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REINFORCED CEMENT CONCRETE

Volume-1 : Study Material with Classroom Practice Questions

Reinforced Cement Concrete

Solutions for Volume : I Classroom Practice Questions

Chapter- 3
Limit State Design- Singly Reinforced Beams

01. Ans: (a)

Sol: For Fe415,

$M_{u\,limit}$ = Equation (1) with $x_{u\,max}$

$= 0.138\ f_{ck}bd^2$

$= 0.138 \times 15 \times 200 \times (500)^2$

$= 103.5$ kN-m

02. Ans: (c)

Sol: Balanced (or) limiting percentage of steel (use $x_{u\,max}$)

$$C = T$$

$$0.36\ f_{ck}\ bx_{u\,max} = 0.87\ f_y\ A_{st}$$

$$0.36\ f_{ck}\ b(0.48d) = 0.87 \times 415\ A_{st}$$

$$0.36 \times 15 \times 200 \times 0.48 \times 300 = 0.87 \times 415\ A_{st}$$

$$A_{st} = 430\text{mm}^2$$

03. Ans: (b)

Sol: $M_u = 138 \times 10^6$ N-mm

$M_u = M_{u\,limit}$

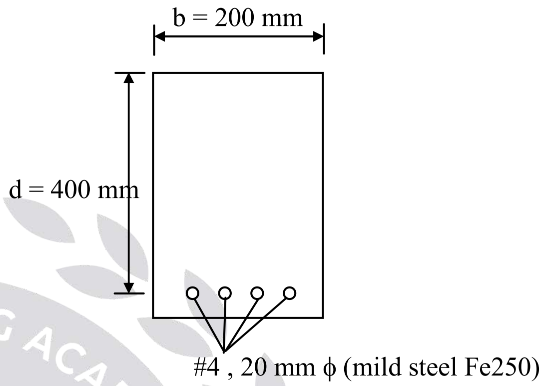
$= 0.138 \times f_{ck}\ bd^2$ – (design as BS)

$138 \times 10^6 = 0.138 \times 20 \times 200 \times d^2$

$d = 500$ mm

04. Ans: (b)

Sol:



i) $x_{umax} = 0.53 \times d$

$= 0.53 \times 400$

$= 212$ mm

ii) x_u = ? C = T

$$0.36 \times f_{ck} \times b \times x_u = 0.87 \times f_y \times A_{st}$$

$$0.36 \times 15 \times 200 \times x_u = 0.87 \times 250 \times 4 \times \left(\frac{\pi}{4} \times 20^2\right)$$

$\Rightarrow 1080x_u = 273318.5$

$x_u = 253.1$ mm

$x_u > x_{u\,max} \Rightarrow$ over reinforced section

Over reinforcement section fails suddenly

To avoid sudden fail decrease the MR to that of a balanced section

$M_{u\,limit} = 0.148 \times f_{ck}\ bd^2$

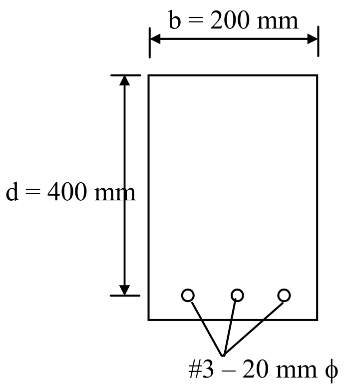
$= 0.148 \times 15 \times 200 \times 400^2$

$= 71040000$ N-mm $= 71.04$ kN-m

$\simeq 72$ kN-m

05. Ans: (d)

Sol:



i) $x_{u\,max} = 0.53 \times d$

$= 0.53 \times 400 = 212$ mm

ii) C = T

$0.36 \times f_{ck} \times b \times x_u = 0.87 \times f_y \times A_{st}$

$$0.36 \times 15 \times 200 \times x_u = 0.87 \times 250 \times \left(3 \times \frac{\pi}{4} \times 20^2\right)$$

$1080\ x_u = 204988.92$

$x_u = 190$ mm

$x_u < x_{max} \Rightarrow$ Under reinforced section

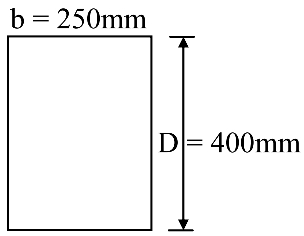
$M_u = 0.36 f_{ck}\ b x_u (d - 0.42 x_u)$

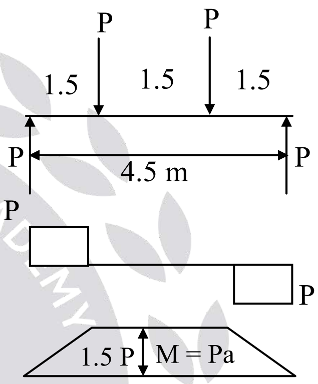
$= 0.36 \times 15 \times 200 \times 190\ (400 - 0.42 \times 190)$

$M_u = 65.7$ kN.m $\simeq 66$ kN-m

06. Ans: 8.86 kN

Sol:





Homogenous beam

$f_{cr} = 2$ MPa

Modulus of rupture/tensile stress of concrete from bending equation

$$\frac{M}{I} = \frac{f}{y}$$

$$\Rightarrow M = f_{cr} \times z \qquad \left[\because z = \frac{bD^2}{6}\right]$$

$$= 2\left[\frac{250 \times 400^2}{6}\right] = 13.33 \times 10^6 \text{N-mm}$$

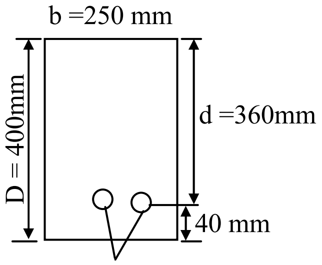
M = P.a

$13.3 = P \times 1.5$

$$P = \frac{13.3}{1.5} = 8.86\,\text{kN}$$

07. Ans: 31.6 kN

Sol:



Reinforced concrete beam

i) $x_{umax} = 0.48d$

$= 0.48 \times 360 = 172.8$ mm

C = T

$0.36f_{ck}bx_u = 0.87\ f_y A_{st}$

$$0.36 \times 20 \times 250 \times x_u = 0.87 \times 415 \times \left(2 \times \frac{\pi}{4} \times 16^2\right)$$

$1800\ x_u = 145186.8$

$x_u = 80.65$ mm

$x_u < x_{max}$

∴ Under reinforced section

$M.R = 0.36f_{ck}\ bx_u\ (d - 0.42x_u)$

$= 0.36 \times 20 \times 250 \times 80.65\ (360 - 0.42 \times 80.65)$

$M_u = 47.5$ kN-m

$M_u = P \times a$

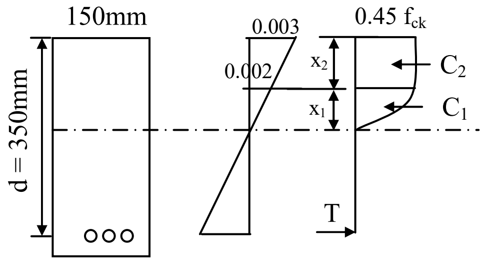
$47.5 = P \times a$

$$P = \frac{47.5}{1.5}$$

P = 31.6 kN

08. Ans: 51 kN-m

Sol:



$x_{u\ max} = 0.48 \times d$

$= 0.48 \times 350$

$= 168$ mm

$M_{u\ limit} = 0.36f_{ck}\ b\ x_{u\ max}\ (d - 0.42\ x_{u\ max})$

$= 0.36 \times 20 \times 150 \times 168(\ 350 - 0.42 \times 168)$

$= 50.70 \times 10^6$ N-m

$= 51$ kN-m

09. Ans: 503 mm^2

Sol: C = T

$0.36\ f_{ck}\ b\ x_{u\ max} = 0.87\ f_y\ A_{st}$

$$A_{st} = \frac{0.36 f_{ck}\ bx_{u\ max}}{0.87 \times f_y}$$

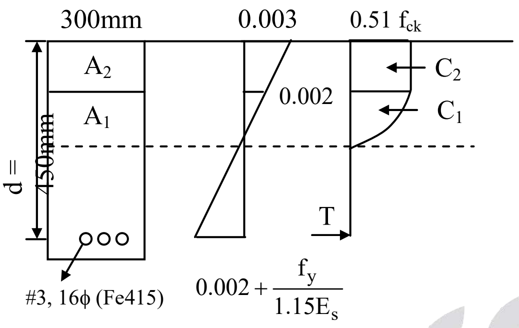
$$= \frac{0.36 \times 20 \times 150 \times 168}{0.87 \times 415}$$

$= 502.53$ mm^2

$A_{st} \simeq 503$ mm^2

10. Ans: 196 mm

Sol:



$x_{umax} = 0.003 \qquad \rightarrow (1)$

$$(d\text{-}x_{umax}) = \left(0.002 + \frac{f_y}{1.1E_s}\right)$$

$$450 - x_{umax} = \left(0.002 + \frac{415}{1.1\times 2\times 10^5}\right) \qquad \rightarrow (2)$$

$$\frac{450 - x_{u\max}}{x_{u\max}} = \frac{0.002 + \frac{415}{1.1\times 2\times 10^5}}{0.003}$$

On solving

$x_{u\ max} = 196.04$ mm

$= 196$ mm

Chapter- 4
Limit State Design– Doubly Reinforced Beams

01. Ans: (c)

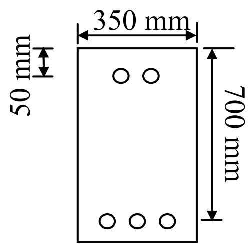
Sol: BM = 300 kN-m

Concrete, $M_{15} = f_{ck} = 15$

Steel, $f_y = 415$

f_{sc} =353.7 MPa

Effective Cover d′ = 50mm



In LSM, we have to use

Factored moment

$M_u = M \times \gamma_f$

Use $\gamma_f = 1.5$

$= 300 \times 1.5 = 450$ kN-m

To calculate $M_{u\ limit}$

$M_{u\ limit} = 0.138\ f_{ck}\ bd^2$

$= 0.138 \times 15 \times 350 \times (700)^2$

$M_{u\ limit} = 355$ kN-m

$M_u = 450$ kN-m

$\therefore\ M_u > M_{ulimit}$

So we need to use ‘DRB’

$M_{ulimit} = 0.87 f_y\ A_{st}\ (d\text{-}0.42\ x_{u\ max})$

$355\times10^6 = 0.87\times415\times A_{st}(700–0.42\times0.48\times700)$

$A_{st} = 1759.31\ mm^2$

for extra moment we need to provide tensile steel & comp. steel

$M_u – M_{u\ limit} = 0.87 f_y(d–d')A_{st2}$

$(450–355)\times10^6 = 0.87 \times 415 A_{st2}(700–50)$

$= 234682.5 A_{st2}$

$A_{st2} = 404.8 \simeq 405\ mm^2$

$A_{st} = A_{st1} + A_{st2} = 2165\ mm^2$

Now our purpose is to calculate 'A_{sc}'

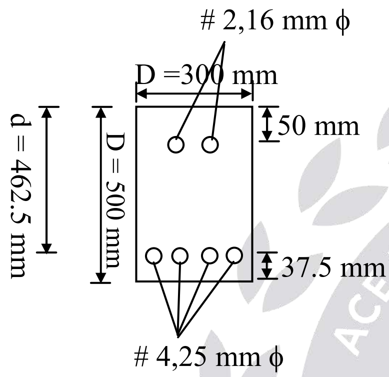
$M_u - M_{ulimit} = f_{sc}A_{sc}(d-d')$

(or) $f_{sc}\,A_{sc} = 0.87\,f_y\,A_{st2}$

$A_{sc} = 413.2\text{ mm}^2$

02. Ans: 271 kN-m

Sol:



b = 300 mm, D = 500 mm, d = 462.5 mm

$f_{ck} = 25\text{ N/mm}^2$, $f_y = 415\text{ N/mm}^2$,

$f_{sc} = 0.8566\,f_y$

$A_{st} = 4\times\frac{\pi}{4}\times 25^2 = 1963.495\text{ mm}^2$

$A_{sc} = 2\times\frac{\pi}{4}\times 16^2 = 402.12\text{ mm}^2$

$\Rightarrow C = T$

$\Rightarrow C_1 + C_2 = T$

$0.36\times f_{ck}\,b\,x_u + f_{sc}A_{sc} = 0.87\,f_y\,A_{st}$

$0.36\times 25\times 300\times x_u + (0.8566\times 415)\times 402.12$

$= 0.87\times 415\times 1963.495$

$x_u = 209.618\text{ mm}$

$x_{u\,max} = 0.48\times d$

$= 0.48\times 462.5 = 222\text{ mm}$

$x_u < x_{u\,max}$

$\therefore$ under reinforced section.

$M_u = 0.36f_{ck}.b.x_u(d-0.42x_u) + f_{sc}A_{sc}(d-d^1)$

$= 0.36\times 25\times 300\times 209.6$

$(462.5-0.42\times 209.6) + (0.8556\times 415)$

$\times 402.12\,(462.5-50)$

$= 270.9\text{ kN-m}$

03. Ans: 18.82 kN/m

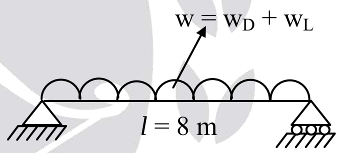
Sol: Working /line moment,

$M = \frac{270.9}{1.5} = 180.6\text{kN}-\text{m}$

Self weight of beam, $w_D = (\gamma_c)\,b\times D$

$= (25\text{kN/m}^3)\times(0.3\times 0.5)$

W = 3.75 kN/m



$M = \frac{(w_D + w_L)\times l^2}{8}$

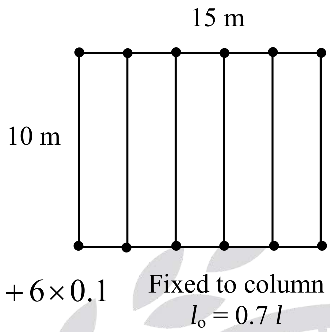
$180.6 = \frac{(3.75 + w_L)\times 8^2}{8}$

$w_L = 18.825\text{ kN/m}$

Chapter- 5
Limit State Design - Flanged Beams

01. Ans: (c)

Sol: For T-beams,



$$b_f = \frac{l_0}{6} + b_w + 6D_f$$

$$= \frac{0.7\times10}{6} + 0.25 + 6\times0.1$$

$= 2.01$ m $\not>$ c = 3 m

$\therefore$ $b_f = 2.01$ m

02. Ans: (d)

Sol: L – beam

$$B_f = \frac{l_0}{12} + b_w + 3D_f$$

$$= \frac{10}{12} + 0.25 + 3\times0.1$$

$= 1.38$ m $\not>$ c = 3 m

$\therefore$ $b_f = 1.38$ m

03. Ans: (d)

Sol: D_f = 100 mm, b_w = 300mm, d = 500mm, c = 3m, l = 6m, l_0 = 3.6 m, b_f = ?

$$b_f = \frac{l_0}{6} + b_w + 6D_f \not> c$$

$$= \frac{3.6}{6} + 0.3 + 6\times0.1$$

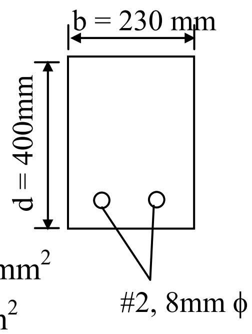
$= 1.5$ m $\not>$ c = 3 m

$= 1.5 \times 1000$ m = 1500 mm

Chapter- 6
Limit State of Collapse - Shear

01. Ans: (b)

Sol:



V_u = 120 kN

f_{ck} = 20 N/mm^2

Main steel, f_y = 415 N/mm^2

Stirrups, f_y = 250 N/mm^2

τ_c = 0.48 N/mm^2

i) 8mm–2 legged Stirrups

$$A_{sv} = 2\times\frac{\pi}{4}\times8^2$$

$= 100.53$ mm^2

$$\tau_v = \frac{V_u}{b\times d} = \frac{120\times10^3}{400\times230}$$

$= 1.3$ N/mm^2

$\tau_v \le \tau_{c\,max}$ – safe in shear

ii) $\tau_v > \tau_c$ – not safe in shear reinforcement

Minimum shear reinforcement is required

$$V_{us} = \frac{(0.87f_y)A_{sv}\times d}{S_v}$$

$V_{us} = V_u - \tau_c b.d$

$= 120\times10^3 - 0.48\times400\times230$

$= 75840$ N = 75.84 kN

$$75.84\times10^3 = \frac{0.87\times250\times100.53\times400}{S_v}$$

S_v = 115 mm c/c

02. Ans: (c)

Sol: T =10.90 kN-m

$V_e = V_u + \frac{1.6T_u}{b}$

$= 120\times10^3 + \frac{1.6\times10.90\times10^6}{230}$

V_e= 196 kN

Design shear force

$V_{us} = V_e - \tau_c.b.d$

$= 196 \times 10^3 - 0.48 \times 230 \times 400$

$V_{us} = 151.84 \times 10^3$N

= 151.84 kN

03. Ans: (d)

Sol: b = 230 m, d = 450 mm

V_u = 50 kN

$f_{ck} = 20$ N/mm^2

$f_y = 250$ N/mm^2

$\tau_{c\,max}$ = 2.8 MPa, τ_c = 0.75 MPa.

$\tau_v = \frac{V_u}{bd} = \frac{50\times10^3}{230\times450}$ = 0.483 MPa

$\tau_v < \tau_{c,max}$ safe in shear.

Provide minimum shear reinforcement.

$\frac{A_{sv}}{bS_v} = \frac{0.4}{0.87f_y}$

$A_{sv} = 2\times\frac{\pi\times8^2}{4}$ = 100.53 mm^2

$S_v = \frac{100.53\times0.87\times250}{0.4\times230}$

= 237.7 mm c/c

$S_v \ngtr 0.75\,d = 0.75 \times 450 = 337.5$ mm

$S_v \ngtr 300$ mm

∴ Provide spacing of 230 mm c/c

04. Ans: (c)

V_u = 100 kN

$\tau_v = \frac{V_u}{b\times d} = \frac{100\times10^3}{230\times450}$ = 0.966

$\tau_v < \tau_{c\,max}$ – shear reinforcement safe

$\tau_v > \tau_c$ not safe in shear reinforcement

Shear reinforcement is required.

Design shear force for shear reinforcement

$V_{us} = V_u - \tau_c\,bd$

$= 100\times10^3 - 0.75\times230\times450$

= 22.375 kN

For vertical stirrups,

$V_{us} = \frac{0.87f_yA_{sv}d}{S_v}$

$S_v = \frac{0.87\times250\times100.53\times450}{22.375\times10^3}$

= 439.75 mm

Min spacing:

i. 439.75 mm

ii. 0.75d = 0.75 ×450 = 337.5 mm

iii. 300 mm

iv. Spacing for min shear reinforcement

$\frac{A_{sv}}{bS_v} = \frac{0.4}{0.87f_y} \Rightarrow S_v = 237.7$ mm

Provide min spacing of 230 mm c/c.

05. Ans: (c)

Sol: $V_u = 150$ kN

$$\tau_v = \frac{150\times10^3}{230\times450} = 1.449 \text{ MPa}$$

$\tau_v < \tau_{c,max}$ – safe in shear reinforcement

$\tau_v > \tau_c \rightarrow$ Shear reinforcement is required.

Design shear force,

$V_{us} = V_u - \tau_c bd$

$= 150\times10^3 - 0.75\times230\times450$

$= 72.375$ KN

Shear force taken by bent-up bars.

$V_{us1} = 0.87 f_y A_{sv} \sin\alpha$

$$= 0.87\times415\times2\times\frac{\pi}{4}\times16^2\times\sin45^o$$

$= 102.66$ kN

$\not> 0.5\, V_{us} = 36.18$ kN

$\therefore\ V_{us1} > 0.5\, V_{us}$

As per IS: 456 ; $V_{us1} \not> 0.5\, V_{us}$. In this case V_{us1} is exceeding $0.5\, V_{us}$. Therefore limit V_{us1} as 36.18 kN, the remaining S.F i.e 36.195 kN should be resisted by vertical stirrups.

Vertical stirrups:

For $V_{us2} = 36.195$ kN

$$36.195\times10^3 = \frac{0.87 f_y A_{sv}.d}{S_v}$$

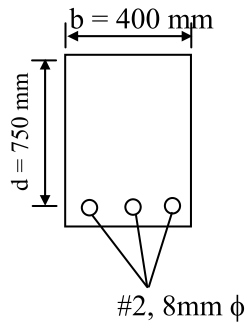
$$S_v = \frac{0.87\times250\times\left(2\times\frac{\pi}{4}\times8^2\right)\times450}{36.195\times10^3}$$

$= 271.708$ mm

Provide minimum center to center spacing of 230 mm c/c

06. Ans: (a)

Sol: Beam -P



$\tau_{c\,max} = 2.1$ MPa

$f_{ck} = 30$ N/mm^2

$\tau_c = 0.75$ MPa

$V_u = 400$ kN

$$\tau_v = \frac{V_u}{b\times d} = \frac{400\times10^3}{750\times400}$$

$\tau_v = 1.33$ N/mm^2

i) $\tau_v < \tau_{c\,max}$ –shear reinforcement safe

ii) $\tau_v > \tau_c$ Minimum shear reinforcement is required

$V_{us} = V_u - \tau_c bd$

$= 400\times10^3 - 0.75\times400\times750$

$V_{us} = 175$ kN

Beam –Q

$V_u = 750$ kN

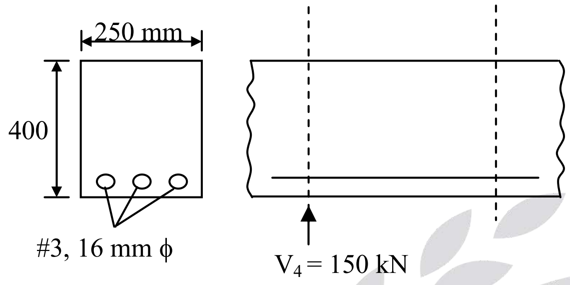
$$\tau_v = \frac{V_u}{b\times d} = \frac{750\times10^3}{750\times400} = 2.5\,\text{N/mm}^2$$

$\tau_v > \tau_{c\,max}$

The beam is not safe in shear. It should be revised.

Chapter- 7
Bond

01. Ans: (c)



Flexural bond:

Steel in tension (sagging moment)

$L_d \ngtr \frac{M_1}{V_u} + l_0 \rightarrow$ continuous beam

$l_0 = 12\ \phi = 12 \times 16$

$\left.\begin{array}{r} = 192\text{mm} \\ d = 400\text{mm} \end{array}\right\}$ Which is greater

Take $l_0 = 400$ mm

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 250 \times 16}{4 \times 1} = 870$ mm

$x_{u,\,max} = 0.53 \times 400 = 212$ mm

$x_u = \frac{0.87 \times 250 \times 3 \times \frac{\pi}{4} \times 16^2}{0.36 \times 15 \times 250}$

$= 97.18$ mm

$x_u < x_{u,max} \rightarrow$ Under reinforcement section.

$M_1 = 0.36 \times 15 \times 250 \times 97.18\ (400 - 0.42 \times 97.18)$

$= 47.12 \times 10^6$ N-mm

$L_d \ngtr \frac{47.12 \times 10^6}{150 \times 10^3} + 400 = 714.15$ mm

$L_d > 714.15$

not safe in bond.

02. Ans: (d)

Sol: $\phi = 12$mm

$f_y = 415$ N/mm^2

$f_{ck} = 30$ N/mm^2, $\tau_{bd} = 2.4$ MPa

$L_d = \frac{\phi\sigma_s}{\tau_{bd} \times 4}$

$= \frac{12 \times 0.87 \times 415}{(1.6 \times \tau_{bd}) \times 4} = 282.0703$

$L_d = 282.0703$ mm

L_d with 90° bend $= 282.0703 - 8\phi$

$= 282.0703 - 8 \times 12$

$= 186.1$ mm

03. Ans: (d)

Sol: Axially loaded short column

$\phi = d = 20$mm, spliced = 16 mm

$f_y = 415$ N/mm^2

$\tau_{bd} = 1.2$ MPa

$\left.\begin{array}{r} \text{lap} \nless l_d \\ \nless 24\phi \end{array}\right\}$ max

Use smaller diameter $\Rightarrow \phi = 16$ mm

$L_d = \frac{\phi\sigma_s}{4 \times \tau_{bd}} = \frac{16 \times 0.87 \times 415}{1.25 \times 4 \times 1.2 \times 1.6}$

$= 601.75$ mm

Lap length $\nless L_d = 601.75$ mm

$\nless 24\ \phi = 384$ mm

Use maximum, i.e., 601.75 mm

04. Ans: (d)

Sol: 1) Pull out (bond fail)

$P_1 = \tau_{bd}[\pi D l]$

2) Breaking of steel bar

$P_2 = \sigma_{st}\left[\frac{\pi}{4}\times D^2\right]$

} minimum

05. Ans: 46.8

Sol: $f_{ck} = 20$ N/mm^2,

$\tau_{bd} = 1.2$ MPa ↑60% - HYSD bars

Steel bar is in tension

$$L_d = \frac{\phi\sigma_s}{4\times\tau_{bd}} = \frac{\phi\times 360}{4\times 1.6\times 1.2} = 46.8\phi$$

06. Ans: 290 mm

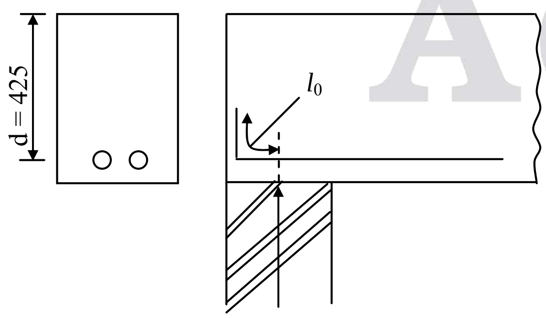
Sol: Given , $V_u = 220$ kN

$A_{st} = 2\times\frac{\pi}{4}\times 16^2 = 402.12$ mm^2

b = 250 mm, d = 425 mm

Fe 415 ,M_{20} ,$\tau_{bd} = 1.2$ MPa

l_0 = ? for 90^o bond



$$L_d = \frac{0.87 f_y\phi}{4\tau_{bd}} = \frac{0.87\times 415\times 16}{4\times 1.6\times 1.2}$$

$= 752.1875$ mm

L_d (req) = 752.1875 – 8 × 16

= 624.1875 mm

$x_{u\,max} = 0.48\times 425 = 204$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}$$

$$= \frac{0.87\times 415\times 402.12}{0.36\times 20\times 250}$$

= 80.65 mm

$x_u < x_{u\,max} \rightarrow$ Under reinforced section

$M_1 = 0.87\, f_y A_{st}\,(d - 0.42\,x_u)$

$= 0.87\times 415\times 402.12\,(425 - 0.42\times 80.65)$

$= 56.78\times 10^6$ N –mm

$$L_d = \frac{1.3M_1}{V} + l_0$$

$$624.1875 = 1.3\times\frac{56.78\times 10^6}{220\times 10^3} + l_0$$

$l_0 = 288.66$ mm

Minimum extension beyond centre of support = 290 mm

Chapter- 8
Limit State of Collapse - Torsion

01. Ans: (d)

Sol: i) size – 300 × 1000 mm

$V_u = 150$ kN; $M_u = 150$ kN

$T_u = 30$ kN -m

$$V_e = V_u + \frac{1.6T_u}{b}$$

$$= 150\times10^3 + \frac{1.6\times30\times10^6}{300} = 310 \text{ kN}$$

$M_{e1} = M_u + M_T$

$$= M_u + \frac{T_u\left[1+\frac{D}{b}\right]}{1.7}$$

$$= 150 + \frac{30\left[1+\frac{1000}{300}\right]}{1.7} = 226.47 \text{ kN-m}$$

02. Ans: (d)

b = 300mm, D = 600 mm

V = 100 kN, M = 100 kN-m

T = 34 kN-m

$M_{e1} = M_u + M_T$

$$= M_u + \frac{T_u\left[1+\frac{D}{b}\right]}{1.7}$$

$$= 100 + \frac{34\left[1+\frac{600}{300}\right]}{1.7}$$

=160 kN-m

03. Ans: (a)

Sol: T = 68 kN-m

$M_{e2} = M_T - M_u$

If $M_T < M_u$ then no need of A_{sc}

$$M_T = \frac{T_u\left(1+\frac{D}{b}\right)}{1.7} = \frac{68\left(1+\frac{600}{300}\right)}{1.7}$$

$= 120$ kN-m

$M_T > M_u$ – additional compression steel is required for M_{e2} i.e $M_{e2} = M_T - M_u$

$= 120 - 100$

$= 20$ kN-m

04. Ans: (a)

Sol: b = 500, D = 700 mm

d = 35 mm, V = 15 kN

M = 100kN-m, T = 10 kN-m

$\tau_c = 1.5$ MPa

If $\tau_{ve} \not> \tau_c$ ignore torsion

If $\tau_{ve} > \tau_c$ consider torsion for A_{st}

$V_e = V_u + V_T$

$$= V_u + 1.6\frac{T_u}{b}$$

$$= 15 + 1.6\left(\frac{10}{0.5}\right)$$

$= 47$ kN

$$\tau_{ve} = \frac{V_e}{b.d} = \frac{47\times10^3}{500\times(700-35)} \approx \frac{47}{0.5\times0.7}$$

$= 0.14$ MPa

$\tau_{ve} < \tau_c$

∴ Design BM for A_{st} is M_u only

$M_u = 100$ kN-m

05. Ans: (d)

Sol: V = 20 kN, T = 9 kN-m

b = 300 mm, M = 200 kN-m

gross depth = 425 mm

cover = 25 mm

$V_e = V_u + V_T$

$$= V_u + 1.6\frac{T_u}{b} = 20 + 1.6\left(\frac{9}{0.3}\right)$$

= 68 kN

06. Ans: (b)

Sol: As $\tau_{ve} < \tau_c$

$T_u = 0$

$M_{e1} = M_u = 200$ kN-m

A_{st} based on M_u only

Chapter- 10
Limit State of Collapse-Compression

01. Ans: (c)

Sol: b = 300 mm

d = 600 mm

$f_y = 415$ MPa

$f_{ck} = 20$ MPa

$P_u = 0.40 f_{ck} A_c + 0.67\, f_y A_{sc}$

$A_{sc} = 0.8\%\, A_g$

$$= \frac{0.8}{100}(300 \times 600) = 1440 \text{ mm}^2$$

$A_c = A_g - A_{sc}$

$= 300 \times 600 - 1440$

$= 178560 \text{ mm}^2$

$P_u = 0.4 \times 20 \times 178560 + 0.67 \times 415 \times 1440$

$P_u = 1829$ kN

02. Ans: (d)

Sol: d = 300 mm; $f_{ck} = 20$ N/mm^2

$f_y = 415$ N/mm^2;

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

$$A_{sc} = \left(\frac{\pi}{4} \times 300^2\right) \times \frac{1}{100} = 706.85 \text{mm}^2$$

$A_c = A_g - A_{sc}$

$$= \left(\frac{\pi}{4} \times 300^2\right) - 706.85$$

$= 69978.98 \text{ mm}^2$

$P_u = 1.05(0.4 \times 20 \times 69978.98 + 0.67 \times 415 \times 706.85)$

= 794.19 kN

03. Ans: (d)

Sol: $A_g = 300 \times 300$ mm

$f_{ck} = 20$ N/mm^2,

$A_c = A_g$ (neglecting A_{sc})

$f_y = 415$ N/mm^2

$A_{sc} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.63$

$P_u = 0.4 \times 20 \times 300 \times 300 + 0.67 \times 415 \times 1256.63$

$= 1069$ kN

04. Ans: (d)

Sol: $m = \frac{E_{strong}}{E_{weak}} = \frac{E_{steel}}{E_{conc}}$

compatability condition for composite (RCC) members

$\delta_s = \delta_c$

$$\frac{P_s l}{A_s E_s} = \frac{P_c l}{A_c E_c}$$

$$\frac{P_s}{P_c} = \frac{A_s}{A_c}\left(\frac{E_s}{E_c}\right) = \frac{1\% A_c}{A_c} \times 10 = 10\%$$

Chapter- 11
Footings

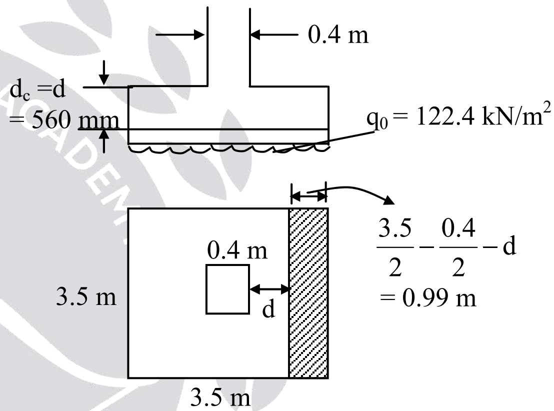
01. Ans: (b)

Sol: B = 3.5m

column size = 400 mm

d = 560 mm

$q_0 = 122.4$ kN/m^2



For one way shear

$V_u = q_0$[hatched area]

$= 122.4\,[0.99 \times 3.5]$

$= 425$ kN

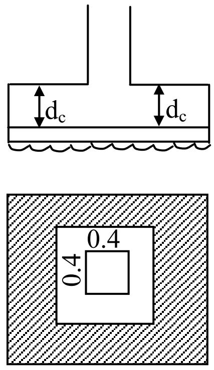
$$\tau_v = \frac{V_u}{b.d_c} = \frac{425 \times 10^3}{3500 \times 560}$$

$= 0.22$ N/mm^2

$= 0.22$ MPa

02. Ans: (c)

Sol:



$$B = 0.4 + \frac{0.56}{2} + \frac{0.56}{2}$$

$= 0.96$

$V_u = q_0$[hatched area]

$= 122.4 \times [3.5^2 - 0.96^2]$

$= 1386$ kN

$$\tau_v = \frac{V_u}{pd} = \frac{1386 \times 10^3}{(4 \times 960)(560)}$$

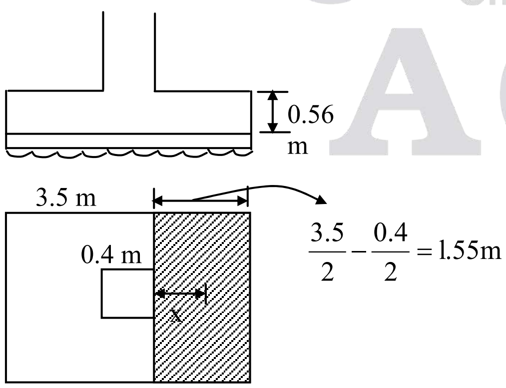
$= 0.64$ MPa

V_u is more for 2-way

2-way shear is critical

03. Ans: (a)

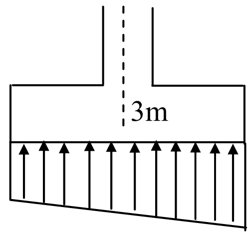
Sol:

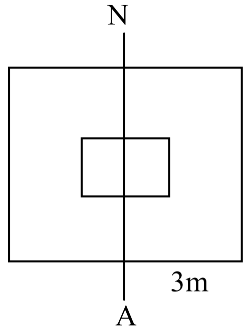


$M_u = q_0$[hatched area $\times \bar{x}$]

$$= 122.4\left[3.5 \times 1.55 \times \frac{1.55}{2}\right] = 515 \text{ kN}$$

04. Ans: (a)





$$\left.\begin{matrix}\sigma_{max} \\ \sigma_{min}\end{matrix}\right\} = \frac{P}{A} \pm \frac{M}{Z}$$

$$= \frac{450}{3 \times 2} \pm \frac{60}{\left(\frac{2 \times 3^2}{6}\right)}$$

$\sigma_{max} = 95$ kN/m^2 compression

$\sigma_{min} = 55$ kN/m^2 compression

As per IS 456 –2000 the assumed pressure distribution below the footing is uniform

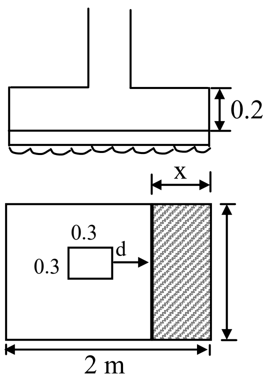
05. Ans: (a)

Sol: $l = 2$m ; d = 200 mm

column size = 300 ×300 mm

$q_0 = 320$ kN

$\tau_v = ?$



$$q_0 = \frac{320}{2 \times 2} = 80 \text{ kN/m}^2$$

$$x = \frac{2}{2} - \frac{0.3}{2} - 0.2$$

$= 1 - 0.15 - 0.2$

$= 0.65$

One way shear $V_u = q_0$ [hatched area]

$= 80[0.65 \times 2] = 104$ kN

$$\tau_v = \frac{V_u}{bd_c} = \frac{104 \times 10^3}{2000 \times 200} = 0.26$$

Chapter- 14
Analysis of Prestressed Concrete Members

01. Ans: (b)

Sol: Prestressing force, P = 2500 kN

Effective span, l = 10 m

udl on the beam, w = 40 kN/m

For load balancing

$$P.e = \frac{w\ell^2}{8}$$

$$(2500)(e) = \frac{(40)(10)^2}{8}$$

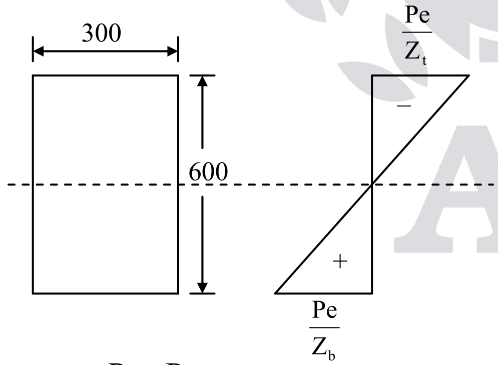
e = 0.2 m = 200 mm

02. Ans: (b)

Sol: $\gamma_c = 24\ kN/m^3$

$\sigma_t = 2$ MPa

$\sigma_b = 20$ MPa



$$\sigma_b = \frac{P}{A} + \frac{Pe}{z} \text{------------------ (1)}$$

$$\sigma_t = \frac{P}{A} - \frac{Pe}{z} \text{------------------ (2)}$$

Adding (1) & (2)

$$20 = \frac{P}{A} + \frac{Pe}{z}$$

$$-2 = \frac{P}{A} - \frac{Pe}{z}$$

$$18 = \frac{2P}{A}$$

P = 1620 kN

$$\sigma_b = \frac{P}{A} + \frac{Pe}{z}$$

$$20 = \frac{1620\times10^3}{300\times600} + \frac{1620\times10^3\times6\times e}{300\times600^2}$$

e = 122 mm

e ≃ 135 mm

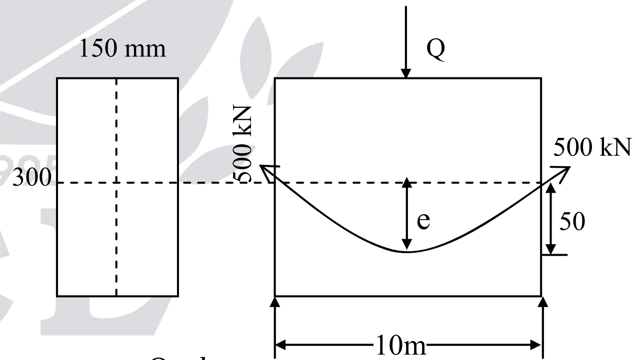
03. Ans: (a)

Sol: 150 × 300 mm

l = 10 m, e at support = 0mm

e = 50 mm (center), P = 500 kN

Q =? (at center of span)



$$Pe = \frac{Q\times l}{4}$$

$$500\times\frac{50}{1000} = \frac{Q\times10}{4}$$

100 = Q × 10

Q = 10 kN

04. Ans: (b)

Sol: Self weight

$w_D = \gamma_c \times b \times D$

$= (24\ kN/m^3) \times 0.15 \times 0.3$

$= 1.08\ kN/m$

P – line at upper kern point ($\sigma_b = 0$)

$$M_D = \frac{w_D l^2}{8} = \frac{1.08\times10^2}{8} = 13.5$$

$$\sigma_b = 0 = \frac{P}{A} + \frac{Pe}{z} - \frac{M_D}{z} - \frac{M_L}{z}$$

$$= \frac{500\times10^3}{300\times150} + \frac{500\times10^3\times50}{\left(\frac{150\times300^2}{6}\right)} - \frac{13.5\times10^6}{\left(\frac{150\times300^2}{6}\right)} - \frac{M_L}{\left(\frac{150\times300^2}{6}\right)}$$

$$0 = 11.11 + 11.11 - 6 - \frac{M_L}{225\times10^4}$$

$M_L = 16.22 \times 225 \times 10^4$

$M_L = 36.5$ kN-m,

$$M_L = \frac{Ql}{4}$$

$$36.5 = \frac{Q\times10}{4}$$

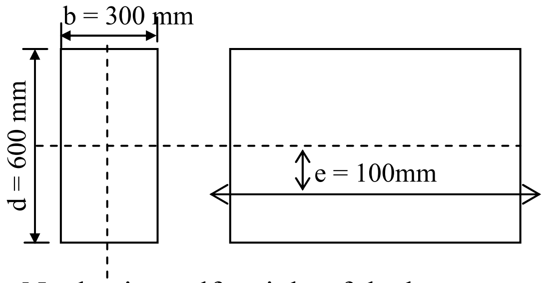
$146 = Q \times 10$

$Q = 14.6$ kN

05. Ans: (c)

Sol: $l = 6$ m, b = 300 mm, d = 600 mm

e = 100 mm, P = 1000 kN,



Neglecting self weight of the beam

$$\sigma_b = \frac{P}{A} + \frac{Pe}{z}$$

$$= \frac{1000\times10^3}{300\times600} + \frac{1000\times10^3\times100}{\left(\frac{300\times(600)^2}{6}\right)}$$

$= 5.55+5.55 = 11.11$ MPa

06. Ans: (b)

Sol: b = 200 mm, D = 250 mm

A = 500 mm^2, P = 1000 MPa

m = 10

$\varepsilon_s = \varepsilon_e$

$$\frac{\sigma_s}{E_s} = \frac{\sigma_c}{E_c}$$

$$\sigma_c = \sigma_s\left(\frac{\varepsilon_c}{\varepsilon_s}\right) = \frac{\sigma_s}{m} = \frac{1000}{10}$$

$\sigma_e = 100$ MPa

Prestressing force on steel = $\sigma_s.A_s$

$= 1000 \times 500 = 500 \times 10^3$N

Compression force in concrete = 500 kN

$= \sigma_c .A_c$

Compression stress in concrete $\sigma_c = \frac{P_c}{A_c}$

$$= \frac{500\times10^3}{200\times250} = 10\ MPa$$

Chapter- 15
Losses of Prestress

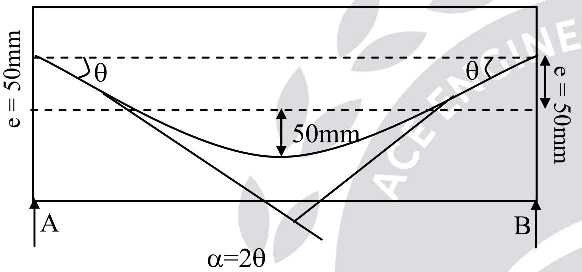
01. Ans: (b)

Sol: l = 10 m, b = 100 mm,

D = 300 mm A = 200 sq-mm,

e = 50 mm, μ = 0.35;

k = 0.0015 per m



Initial stress in wires = 1200 MPa

Loss of stress in wires = σ(μα + kx)

= 1200[0.35 × α + 0.0015×10]

From equation of parabola

$$\theta = \frac{4\times0.1}{10} = 0.04 \text{ radians}$$

$\alpha = 2 \times \theta = 0.08$

Loss = 1200[0.35 × 0.08 + 0.0015 × 10]

Loss of stress = 51.6 MPa

$$\%\text{ loss of stress} = \frac{51.6}{1200}\times100$$

$= 4.28 \simeq 4.3\%$

02. Ans: (b)

Sol:

Tensioning from both the ends % loss of stress

$$= \frac{\%\text{loss of stress}}{2} = \frac{4.28}{2} = 2.15$$

03. Ans: (b)

Sol: Straight tendon tensioned from one end

Loss of stress in wires = σ[μα + kx]

($\because$ α = 0)

1200(0.35 × (0) + 0.0015 × 10) = 18 MPa

$$\%\text{ of loss} = \frac{18}{1200}\times100$$

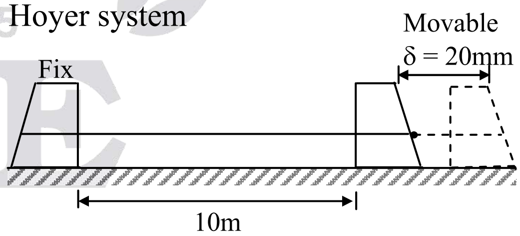
= 1.5%

If tensioned from two ends

$$\frac{\%\text{of loss}}{2} = \frac{1.5}{2} = 0.75\%$$

04. Ans: (c)

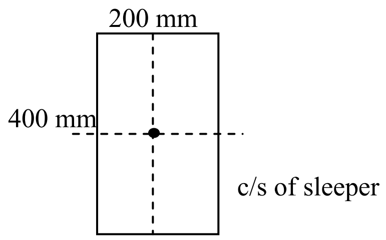
Sol: Hoyer system



$$\delta = \frac{PL}{AE} \qquad \left(\text{as } \sigma = \frac{P}{A}\right)$$

Prestress induced in steel wire, $\sigma = \frac{\delta E}{L}$

$$\sigma = \frac{20\times2\times10^5}{10,000} = 400 \text{ MPa}$$



Eccentricity of Prestress, e = 0

Prestressing force in steel wire = P = $\sigma_s.A_s$

= 400 × 500 mm^2

= 200 kN

$$f_c = \frac{P}{A} + \frac{Pe}{I}(e) = \frac{200\times10^3}{200\times400} = 2.5\text{ MPa}$$

Loss due to elastic shortening

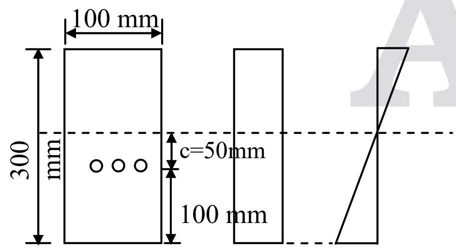
$$= m \times f_c = \left(\frac{E_s}{E_c}\right) f_c$$

$$\sigma = \left(\frac{200{,}000}{20{,}000}\right)\times 2.5 = 25\text{ MPa}$$

$$\%\text{ loss of Prestress} = \frac{25}{400}\times100 = 6.25\%$$

05. Ans: (d)

Sol:



$$f_c = \frac{P}{A} + \frac{P}{I}(e)^2$$

Initial stress in steel wire =1200 MPa

Prestressing force in each steel wire P = $\sigma_s.\ A_s$

P = 1200 × 50 = 60 kN

$$f_c = \frac{60\times10^3}{100\times300} + \frac{60\times10^3}{\left(\frac{100\times300^3}{12}\right)}\times(50)^2$$

f_c = 2.66 MPa

Simultaneous tensioning = loss of prestress is zero

06. Ans: (a)

Sol: Successive tensioning of the 3 cables

$$= \frac{n(n-1)}{2}(m.f_c)$$

$$= \frac{3(3-1)}{2}(6\times2.66)$$

= 48.0 MPa

$$\%\text{ of loss} = \frac{48.0}{1200}\times100 = 4\%$$

(or) For pretensioning system

Loss = n(m×f_c)

= 3(6 × 2.66) = 48.0 MPa

07. Ans: (c)

Sol: Anchorage slip = 3 mm

l = 30 m, σ = 1200 MPa

E = 2.1 × 10^5 MPa

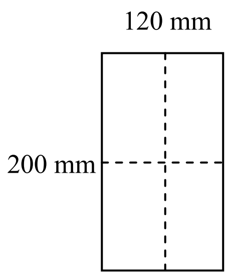
$$E = \frac{\delta E}{l} = \frac{3\times2.1\times10^5}{30\times10^3}$$

σ = 21 MPa

$$\%\text{ of loss} = \frac{21}{1200}\times100 = 1.73\%$$

08. Ans: (b)

Sol:



P = 150 kN, e = 20 mm

A = 187.5 mm^2

$E_s = 2.1 \times 10^5$ MPa

$E_c = 3.0 \times 10^4$ MPa

$$f_c = \frac{P}{A} + \frac{Pe}{I}.e$$

$$= \frac{150\times10^3}{187.5} + \frac{150\times10^3\times20^2}{\left(\frac{120\times200^3}{12}\right)}$$

= 800 + 0.75 MPa

f_c = 800.75 MPa

loss due to elastic shortening = $m.f_c$

$$= \left(\frac{E_s}{E_c}\right).f_c$$

$$= \frac{2.1\times10^5}{3.0\times10^4}\times7 = 4.9$$

Percentage loss in the prestressing steel due to elastic deformation

$$= \frac{4.9}{800.75}\times100$$

= 6.12%

09. Ans: (c)

Sol: $\varepsilon = \varepsilon_{shrink} + \varepsilon_{creep}$

= 0.0008

Loss of prestress on steel = $\varepsilon \times E_s$

= $0.0008 \times 200 \times 10^3$

= 160 MPa

Stress remaining after loss = Initial stress–Loss

= 200 – 160

= 40 MPa