CBCS SCHEME

USN 1CR17CV009

17CV/CT51

Fifth Semester B.E. Degree Examination, Dec.2019/Jan.2020
Design of RC structural Elements

Time: 3 hrs. Max. Marks: 100

Note: *Answer FIVE full questions, choosing ONE full question from each module.*

Module-1

1 a. Define characteristic strength of materials and characteristic loads with sketches. (04 Marks)

b. Distinguish between : (i) Balanced section, (ii) Under-reinforced section and (iii) Over reinforced section with sketches. Which section is preferable and why? (10 Marks)

c. Derive an expression for $\bar{y}$, the depth of centre of compressive force from the extreme compressive fiber for a singly reinforced rectangular beam section. (06 Marks)

OR

2 a. What are the assumption made in the limit state of design for collapse in flexure in singly reinforced beam section. (04 Marks)

b. A simply supported beam has a rectangular section of size $300mm \times 650mm$ and carries a uniformly distributed load of 15 kN/m over a clear span of 5.5 m. It is reinforced with 4 bars of 25 mm diameter bar. Use M25 concrete and Fe 500 grade HYSD bars. Compute short and long term deflections of the beam. (16 Marks)

Module-2

3 a. A reinforced concrete Cantilever beam 2 m long and having cross section of size $240mm \times 400mm$ is reinforced with 4 bars of 16 mm diameter at top on tension side. The beam is designed to support a concentrated load of 3 kN at the free end in addition to uniformly distributed load on it. Determine the permissible uniformly distributed load, the beam can carry on it. Use M20 grade concrete and Fe 415 grade steel. (10 Marks)

b. A doubly reinforced beam section is 300 mm wide and 500 mm deep to the centre of tensile reinforcement. It is reinforced with compression reinforcement of 300 mm^2 at an effective cover of 50 mm and tension reinforcement of 1800 mm^2. Determine the safe moment of resistance of the section. M20 grade concrete and Fe 500 grade steel is used. (10 Marks)

OR

4 a. A singly reinforced concrete slab 150 mm thick is reinforced with 10 mm diameter bars at 200 mm centres located at an effective depth of 125 mm. M20 grade concrete and Fe415 grade HYSD bars are used. Estimate the ultimate moment of resistance of the section. (04 Marks)

b. A rectangular RC section of size 300×600mm effective is reinforced with 4 bars of 25 mm diameter HYSD bar of grade Fe 415. Two of the tension bars are bent at 45° near the support section. The beam is provided with double legged vertical links of 8 mm diameter at 150 mm centres near supports. Using M-25 grade concrete, compute the ultimate shear strength of the support section. (08 Marks)

c. A simply supported T-beam of depth of 450 mm has a flange width of 1000 mm and depth of 120 mm. It is reinforced with 6 – 20 mm diameter bars on tension side with a clear cover of 30 mm. M20 grade concrete and Fe415 grade steel are used. Calculate moment of resistance of beam. Take, $b_w = 300$ mm. (08 Marks)

Module-3

5 a. Design a singly reinforced beam simply supported at its two ends for flexural reinforcement. The clear span of beam is 5.6 m, the intensity of uniformly distributed superimposed dead and live loads are 18 kN/m and 26 kN/m. Use M-25 grade of concrete and HYSD steel of Fe500 grade. The beam should meet the durability requirement for exposed conditions of 'Severe' atmospheric and fire resistance of one and half hour. (08 Marks)

b. Design a doubly reinforced rectangular beam of size 300mm × 600mm simply supported at both ends. Check for deflection need not be calculated. The effective span is 5.6 m. The beam carries a service imposed load of 24 kN/m and super imposed dead load of 16 kN/m. Use M20 grade of concrete and HYSD steel of Fe415 grade. (12 Marks)

OR

6 Design an intermediate T-beam for a hall measuring 6.5m×12m (clear dimensions). Beams are spaced at 3 m C/C. Depth of slab is 150 mm. Super imposed live load on slab is 4.0 kN/m^2, finishes is 1.0 kN/m^2. Check for deflection also. Use M20 grade concrete and HYSD bar of Fe500 grade. Sketch the reinforcement details. (20 Marks)

Module-4

7 Design a slab for a class room of dimension 4m × 6m (supported on all the four edges) with two adjacent edges discontinuous. Live load = 3 kN/m^2, Floor finish = 1 kN/m^2; Bearing = 300 mm. Use M20 grade concrete and Fe500 grade steel. Check for deflection need not be done. (20 Marks)

OR

8 Design the two flight dog legged stair for a hall of dimension (clear) 3m × 5m between the floors. The floor to floor height is 3.2 m and rise is 160 mm. Also check for deflection. Use M20 grade concrete and Fe500 grade steel. Sketch the reinforcement details of one flight. (20 Marks)

Module-5

a. Design the necessary reinforcement for RC column 450mm × 600mm to carry an axial load of 2000 kN. The length of the column is 3.5 m. Use M25 grade concrete and Fe415 grade steel. Sketch the reinforcement details. (10 Marks)

b. A rectangular column 300 mm wide and 500 mm deep is subjected to an axial factored load of 1200 kN and a factored moment of 200 kN-m. Calculate the necessary reinforcement distributing equally on all four sides. Sketch the reinforcement details. Adopt M25 and Fe500 grade materials. (10 Marks)

OR

Design a square footing of flat type for a column of size 400mm × 400mm to carry an axial dead load of 800 kN and a live load of 1000 kN without any moment. Safe bearing capacity of soil is 180 kN/m^2. Adopt M20 grade concrete and Fe 500 grade steel. Sketch the footing showing the details of reinforcement. (20 Marks)

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SOLUTIONS

MODULE 1

1 a.

→ Characteristic strength

In any structure, there will be variation in strength of materials used. The difference in strength of different test sample may be small. These variations are expressed statistically.

The characteristic strength means that "value of strength of material below which not more than 5% of the test results are expected to fall."

Characteristic strength = Mean strength − K × Standard deviation

$$f_{ck} = f_m - K \times S_d$$

K is constant = 1.65

Sd - Standard deviation for a set of test results.

Characteristic load / Design load

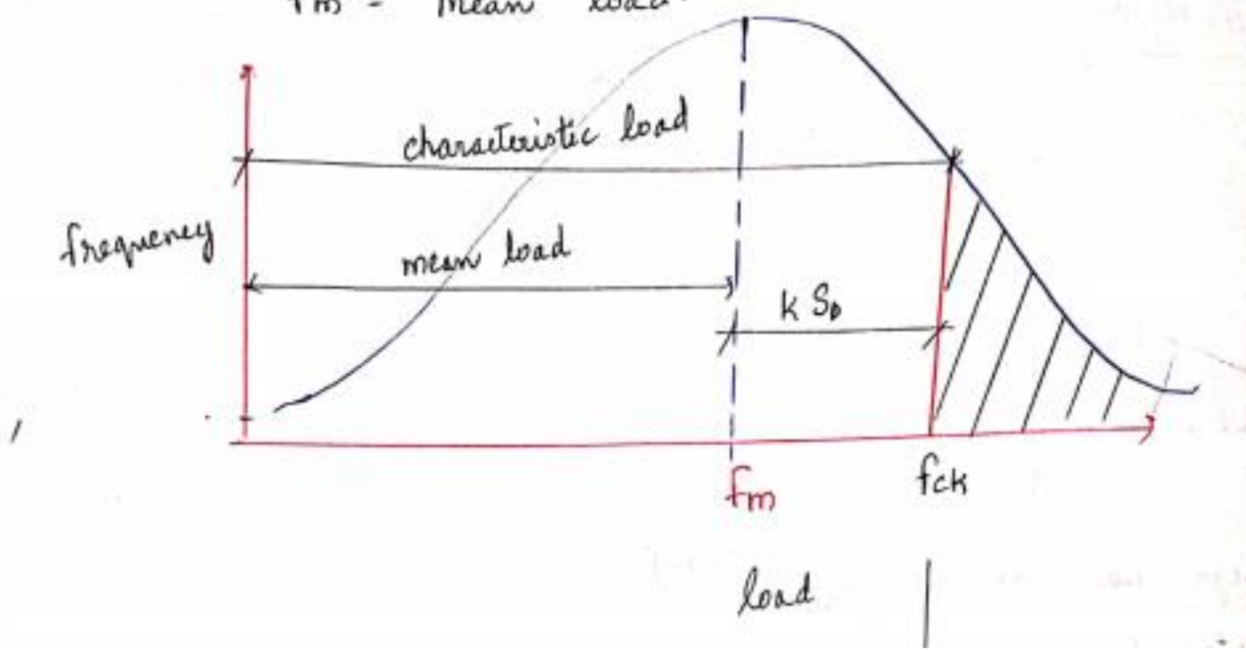
It is that value of load which has 95% probability of not being exceeded during life of structure.

$$f_{ck} = f_m + K \times S_d$$

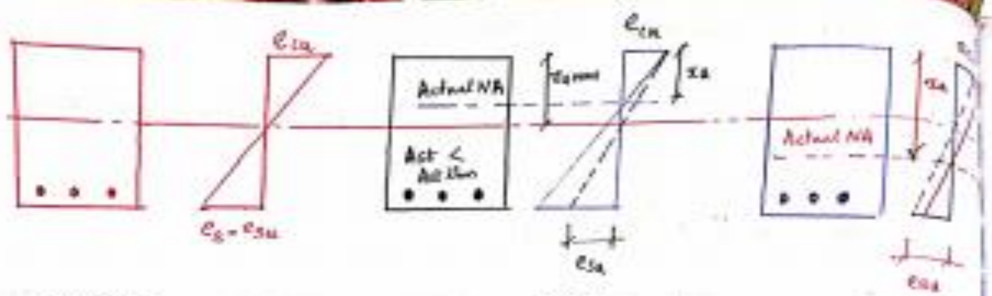
$$K = 2.65$$

f_{ck} - characteristic load

f_m - mean load.



1 b.



BALANCED SECTION

$e_s > e_{su}$

UNDER REINFORCED SECTION

OVER REINFORCED SECTION

$e_s < e_{su}$

Balanced section: The strain in steel and strain in concrete reach their maximum value simultaneously $e_c = e_{cu}$ & $e_s = e_{su}$. The percentage of steel in this section is known as critical or limiting steel percentage ($P_{t\,lim}$). The depth of neutral axis $x_u = x_{u\,max}$.

Under reinforced section: It is one which 'pt' is less than critical or limiting percentage. Due to this actual NA is above the balanced NA and $x_u < x_{u\,max}$. Hence stress in steel required reaches first than concrete.

Beam fails by excess yielding of steel

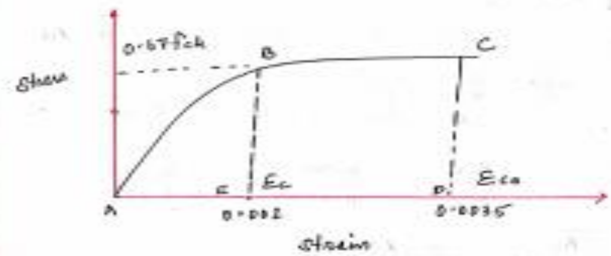
Before beam fails it gives sufficient warning

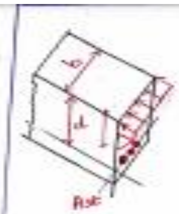
Over reinforced section:

In this type of beam cross section, the percentage of steel is greater than what is required for balanced section. Hence stress in concrete reaches first than steel. Beam fails by crushing of concrete in compression zone. Hence this type of failure is sudden and it would not give warning before it fails.

1. c

structure = 0.67 fy f_{ck}





Fig(1) Stress strain curve for concrete

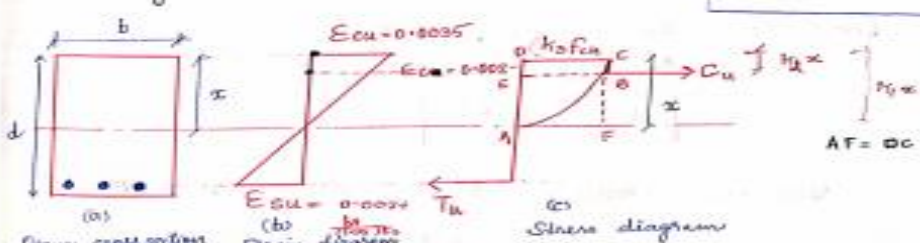


Fig (2) Strain and stress variation across sections

The stress block ABCDEA has parabolic part same as stress - strain curve from fig(1) ie AB part and linear part BC

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The total compressive force - Cu which is below the top fibre can be expressed in stress block parameters k_1, k_2, k_3.

Where k_1 - Shape factor = $\frac{\text{Area of stress block}}{\text{Area of rectangle}}$

$$= \frac{ABCD}{AFCD}$$

$$k_1 = \frac{\text{area of } ABCD}{\text{area of } (x \times CD)} \quad \text{—— (1)}$$

Stress block parameters k_1, k_2, k_3.

Where k_1 - shape factor = $\dfrac{\text{Area of stress block}}{\text{Area of rectangle}}$

$$= \frac{ABCD}{AFCD}$$

$$k_1 = \frac{\text{area of } ABCD}{\text{area of } (x \times CD)} \quad \text{—— ①}$$

Ultimate strain in concrete $= 0.0035 = \varepsilon_{cu} = AD$

$\therefore$ Strain after yielding in concrete $= 0.002 = \varepsilon_{c0} = AE$

Ratio $\dfrac{AE}{AD} = \dfrac{\varepsilon_c}{\varepsilon_{cu}} = \dfrac{0.002}{0.0035} = \dfrac{4}{7}$

$$\boxed{AE = \frac{4}{7} AD}$$

Similarly $\dfrac{ED}{AD} = \dfrac{0.0035 - 0.002}{0.0035} = \dfrac{3}{7}$

$$\boxed{ED = 3/7\ AD}$$

Now area $ABCD$ = Area of ABE + Area of $BEDC$

$= \frac{2}{3} AE \times BE + (ED \times CD)$

$= \frac{2}{3} \times \frac{4}{7} AD \times CD + \frac{3}{7} AD \times CD$

$= \frac{8}{21} (AD \times CD) + \frac{3}{7} AD \times CD$

$= \frac{17}{21} (AD \times CD)$

Substituting in eq ①

$$k_1 = \frac{ABCD}{x \times CD} = \frac{17}{21} \frac{AD \times CD}{AD \times CD} = \frac{17}{21}$$

$$\boxed{k_1 = \frac{17}{21}}$$

Resultant compressive force is located at depth of $k_2 x$

(k_2 - depth factor)

$k_2 x = k_2 AD =$

$$\frac{\text{area of parabola} \times \bar{x}_1 + \text{area of rectangle} \times \bar{x}_2}{\text{area of } ABCD}$$

$$= \frac{\frac{8}{21} (AD \times CD) \cdot \left(ED + \frac{3}{8} AE\right) + \frac{3}{7} (AD \times CD) \left(\frac{1}{2} ED\right)}{\frac{17}{21} (AD \times CD)}$$

$$= \frac{\frac{8}{21} \left\{ \frac{3}{7} + \frac{3}{8} \times \frac{4}{7} \right\} + \frac{3}{7} \left(\frac{1}{2} \times \frac{3}{7} \right)}{\frac{17}{21}}$$

$= \frac{99}{238} = 0.416 \approx 0.42$

2 a

* Durability is taken care by providing appropriate grade of concrete, nominal cover for various exposure conditions, cement content etc.

* Assumptions made in limit state method
* The plane section remains plane even after bending
* Material is homogeneous and isotropic.
* Maxi strain in the concrete at outer most compression fibre is taken as 0.0035 in bending.
* The tensile strength of concrete is ignored
* The compressive strength of concrete is assumed to be 0.67 times the characteristic strength.
* The maxi strain in the tension reinforcement in the section at failure shall not be less than $\frac{fy}{1.15 E_s} + 0.002$
* Partial factor of safety for concrete is 1.5 and steel is 1.15.

2 b

Step 1: Properties of concrete section

y_t = $D/2$ = 300 mm, I_{gr} = $bD^3/12$ = $300(600)^3/12$ = $5.4(10^9)$ mm^4

Step 2: Properties of cracked section

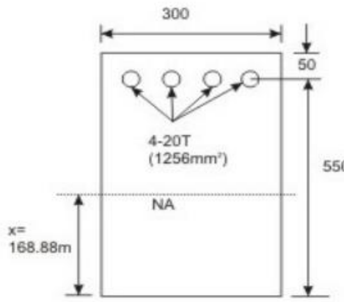


Fig. 7.17.5: TQ. 3 (cracked section, E_c=E_c)

f_{cr} = $0.7\sqrt{20}$ (cl. 6.2.2 of IS 456) = 3.13 N/mm^2

y_t = 300 mm

M_r = $f_{cr}\, I_{gr} / y_t$ = $3.13(5.4)(10^9)/300$ = $5.634(10^7)$ Nmm

E_s = 200000 N/mm^2

E_c = $5000\sqrt{f_{ck}}$ (cl. 6.2.3.1 of IS 456) = 22360.68 N/mm^2

m = E_s / E_c = 8.94

Taking moment of the compressive concrete and tensile steel about the neutral axis (Fig.7.17.5):

$$300\, x^2/2 = (8.94)(1256)(550 - x) \text{ or } \quad x^2 + 74.86\, x - 41171.68 = 0$$

This gives x = 168.88 mm and z = $d - x/3$ = $550 - 168.88/3$ = 493.71 mm.

I_r = $300(168.88)^3/3 + 8.94(1256)(550 - 168.88)^2$ = $2.1126(10^9)$ mm^4

M = $wl^2/2$ = $20(4)(4)/2$ = 160 kNm

$$I_{eff} = \frac{I_r}{(1.2) - (\frac{5.634}{16})(\frac{493.71}{550})(1 - \frac{168.88}{550})(1)} = 1.02\ I_r = 2.1548\,(10^9)\,\text{mm}^4$$

This satisfies $I_r \le I_{eff} \le I_{gr}$. So, I_{eff} = $2.1548(10^9)$ mm^4.

Step 3: Short-term deflection (sec. 7.17.5)

E_c = 22360.68 N/mm^2 (cl. 6.2.3.1 of IS 456)

Short-term deflection = $wl^4/8E_cI_{eff}$

= $20(4^4)(10^{12})/8(22360.68)(2.1548)(10^9)$ = 13.283 mm

So, short-term deflection = 13.283 mm (1)

Step 4: Deflection due to shrinkage (sec. 7.17.6)

$$k_4 = 0.72(0.761)/\sqrt{0.761} = 0.664$$

$$\psi_{cs} = k_4\, \varepsilon_{cs} / D = (0.664)(0.0003)/600 = 3.32(10)^{-7}$$

k_3 = 0.5 (from sec. 7.17.6)

$$\alpha_{cs} = k_3\, \psi_{cs}\, l^2 = (0.5)(3.32)(10)^{-7}(16)(10^6) = 2.656 \text{ mm} \quad (2)$$

Step 5: Deflection due to creep (sec. 7.17.7)

Step 5a: Calculation of $\alpha_{1cc(perm)}$

Assuming the age of concrete at loading as 28 days, cl. 6.2.5.1 of IS 456

Step 5: Deflection due to creep (sec. 7.17.7)

Step 5a: Calculation of $\alpha_{1cc(perm)}$

Assuming the age of concrete at loading as 28 days, cl. 6.2.5.1 of IS 456
gives

θ = 1.6

So, $E_{cc} = E_c/(1 + \theta)$ = 8600.2615 N/mm^2

$m = E_s/E_{cc}$ = 200000/8600.2615 = 23.255

Step 5b: Properties of cracked section

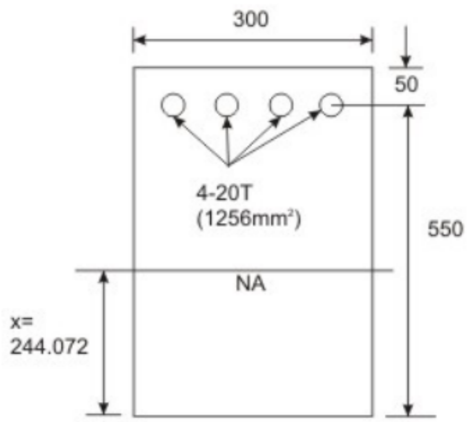


Fig. 7.17.6: TQ. 3 (cracked section, E_c=E_{cs})

From Fig.7.17.6, taking moment of compressive concrete and tensile steel about the neutral axis, we have:

$300\ x^2/2 \ = \ (23.255)(1256)(550 - x)$

or $\quad x^2 + 194.72\ x - 107097.03 \ = \ 0$

solving we get $\ x \ = \ 244.072$ mm

$z \ = \ d - x/3 \ = \ 468.643$ mm

$I_r \ = \ 300(244.072)^3/3 \ + \ (23.255)(1256)(550 - 468.643)^2$

$\quad = \ 1.6473(10)^9 \ \text{mm}^4$

$M_r \ = \ 5.634(\,10^7)$ Nmm (see Step 2)

$M \ = \ w_{perm}\ l^2/2 \ = \ 4.5(4^2)/2 \ = \ 36$ kNm

$$I_{eff} \ = \ \frac{I_r}{1.2 - (\frac{5.634}{3.6})(\frac{468.643}{550})(1 - \frac{244.072}{550})(1)} \ = \ 2.1786\ I_r \ = \ 3.5888(10^9)\ \text{mm}^4$$

Since this satisfies $I_r \le I_{eff} \le I_{gr}$, we have, $I_{eff} = 3.5888(10^9)$ mm^4. For the value of I_{gr} please see Step 1.

Step 5c: Calculation of $\alpha_{1cc(perm)}$

$$\alpha_{1cc(perm)} = (w_{perm})(l^4)/(8E_{cc}I_{eff}) = 4.5(4)^4(10)^{12}/8(8600.2615)(3.5888)(10^9)$$

$$= 4.665 \text{ mm}$$

(3)

Step 5d: Calculation of $\alpha_{1(perm)}$

$$\alpha_{1(perm)} = (w_{perm})(l^4)/(8E_c I_{eff}) = 4.5(4)^4(10)^{12}/8(22360.68)(3.5888)(10^9)$$

$$= 1.794 \text{ mm}$$

(4)

Step 5e: Calculation of deflection due to creep

$$\alpha_{cc(perm)} = \alpha_{1cc(perm)} - \alpha_{1(perm)}$$

$$= 4.665 - 1.794 = 2.871 \text{ mm}$$

(5)

Moreover: $\alpha_{cc(perm)} = \alpha_{1cc(perm)}(\theta)$ gives $\alpha_{cc(perm)} = 1.794(1.6) = 2.874$ mm.

Step 6: Checking of the two requirements of IS 456

Step 6a: First requirement

Maximum allowable deflection = 4000/250 = 16 mm

The actual deflection = 13.283 (Eq.1 of Step 3) + 2.656 (Eq.2 of Step 4)

+ 2.871 (Eq.5 of Step 5e) = 18.81 > Allowable 16 mm.

Step 6b: Second requirement

The allowable deflection is lesser of span/350 or 20 mm. Here, span/350 = 11.428 mm is the allowable deflection. The actual deflection = 1.794 (Eq.4 of Step 5d) + 2.656 (Eq.2 of Step 4) + 2.871 (Eq.5 of step 5e) = 7.321 mm < 11.428 mm.

MODULE 2

3 a

Given

$D = 450$ mm

$b = 250$ mm

3 16 dia

effective cover = 50 mm

$l = 6$ m

$f_{ck} = 20$

$f_y = 250$

$d = D -$ effective cover

$d = 450 - 50$

$d = 400$ mm

$$A_{st} = 3 \times \frac{\pi}{4} 16^2 = 603.18 \text{ mm}^2$$

Step 1: Depth of N.A Pg 96 IS 456

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

$$= \frac{0.87 \times 250 \times 603.18}{0.36 \times 20 \times 250 \times 400}$$

$$\frac{x_u}{d} = 0.182$$

Step 2: Compare with maximum value Pg 70

$$\left(\frac{x_{u\,max}}{d}\right)_{250} = 0.53$$

$\frac{x_{u\,max}}{d} > \frac{x_u}{d}$

Step 3:- Moment Resistance [Ultimate].

$Mu = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$

$= 0.87 \times 250 \times 603.18 \times 400 \left(1 - \frac{603.18 \times 250}{250 \times 400 \times 20}\right)$

$Mu = 48520051.03\ Nm$

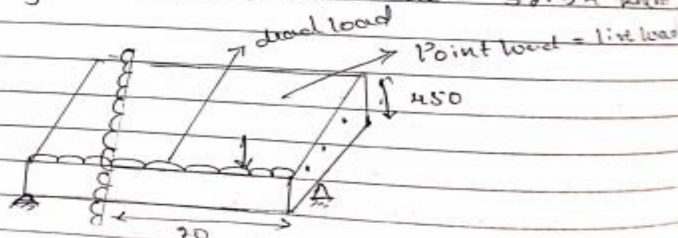
$Mu = \underline{\underline{48.52}}\ kNm$

Step 4: Working moment of Resistance $= \frac{Mu}{P.S.F}$

P.S.F = of concrete = 1.5

$= \frac{48.52}{1.5}$

Working moment of Resistance = 32.34 kNm



Density of Concrete = 25 kN/m^3

Dead load w_D = Density of Concrete × area of beam

$w_D = 25 \times 0.25 \times 0.450$

$w_D = 2.81\ kN/m$

moment due to dead load

Moment due to live load $= M_L = W_L l$

WKT,

Working moment of Resistance $= M_D + M_L$

$32.34 = 12.645$

$M_L = 19.695\ kN/m$

$M_L = W_L \times l$

$19.695 = W_L \times 6$

$W_L = \underline{\underline{3.28}}\ kN.$

3 b

$A_{sc} = 2 \times \frac{\pi}{4} \times 20^2 = 628.32 mm^2$

$A_{st} = 3 \times \frac{\pi}{4} \times 25^2 = 1472.62 mm^2$

$M_u = ?$

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

$f_y = 415 N/mm^2$

$\frac{x_u}{d} max = 0.48$ Pg 70 · IS 456

Limit neutral axis depth as $x_{u,max} = 0.48 \times 450$
$= 216 mm$

Let us find f_{sc} design stress in compression reinforcement corresponding to strain $0.0035 \frac{(x_{u,max} - d')}{x_{u,max}}$ (Pg. 96, G1.2) IS456

$$f_{sc} = \left[0.0035 \frac{(x_{u,max} - d')}{x_{u,max}} \right] \times 2 \times 10^5$$

$$= 0.0035 \frac{(216 - 40)}{216} \times 2 \times 10^5$$

$$= 570 N/mm^2 \quad -(1)$$

From IS456 : 2000, Pg. 70 C.L. 38.1, Graph No. 23A

for a strain $0.0035 \frac{(x_{u,max} - d')}{x_{u,max}}$

$= 0.0035 \left(\frac{216-40}{216}\right) = 0.00285$

For 0.00285 strain, $0.86 f_y = 0.86 \times 415 = 356.9\ N/mm^2$ — ②

Compare ① and ② least value of $f_{sc} = 356.9 N/mm^2$

$\simeq 360 N/mm^2$

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

$= \frac{0.87 \times 415 \times 1472.62}{0.36 \times 20 \times 230} - \frac{360 \times 628.32}{0.36 f_{ck} b}$

$= 184.47\ mm$

- Recalculate value of f_{sc} with $x_u = 184.47 mm$

$f_{sc} = 0.0035 \left(\frac{184.47 - 40}{184.47}\right) = 0.00274$

$\therefore$ from graph, $f_{sc} = 360 N/mm^2$

- Compare x_u & $x_{u,max}$

$M_u =$ ~~0.36 $x_{u,m}$~~

$0.36 \frac{x_u}{d} \left(1 - 0.42 \frac{x_u}{d}\right) bd^2 f_{ck} + f_{sc} A_{sc} (d-d')$

$= 0.36 \times \frac{184.47}{450} \left(1 - 0.42 \times \frac{184.47}{450}\right) 230 \times 450^2 \times 20 +$

$360 \times 628.32 \times (450 - 40)$

$= 206.54 \times 10^6\ N\ mm$

$M_u = M_R =$ (moment of resistance) $= 206.54 \times 10^6 Nmm$

4 a

4(a) Slab thickness = 150mm.

Ast =

d = 10mm.

Spacing = 200mm c/c.

d = 125mm.

M20.

Fe 415

$$\text{Spacing} = \frac{\text{Area of one bar}}{Ast} \times 1000$$

$$Ast = \frac{\pi/4 \times 10^2 \times 1000}{200}$$

$$= 392.69\,mm^2$$

$$M_u = 0.87 f_y\, Ast\, d \left[1 - \frac{Ast\, f_y}{f_{ck}\, b d}\right]$$

$$= 0.87 \times 415 \times 392.6 \times 125 \times$$

$$\left[1 - \frac{392.69 \times 415}{25 \times 1000 \times 125}\right]$$

$$= 55.7\ kNm$$

4 b

Rectangular RC section.

300 x 600 mm.

$A_{st} = 4 \times \pi/4 \times 25^2 = 1963.45$

Fe 415, $f_y = 415\ N/mm^2$

Ultimate shear strength of support section

– Shear resistance of concrete

$V_{uc} = \tau_c b d$

τ_c - $p_t = \frac{100 \times A_{st}}{bd}$

$= \frac{100 \times 1963.45}{300 \times 600}$

$= 1.09.$

$\tau_c = 0.64\ N/mm^2$

$= 0.64 \times 300 \times 600 = 115.2\ kN$ – (1)

Shear resistance of steel.

Shear resistance of 2 legged stirrups + bent up bars.

= Shear resistance of 2 legged stirrups

$= \frac{0.87 \times 415 \times 2 \times \pi/4 \times 8^2 \times 600}{150}$ – (2) $= 145.186\ kN$

Shear resistance of bent up bars

$= 0.87 \times 415 \times \pi/4 \times 25^2 \times 2 \times \sin 45$

$= 125.32\ kN$ – (3)

$= 385.7\ kN$

4 c

4(c) $d = 450m$

$b_f = 1000\ mm$

$D_f = 120mm$

$A_{st} = 6 \times \pi/4 \times 20^2 = 1884.75$

$b_w = 300mm$

Determine 'x_u'

Assume 'x_u' in flange

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times 1884.75}{0.36 \times 25 \times 1000}$$

$75.61\ mm$

Compare x_u & D_f

Find $M_u = ?$

$x_u < x_{umax}$

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b_f d f_{ck}} \right]$$

$$= 0.87 \times 415 \times 1884.95 \times 450 \times \left[1 - \frac{1884.95 \times 415}{1000 \times 450 \times 20} \right]$$

$$= \underline{\underline{279.6 \text{ kNm}}}$$

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$$Ast = \frac{\pi/4 \times 10^2 \times 1000}{200}$$

$$= 392.69 mm^2$$

$$Mu = 0.87 f_y \, Ast \, d \left[1 - \frac{Ast \, f_y}{f_{ck} \, bd}\right]$$

$$= 0.87 \times 415 \times 392.6 \times 125 \times$$

$$\left[1 - \frac{392.69 \times 415}{25 \times 1000 \times 125}\right]$$

$$= 55.7 \text{ kNm}$$

MODULE 3

5 a

Step 1 (a): Fixing up the depth of the section.

Taking $\frac{L}{d} = 20$, for SSB [Refer 23.2.1, pg 37]

$$d = \frac{L}{20} = \frac{5}{20} = 0.25 \text{ m} = 250 \text{ mm}$$

Providing a cover of 25 mm, overall depth D = 250 + 25 = 275 mm

Dimensions of the section.

Width b = 230 mm

depth d = 250 mm

Step 1 (b): Check for lateral stability/lateral buckling

Refer page 39, clause 23.3

Allowable l = 60b or $\frac{250\, b^2}{d}$

Allowable l = 60b = 13800 mm = 13.8 m

Or $\frac{250\, b^2}{d}$ = 52900 mm = 52.9 m

Allowable l = Lesser of the two values

= 13.8 m

Actual l of the beam (5m) < Allowable value of l. Hence ok

Step 2: Effective span

Referring class 22.2 page 34,

Effective span l_e = clear span + d

Or l_e = clear span + $\frac{1}{2}$ support thickness + $\frac{1}{2}$ support thickness

= clear span + $\frac{t_s}{2} + \frac{t_s}{2}$

Whichever is lesser.

l_e = 5000 + 250 mm = 5250 mm

Or $l_e = 5000 + \frac{230}{2} + \frac{230}{2}$ = 5230 mm

Therefore l_e = 5230 mm

Step 3: Calculation of loads:

Consider 1m length of the beam

a. Dead load = (0.23 x 0.275 x 1m x 25 kN/m^3) = 1.58 kN/m
b. Live load = 25 kN/m

Total working load w = 26.58 kN/m

Factored moment $M_u = \frac{W_u \times l_e^2}{8} = \frac{40 \times 5.23^2}{8}$ = 136.76 kN-m

Factored shear = $\frac{40 \times 5.23}{2}$ = 104.6 kN

Step 4: Check for depth based on flexure or bending moment consideration

Assuming the section to be nearly balanced, and equating M_u to M_{ulim},

$M_u = M_{ulim}$ = 136.76 kN-m

Using the equation G 1.1 (c), Annexure G IS 456-2000

$$M_{ulim} = 0.36\frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)bd^2 f_{ck}$$

$$136.76 \times 10^6 = 0.36 \times 0.48\,(1 - 0.42 \times 0.48)230d^2 \times 20$$

d = 464.21 mm

Assumed depth d is less than the required depth of 464 mm. Hence revise the section

Assume

d = 500 mm

b = 230 mm

Loads:

Dead load = 0.23 x 0.525 x 1 x 25 = 2.875 kN/m

Live load = 25 kN/n

Total working load = 27.875 kN/m

Factored load = 27.875 x 1.5 = 41.8 ≈ 42 kN/m

Factored moment $M_u = \frac{W_u \times l_e^2}{8} = \frac{42 \times 5.23^2}{8}$ = 143.6 kN-m

Factored shear = $\frac{42 \times 5.23}{}$ = 109.83 kN

Check for depth based on flexure

$M_u = M_{ulim}$ = 143.6 kN-m

Using the equation G 1.1 (c)

$$M_{ulim} = 0.36 \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2 f_{ck}$$

$$143.6 \times 10^6 = 0.36 \times 0.48 \, (1 - 0.42 \times 0.48) 230 d^2 \times 20$$

d = 475.68 mm

Assumed depth is greater than the required depth of 475.68 mm.

Required 'd' = 476 mm and Assumed 'd' = 500 mm

Hence ok.

Therefore we shall continue with d = 500 mm and D = 525 mm

Check whether the section is under reinforced

Actual moment acting M_u = 143.6 kM-m

Actual moment acting M_u = 143.6 kN-m

Using equation G 1.1 (c)

$$M_{ulim} = 0.36\frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) bd^2 f_{ck}$$

$$M_{ulim} = 0.36 \times 0.48\,(1 - 0.42 \times 0.48)230 \times 500^2 \times 20$$

= 158.66 kN-m

$M_u < M_{ulim}$

Hence the section is under reinforced

Step 5: Calculation of steel:

Since the section is under reinforced we have,

Using equation G 1.1 (b)

$$M_u = 0.87 f_y\, A_{st}\, d\left(1 - \frac{A_{st} f_y}{bd\, f_{ck}}\right)$$

$$143.6 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500\left(1 - \frac{A_{st} \times 415}{230 \times 500 \times 20}\right)$$

Solving the quadratic equation, A_{st} = 960.33 mm^2 ≈ 960 mm^2

Choosing 8 mm diameter bars,

Area of 1 bar = $\frac{\pi}{4} \times 8^2$ = 50.265 mm^2

Therefore number of bars of 8mm required = 19.10 = 20 bars

Distance between any two bars

Minimum distance between two bars is greater of the following:

a. Size of the aggregate + 5 mm
   20 mm + 5 mm
b. Size of the bar (whichever is greater)

Therefore minimum distance = 25 mm

Distance between bars = $\frac{230-2\times25-2\times8}{19}$ = 8.63

1.63 < 25. Therefore 8 mm dia bars cannot be provided.

Let us choose 16 mm dia bars.

Area of 1 bar = $\frac{\pi}{4} \times 16^2$ = 201.06 mm^2

Therefore number of bars of 16 mm required = 4.77 = 5 bars

Distance between bars = $\frac{230-2\times25-2\times8-5\times16}{4}$ = 21 mm

Minimum distance required = 25 mm

Therefore 16 mm dia cannot be used.

Let us choose 25 mm dia bars.

Area of 1 bar = $\frac{\pi}{4} \times 25^2$ = 490.890 mm^2

Therefore number of bars of 25mm required = 1.95 = 2 bars

Distance between the bars = $\frac{230-2\times25-2\times8-2\times25}{1}$ = 114 mm

Check for $A_{st\ min}$

$$A_{st\ min} = \frac{0.85bd}{0.87f_y}$$

$$A_{st\ min} = \frac{0.85 \times 230 \times 500}{0.87 \times 415} = 270.7\ mm^2$$

Check for $A_{st\ max}$

$A_{st\ max} = 0.04 \times b \times D = 4830\ mm^2$

A_{st} provided = 982 mm^2

$A_{st\ min} < A_{st} < A_{st\ max}$

Hence ok.

Check for shear

Factored load = 42 kN/m

Support reaction = $\frac{wl}{2} = \frac{42 \times 5}{2}$ = 105 kN

V_u = 105 kN

$\tau_v = \frac{V_u}{bd}$ = 0.913 N/mm^2

$P_t = \frac{100A_{st}}{bd} = \frac{100\times982}{230\times500}$ = 0.8539

From table 19, IS 456-2000 page 73

τ_c = 0.58 N/mm^2

From table 20, IS 456-2000 page 73

$\tau_{c\ max}$ = 2.8 N/mm^2

$\tau_c < \tau_v < \tau_{c\ max}$

Hence design of shear reinforcement is required

Selecting 2 leg vertical stirrups of 8 mm diameter, Fe 415 steel,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100 \text{ mm}^2$$

V_c = Shear force taken up by the concrete

$$= \frac{\tau_c bd}{1000} = \frac{0.28 \times 230 \times 500}{1000} = 66.7 \text{ kN}$$

V_u = 105 kN

$V_{us} = V_u - V_c$

= 105 – 66.7 = 38.3 kN

$V_{us} = \frac{0.87 \times f_y \times A_{sv} \times d}{S_v}$ from clause 40.4

$$38.3 \times 10^3 = \frac{0.87 \times 415 \times 100 \times 500}{S_v}$$

S_v = 471.3 mm

Check for maximum spacing

Maximum spacing = 0.75d or 300mm whichever is lesser

Maximum spacing = 375 or 300mm

Therefore maximum spacing allowed = 300mm

Let us provide 8 mm dia 2-leg vertical stirrups at a spacing of 300 mm.

Check for $A_{sv\,min}$:

A_{sv} provided = 100 mm^2

$$A_{sv\,min} = \frac{0.4 b S_v}{0.87 f_y} = 76.44 \text{ mm}^2$$

A_{sv} provided > $A_{sv\,min}$

Hence ok.

Check for deflection:

Allowable $\frac{l}{d}$ = Basic $\frac{l}{d}$ x M_t x M_c x M_f

Basic $\frac{l}{d}$ = 20 as the beam is simply supported

To determine M_t

$f_s = 0.58 \times 415 \times \frac{960}{982} = 235.3$ N/mm^2

from fig 4, M_t = 1

To determine M_c

From fig 5, M_c = 1 [since there is no compression reinforcement]

To determine M_f

$\frac{b_w}{b_f} = 1$ [since it is rectangular section $b_w = b_f$]

Therefore allowable l/d = 20 x 1 x 1 x 1 = 20

Actual l/d = $\frac{5230}{500}$ = 10.46 < Allowable l/d.

Hence ok.

5 b

a. Data

$b = 250$ mm

$D = 500$ mm

$d = 450$ mm

$d' = 50$ mm

$l_e = 5$ m

$w = 40$ kN/m and $W_u = 40 \times 1.5 = 60$ kN/m

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

$f_y = 415$ N/mm^2

$E_s = 2 \times 10^5$ N/mm^2

b. Ultimate moments and shear forces

$$M_u = \frac{W_u \times l_e^2}{8} = \frac{60 \times 5^2}{8} = 187.5 \text{ kN-m}$$

$$V_u = \text{Factored shear} = \frac{W_u \times l_e}{2} = 150 \text{ kN}$$

c. Determination of M_{ulim} and f_{sc}

$$M_{ulim} = 0.36\frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)bd^2 f_{ck}$$

$$M_{ulim} = 0.36 \times 0.48\,(1 - 0.42 \times 0.48)250 \times 450^2 \times 20$$

$= 140$ kN.m

Since $M_u > M_{u\,lim}$, design a doubly reinforced section

$(M_u - M_{u\,lim}) = 187.5\text{-}140 = 47.5$ kN.m

$$f_{sc} = \epsilon_{sc} \times E_s$$

Where, $\epsilon_{sc} = \left\{\frac{0.0035(x_{u\,max} - d')}{x_{u\,max}}\right\}$

$$f_{sc} = \left\{\frac{0.0035(x_{u\,max} - d')}{x_{u\,max}}\right\} E_s$$

$$= \left\{\frac{0.0035[(0.48 \times 450)-50]}{0.48 \times 450}\right\} 2 \times 10^5$$

$= 538$ N/mm^2

But $f_{sc} \ngtr 0.87 f_y = (0.87 \times 415) = 361$ N/mm^2

Therefore $f_{sc} = 361$ N/mm^2

steel $A_{sc} = \left[\frac{(M_u - M_{u\,lim})}{f_{sc}(d-d')}\right]$

$$= \left[\frac{(47.5 \times 10^6)}{361 \times 400}\right] = 329 \text{ mm}^2$$

Provide 2 bars of 16mm diameter ($A_{sc} = 402$ mm^2)

$$A_{st2} = \left(\frac{A_{sc} f_{sc}}{0.87 f_y}\right) = \left(\frac{329 \times 361}{0.87 \times 415}\right) = 329 \text{ mm}^2$$

$$A_{st1} = \left[\frac{0.36 f_{ck} b x_{u\,lim}}{0.87 f_y}\right]$$

$$= \left[\frac{0.36 \times 20 \times 250 \times 0.48 \times 450}{0.87 \times 415}\right] = 1077 \text{ mm}^2$$

Total tension reinforcement $= A_{st} = (A_{st1} + A_{st2})$

$= (1077 + 329)$

$= 1406$ mm^2

Provide 3 bars of 25mm diameter ($A_{st} = 1473$ mm^2)

d. Shear reinforcements

$\tau_v = (V_u/bd) = (150 \times 10^3) / (250 \times 450) = 1.33$ N/mm^2

$$P_t = \frac{(100 A_s)}{bd} = \frac{100 \times 1473}{250 \times 450} = 1.3$$

Referring table 19 of IS : 456 – 2000 ,

$\tau_c = 0.68$ N/mm^2

$\tau_{cmax} = 2.8$ N/mm^2 for M20 concrete from table 20 of IS 456-2000

Since $\tau_c < \tau_v < \tau_{cmax}$, shear reinforcements are required.

$V_{us} = [V_u - (\tau_c bd)]$

$V_{us} = [V_u - (\tau_c bd)]$

$= [150-(0.68 \times 250 \times 450)10^{-3}] = 73.5$ kN

Using 8 mm diameter 2 legged stirrups,

$$S_v = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}} = \frac{0.87 \times 415 \times 2 \times 50 \times 450}{73.5 \times 10^3} = 221 \text{mm}$$

Maximum spacing is 0.75d or 300 mm whichever is less

$S_v > 0.75d = (0.75 \times 450) = 337.5$ mm

Adopt a spacing of 200 mm near supports gradually increasing to 300 mm towards the centre of the span.

e. Check for deflection control

$(l/d)_{actual} = (5000/450) = 11.1$

$(l/d)_{allowable} = [(l/d)_{basic} \times M_t \times M_c \times M_f]$

$P_t = 1.3$ and $P_c = [(100 \times 402) / (250 \times 450)] = 0.35$

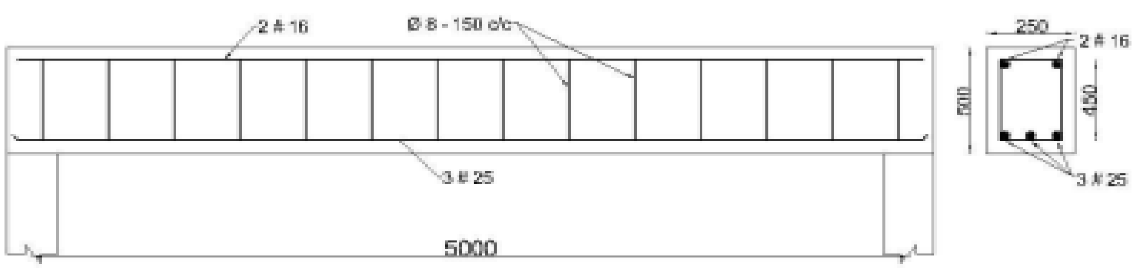
Refer Fig 4, $M_t = 0.93$
Fig 5, $M_c = 1.10$
Fig 6, $M_f = 1.0$

$(l/d)_{allowable} = [(20 \times 0.93 \times 1.10 \times 1] = 20.46$

$(l/d)_{actual} < (l/d)_{allowable}$

Hence deflection control is satisfied.

f. Reinforcement details



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Step 1: Fix the depth

39, cl 24.1

$$\frac{l}{d} = 40 \times 0.8$$

$$d = \frac{4000}{40 \times 0.8}$$

$$\boxed{d = 125mm}$$

assuming 10mm ϕ bars, clear cover of 15mm

$$D = 125 + 15 + \frac{10}{2}$$

$$\boxed{D = 145mm}$$

Step 2: effective length

34, 22.2

$$l_{eff} = l + d \quad \text{or} \quad l_{eff} = l + \frac{300}{2} + \frac{300}{2}$$

$$= 4 + 0.125 \qquad = 4 + 0.3$$

step 3: load calculation

dead load = $25 \times 0.145 \times 1 = \cancel{88}\ 3.625\,kN/m$

live load $= 3\,kN/m^2 \times 1 = 3\,kN/m$

floor finish $= 1\,kN/m^2 \times 1 = 1\,kN/m$

Total load $= 3.625 + 3 + 1 = 7.625\,kN/m$

factored load = $11.44\,kN/m$

step 4: Bending moment and s.v

P 90, Annex-D

$M_x = \alpha_x \cdot w \cdot l_x^2$ $\qquad$ $M_y = \alpha_y \cdot w \cdot l_x^2$

table 26, case 4

negative moments @ continuous edge

$M_x^{(-)} = \alpha_x^{(-)} \cdot w \cdot l_x^2$

$= 0.0625 \times 11.44 \times 4.0125^2$

$\boxed{M_x^{(-)} = 12.17\,kN\text{-}m}$

$M_y^{(-)} = \alpha_y^{(-)} \cdot w \cdot l_x^2$

$= 0.047 \times 11.44 \times 4.125^2$

$\boxed{M_y^{(-)} = 9.15\,kN\text{-}m}$

positive moment at mid span

$M_x^{(+)} = \alpha_x^{(+)} \cdot w \cdot l_x^2$

$= 0.047 \times 11.44 \times 4.125^2$

$\boxed{M_x^{(+)} = 9.15\,kN\text{-}m}$

$M_y^{(+)} = \alpha_y^{(+)} \cdot w \cdot l_x^2$

$= 0.035 \times 11.44 \times 4.1$

$\boxed{M_y^{(+)} = 6.81\,kN\text{-}m}$

$\boxed{M_x^{(+)} = 9.15 kN\text{-}m}$ $\boxed{M_y^{(+)} = 6.81 kN\text{-}m}$

Step 5: check for depth

$$M_{max} = M_{u,lim} = 0.36 \times \frac{x_{u,max}}{d}\left(1 - 0.42 \times \frac{x_{u,max}}{d}\right) . b . d^2 . f_{ck}$$

$$12.17 \times 10^6 = 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 20$$

$$\boxed{d = 66.41mm}$$

∵ d req < d provided

d mm is safe

Step 6: Ast calculation

$$M_u = 0.87 \times f_y \times A_{st} \times d \left(1 - \frac{f_y \times A_{st}}{b . d . f_{ck}}\right)$$

$A_{st_y}^{(-)} = 210.07 mm^2$

$A_{st_x}^{(-)} = 282.95 mm^2$

$= \frac{...}{282.95} \times 1000$

$= 277.57mm$

$\approx 270mm$

$A_{st_x}^{(+)} = 210.07mm^2$

$A_{st_y}^{(+)} = 154.87mm^2$

spacing $= 373.87mm$

$\approx 370mm$

spacing $: 507.13mm$

$\approx 500mm$

∴ provide spacing should be a min of

i) 300mm

ii) $3 \times d_{eff} = 3 \times 125 = 375mm$

iii) 270m

∴ provide 10mm ϕ @270mm c/c as main reinforcement along both directions i.e x and y directions

Step 7: Check for shear

$$\tau_v = \frac{V_F}{b . d} = \frac{\frac{w \times D_{eff}}{2}}{b \times d} = \frac{\frac{11.44 \times 4.125}{2} \times 10^3}{1000 \times 125} = 0.19 N/mm^2$$

$$P_t = \frac{100 . A_{st}}{b . d}$$

$$A_{st} \text{ here} = \frac{\pi/4 \times 10^2}{270} \times 1000 = 290.89mm^2$$

$$P_t = \frac{100 \times 290.89}{1000 \times 125} = 0.23$$

#73, table 19,

$\tau_c = 0.34\ N/mm^2$

$\because\ \tau_v < \tau_c$

It is safe against shear

Step 8: Torsional Reinforcement
provided in 4 layers

#90, Annex D

Torsional reinforcement $= \frac{3}{4}(A_{st})$ D 1.8

$= \frac{3}{4} \times 290.89$

$= 218.17\ mm^2$

spacing $= \frac{\pi/4 \times 6^2}{218.17} \times 1000 \approx 120mm$

$\therefore$ provide 6mm ϕ @ 120mm c/c as torsional reinforcement

side length of the torsion mesh

$= \frac{1}{5}(4.125)$

$= 825mm$

Step 9: check for deflection

$\left(\frac{l}{d}\right)_{actual} = \frac{4.125}{0.125} = 33$

$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times k_t$

$k_t = 0.58\, f_y \times \frac{(A_{st})_{req}}{(A_{st})_{provided}}$

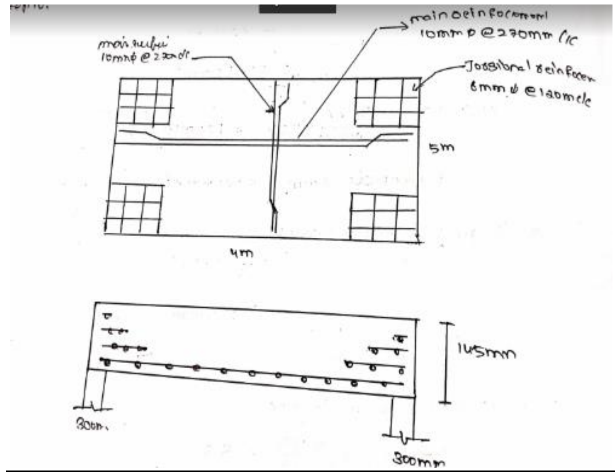
#38, fig 4 corresponding to $p_t \simeq 0.23$, and $f_s \simeq 240$

$k_t \approx 1.6$

$\therefore \left(\frac{l}{d}\right)_{max} = 40 \times 0.8 \times 1.6 = 51.2$

$\because \left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{actual}$

It is safe against deflection



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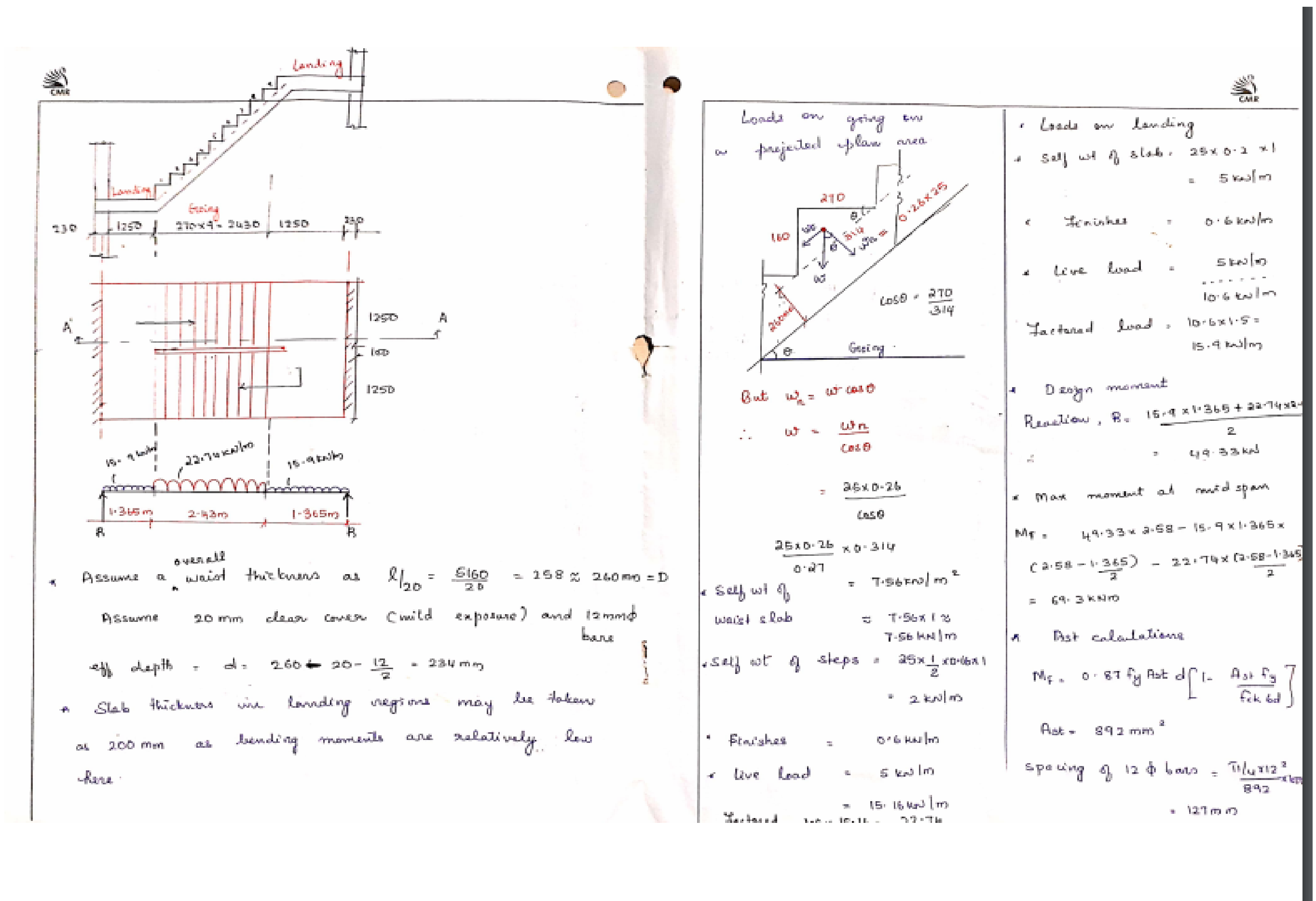


* Assume a overall waist thickness as $l/20 = \frac{5160}{20} = 258 \approx 260\text{mm} = D$

Assume 20 mm clear cover (mild exposure) and 12mm ϕ bars

eff depth = $d = 260 - 20 - \frac{12}{2} = 234$ mm

* Slab thickness in landing regions may be taken as 200 mm as bending moments are relatively low here.

Loads on going on projected plan area

But $w_n = w \cos\theta$

$\therefore w = \frac{w_n}{\cos\theta}$

$= \frac{25 \times 0.26}{\cos\theta}$

$\frac{25 \times 0.26}{0.27} \times 0.314$

* Self wt of waist slab $= 7.56 kN/m^2$

$= 7.56 \times 1 = 7.56$ kN/m

* Self wt of steps $= 25 \times \frac{1}{2} \times 0.16 \times 1$

$= 2$ kN/m

* Finishes = 0.6 kN/m
* Live load = 5 kN/m

= 15.16 kN/m

Factored load = 15.16 × ... = 22.74

* Loads on landing
* Self wt of slab = 25 × 0.2 × 1 = 5 kN/m
* Finishes = 0.6 kN/m
* Live load = 5 kN/m

10.6 kN/m

Factored load = 10.6 × 1.5 = 15.9 kN/m

* Design moment

Reaction, $R = \frac{15.9 \times 1.365 + 22.74 \times 2.43 ...}{2}$

= 49.33 kN

* Max moment at mid span

$M_f = 49.33 \times 2.58 - 15.9 \times 1.365 \times (2.58 - \frac{1.365}{2}) - 22.74 \times \frac{(2.58 - 1.365)^2}{2}$

= 69.3 kNm

* Ast calculations

$M_f = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} b d}\right]$

$A_{st} = 892 mm^2$

Spacing of 12 ϕ bars $= \frac{\pi/4 \times 12^2}{892} \times 1000$

= 127 mm

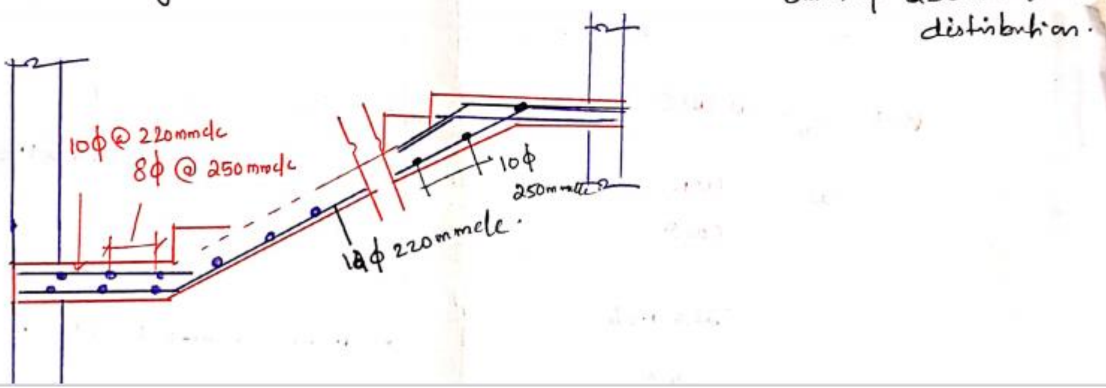
Distribution bars

$$= \frac{0.12}{100} \times 1000 \times 260 = 312 \, mm^2$$

Spacing of 10 mm ϕ bars

$$= \frac{\pi/4 \times 10^2}{312} = 251 \, mm$$

Provide 10 mmϕ as distribution reinf. At support 'top' to resist negative moments on account of partial fixity-landing 8mm ϕ 250 mmc/c as distribution.



MODULE 5

9.a

a) 450mm × 450mm

$f_{ck} = 25 N/mm^2$

$f_y = 415 N/mm^2$

P = 1600kN

$P_u = 1600 \times 1.5 = 2400 kN$

Step 1: Asc calculation

31, 39.3

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

$$2400 \times 10^3 = 0.4 \times 25 \times (450^2 - A_c) + 0.67 \times 415 \times A_{sc}$$

$$A_{sc} = 1398.99 mm^2$$

assuming 24mm ϕ bars,

$$\text{no. of bars} = \frac{1398.99}{\frac{\pi}{4} \times 24^2} = 3.09$$

$$\approx 4 \text{ bars}$$

∴ provide, #4, 24mm ϕ as main bars

... of 8mm ϕ

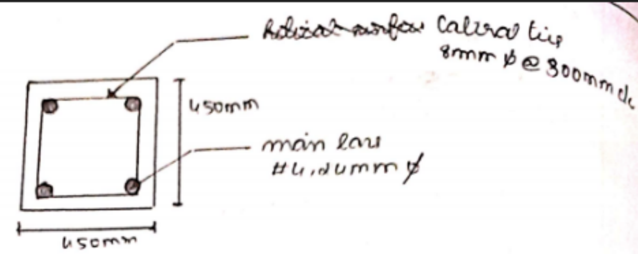
spacing, i) 16 × d = 16 × 24 = 384mm

ii) b = 450mm

iii) 300mm

} least

∴ provide 8mm ϕ lateral tie @ 300mm c/c



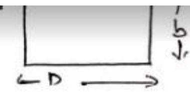
9 b

$P_u = 1200\,kN$

$M_u = 160\,kN\text{-}m$

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

$f_y = 415\,N/mm^2$



Step 1: slenderness ratio

$$= \cancel{\frac{3.6}{0.4}} = 9 \quad \frac{l_{eff}}{D} = \frac{3.6}{0.4} = 9$$

short column $\quad \frac{l_{eff}}{b} = \frac{3.6}{0.3} = 12 \quad$ } # 41, 25.1.2

Step 2: min eccentricity

$$e_{min} = \frac{l_{eff}}{500} + \frac{D}{30} \quad / \quad \frac{l_{eff}}{500} + \frac{d}{30}$$

$$= \frac{3600}{500} + \frac{400}{30} \qquad \frac{3600}{500} + \frac{300}{30}$$

$e_{min} = 20.53\,mm$ or $20\,mm$ or 17.2

considering 25mm ϕ bar

$$d' = 40 + \frac{25}{2} = 52.5$$

Now, $\frac{d'}{D} = \frac{52.5}{400} = 0.131$

chart 33, # 118. SP-16 (rect. distribution in rect. section)

$$\frac{P_u}{f_{ck} \cdot b D} = \frac{1200 \times 10^3}{25 \times 300 \times 400} = 0.4$$

$$\frac{M_u}{f_{ck} \cdot b \cdot D^2} = \frac{160 \times 10^6}{25 \times 300 \times 400^2} = 0.13$$

$\therefore \quad \frac{p}{f_{ck}} \approx 0.11\%$

$$p = 0.11 \times 25 = 2.75\%$$

$$A_s = \frac{p \cdot b \cdot D}{100} = \frac{2.75 \times 300 \times 400}{100} = 3300\,mm^2$$

no. of bars = $\dfrac{A_s}{\text{area of one bar}} = \dfrac{3300}{\frac{\pi}{4} \times 24^2} = 7.29$

≈ 8 bar

∴ provide #8, 20mm ϕ on two sides as main bars

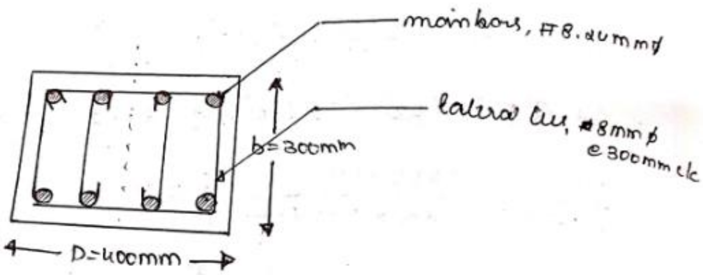
Step v: lateral ties

provide 8mm ϕ lateral ties

spacing
i) 300mm
ii) b = 300mm
iii) $16 \times \text{bar}\ \phi = 16 \times 24 = 384$mm

} minimum

∴ provide 8mm ϕ @ 300mm c/c as lateral ties



10

* Step I :- Size of footing

Load on column = 600kN

Assume 10% as load / wt of footing

$= \frac{10}{100} \times 600 = 60$kN

Total load = 660kN

Required area of footing $= \dfrac{660 \times 10^3}{SBC}$

$= \dfrac{660 \times 10^3}{200 \times 10^3}$

$= 3.3\text{m}^2$

Assume a square footing : $L \times B \approx \sqrt{3.3}$

$= 1.82\text{m} \approx 1.85\text{m}$

$L \times B = 1.85 \times 1.85$

Net upward pressure in soil $= \dfrac{600}{1.85 \times 1.85}$

$= 175.3\ kN/m^2$

$<< 200\ kN/m^2$

Hence O.K.

So, factored upward pressure $= 1.5 \times 175.3$

$= 263\ kN/m^2$

Factored load from column $= 1.5 \times 600$

$= 900\ kN$

Step 2 :- Two way shear.

Assume an uniform overall thickness of footing $D = 450mm$

using 12mm for main steel, eff thickness of footing

$d = 450 - 50 - \frac{12}{2} = \cancel{388}\ 394\ mm.$

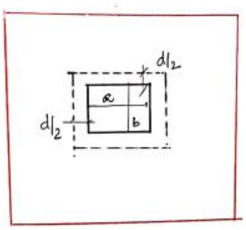
The critical section for the two way shear at a distance $d/2$ from the face of column.

Punching area of footing

$= (a+d)(b+d)$

$= (0.23 + 0.394)(0.23 + 0.394)$

$= 0.382\ m^2$



Punching shear force =

Factored load − Factored upward pressure × Punching area of footing

$= 900 \times 10^3 - (263 \times 0.394) = 799.53\ kN$

Perimeter of critical section $= 4(a+d)$

$= 4(0.23 + 0.394)$

$= 2.448\ m$

Punching shear stress $\tau_v = \dfrac{\text{Punching shear force}}{\text{perimeter} \times \text{eff thickness}}$

$= \dfrac{799.53 \times 10^3}{2.448 \times 0.394 \times 10^6} = 0.842\ N/mm^2$

Allowable shear stress $\tau_c = 0.25\sqrt{f_{ck}}$ Pg. 58 31.6.3.1

$= 0.25\sqrt{20} = 1.12 N/mm^2$

$\tau_v < \tau_c$ Hence safe.

Hence assumed thickness is sufficient.

Effective depth for lower reinf layer = $450-50-6 = 394$ mm

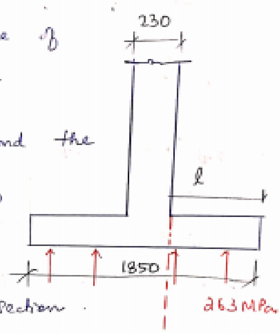
" " " upper reinf layer = $450-50-12-6 = 382$ mm

Step 3 :~ Design for flexure

The critical section for bending occurs at face of column

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

Factored upward pressure of soil $p_u = 263\ kN/m^2$



projection of footing beyond the column face, $l = \dfrac{1850-230}{2}$

$= 810mm$

Hence BM in critical section.

$Mu = \dfrac{P_u l^2}{2} = \dfrac{263 \times 0.81^2}{2} = 86.28\ kNm$

Ast calculations.

$Mu = 0.87 f_y A_{st} d \left[1 - \dfrac{f_y A_{st}}{f_{ck} bd}\right]$

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

$A_{st} = 648.42 mm^2$

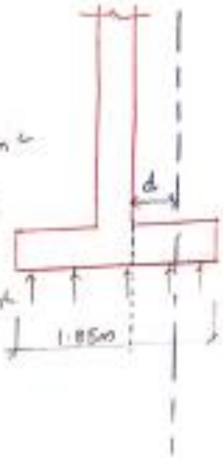
Step 4 :~ One way shear

The critical section for one way shear occurs at a distance 'd' from the face of column.

For a cantilevered slab, total shear force along

$= 263 \times 1.85 \times (0.81 - 0.382)$

$= 208.24\,kN$



$\tau_v = \dfrac{V_u}{bd} = \dfrac{208.24 \times 10^3}{1850 \times 382} = 0.3\,N/mm^2$

Use Table 61 of SP16/, find p_t, Table 19, IS456

for $\tau_c = \tau_v = 0.3\,N/mm^2$, $f_{ck} = 20\,N/mm^2$

$p_t = \dfrac{100 A_{st}}{bd} = 0.175\%$

A_{st} provided $= \dfrac{0.175}{100} \times 1000 \times 382 = 669\,mm^2$

Spacing $= 1000 \times \dfrac{A_{st} \text{ of one bar}}{A_{st} \text{ of one bar provided}} = 169.05 \approx 165\,mm\ c/c$

$165 = 1000 \times \dfrac{\pi/4 \times 12^2}{A_{st} \text{ provided}} =$

A_{st} pro $= 685.4\,mm^2$

Step 5:- Check for development length.

L_d for $12\,mm\ \phi$ bars

$=$ cl. 26.2.1, $L_d = \dfrac{\phi \sigma_s}{4 \tau_{bd}}$

$\tau_{bd} = 1.2\,N/mm^2$ Table 26.2.1.1

$\sigma_s =$

$L_d = 47\phi = 47 \times 12 = 564\,mm$

Provide 60 mm side cover, total length available from critical section $= \frac{1}{2}(L-a) - 60 = \frac{1}{2}(1850 - 230) - 60$

$= 750\,mm > L_d$

Hence OK

Step ⑥ :- Check for bearing stress

Load is assumed to disperse from column base to footing base at rate of 2H : 1V

Hence the side of area of dispersion at the bottom of footing = 230 + 2(2×450) = 2030 mm

CL. 34.4 Bearing pressure on loaded area shall not exceed permissible bearing stress in direct comp multiplied by a value equal to $\sqrt{\frac{A_1}{A_2}} \not> 2$

$A_1 = 1.85 \times 1.85 = 3.4225 m^2$

$A_2 = 0.23 \times 0.23 = 0.0529 m^2$

$\sqrt{\frac{A_1}{A_2}} = 8.04 > 2$, But limit $\sqrt{\frac{A_1}{A_2}} = 2$

$\therefore$ permissible stress $= 0.45 f_{ck} \sqrt{\frac{A_1}{A_2}} = 0.45 \times 20 \times \sqrt{\frac{A_1}{A_2}}$

$= 0.45 \times 20 \times 2$

$= 18 N/mm^2$

Actual bearing stress = $\frac{\text{Factored load}}{\text{Area at column base}}$

$= \frac{900 \times 10^3}{230 \times 230} = 17.01 N/mm^2$

Since the actual stress is less than permissible stress, $17.01 < 18 N/mm^2$, design for bearing stress is satisfactory.