VTU EDUSAT PROGRAMME – 17
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Lecture Notes on Design of Columns

DESIGN OF RCC STRUCTURAL ELEMENTS - 10CV52

(PART – B, UNIT – 6)

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DESIGN OF RCC STRUCTURAL ELEMENTS - 10CV52
Syllabus

PART - B

UNIT – 6

DESIGN OF COLUMNS: General aspects, effective length of column, loads on columns, slenderness ratio for columns, minimum eccentricity, design of short axially loaded columns, design of column subject to combined axial load and uniaxial moment and biaxial moment using SP – 16 charts.

5 Hours

UNIT – 8

DESIGN OF STAIR CASES: General features, types of stair case, loads on stair cases, effective span as per IS code provisions, distribution of loading on stairs, Design of stair cases with waist slabs.

6 Hours

REFERENCE BOOKS

1. **Limit State Design of Reinforced concrete-**by P.C. Varghese, PHI Learning Private Limited 2008-2009
2. **Fundamentals of Reinforced concrete Design-**by M.L.Gambhir, PHI Learning Private Limited 2008-2009.
3. **Reinforced concrete Design-**by Pallai and Menon, TMH Education Private Limited,
4. **Reinforced concrete Design-**by S.N.Shinha, TMH Education Private Limited,
5. **Reinforced concrete Design-**by Karve & Shaha, Structures Publishers, Pune.
6. **Design of RCC Structural Elements** S. S. Bhavikatti, Vol-I, New Age International Publications, New Delhi.
7. **IS: 456-2000 and SP:16**

Design of Columns

UNIT-6

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.

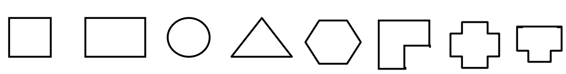
The more general terms compression members and members subjected to combined axial load and bending are sometimes used to refer to columns, walls, and members in concrete trusses or frames. These may be vertical, inclined, or horizontal. A column is a special case of a compression member that is vertical.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

Classification of columns

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

1. Based on shape

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



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 carry less load compared to long column.

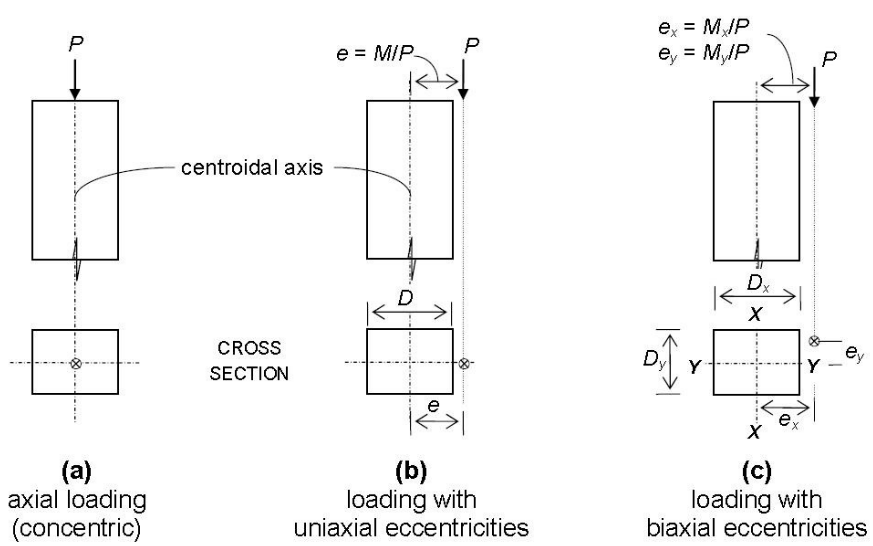
3. Based on pattern of lateral reinforcement

- 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 carry 5% extra load at failure compared to similar tied column.

4. Based on type of loading

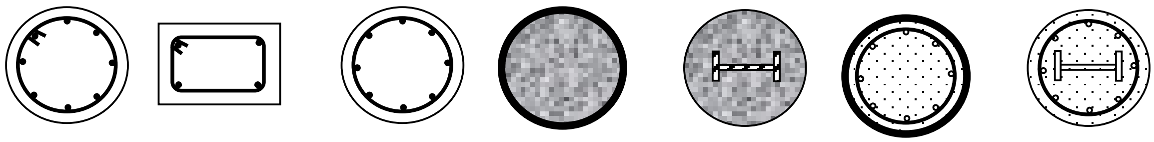
- Axially loaded column or centrally or concentrically loaded column (P_u)
- A column subjected to axial load and unaxial bending ($P_u + M_{ux}$) or ($P + M_{uy}$)
- A column subjected to axial load and biaxial bending ($P_u + M_{ux} + M_{uy}$)



Different loading situations in columns

5. Based on materials

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



RCC-Tied RCC spiral Composite columns

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.

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

Effective length of compression member

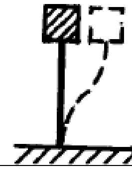
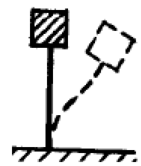
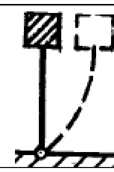
Column or strut is a compression member, the effective length of which exceeds three times the least lateral dimension. For normal usage assuming idealized conditions, the effective length of in a given plane may be assessed on the basis of Table 28 of IS: 456-2000. Following terms are required.

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

Table.Effective length of compression member

Sl. No.	Degree of End Restraint of Compression Members	Figure	Theo. Value of Effective Length	Reco. Value of Effective Length
1	Effectively held in position and restrained against rotation in both ends		0.50 l	0.65l
2	Effectively held in position at both ends, restrained against rotation at one end		0.70 l	0.80l

3	Effectively held in position at both ends, but not restrained against rotation		1.0 l	1.0l
4	Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position		1.0 l	1.20l
5	Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position		-	1.5l
6	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		2.0 l	2.0l
7	Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end		2.0 l	2.0l

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),

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.

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.

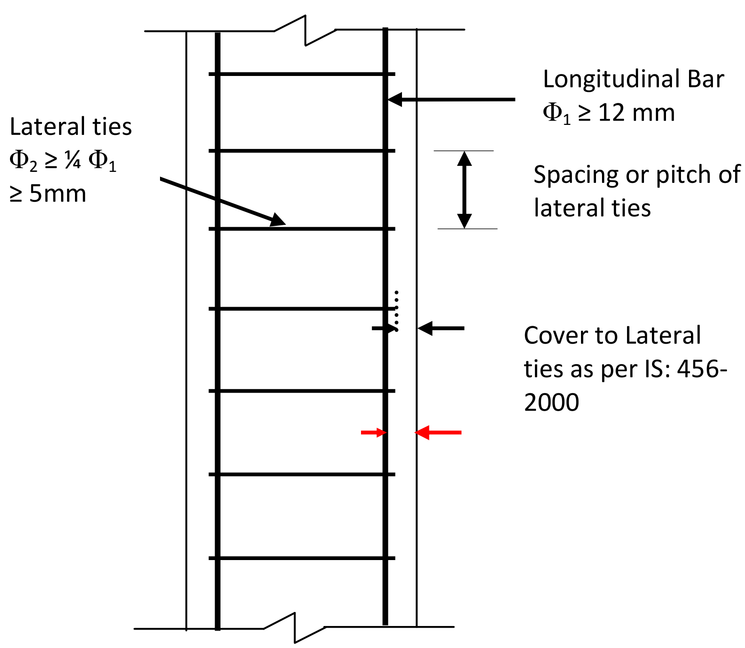
5. The bars shall not be less than 12 mm in diameter

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.



Transverse reinforcement

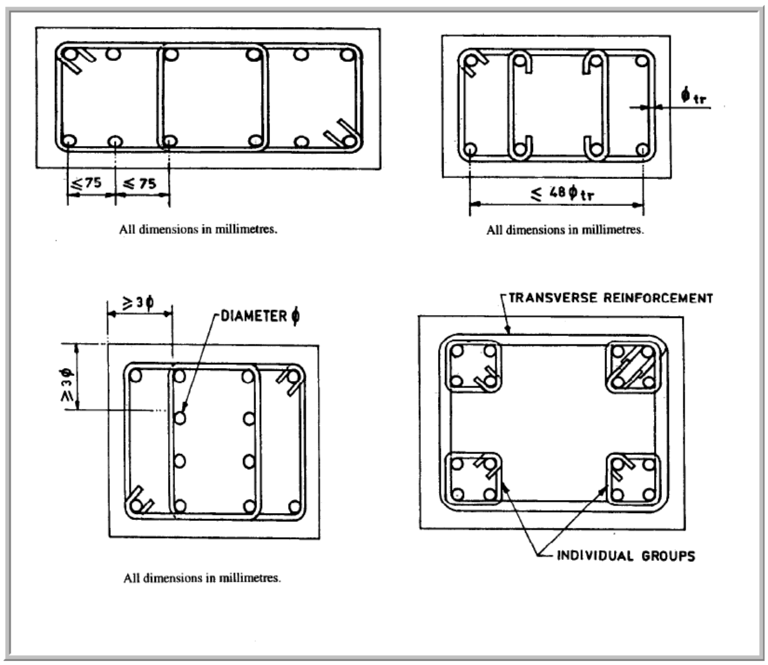
A reinforced concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling.

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.

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).



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 onefourth of the diameter of the largest longitudinal bar, and in no case less than 6 mm.

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.

LIMIT STATE OF COLLAPSE: COMPRESSION

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:456-200. 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 y 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 equal to 1.15 shall be applied.

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.

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.

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/mm.

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.

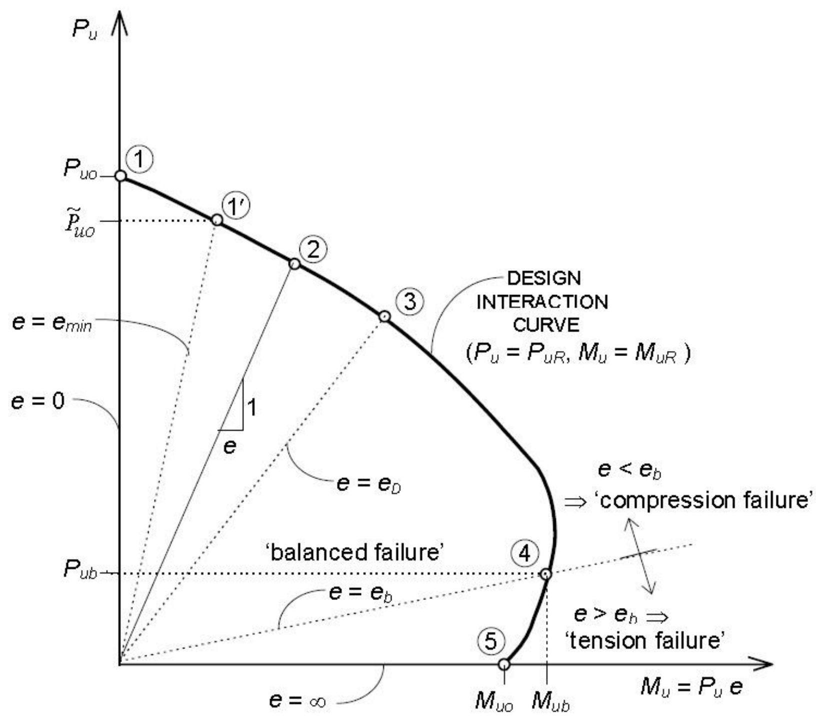
(b) Rectangular sections with reinforcement distributed equally on four sides (Charts 39 – 50): two outer rows (with area $0.3A_s$ each) and four inner rows (with area $0.1A_s$ each) have been considered in the calculations ; however, the use of these Charts can be extended, without significant error, 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^1/ D ratio for each grade (namely 0.05, .0.10, 0.15, 0.20). For intermediate values of d^1/ 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



Typical P_u — M_u interaction diagram

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 1^1 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 ($e = \infty$, $P_{uR} = 0$); the resulting ultimate moment of resistance is denoted M_{uo} and the corresponding NA depth takes on a minimum value $x_{u,\, min}$.

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

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:

$[M_{ux}/M_{ux1}]^{\alpha n} + [M_{uy}/M_{uy1}]^{\alpha n} \leq 1$, 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 αn is related to P_u/P_{uz}, where $P_{uz} = 0.45\ f_{ck}.A_c + 0.75\ f_y\ A_{sc}$

For values of P_u/P_{uz} = 0.2 to 0.8, the values of αn 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.

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

1. **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 MPa, f_y= 415 MPa
Area of steel $A_{SC} = 6 \text{ x } \pi \text{ x } 20^2/4 = 6 \text{ x } 314 = 1884 \text{ mm}^2$
Percentage of steel = 100Asc/bD = 100x1884/300x400 = 1.57 %
Area of concrete $A_c = A_g - A_{sc} = 300 \text{ x } 400 - 1884 = 118116 \text{ mm}^2$
Ultimate load carried by the column
P_u = $0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$
0.4x20x118116 + 0.67x415x1884
944928 + 523846 = 1468774 N = 1468. 8 kN
Therefore the safe load on the column = 1468.8 /1.5 = 979.2 kN

2. **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 MPa, f_y= 415 MPa, P = 980 kN
Area of steel A_{SC} = ?

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

Ultimate load carried by the column
P_u = $0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$
980 x 1.5 x 1000 = 0.4x20x (300 x 400 – A_{SC}) + 0.67x415 A_{SC}
= 960000 - 8 A_{SC} + 278.06 A_{SC}
A_{SC} =1888.5 mm^2,
Percentage of steel = 100Asc/bD = 100x1888.5 /300x400 = 1.57 % which is more than 0.8% and less than 6% and therefore ok.

Use 20 mm dia. bas, 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
$P_u = 0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_{sc}$
$980 \times 1.5 \times 1000 = 0.4 \times 20 \times 0.99\ A_g + 0.67 \times 415 \times 0.01A_g$
$= 7.92\ A_g + 2.78\ A_g = 10.7A_g$
$A_g = 137383\ mm^2$

Let us design a square column:
$B = D = \sqrt{A_g} = 370.6$ 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 400x 400 mm.

Area of steel required is $A_g = 1373.8\ mm^2$. Provide 4 bar of 22 mm diameter. Steel provided is $380 \times 4 = 1520\ mm^2$

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 = ¼ diameter of main steel = 22/4 =5.5mm or 6 mm, whichever is greater. Provide 6 mm.

Spacing: < 300 mm, < 16 x22 = 352mm, < LLD = 400mm. Say 300mm c/c

Design of circular column:

Here $A_g = 137383\ mm^2$
$\pi \times D^2/4 = A_g$, D= 418.2 mm say 420 mm. This satisfy the minimum eccentricity of 20m
Also provide 7 bars of 16 mm, $7 \times 201 = 1407\ mm^2$

Design of Transverse steel:

Dia of tie = ¼ dia of main steel = 16/4 = 4 mm or 6 mm, whichever is greater. Provide 6 mm.

Spacing: < 300 mm, < 16 x16 = 256 mm, < LLD = 420mm. 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 Fe 415 respectively.

Given:

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

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
$P_u = 0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_{sc}$
$2500 \times 1000 = 0.4 \times 20 \times 0.99\ A_g + 0.67 \times 415 \times 0.01 A_g$
$= 7.92\ A_g + 2.78\ A_g = 10.7 A_g$
$A_g = 233645\ mm^2$

If it is a square column:

$B = D = \sqrt{A_g}$ =483 mm. However provide rectangular column of size 425 x 550mm. The area provided=333750 mm^2

Area of steel = 2336 mm^2, Also provide 8 bars of 20 mm, 6 x 314 = 2512 mm^2

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

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

Check for minimum eccentricity:

<u>In the direction of longer direction</u>

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

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

<u>In the direction of shorter direction</u>

$e_{min,\ y} = l_{uy}/500 + b/30 = 3000/500 + 425/30 = 20.17$ mm or 20mm whichever is greater.

$e_{min,\ x}$ = 20.17 mm < 0.05b = 0.05 x 425 =21.25 mm. O.K

Design of Transverse steel:

Dia of tie = ¼ dia of main steel = 20/4 = 5 mm or 6 mm, whichever is greater. Provide 6 mm or 8 mm.

Spacing: < 300 mm, < 16 x20 = 320 mm, < LLD = 425mm. 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 MPa, f_y= 415 MPa, P_u = 2500kN

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
$P_u = 0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_{sc}$
$2500 \times 1000 = 0.4 \times 20 \times 0.99\ A_g + 0.67 \times 415 \times 0.01 A_g$
$= 7.92\ A_g + 2.78\ A_g = 10.7A_g$
$A_g = 233645\ mm^2$

$\pi \times D^2/4 = A_g$, D = 545.4 mm say 550 mm.

Area of steel = 2336 mm^2, Also provide 8 bars of 20 mm, 6 x 314 = 2512 mm^2

Check for shortness: Ends are hinged $l_{ex} = l_{ey} = l$ = 3000 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 = 24.22mm or 20mm whichever is greater.

e_{min} = 24.22 mm < 0.05D = 0.05 x 550 =27.5 mm. O.K

Design of Transverse steel:

Diameter of tie = ¼ dia of main steel = 20/4 = 5 mm or 6 mm, whichever is greater. Provide 6 mm or 8 mm.

Spacing: < 300 mm, < 16 x20 = 320 mm, < LLD = 550mm. 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 mm, b = 450 mm, D = 600 mm, P =2000kN, M20, Fe415

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

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

Check for minimum eccentricity:

<u>In the direction of longer direction</u>

$e_{min, x} = l_{ux}/500 + D/30 = 3000/500 + 600/30 = 26$ mm or 20mm whichever is greater.

$e_{min, x} =$ 26 mm < 0.05D = 0.05 x 600 =30 mm. O.K

<u>In the direction of shorter direction</u>

$e_{min, y}= l_{uy}/500 + b/30 = 3000/500 + 450/30 = 21$ mm or 20mm whichever is greater.

$e_{min, x} =$ 21 mm < 0.05b = 0.05 x 450 =22.5 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 = service load × partial load factor
= 2000 × 1.5 = 3000 kN

Design of Longitudinal Reinforcement
$P_u \quad = 0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$ or
$P_u \quad = 0.4\, f_{ck}\, A_c + (0.67\, f_y - 0.4 f_{ck})\, A_{sc}$

$$3000 \times 10^3 = 0.4 \times 20 \times (450 \times 600) + (0.67 \times 415 - 0.4 \times 20)A_{sc}$$
$$= 2160 \times 10^3 + 270.05A_{sc}$$

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

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 4–25φ at corners ie, $4 \times 491 = 1964\ mm^2$

and 4–20φ additional ie, $4 \times 314 = 1256\ mm^2$

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

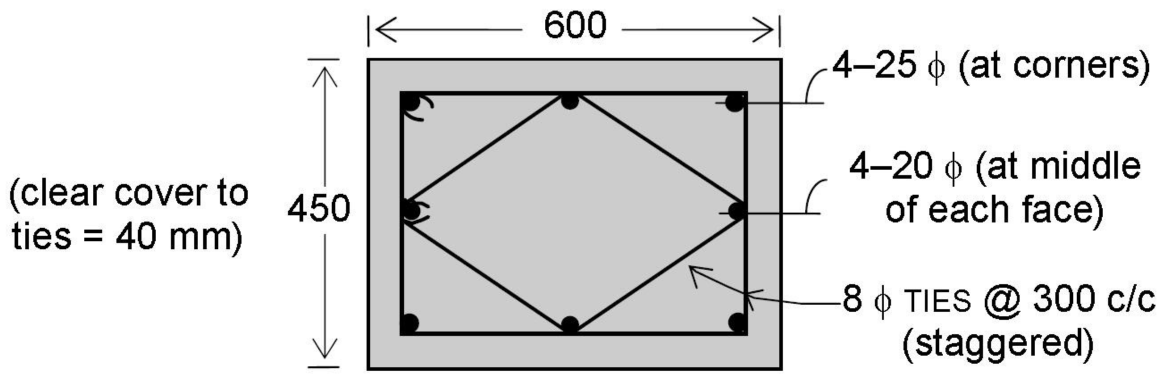
$$\Rightarrow p = (100\times3220) / (450\times600) = 1.192 > 0.8 \text{ (minimum steel), OK.}$$

Design of transverse steel

Diameter of tie = ¼ diameter of main steel = 25/4 =6.25 mm or 6 mm, whichever is greater. Provide 6 mm.

Spacing: < 300 mm, < 16 x 20 = 320 mm, < LLD = 450mm. Say 300 mm c/c
Thus provide ties 8mm @ 300 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 x 450 mm
Concrete mix M 25
Characteristic strength of steel 415 N/mm²
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 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 x 1000)/ (25 x 450 x 450) = 0.494

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

a) Reinforcement on two sides,

Referring to Chart 33,
p/f_{ck} = 0.09

Percentage of reinforcement,
p = 0.09 x 25 = 2.25 %
As = p bD/100 = 2.25 x 450 x 450/100
= 4556 mm^2

b) Reinforcement on four sides
from Chart 45,
p/f_{ck} = 0.10
p = 0.10 x 25 = 2.5 %
A_s = 2.5 x 450 x 450/100 = 5063 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/mm^2
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 mm
d^1/D – 52.5/50 = 0.105
Charts for *d'/D* = 0.10 will be used.

(a) Column *with hoop reinforcement*

P_u/f_{ck} D D = (1600 x 1000)/ (20 x 500 x 500) = 0.32

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

Referring to *Chart 52,* for f_y = 250 N/mm^2
p/f_{ck} = 0.87

Percentage of reinforcement,
p = 0.87 x 20 = 1.74 %
As = 1.74 x (π x 500^2/4)/100 = 3416 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 x 1000)/ (20 x 500 x 500) = 0.31

M_u/f_{ck} D x D^2 =125/1.05 x 10^6 /(20 x 500 x 500^2) = 0.048

Hence, From Chart 52, for f_y = 250 N/mm^2,

p/f_{ck} = 0.078

p = 0.078 x 20 = 1.56 %
As = 1.56 x(π x 500 x 500/4)/100 = 3063 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,
Core diameter = 500 - 2 (40 - 8) = 436 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} π x 428 /(π/4 x 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 coda1 requirement,

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

For 8 mm dia bar,
$s_h \leq 0.09 \times 50 / 0.0091 = 49.7$ 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/mm^2
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'/D = 0.1 will be used.

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

Referring to chart 44

M_u/f_{ck} b x D^2 = 0.09
M_{ux1} = 0.09 x 15 x 400 x 600^2) = 194.4 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 x D^2 = 0.083
M_{ux1} = 0.083 x 15 x 600 x 400^2) = 119.52 kN.m

Calculation of P_{uz} :
Referring to Chart 63 corresponding to
p = 1.2, f_y = 415 and f_{ck} = 15,
$P_{uz}/A_g = 10.3$

P_{uz} = 10.3 x 400 x 600 = 2472 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 Churn 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

$P_u/P_{uz} = 0.65$ 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%

Thus provide As = 1.3x400x600/100 = 3120 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
$M_u/f_{ck}\, b \times D^2 = 0.095$
$M_{ux1} = 0.095 \times 15 \times 400 \times 600^2) = 205.2$ kN.m

Referring to Chart 45,
$M_u/f_{ck}\, b \times D^2 = 0.085$
$M_{ux1} = 0.085 \times 15 \times 600 \times 400^2) = 122.4$ kN.m

Chart 63 : $P_{uz}/A_g = 10.4$
$P_{uz} = 10.4 \times 400 \times 600 = 2496$ 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.

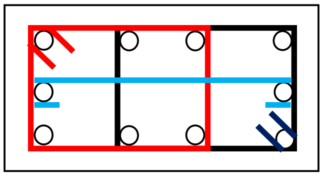
Using Boris load contour equation as per IS:456-2000

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

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

As = 3120 mm^2. Provide 10 bars of 20 mm dia. Steel provided is 314 x 10 = 3140 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 kN, M_{ux} = 125 kNm, M_{uy} = 75 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 mm, b = 300 mm, A_s = 2946 mm^2 M_{ux} = 125 kNm, M_{uy} = 75 kNm, f_{ck} = 25 MPa, f_y = 415 MPa

Applied eccentricities

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

$e_y = M_{uy}/P_u = 75 \times 10^3/1400 = 53.6$ 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 kN,

M_{ux1} = 187 kNm

M_{uy1} = 110 kNm

Values of P_{uz} and α_n

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

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

$= (1687500 + 883800)$N = 2571 kN

$\Rightarrow P_u/P_{uz} = 1400/2571 = 0.545$ (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.