)^2 = 0.375 \text{ m}^2$ here $a = b$ =side of column

Punching shear force = Factored load – (Factored upward pressure x punching area of footing) = $900 - (263 \times 0.375) = 801.38 \text{ kN}$

Perimeter of the critical section = $4(a+d) = 4(230+ 382) = 2448 \text{ mm}$

Therefore, nominal shear stress in punching or punching shear stress ζ_v is computed as

$$\zeta_v = \frac{\text{Punching shear force}}{\text{perimeter} \times \text{effective thickness}}$$

$$= \frac{801.38 \times 1000}{2448 \times 382} = 0.86 \text{ N/mm}^2$$

Allowable shear stress = $k_s \cdot \zeta_c$

where $\zeta_c = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{20} \approx 1.12 \text{ N/mm}^2$

and, $k_s = (0.5 + \beta_c) = \left(0.5 + \frac{0.23}{0.23}\right) = 1.0$; Hence, adopt $k_s=1$

Thus, Allowable shear stress = $k_s \cdot \zeta_c = 1 \times 1.12 = 1.12 \text{ N/mm}^2$

Thus, Allowable shear stress = $k_s \cdot \zeta_c = 1 \times 1.12 = 1.12 \text{ N/mm}^2$

Since the punching shear stress (0.86 N/mm^2) is less than the allowable shear stress (1.12 N/mm^2), the assumed thickness is sufficient to resist the punching shear force.

Hence, the assumed thickness of footing $D = 450 \text{ mm}$ is sufficient.

The effective depth for the lower layer of reinforcement, $d = 450 - 50 - 6 = 396 \text{ mm}$, and the effective depth for the upper layer of reinforcement, $d = 450 - 50 - 12 - 6 = 382 \text{ mm}$.

Step 3: Design for flexure

The critical section for flexure occurs at the face of the column (Fig. 7).

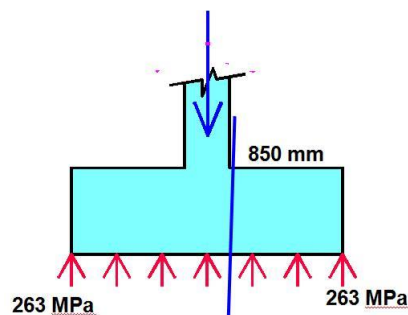


Fig. 7 Critical section for flexure

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

Factored upward pressure of soil, $p_u = 263 \text{ kN/m}^2$

Projection of footing beyond the column face, $l = (1850 - 230)/2 = 810 \text{ mm}$ Hence, bending moment at the critical section in the footing is

$$M_u = \frac{p_u l^2}{2} = \frac{263 \times 0.81^2}{2} = 86.28 \text{ kN-m} \quad \text{/m width of footing}$$

The area of steel A_{st} can be determined using the following moment of resistance relation for under reinforced condition given in Annex G – 1.1 b of IS 456 :2000.

Considering 1m width of footing,

$$86.28 \times 10^6 = 0.87 \times 415 \times A_{st} \times 382 \left[1 - \frac{415 \times A_{st}}{1000 \times 382 \times 20} \right]$$

Solving the above quadratic relation, we get

$$A_{st} = 648.42 \text{ mm}^2 \text{ and } 17,761.01 \text{ mm}^2$$

Selecting the least and feasible value for A_{st} , we have

$$A_{st} = 648.42 \text{ mm}^2$$

The corresponding value of $p_t = 0.17 \%$

Hence from flexure criterion, $p_t = 0.17 \%$

Step 4: One-way shear

The critical section for one-way shear occurs at a distance 'd' from the face of the column (Fig.8)

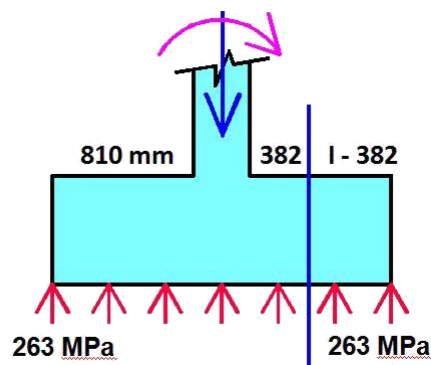


Fig. 8 Critical section for one way shear

For the cantilever slab, total Shear Force along critical section considering the entire width B is $V_u = p_u B (l - d)$

$$\begin{aligned} &= 263 \times 1.85 \times (0.81 - 0.382) \\ &= 208.24 \text{ kN} \end{aligned}$$

The nominal shear stress is given by

$$\begin{aligned} V_u &= p_u B (l - d) \\ &= 263 \times 1.85 \times (0.81 - 0.382) \\ &= 208.24 \text{ kN} \end{aligned}$$

From Table 61 of SP 16, find the p_t required to have a minimum design shear strength

$$\zeta_C = \zeta_V = 0.30 \text{ N/mm}^2 \text{ with } f_{ck} = 20 \text{ N/mm}^2.$$

For $p_t = 0.175\%$ the design shear strength ζ_C is $0.30 \text{ N/mm}^2 = \zeta_V = 0.30 \text{ N/mm}^2$. Hence from one-way shear criterion, $p_t = 0.175\%$

Comparing p_t from flexure and one-way shear criterion, provide $p_t = 0.175\%$ (larger of the two values)

Hence,

$$A_{st} = \frac{p_t}{100} b d = \frac{0.175}{100} 1000 \times 382 = 669 \text{ mm}^2$$

Provide ϕ 12 mm dia bars at 140 mm c/c.

Therefore, $A_{st} \text{ provided} = 808 \text{ mm}^2 > A_{st} \text{ required} (609 \text{ mm}^2)$. Hence O.K.

Step 5: Check for development length

Sufficient development length should be available for the reinforcement from the critical section. Here, the critical section considered for L_d is that of flexure. The development length for 12 mm dia bars is given by

$$L_d = 47 \phi = 47 \times 12 = 564 \text{ mm.}$$

Providing 60 mm side cover, the total length available from the critical section is

$$\frac{1}{2}(L - a) - 60 = \frac{1}{2}(1850 - 230) - 60 = 750 \text{ mm} > L_d. \text{ Hence O.K.}$$

Step 6: Check for bearing stress

The load is assumed to disperse from the base of column to the base of footing at rate of 2H : 1V. Hence, the side of the area of dispersion at the bottom of footing = $230 + 2(2 \times 450) = 2030$ mm. Since this is lesser than the side of the footing (i.e., 1850 mm)

$$A_1 = 1.85 \times 1.85 = 3.4225 \text{ m}^2$$

The dimension of the column is 230 mm x 230 mm. Hence, $A_2 = 0.230 \times 0.230 = 0.0529 \text{ m}^2$

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{3.4225}{0.0529}} = 8.04 > 2$$

Hence, Limit the value of $\sqrt{\frac{A_1}{A_2}}$

Permissible bearing stress =

$$0.45 f_{ck} \sqrt{\frac{A_1}{A_2}}$$

$$= 0.45 \times 20 \times 2 = 18 \text{ N/mm}^2$$

$$\text{Actual bearing stress} = \frac{\text{Factored load}}{\text{Area at column base}} = \frac{900 \times 1000}{230 \times 230} = 17.01 \frac{\text{N}}{\text{mm}^2}$$

Since the Actual bearing stress (17.01 N/mm²) is less than the Permissible bearing stress (18 N/mm²), the design for bearing stress is satisfactory.

Appropriate detailing should be shown both in plan and elevation for the footing as per the recommendations given in SP 34.

Example 2

Design an isolated footing for an R.C. column of size 300 mm x 300 mm which carries a vertical load of 800 kN together with a uniaxial moment of 40 kN-m. The safe bearing capacity of soil is 250 kN/m². Use M25 concrete and Fe 415 steel.

Solution**Step 1: Size of footing**

Load on column = 800 kN

Extra load at 10% of load due to self-weight of soil = 80 kN

Hence, total load, $P = 880$ kN

Let us provide a square isolated footing, where $L=B$

Equating the maximum pressure of the footing to SBC of soil,

i.e.,

$$\frac{P}{A} + \frac{M}{Z} = \text{SBC}$$

$$\frac{880}{B^2} + \frac{40 \times 6}{B^3} = 250$$

On solving the above equation, and taking the least and feasible value, $B = 2$ m

Hence, provide a square footing of size 2 m x 2 m

The maximum and minimum soil pressures are given by

$$p_{max} = \frac{800}{2^2} + \frac{40 \times 6}{2^3} = 230 \frac{kN}{m^2} < 250 \frac{kN}{m^2} \text{ O.K.}$$

$$p_{min} = \frac{800}{2^2} - \frac{40 \times 6}{2^3} = 170 \frac{kN}{m^2} > \text{Zero O.K.}$$

Hence, factored upward pressures of soil are,

$$p_{u,max} = 345 \text{ kN/m}^2 \text{ and } p_{u,min} = 255 \text{ kN/m}^2$$

Further, average pressure at the center of the footing is given by

$$p_{u,avg} = 300 \text{ kN/m}^2$$

and, factored load, $P_u = 900$ kN, factored uniaxial moment, $M_u = 60$ kN-m

Step 2: Two-way shear

Assume a uniform overall thickness of footing, $D = 450$ mm

Assuming 16 mm diameter bars for main steel, effective thickness of

$$\text{footing 'd' is } d = 450 - 50 - 16 - 8 = 376 \text{ mm}$$

The critical section for the two-way shear or punching shear occurs at a distance of $d/2$ from the face of the column (Fig. 9), where a and b are the dimensions of the column.

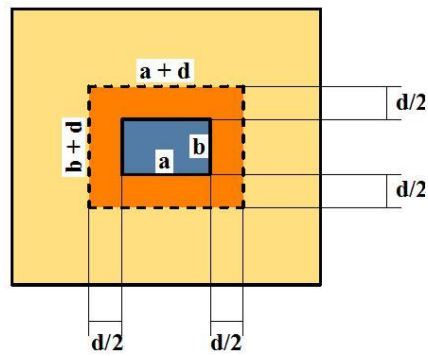


Fig. 9 Critical section in two-way shear

Hence, punching area of footing = $(a + d)^2 = (0.30 + 0.376)^2 = 0.457 \text{ m}^2$ where $a = b =$ side of column

Punching shear force = Factored load – (Factored average pressure \times punching area of footing) = $1200 - (300 \times 0.457) = 1062.9 \text{ kN}$

Perimeter along the critical section = $4(a + d) = 4(300 + 376)$
 $= 2704 \text{ mm}$

Therefore, nominal shear stress in punching or punching shear stress ζ_v is computed as

$$\zeta_v = \frac{\text{Punching shear force}}{\text{perimeter} \times \text{effective thickness}}$$

$$= \frac{1062.9 \times 1000}{2704 \times 376} = 1.05 \text{ N/mm}^2$$

Allowable shear stress = $k_s \cdot \zeta_c$

where $\zeta_c = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{25} = 1.25 \text{ N/mm}^2$

and, $k_s = (0.5 + \beta_c) = (0.5 + \frac{0.30}{0.30}) = 1.0$; Hence, adopt $k_s = 1$

Thus, Allowable shear stress = $k_s \cdot \zeta_c = 1 \times 1.25 = 1.25 \text{ N/mm}^2$

Since the punching shear stress (1.05 N/mm^2) is less than the allowable shear stress (1.25 N/mm^2), the assumed thickness is sufficient to resist the punching shear force. Hence, the assumed thickness of footing $D = 450 \text{ mm}$ is sufficient.

The effective depth for the lower layer of reinforcement, $d = 450 - 50 - 8 = 392 \text{ mm}$, and the effective depth for the upper layer of reinforcement, $d = d = 450 - 50 - 16 - 8 = 376 \text{ mm}$.

Step 3: Design for flexure

The critical section for flexure occurs at the face of the column (Fig. 10).

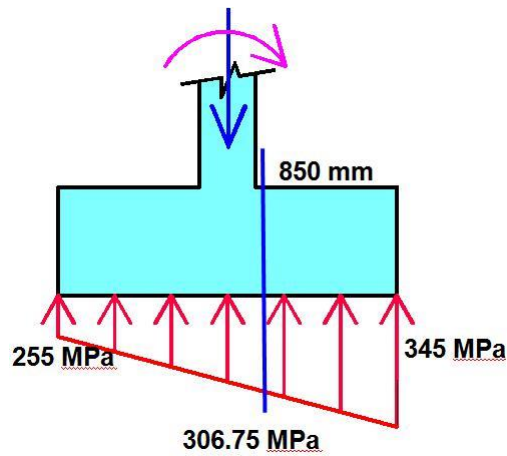


Fig. 10 Critical section for flexure

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

Factored maximum upward pressure of soil, $p_{u \max} = 345 \text{ kN/m}^2$

Factored upward pressure of soil at critical section, $p_u = 306.75 \text{ kN/m}^2$

Projection of footing beyond the column face, $l = (2000 - 300)/2 = 850 \text{ mm}$

Bending moment at the critical section in the footing is

$$M_u = [\text{Total force}] \times [\text{Distance of CG from critical section}]$$

$$M_u = \left[\left(\frac{345 + 306.75}{2} \right) 0.85 \right] \times \left[\left(\frac{2 \times 345 + 306.75}{345 + 306.75} \right) \times \frac{0.85}{3} \right]$$

$M_u = 119.11 \text{ kN-m/ m width of footing}$

The area of steel A_{st} can be determined using the following moment of resistance relation for under reinforced condition given in Annex G – 1.1 b of IS 456 :2000.

Considering 1m width of footing,

$$119.11 \times 10^6 = 0.87 \times 415 \times A_{st} \times 376 \left[1 - \frac{415 \times A_{st}}{1000 \times 376 \times 25} \right]$$

Solving the quadratic equation,

$$A_{st} = 914.30 \text{ mm}^2 \text{ and } 21,735.76 \text{ mm}^2$$

Selecting the least and feasible value, $A_{st} = 914.30 \text{ mm}^2$

The corresponding value of $p_t = 0.24 \%$

Hence from flexure criterion, $p_t = 0.24 \%$

Step 4: One-way shear

The critical section for one-way shear occurs at a distance of 'd' from the face of the column (Fig. 11).

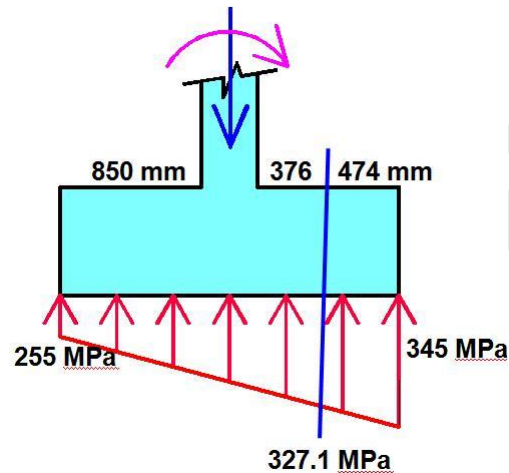


Fig. 11 Critical section for one-way shear

Factored maximum upward pressure of soil, $p_{u \max} = 345 \text{ kN/m}^2$

Factored upward pressure of soil at critical section, $p_u = 327.1 \text{ kN/m}^2$

For the cantilever slab, total Shear Force along critical section considering the entire width B is

$$V_u = [\text{Total force}] \times [(1 - d) \times B]$$

$$V_u = \left[\frac{345 + 327.1}{2} \right] \times [(0.85 - 0.376) \times 2]$$

$$V_u = 318.58 \text{ kN}$$

The nominal shear stress is given by

$$\zeta_v = \frac{V_u}{B d} = \frac{318.58 \times 1000}{2000 \times 376} = 0.42 \text{ N/mm}^2$$

From 19 of IS 456 :2000, find the p_t required to have a minimum design shear strength $\zeta_C = \zeta_v = 0.42 \text{ N/mm}^2$ with $f_{ck} = 25 \text{ N/mm}^2$.

For $p_t = 0.365 \%$ the design shear strength ζ_C is $0.42 \text{ N/mm}^2 = \zeta_v = 0.42 \text{ N/mm}^2$.

Hence from one-way shear criterion, $p_t = 0.365 \%$

Comparing p_t from flexure and one-way shear criterion, provide $p_t = 0.365 \%$ (larger of the two values)

Hence,

$$A_{st} = \frac{P_t}{100} b d = \frac{0.365}{100} 1000 \times 376 = 1372.4 \text{ mm}^2$$

Provide ϕ 16 mm dia bars at 140 mm c/c.

Therefore, $A_{st} \text{ provided} = 1436 \text{ mm}^2 > A_{st} \text{ required} (1372.4 \text{ mm}^2)$. Hence O.K.

Step 5: Check for development length

Sufficient development length should be available for the reinforcement from the critical section.

Here, the critical section considered for L_d is that of flexure.

The development length for 16 mm dia bars is given by

$$L_d = 47 \phi = 47 \times 16 = 752 \text{ mm.}$$

Providing 60 mm side cover, the total length available from the critical section is

$$\frac{1}{2}(L - a) - 60 = \frac{1}{2}(2000 - 300) - 60 = 790 \text{ mm} > L_d$$

Hence O.K.

Step 6: Check for bearing stress

The load is assumed to disperse from the base of column to the base of footing at rate of 2H: 1V.

Hence, the side of the area of dispersion at the bottom of footing = $300 + 2(2 \times 450) = 2100$ mm.

Since this is lesser than the side of the footing (i.e., 2000 mm),

$$A_1 = 2 \times 2 = 4 \text{ m}^2$$

The dimension of the column is 300 mm x 300 mm.

$$\text{Hence, } A_2 = 0.30 \times 0.30 = 0.09 \text{ m}^2$$

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{4}{0.09}} = 6.67 > 2$$

Hence, Limit the value of—

$$\sqrt{\frac{A_1}{A_2}} = 2$$

Permissible bearing stress =

$$0.45 f_{ck} \sqrt{\frac{A_1}{A_2}}$$

$$= 0.45 \times 25 \times 2 = 22.5 \text{ N/mm}^2$$

Actual bearing stress =

$$\frac{\text{Factored load}}{\text{Area at column base}} = \frac{1200 \times 1000}{300 \times 300} = 13.33 \text{ N/mm}^2$$

Since the Actual bearing stress (13.33 N/mm²) is less than the Permissible bearing stress (22.5 N/mm²), the design for bearing stress is satisfactory.

Appropriate detailing should be shown both in plan and elevation for the footing as per the recommendations given in SP 34.

Model Questions:

1. What is necessity of transverse reinforcement in columns
2. Design RCC column having unsupported length 2.75 m to support a load of 2000 KN using M20 concrete and Fe 415 steel square column 400mm sides carries a load of 900kN. Design a footing SBC of soil 100kN/m^2 . Adopt M20 concrete Fe415 steel. Check the necessary conditions.
3. Explain difference between short column and long column
4. What are the advantages of Providing pedestal to columns?
5. A column 300mm x 400mm is to support an ultimate load of 1200kN and M_u 200kN-m. Find steel using M20 concrete Fe415 steel, assuming effective cover 50mm. Sketch the reinforcement details.
6. Design a rectangular isolated footing of uniform thickness for a column 400 mm x 600 mm to support a load of 600 kN, SBC of soil is 120 kN/m^2 . Adopt M20 concrete and Fe 415 steel. Consider the size of footing 2.5 m x 2.3 m. Also check the requirement with all necessary checks