Column Design Reference Document

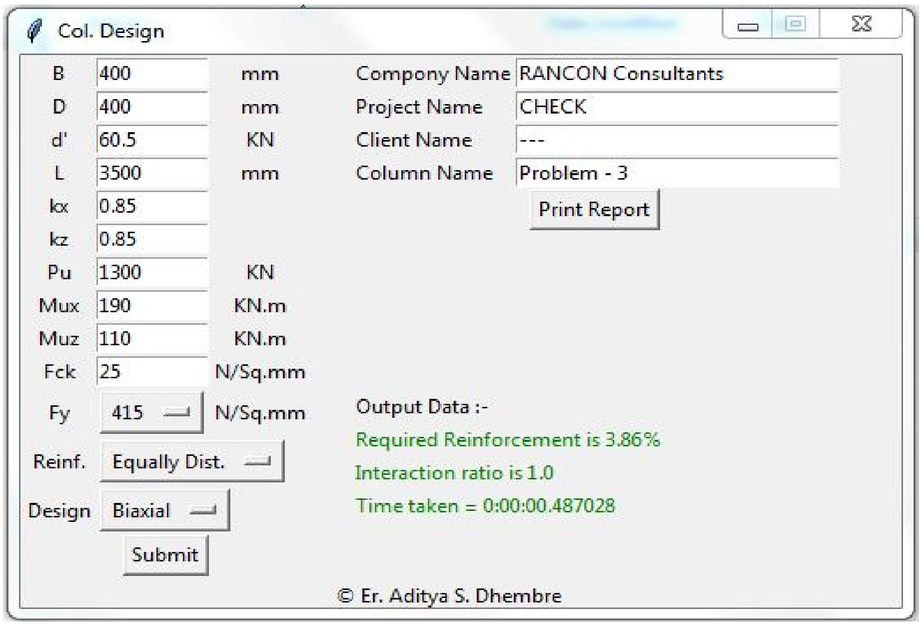

Fig. - Window View of Column Design Application

Ref. Book	=	Reinforced Concrete Design by Pillai and Menon
Cover Page ref	=	Page No. - 9

Problem 1	=	Page No.-2
Ref. in book	=	Example 13.3 Page No. 634
Ref.	=	Page No. - 5
Ast by Ref. book	=	3111
Ast by Program	=	3132.00
Difference	=	0.67%

Problem 2	=	Page No.-3
Ref. in book	=	Example 13.12 Page No. 667
Ref.	=	Page No. - 6
Ast by Ref. book	=	3960
Ast by Program	=	3906
Difference	=	-1.38%

Problem 3	=	Page No.-4
Ref. in book	=	Example 13.15 Page No. 677
Ref.	=	Page No. - 7
Ast by Ref. book	=	5892
Ast by Program	=	6176
Difference	=	4.60%

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******************   Column  Design   ******************
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Basic Info >>>

Compony Name     =      RANCON Consultants
Project Name     =      CHECK
Client Name      =      ---
Column Name      =      Problem - 1

Input...

Width            =      450      mm
Depth            =      600      mm
Eff. Cover       =      60       mm
Uns. Length      =      3000     mm
Kx               =      1
kz               =      1
Axial Force      =      3000     KN
Moment in x      =      0        KN.m
Moment in z      =      0        KN.m
Conc. Grade      =      M20
Steel Grade      =      Fe415
Reinf. Type      =      Equally Dist.
Design as        =      Axial

Solution >>>

1) Check for Slender
        Le/D = 5.0 < 12
        Le/b = 6.67 < 12
        Hence Column is Short

2) Minimum Eccenrticity
        emin = max(L/500 + D/30,20) mm
        ex,min = N.A mm
        ez,min = N.A mm
        Mx_for_ex = N.A KN.m
        Mz_for_ez = N.A KN.m

3) Final Forces
        M       = max(M_for_e, Mu)
        Mx      = N.A KN.m
        Mz      = N.A KN.m
        P       = 3000 KN

4) Area of Steel required
        Ast = max(Ast.req , Ast.min) < Ast,max
        Ast = 3132.0 Sq.mm

5) Check
        Mxr     = 90.17 KN.m > Mux...Ok
        Mzr     = 67.63 KN.m > Muz...Ok
        Interaction ratio

        (Mxu/Mxr)^a + (Mzu/Mzr)^a = N.A <= 1

        Hence OK.

Output >>>

Provide 3132.0 Sq.mm Reinforcement.

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*******************   Column  Design   *******************
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Basic Info >>>

Compony Name       =        RANCON Consultants
Project Name       =        CHECK
Client Name        =        ---
Column Name        =        Problem - 2

Input...

Width              =        300       mm
Depth              =        600       mm
Eff. Cover         =        60        mm
Uns. Length        =        0         mm
Kx                 =        1
kz                 =        1
Axial Force        =        1400      KN
Moment in x        =        280       KN.m
Moment in z        =        0         KN.m
Conc. Grade        =        M20
Steel Grade        =        Fe415
Reinf. Type        =        Equally Dist.
Design as          =        Uniaxial

Solution >>>

1) Check for Slender
        Le/D = 0.0 < 12
        Le/b = 0.0 < 12
        Hence Column is Short

2) Minimum Eccenrticity
        emin = max(L/500 + D/30,20) mm
        ex,min = 20.0 mm
        ez,min = N.A mm
        Mx_for_ex = 28.0 KN.m
        Mz_for_ez = N.A KN.m

3) Final Forces
        M       = max(M_for_e, Mu)
        Mx      = 280.0 KN.m
        Mz      = N.A KN.m
        P       = 1400 KN

4) Area of Steel required
        Ast = max(Ast.req , Ast.min) < Ast,max
        Ast = 3906.0 Sq.mm

5) Check
        Mxr     = 280.21 KN.m > Mux...Ok
        Mzr     = N.A KN.m > Muz...Ok
        Interaction ratio

        (Mxu/Mxr)^a + (Mzu/Mzr)^a = N.A <= 1

        Hence OK.

Output >>>

Provide 3906.0 Sq.mm Reinforcement.

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******************   Column  Design   ******************
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Basic Info >>>

Compony Name      =      RANCON Consultants
Project Name      =      CHECK
Client Name       =      ---
Column Name       =      Problem - 3

Input...

width             =      400      mm
Depth             =      400      mm
Eff. Cover        =      60.5     mm
Uns. Length       =      3500     mm
Kx                =      0.85
kz                =      0.85
Axial Force       =      1300     KN
Moment in x       =      190      KN.m
Moment in z       =      110      KN.m
Conc. Grade       =      M25
Steel Grade       =      Fe415
Reinf. Type       =      Equally Dist.
Design as         =      Biaxial

Solution >>>

1) Check for Slender
        Le/D = 7.44 < 12
        Le/b = 7.44 < 12
        Hence Column is Short

2) Minimum Eccenrticity
        emin = max(L/500 + D/30,20) mm
        ex,min = 20.33 mm
        ez,min = 20.33 mm
        Mx_for_ex = 26.429 KN.m
        Mz_for_ez = 26.429 KN.m

3) Final Forces
        M        = max(M_for_e, Mu)
        Mx       = 190.0 KN.m
        Mz       = 110.0 KN.m
        P        = 1300 KN

4) Area of Steel required
        Ast = max(Ast.req , Ast.min) < Ast,max
        Ast = 6176.0 Sq.mm

5) Check
        Mxr      = 262.53 KN.m > Mux...Ok
        Mzr      = 262.53 KN.m > Muz...Ok
        Interaction ratio

        (Mxu/Mxr)^a + (Mzu/Mzr)^a = 0.16408082679235023 <= 1

        Hence OK.

Output >>>

Provide 6176.0 Sq.mm Reinforcement.

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$$P_{u0} = 0.447 f_{ck} A_g + (f_{sc} - 0.447 f_{ck}) A_{sc} \tag{13.16}$$

$$\text{with } f_{sc} = \begin{cases} 0.870\, f_y & \text{for Fe 250} \\ 0.790\, f_y & \text{for Fe 415} \\ 0.746 f_y & \text{for Fe 500} \end{cases} \tag{13.16a}$$

However, as explained in Section 13.2.2, the Code requires all columns to be designed for 'minimum eccentricities' in loading. Hence, Eq. 13.16 cannot be directly applied. Nevertheless, where the calculated minimum eccentricity (in any plane) does not exceed 0.05 times the lateral dimension (in the plane considered), the Code (Cl. 39.3) permits the use of the following simplified formula, obtained by reducing P_{u0} (from Eq. 13.16) by approximately 10 per cent[†] [Ref. 13.7]:

$$\tilde{P}_{u0} = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc} \tag{13.17}$$

where $\tilde{P}_{u0}$ denotes the design strength in uniaxial compression permitted by the Code (including the effect of minimum eccentricities). It is found that the use of Eq. 13.17 results in a conservative design, compared to the rigorous design involving axial compression and biaxial bending with the minimum eccentricities.

As mentioned earlier, the Code (Cl. 39.4) permits the load capacity be enhanced by 5 per cent when spiral reinforcement is provided, conforming to Eq. 13.15.

Example 13.3

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.m and is braced against sideway in both directions. Use M 20 concrete and Fe 415 steel.

Solution

Short Column or Slender Column?

- Given: $lx = ly = 3000$ mm, $D_y = 450$ mm, $D_x = 600$ mm

$$\text{slenderness ratios } \begin{cases} l_{ex}/D_x = k_x l_x / D_x = k_x \times 3000/600 = 5k_x \\ l_{ey}/D_y = k_y l_y / D_y = k_y \times 3000/450 = 6.67k_y \end{cases}$$

As the column is braced against sideway in both directions, effective length ratios k_x and k_y are both less than unity, and hence the two slenderness ratios are both less than 12.

- Hence, the column may be designed as a *short column*.

[†] The reduction works out as 10 per cent with respect to Fe 415/Fe 500 grades of steel. However, with respect to Fe 250 steel, the reduction in f_{sc} is as high as 30 per cent, while the reduction in f_{ck} is 10 per cent.

Minimum Eccentricities [Eq. 13.8]

$$e_{x,\min} = \frac{3000}{500} + \frac{600}{30} = 26.0 \text{ mm } (> 20.0 \text{ mm})$$

$$e_{y,\min} = \frac{3000}{500} + \frac{450}{30} = 21.0 \text{ mm } (> 20.0 \text{ mm})$$

- As $0.05D_x = 0.05 \times 600 = 30.0 \text{ mm} > e_{x,\min} = 26.0$ mm and $0.05D_y = 0.05 \times 450 = 22.5 \text{ mm} > e_{y,\min} = 21.0$ mm, the Code formula for axially loaded short columns can be used.

Factored Load

- P_u = *service load × partial load factor* $= 2000 \times 1.5 = 3000$ kN

Design of Longitudinal Reinforcement

- $P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$ [Eq. 13.17]

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

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

- 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:** $4 \times 491 = 1964 \text{ mm}^2$
  and **4–20 ϕ additional:** $4 \times 314 = 1256 \text{ mm}^2$

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

$\Rightarrow p = (100 \times 3220) / (450 \times 600) = 1.192 > 0.8$ (minimum reinf.) —OK.

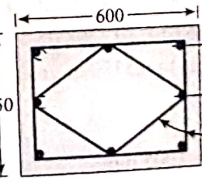


Fig. 13.10 *Example 13.3*

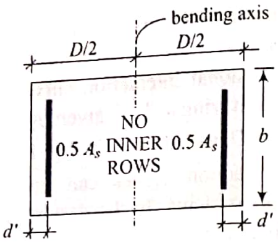
Lateral Ties

- Tie diameter $\phi_t > \begin{cases} 25/4 \\ 6 \text{ mm} \end{cases}$: provide 8 mm dia;

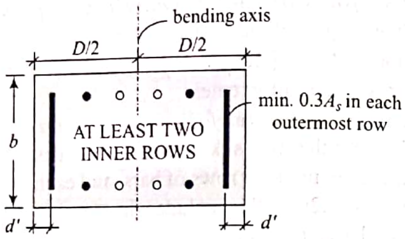
Tie spacing $s_t < \begin{cases} 450 \text{ mm} \\ 16 \times 20 = 320 \text{ mm} \\ 300 \text{ mm} \end{cases}$: provide 300 mm.

∴ Provide **8 ϕ ties @ 300 c/c**

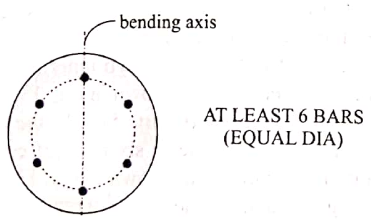
The detailing of reinforcement is shown in Fig. 13.10



(a) rectangular section with "*reinforcement distributed equally on two sides*"



(a) rectangular section with "*reinforcement distributed equally on all sides*"



(c) circular column section

Fig. 13.22 *Reinforcement arrangements for which SP : 16 Charts are applicable*

(as in Examples 13.9 and 13.11) for the section chosen, and thereby to verify the safety of the section; if required, the design should be suitably revised, to make it more economical.

There are other situations, encountered in practice, which are not amenable for the use of SP : 16 Charts. These include cases of

- unsymmetrically arranged reinforcement in rectangular sections;
- non-rectangular and non-circular sections—such as L-shaped, T-shaped, H-shaped, cross shaped sections, etc.

In such cases, it becomes necessary to construct proper interaction diagrams in order to obtain accurate and reliable solutions.

Example 13.12

Using the design aids given in SP : 16, design the longitudinal reinforcement in a rectangular reinforced concrete column of size 300 mm × 600 mm subjected to a factored load of 1400 kN and a factored moment of 280 kNm with respect to the major axis. Assume M 20 concrete and Fe 415 steel.

Solution

- *Given*: b = 300 mm, D = 600 mm, f_{ck} = 20 MPa, f_y = 415 MPa, P_u = 1400 kN, M_{ux} = 280 kNm
- *Arrangement of bars*: as D = 600 mm, the spacing between the corner bars will exceed 300 mm; hence inner rows of bars have to be provided to satisfy detailing requirements [refer Section 13.2.3]. Assuming two or more inner rows, the SP : 16 Charts for "equal reinforcement on four sides" can be made use of [Fig. 13.22 (b)].
- Assuming an effective cover d' = 60 mm,

  $\Rightarrow d'/D = 60/600 = 0.1$

$$p_u = \frac{P_u}{f_{ck}bD} = \frac{1400 \times 10^3}{20 \times 300 \times 600} = 0.389$$

$$m_u = \frac{M_{ux}}{f_{ck}bD^2} = \frac{280 \times 10^6}{20 \times 300 \times 600^2} = 0.130$$

- Referring to Chart 44 ($d'/D = 0.10$) of SP : 16, it can be observed that, the coordinates $p_u = 0.389$, $m_u = 0.130$ would lie on a design interaction curve with

  $p/f_{ck} \approx 0.11$

  $\Rightarrow p_{reqd} = 0.11 \times 20 = 2.2$

  $\Rightarrow A_{s,\,reqd} = 2.2 \times 300 \times 600/100 = \mathbf{3960\ mm^2}$

Detailing of longitudinal reinforcement

- The design chart used refers to the case of "equal reinforcement on four sides" [Fig. 13.22(b)],

  Outermost rows

  Minimum area required in each outermost row = 0.3 × 3960 = 1188 mm²

  Provide 2 – 28 ϕ: area = 616 × 2 = 1232 mm² > 1188 mm²

  Inner rows

  Total area required = 3960 – (1232 × 2) = 1496 mm²

  Provide 4 – 22 ϕ in two inner rows: area = 380 × 4 = 1520 mm² > 1496 mm²
- Total area provided = (1232 × 2) + 1520 = 3984 mm² > 3960 mm²

  ($\Rightarrow p$ = 100 × 3984/(300 × 600) = 2.213)

  Assuming 8 mm ties, effective cover = 40 + 8 + (28/2) = 62 mm ≈ 60 mm—OK

Code Procedure for Columns subject to Biaxial Bending

1. Given P_u, M_{ux}, M_{uy}, verify that the eccentricities $e_x = M_{uy}/P_u$ and $e_y = M_{ux}/P_u$ are not less than the corresponding minimum eccentricities (refer Section 13.5.1).
2. Assume a trial section for the column.
3. Determine M_{ux1} and M_{uy1}, corresponding to the given P_u (using appropriate design aids). Ensure that M_{ux1} and M_{uy1} are significantly greater than M_{ux} and M_{uy} respectively; otherwise, suitably redesign the section†.
4. Determine P_{uz} [Eq. 13.39], and hence α_n [Eq. 13.40].
5. Check the adequacy of the section [Eq. 13.38]; if necessary, redesign the section and check again.

Selection of Trial Section Generally, in practice, the cross-sectional dimensions of the column are tentatively fixed in advance, and the structural analysis is performed on the basis of these dimensions. Indeed, the biaxial moments obtained from frame analyses (considering various load combinations) are based on the assumed cross-sectional dimensions (required for stiffness calculations). Hence, in the selection of the 'trial section' for the design of biaxially loaded columns, it is only reinforcement details that need to be suitably assumed in practical situations.

One simple way of doing this is by designing the trial section for uniaxial eccentricity, considering a moment of approximately 15 per cent‡ in excess of the resultant moment, i.e.,

$$M_u \equiv 1.15\sqrt{M_{ux}^2 + M_{uy}^2} \qquad (13.41)$$

This bending moment should be considered to act with respect to the major principal axis if $M_{ux} \geq M_{uy}$; otherwise, it should be with respect to the minor principal axis. The reinforcement may be assumed to be distributed equally on all sides of the section.

Example 13.15

A corner column (400 mm × 400 mm), located in the lowermost storey of a system of braced frames, is subjected to factored loads: P_u = 1300 kN, M_{ux} = 190 kNm and M_{uy} = 110 kNm. The unsupported length of the column is 3.5 m. Design the reinforcement in the column, assuming M 25 concrete and Fe 415 steel.

Solution

- *Given*: $D_x = D_y$ = 400 mm, l = 3500 mm, P_u = 1300 kN, M_{ux} = 190 kNm, M_{uy} = 110 kNm, f_{ck} = 25MPa, f_y = 415MPa.

† This is usually achieved by increasing the percentage of reinforcement and/or improving the grade of concrete; the dimensions may also be increased, if required.

‡ Lower percentages (up to 5 per cent) can be assumed if the axial loading level (P_u/P_{uz}) is relatively high.

Slenderness ratios

- Assuming an effective length ratio of 0.85 for the braced column,

$l_{ex} = l_{ey}$ = 0.85 × 3500 = 2975 mm

$\Rightarrow l_{ex}/D_x = l_{ey}/D_y$ = 2975/400 = 7.44 < 12

Hence the column may be designed as a *short column*.

Check minimum eccentricities

- Applied eccentricities: e_x = 190 × 10^3/1300 = 146 mm

e_y = 110 × 10^3/1300 = 84.6 mm

Minimum eccentricities as per Code [Eq. 13.8]:

$e_{x,min} = e_{y,min}$ = 3500/500 + 400/30 = 20.3 mm > 20 mm

As the minimum eccentricities are less than the applied eccentricities, no modification to M_{ux}, M_{uy} is called for.

Trial section: Longitudinal reinforcement

- Designing for uniaxial eccentricity with P_u = 1300 kN and

$$M_u \approx 1.15\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.15\sqrt{190^2 + 110^2} = 252 \text{ kNm}$$

- Assuming d' = 60 mm,

d'/D = 60/400 = 0.15

$$\frac{P_u}{f_{ck}bD} = \frac{1300 \times 10^3}{25 \times 400^2} = 0.325$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{252 \times 10^6}{25 \times 400^3} = 0.157$$

- Referring to chart 45 of SP : 16 ("equal reinforcement on all sides"),

p/f_{ck} = 0.14

$\Rightarrow p_{reqd}$ = 0.14 × 25 = 3.5

[Note: This relatively high percentage of steel is particularly acceptable for a column located in the lowermost storey of a tall building.]

$\Rightarrow A_{s,reqd}$ = 3.5 × 400^2/100 = 5600 mm^2

- **Provide 12 – 25 ϕ**: A_s = 491 × 12 = 5892 mm^2 > 5600 mm^2. The arrangement of bars is shown in Fig. 13.29.

Uniaxial moment capacities: M_{ux1}, M_{ux2} [Here, due to symmetry, $M_{ux1} = M_{ux2}$]

$$\frac{P_u}{f_{ck}bD} = 0.325 \text{ (as calculated earlier)}$$

- $p_{provided} = 5892 \times 100/400^2 = 3.68$

  $\Rightarrow p/f_{ck} = 3.68/25 = 0.147$

  $d' = 40 + 8 + 25/2 = 60.5$ mm (assuming a clear cover of 40 mm and 8 mm ties)

  $\Rightarrow d'/D = 60.5/400 = 0.151 \approx 0.15$

- Referring to Chart 45 ($d'/D = 0.15$),

$$\frac{M_{ux1}}{f_{ck} bD^2} = 0.165$$

$$\Rightarrow M_{ux1} = M_{uy1} = 0.165 \times 25 \times 400^3 = 264 \times 10^6 \text{ Nmm}$$
$$= 264 \text{ kNm}$$

which is significantly greater than $M_{ux} = 190$ kNm and $M_{uy} = 110$ kNm

Values of P_{uz} and α_n

- $P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$ [Eq. 13.40]

  $= (0.45 \times 25 \times 400^2) + (0.75 \times 415 - 0.45 \times 25) \times 5892$

  $= (1800 \times 10^3 + 1767.6 \times 10^3)$ N $= 3568$ kN

  $\Rightarrow P_u/P_{uz} = 1300/3568 = 0.364$ (which lies between 0.2 and 0.8)

$$\Rightarrow \alpha_n = 1.0 + \frac{0.364 - 0.2}{0.8 - 0.2}(2.0 - 1.0) = 1.273$$

[Alternatively, Eq. 13.40 may be used].

Check safety under biaxial loading

- $$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{190}{264}\right)^{1.273} + \left(\frac{110}{264}\right)^{1.273}$$
$$= 0.658 + 0.328 = 0.986 < 1.0$$

Hence, the trial section is safe under the applied loading.

Transverse reinforcement

- The minimum diameter ϕ_t and maximum spacing s_t of the lateral ties are specified by the Code [Eq. 13.9, 13.10]:

$$\phi_t > \begin{cases} 25/4 = 6.25 \text{ mm} \\ 6 \text{ mm} \end{cases}$$

$\Rightarrow$ Provide 8 ϕ ties

$$s_t < \begin{cases} D = 400 \text{ mm} \\ 16 \times 25 = 400 \text{ mm} \\ 300 \text{ mm} \end{cases}$$

Provide 8 ϕ ties@ 300 c/c as shown in Fig. 13.29.

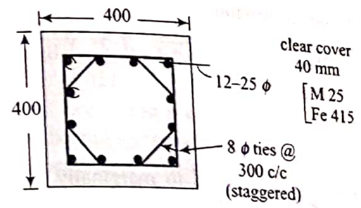


Fig. 13.29 *Example 13.15*

Example 13.16

Verify the adequacy of the short column section Fig. 13.15(a) under the following load conditions:

$P_u = 1400$ kN, $M_{ux} = 125$ kNm, $M_{uy} = 75$ kNm

The design interaction curves [Fig. 13.19, 13.20] derived earlier may be used for this purpose. Assume that the column is a 'short column'.

Solution

- *Given*: $D_x = 500$ mm, $D_y = 300$ mm, $A_s = 2946$ mm² $M_{ux} = 125$ kNm, $M_{uy} = 75$ kNm, $f_{ck} = 25$ MPa, $f_y = 415$ MPa [refer Example 13.5].

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 Example 13.10 and 13.11 [also see Fig. 13.20], corresponding to $P_u = 1400$ kN,

  $M_{ux1} = 187$ kNm

  $M_{uy1} = 110$ kNm

Values of P_{uz} and α_n

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

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

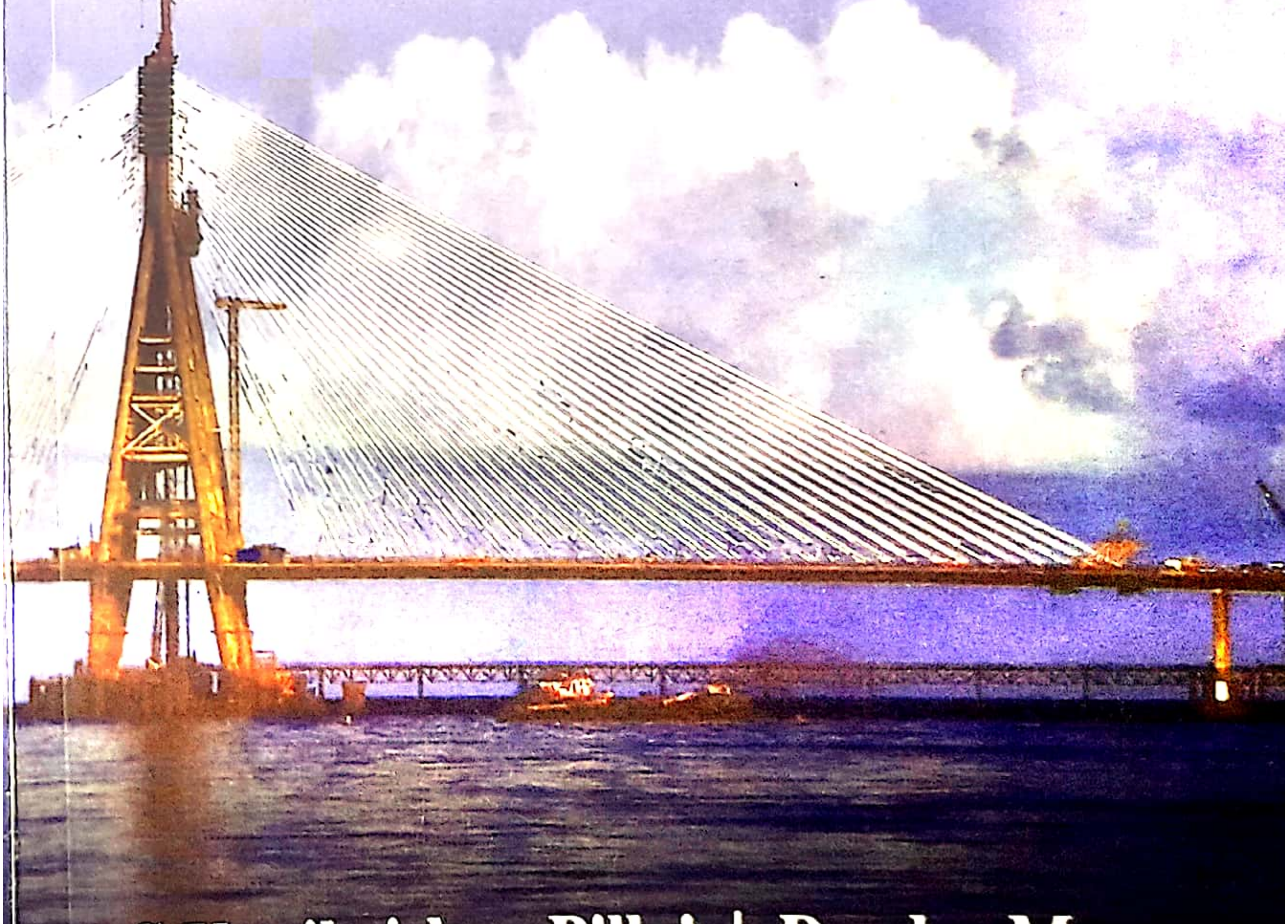
  $= (1687500 + 883800)$ N $= 2571$ kN

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

$$\Rightarrow \alpha_n = 1.0 + \frac{0.545 - 0.2}{0.8 - 0.2}(2.0 - 1.0) = 1.575$$

Reinforced Concrete Design

Third Edition



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