27/6/19

UNIT - 1

Compressive Strength of Concrete :

It is defined as characteristic Compressive strength of 150mm Cubes at the age of 28 days → M_{20} grade

M → Mix proportion

20 → characteristic Compressive strength of Concrete in MPa

1. Ordinary Concrete - M_{10} to M_{20}
2. Standard Concrete - M_{25} to M_{55}
3. High strength Concrete - M_{60} to M_{80}

Mix proportions:

M_5 - 1 : 5 : 10

$M_{7.5}$ - 1 : 4 : 8

M_{10} - 1 : 3 : 6

M_{15} - 1 : 2 : 4

M_{20} - 1 : 1½ : 3

Steel reinforcement : Steel bars are essentially used in tension zone of flexural members to resist the tensile stresses.

Why steel is used as reinforcement?

→ Its tensile strength is very high.

→ It can develop good bond with Concrete.

→ The thermal Coefficient value is more.

→ It is easily available.

Reinforcement Serves the following functions:

1. To resist the bending tension.
2. To increase the load Carrying Capacity.
3. To resist the diagonal tension due to shear
4. To reduce the shrinkage of Concrete.
5. To resist wide cracks & spiral cracks.

Types of reinforcement:

1. Mild steel & medium tensile steel.
2. High yield strength deformed steel bars [HYSD
3. Steel wire fabrics.
4. Structural steel.

Mild steel bars: These are plain bars of grade Fe 250 where,

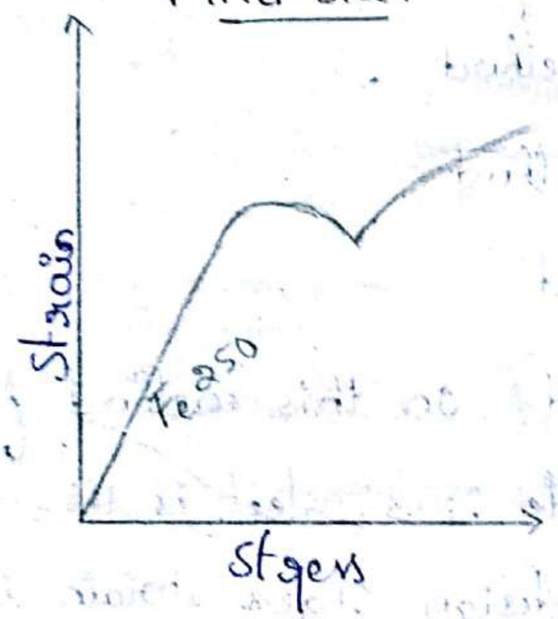
Fe → Ferrous

250 → yield strength (or) yield stress

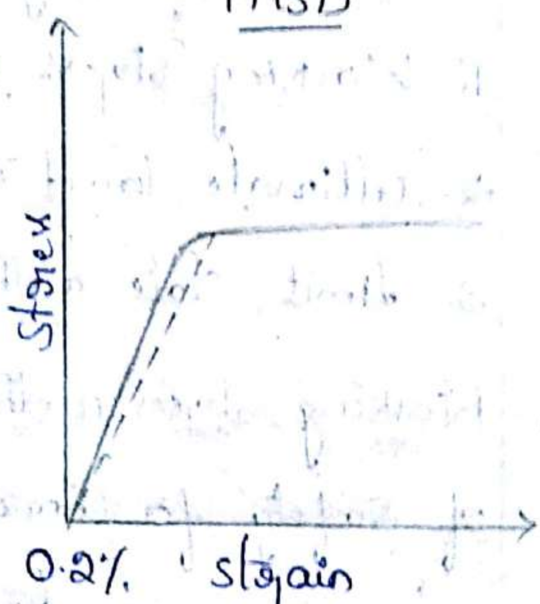
HYSD bars:

They are having high yield strength of grade Fe 415 or Fe 500.

Mild steel



HYSD



→ Yield stress is given by 0.2% of proof stress

→ The young's modulus of all types of steel are 2×10^5 N/mm^2.

→ Unit wt. of steel is 78.5 kN/m^3

Types of loads:

1. Dead load - Self wt (Constant)
2. Live load - moving (Assume).
3. Wind load
4. Earthquake load
5. Snow load

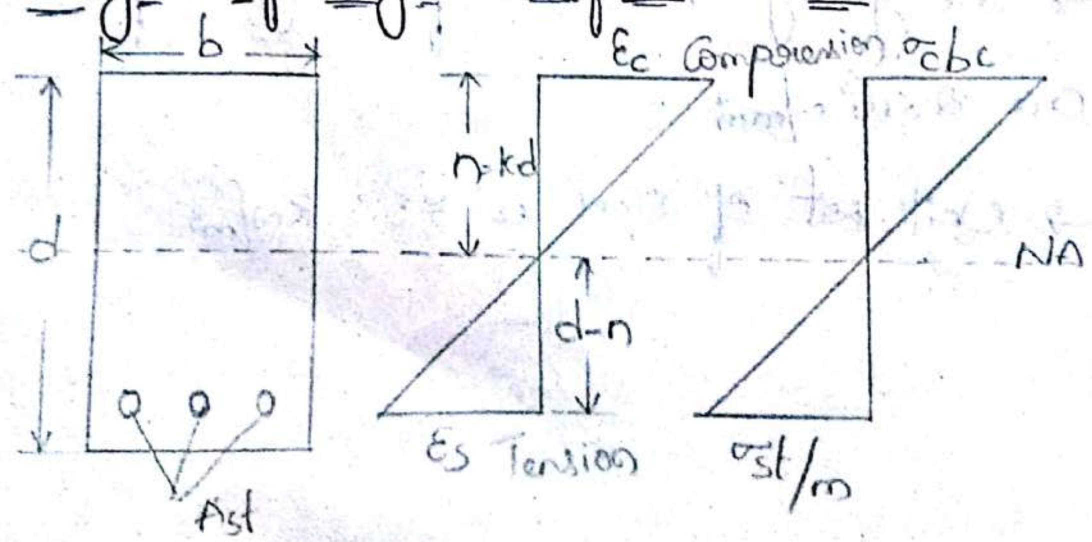
22/6/18

Methods of design of reinforced Concrete:

1. Working stress method.
2. ultimate load method
3. Limit state method

Working stress method: In this method factor of safety for Concrete is 3, steel is 1.5. In this only beams can be design. stress strain is linear.

Analysis of Single reinforced Sections:



If the reinforcement bars are provided only on tension side in the beam section. It is called as single reinforced beam sections.

1. When the stresses developed in the concrete section are known.

Let,

σ_{cbc} = Compressive stress in concrete due to bending.

σ_{st} = Tensile stress in steel

A_{st} = Area of tension steel

ε_c = Maximum strain in concrete.

ε_s = Maximum strain at the centroid of steel.

d = Effective depth.

b = width of the member.

n = Depth of neutral axis = K.d

K = Neutral axis depth factor

m = Modular ratio = $280/3\sigma_{cbc}$

Since the strains in concrete & steel are proportional to their distances from neutral axis,

$$\frac{\varepsilon_c}{\varepsilon_s} = \frac{n}{d-n}$$

$$\frac{d-n}{n} = \frac{\varepsilon_s}{\varepsilon_c}$$

$$\left(\frac{d}{n}\right)-1 = \frac{\sigma_{st}}{E_s} \times \frac{E_c}{\sigma_{cbc}}$$

$$= \frac{\sigma_{st}}{\sigma_{cbc}} \cdot \frac{1}{m} \qquad \left[\because \varepsilon_s = \frac{\sigma_{st}}{E_s}, \ \varepsilon_c = \frac{\sigma_{cbc}}{E_c}\right]$$

$$\left[\because m = \frac{E_s}{E_c}\right]$$

modular ratio

$$\Rightarrow \left(\frac{d}{n}\right)-1 = \frac{\sigma_{st}}{m.\sigma_{cbc}}$$

$$\boxed{m = \frac{280}{3\sigma_{cbc}}}$$

$$\Rightarrow \frac{d}{n} = 1+\frac{\sigma_{st}}{m.\sigma_{cbc}}$$

$$n = \left[\frac{1}{1+\frac{\sigma_{st}}{m.\sigma_{cbc}}}\right] d$$

$$n = K.d$$

where $K = \dfrac{1}{1+\dfrac{\sigma_{st}}{m.\sigma_{cbc}}}$

2. When the dimensions of the beam & the reinforcements are given:

The depth of neutral axis can be obtained by considering the equilibrium of internal forces of compression & tension

Force of compression = Force of tension

Force of compression C = Average stress × area of beam in compression

$$= \frac{\sigma_{cbc}}{2} \times b \times n$$

Force of tension T = permissible stress × Area of steel.

$$= \sigma_{st} \times A_{st}$$

$$= m.\sigma_{cbc}.\left(\frac{d-n}{n}\right) \times A_{st}$$

$$\Rightarrow \frac{\cancel{\sigma_{cbc}}}{2} \times b \times n = m.\cancel{\sigma_{cbc}}.\left(\frac{d-n}{n}\right).A_{st}$$

$$\Rightarrow \boxed{\frac{bn^2}{2} = m.A_{st}(d-n)}$$

$$\Rightarrow \cancel{\frac{bn^2}{2} = m.A_{st}(d-n)}$$

$$\Rightarrow \frac{bn^2}{2} + m.A_{st}n - m.A_{st}d = 0$$

By solving the above quadratic equation,

$$n = \frac{-m.Ast + \sqrt{(m.Ast)^2 - 2.b.mAstd}}{b}$$

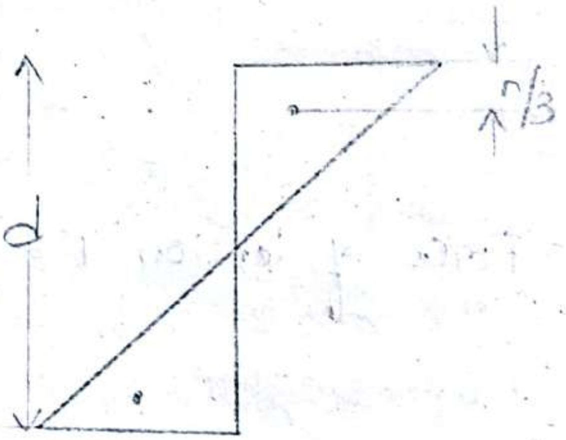
Lever arm:

The forces of compression & tension form a couple. The distance between the lines of action of compression & tension forces is called as lever arm.

$$z = d - \frac{n}{3}$$

$$= d - \frac{k.d}{3}$$

$$z = d\left(1 - \frac{k}{3}\right)$$



$$\boxed{z = j.d}$$

$\therefore$ Lever arm factor $\boxed{j = 1 - \frac{k}{3}}$

Moment of resistance:

Moment of resistance = Total compression or tension $\times$ lever arm

Total compressive force $C = \frac{1}{2} \times \sigma_{cbc} \times b \times n$

Total tensile force $T = \sigma_{st} \cdot A_{st}$

Moment of resistance (MR) = $C \cdot z$

$$= \frac{1}{2} \cdot \sigma_{cbc} \times b \times n \times (J \cdot d)$$

$$= \frac{\sigma_{cbc}}{2} \times b \times kd \times jd$$

$$= \frac{1}{2} kj \cdot \sigma_{cbc} \cdot bd^2$$

$$\therefore \boxed{MR = Q \cdot bd^2}$$

Q = moment of resistance Constant

$\therefore$ m, k, j, Q are also design Constants.

Types of section:

1. Balanced Section.
2. under reinforced Section.
3. Over reinforced Section

Balanced Section:

A reinforced Concrete Section in which steel & Concrete reach their maximum allowable stresses simultaneously is called balanced or Critical section. Neutral axis Corresponding to balanced section is called as Critical neutral axis.

The percentage steel corresponding to balanced section is called critical percentage of steel.

Under reinforced Section:

A section in which the area of steel reinforcement provided is less than that is required for a balanced section is called as under reinforced section.

$$\boxed{\text{Moment of resistance} = \sigma_{st} \cdot A_{st}\left(d - \frac{n_a}{3}\right)}$$

Over reinforced Section:

A section in which the area of steel reinforcement provided is more than that is required for a balanced section is called as ~~under~~ over reinforced section.

$$\boxed{MR = \frac{1}{2}\sigma_{cbc} \times b \times n_a\left(d - \frac{n_a}{3}\right)}$$

- A reinforced concrete beam 250mm wide, 475mm overall depth is reinforced with 3 bars of 16mm at an effective cover of 50mm using M_{20} grade concrete & Fe 415 steel. Find the depth of NA

Sol Given data,

$b = 250mm$

$d = 475 - 50$

$= 425mm$

$A_{st} = 3 \times \frac{\pi}{4}(16)^2$

$A_{st} = 603.19 mm^2$

$\sigma_{cbc} = 7 N/mm^2$ [∵ From table no: 21 for M_{20} grade]

$\sigma_{st} = 230 N/mm^2$

[∵ From table no: 22]

$\frac{bn^2}{2}$ Depth of actual neutral axis (n_a):-

$$\frac{bn^2}{2} = m \cdot A_{st}(d - n)$$

$$\frac{250 \times n^2}{2} = 13.33 \times 603.19(425 - n)$$

$$n^2 = 64.32(425 - n)$$

$$n^2 - 27337.78 + 64.32n = 0$$

$$\therefore \boxed{n_a = 136.28 mm}$$

2. A reinforced concrete beam of 300mm wide by 550mm overall depth is reinforced with 4 bars of 20mm ϕ at an effective depth of 50mm

Using M_{20} grade concrete & Fe 415 steel. Estimate the moment of resistance of the section

Sol Given data,

$b = 300mm$

$D = 550mm$

Effective depth $d = 550 - 50$

$= 500mm$

Area of tension steel $A_{st} = 4 \times \frac{\pi}{4}(20)^2$

$= 1256.6 mm^2$

$\sigma_{st} = 230 N/mm^2$

$\sigma_{cbc} = 7N/mm^2$

Design constant for M_{20} & Fe 415

$$m = \frac{280}{3\sigma_{cbc}} = 13.33$$

Critical depth of neutral axis,

$$n_c = k \cdot d$$

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m \times \sigma_{cbc}}}$$

$$= \frac{1}{1 + \frac{230}{13.33 \times 7}}$$

$k = 0.289$

$\therefore \; n_c = k \cdot d$

$= 0.289 \times 500$

$\therefore \; \boxed{n_c = 144.5 mm}$

Depth of actual neutral axis (n_a):

$$\frac{bn^2}{2} = m \cdot Ast (d - n_a)$$

$$\frac{300n^2}{2} = 13.33 \times 1256.6 (500 - n_a)$$

$$\boxed{n_a = 187 mm}$$

$n_a < n_c \rightarrow$ under reinforced

$n_a > n_c \rightarrow$ Over reinforced

$$\boxed{n_a > n_c}$$

$\therefore$ The section is over reinforced.

Moment of resistance (MR): $\frac{1}{2} \sigma_{cbc} \times b \times n_a \left(d - \frac{n_a}{3}\right)$

$$\Rightarrow \frac{1}{2} \times 7 \times 300 \times 187 \left(500 - \frac{187}{3}\right)$$

$\therefore \; \boxed{MR = 85.9 KN\text{-}m}$

3. A reinforced Concrete beam 300mm x 600mm, Over all depth is reinforced with 4 bars of 20mm at an effective Cover of 50mm. What uniformly distributed load of this beam can carry excluding Self wt, over a S.S span of 5m. Assume M_{25} grade & Fe 415 steel.

Sol

Given data,

$b = 300mm$

$D = 600mm$

Effective Cover $d = 600 - 50$

$= 550mm$

Area of tension steel $A_{st} = 4 \times \frac{\pi}{4}(20)^2$

$= 1256.6 \, mm^2$

$\sigma_{st} = 230 \, N/mm^2$ [∵ From table 22]

$\sigma_{cbc} = 8.5 \, N/mm^2$ [∵ from pg: no 81 Table 21]

Design Constant for M_{25} grade & Fe 415

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 8.5} = 10.98$$

Critical depth of neutral axis;

$n_c = k \cdot d$

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m \sigma_{cbc}}} = \frac{1}{1 + \frac{230}{10.98 \times 8.5}}$$

$$K = 0.28$$

$$n_c = 0.28 \times 550$$

$$\boxed{n_c = 158.76 mm}$$

Depth of actual neutral axis (n_a):

$$\frac{bn^2}{2} = m.A_{st}(d - n_a)$$

$$\frac{300n^2}{2} = 10.98 \times 1256.6(550 - n_a)$$

$$\boxed{n_a = 183.59 mm}$$

$$n_a > n_c$$

$\therefore$ The section is over reinforced section.

$$MR = \frac{1}{2} \times \sigma_{cbc} \times b \times n_a \left(d - \frac{n_a}{3}\right)$$

$$= \frac{1}{2} \times 8.5 \times 300 \times 183.59 \left(550 - \frac{183.59}{3}\right)$$

$$\boxed{MR = 114.42 \, kN\text{-}m}$$

Maximum load:

Let w kN/m be the uniformly distributed load beam can carry.

$$\text{Max BM} = \frac{wl^2}{8} \Rightarrow \frac{w \times 5^2}{8} = 3.125w$$

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$MR = max \cdot BM$

$114.42 = 3.125\,w$

$w = 36.51$ [Including self wt]

Self wt. of the beam $= 0.3 \times 0.6 \times 1 \times 25$

$= 4.5\ kN/m$

Uniformly distributed load excluding self wt.

$= 36.51 - 4.5$

$= 32.01\ kN/m$

4. Design a RCC beam 230mm wide to resist a BM of 30kN-m. Use M_{20} & Fe 415.

Sol Given data,

$b = 230mm$

$BM = 30kN\text{-}m$

$\sigma_{st} = 230\ N/mm^2$

$\sigma_{cbc} = 7\ N/mm^2$

Design Constants for M_{20} grade & Fe 415 steel

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m \cdot \sigma_{cbc}}} = \frac{1}{1 + \frac{230}{13.33 \times 7}} = 0.289$$

$$\boxed{k = 0.289}$$

$$j = 1 - \frac{k}{3}$$

$$= 0.904$$

$$Q = \frac{1}{2} \sigma_{cbc} \times k \times j$$

$$= \frac{1}{2} \times 7 \times 0.289 \times 0.904$$

$$= \underline{0.914}$$

Depth required,

$$MR = Qbd^2$$

$$d = \sqrt{\frac{BM}{Q \times b}} \quad [\because MR = max. BM]$$

$$= \sqrt{\frac{30 \times 10^6}{0.914 \times 230}}$$

$$\boxed{d = 377.8\,mm} \simeq 380mm$$

<u>Area</u> <u>of</u> <u>tensile</u> <u>steel</u> <u>Ast</u> :

$$MR = Ast \cdot \sigma_{st} \times j \cdot d$$

$$30 \times 10^6 = Ast \times 230 \times 0.904 \times 380$$

$$30 \times 10^6 = 79009.6\, Ast$$

$$\boxed{Ast = 379.7\,mm}$$

$d = 377.8 \approx 380mm$

$D = 380 + 40$

$= 420mm$ [$\because$ Assume 40mm]

Let,

Assume 12mm ϕ bars to find out the no. of bars

$A_{st} = \text{No. of bars} \times \frac{\pi}{4}(12)^2$

$379.9 = 113.09 \times \text{No. of bars}$

$\text{No. of bars} = \frac{379.9}{113.09}$

$= 3.35 \approx 4$ bars

$\therefore$ No. of bars $= 4$

17/17

5. Double reinforced beam is to be design having an overall c/s dimensions of 250mm x 400mm with an effective span of 4m. The beam has to support and uniformly distributed dead load of 25 KN/m together with a live load of 20KN/m in addition to its self wt adopting M_{20} grade concrete & Fe 415. Design suitable reinforcement in the beam

Effective span $L = 4m$

Breadth of beam $b = 250mm$

Overall depth $D = 400mm$

dead load $= 2.5 \; kN/m$

Live load $= 20 \; kN/m$

M_{20} grade Concrete & Fe 415 HYSD bars,

$\sigma_{cbc} = 7 \; N/mm^2$

$Q = 0.91$

$j = 0.90$

$\sigma_{st} = 230 \; N/mm^2$

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$Q = \frac{1}{2} \times \sigma_{cbc} \times k \times j$$

$\cancel{\frac{1}{2} \times 7 \times}$

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m \cdot \sigma_{cbc}}} = \frac{1}{1 + \frac{230}{13.33 \times 7}}$$

$$\boxed{k = 0.289}$$

$$j = 1 - \frac{k}{3}$$

$$= 1 - \frac{0.289}{3}$$

$j = 0.904$

$Q = \frac{1}{2} \times 7 \times 0.289 \times 0.904$

$= 0.914$

~~Depth required~~

~~MRu~~

Loads:

Self wt. of beam $= 0.25 \times 0.4 \times 25$

$W = 2.5\ kN/m$

Dead Load $= 2.5$

Live Load $= 20.00$

total load $W = 25\ kN/m$

Adopt an effective Cover of 50mm

Effective depth $= 400 - 50$

$d = 350mm$

Bending moments & shear force

$M = \frac{wl^2}{8} = \frac{25 \times 4^2}{8} = 50\ kN\text{-}m$

Depth required.

$MR = Qbd^2$

$= 0.914 \times 0.25 \times (3.5 \times 10^{-4})^2$

$M_R = 27.8 \text{ kN-m} \longrightarrow M_1$

M_1 greater than M. So, it is design by doubly reinforced section.

Balance moment $M_2 = M - M_1$

$= 50 - 27.8$

$= 22.2 \text{ kN-m}$

Tension steel required for balanced singly reinforced section,

$$A_{st_1} = \frac{M_1}{\sigma_{st} \cdot j \cdot d} \Rightarrow \frac{27.8 \times 10^6}{230 \times 0.904 \times 350}$$

$= 382 \text{ mm}^2$

Additional tension steel for balanced moment,

$$A_{st_2} = \frac{M_2}{\sigma_{st}(d - d_c)}$$

$$= \frac{22.2 \times 10^6}{230 \times (350 - 50)}$$

$= 321.7 \simeq 322 \text{ mm}^2$

Total tensile steel bars,

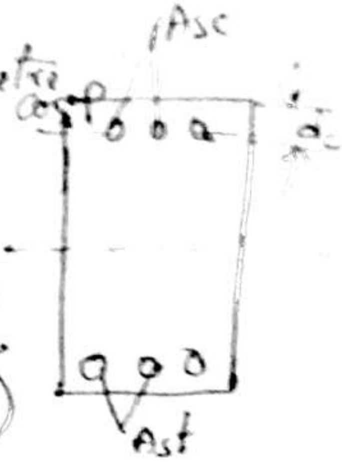
$A_{st} = A_{st_1} + A_{st_2}$

$A_{st} = 322 + 381$

$= 703 \, mm^2$

Provide 4 bars of 16mm diameter

Compression reinforcement.



$$A_{sc} = \frac{m . A_{st_2} (d - n_c)}{(1.5m - 1)(n_c - d_c)}$$

$n_c = 0.289 d$

$= 0.289 \times 350$

$= \cancel{99.015} \; 101.15$

$$A_{sc} = \frac{13.33 \times 322 (350 - \cancel{99.015} \; 101.15)}{(1.5 \times 13.33 - 1)(\cancel{99.015} \; 101.15 - 50)}$$

$= 1096.8 \, mm^2$

Assume 16mm ϕ bars to find out the no of bars

$A_{sc} = \text{No. of bars} \times \frac{\pi}{4} (16)^2$

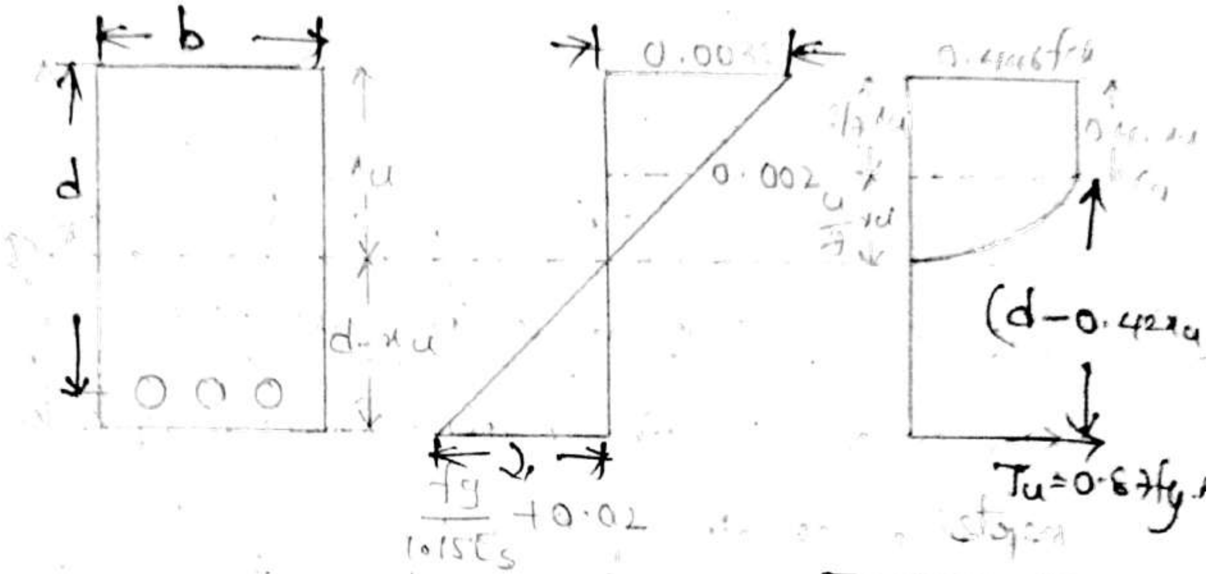
$1096.3 = \text{No. of bars} \times 201.06$

No. of bars $= 5.46 \approx 6$ bars

8, 12, 20, 16, 18, 22, 24, 25, 32, 36 — ϕ

UNIT-II

Limit state Method



$$\boxed{\frac{0.67f_{ck}}{1.5}=0.446f_{ck}}$$

$$\frac{0.002}{0.0035}=\frac{x}{x_u}$$

$$\boxed{x=\frac{4}{7}x_u}$$

Area of Concrete = parabolic + rectangle

$$=\frac{2}{3}bh+\frac{3}{7}x_u\times 0.446f_{ck}$$

$$=\left(\frac{2}{3}\times\frac{4}{7}x_u\times 0.446f_{ck}\right)$$

$$+\frac{3}{7}x_u\times 0.446f_{ck} \quad 0.1911$$

$$=0.36f_{ck}\cdot x_u$$

Distance of Centroid of stress block from the top fibre:

$$x_c=\left(\frac{3}{7}x_u\times 0.446f_{ck}\right)\left(\frac{1}{2}\times\frac{3}{7}x_u\right)+$$

$$+\left(\frac{2}{3}\times\frac{4}{7}x_u\times 0.446 f_{ck}\right)\left(\frac{3}{8}\times\frac{4}{7}x_u+\frac{3}{7}x_u\right)$$

$$\boxed{x_c = 0.42\,x_u}$$

Stress Block parameters:

Stress - strain Curve at the Crushing of Concrete is assumed to be parabolic shape upto 0.002 strain & then Constant upto the maximum strain of 0.0035.

The strain varies linearly across the depth of the section.

4/7/17 **Depth of Neutral axis (x_u):**

Force of Compression = Force of Tension

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

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

17 Moment of resistance :-

Moment = Force × distance

$$\text{Stress} = \frac{\text{Force}}{\text{Area}}$$

Moment = stress × Area × distance

(i) For balanced reinforced section,

$$x_u = x_{umax}$$

In this Concrete & steel both are failed

So,

$$\boxed{M_R = 0.36 f_{ck} \cdot b \times x_u (d - 0.42 x_u)}$$

→ for Concrete.

for steel,

$$\boxed{M_R = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)}$$

(ii) For under reinforced section,

$$x_u < x_{umax}$$

$$\boxed{M_R = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)}$$

Steel fails in this section

(iii) for over reinforced Section ($x_u > x_{umax}$)

In this section Concrete fails.

So, the moment of resistance is,

$$\boxed{MR = 0.36 f_{ck} \cdot x_{u \cdot max} \times b \,(d - 0.42 x_{umax})}$$

x_u = Actual depth of Neutral axis,

$x_{u,max}$ = Max. depth of Neutral axis.

⇒ From pg:70 in Code book:

f_y	x_{umax}/d
250	0.53
415	0.48
500	0.46

for Fe 250 steel, $x_{umax} = 0.53d$

for Fe 415 steel, $x_{umax} = 0.48d$

for Fe 500 steel, $x_{umax} = 0.46d$.

Limiting value of MR: ($M_{u,limit}$)

Since the maximum depth of NA is limited to avoid brittle failure, the maximum value of moment of resistance is also limited.

$$\overset{M_{u,lim}}{\cancel{MR}} = 0.36 f_{ck} \times b \times x_u (d - 0.42 x_u)$$

for Fe 250 steel, $x_{umax} = 0.53d$ [from Code book]

$$\overset{M_{u,lim}}{\cancel{MR}} = 0.36 f_{ck} \times b (0.53d)(d - (0.42 \times 0.53d))$$

$$\therefore \boxed{\overset{M_{u,lim}}{\cancel{MR}} = 0.148 f_{ck} b d^2}$$

for Fe 415 steel, $x_{umax} = 0.48d$ [from Code book].

$$\overset{M_{u,lim}}{\cancel{MR}} = 0.36 f_{ck} \times b \times x_u (d - 0.42 x_u)$$

$$= 0.36 f_{ck} \times b \times 0.48d (d - 0.42(0.48d))$$

$$\boxed{M_{u,lim} = 0.138 f_{ck} b d^2}$$

for Fe 500 steel, $x_{umax} = 0.46d$

$$M_{u,lim} = 0.36 f_{ck} \times b \times 0.46d (d - 0.42(0.46d))$$

$$= 0.133 f_{ck} b d^2$$

1. Find the moment carrying a singly reinforced beam of 230mm x 480mm effective depth reinforced with 3 bars of 20mm diametre. Concrete is of M_{20} grade & Fe 415 steel

Sol

Given data,

$b = 230mm$

$d = 480mm$

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

$f_y = 415 N/mm^2$

$A_{st} = \frac{\pi}{4}(20)^2 \times 3$

$\boxed{A_{st} = 942.47mm^2}$

Depth of Neutral axis:

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

$= \frac{0.87 \times 415 \times 942.47}{0.36 \times 20 \times 230}$

$\boxed{x_u = 205.48mm}$

$x_u max = 0.48d$ [for Fe 415 steel]

$= 0.48(480)$

$\therefore \boxed{x_u max = 230.4 mm}$

$\therefore \quad x_u < x_u max$

So, the section is under reinforced.

<u>Moment</u> of <u>resistance</u>:

$$MR = 0.87 fy . Ast (d - 0.42 x_u)$$

$$= 0.87 \times 415 \times 942.47 (480 - 0.42(205.43$$

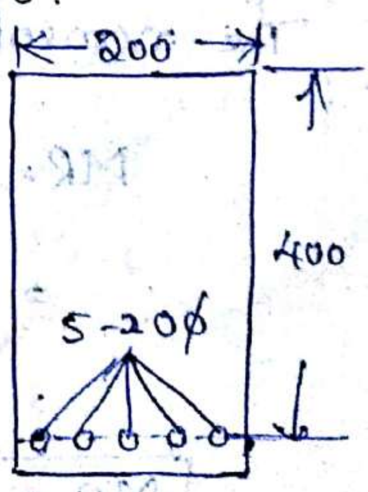
$$\boxed{MR = 183.967 \times 10^6 N/mm^2}$$

$$MR = 133.97 \times 10^6 N\text{-}mm$$

$$= 133.97 kN\text{-}m$$

2. Find the ultimate moment of resistance of singly reinforced beam 200mm × 400mm effective depth reinforced with 5 bars of 20mm ϕ. Concrete is of M_{20} grade & steel Fe 250.

Sol Given data,



$b = 200mm$

$d = 400mm$

$fck = 20 N/mm^2$

$fy = ~~415~~ 250 N/mm^2$

$A_{st} = 5 \times \frac{\pi}{4}(20)^2$

$\boxed{A_{st} = 1570.79 mm^2}$

Depth of Neutral axis:

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

$$= \frac{0.87 \times 250 \times 1570.79}{0.36 \times 20 \times 200}$$

$\boxed{x_u = 237.25 mm}$

$x_{u,max} = 0.53 d$ [for Fe 250 steel]

$= 0.53 \times 400$

$\boxed{x_{u,max} = 212 mm}$

$\therefore x_u > x_{u,max}$

So, the section is over reinforced

The moment of resistance is:

$MR = 0.36 f_{ck} x_{u,max} \times b (d - 0.42 x_{u,max})$

$= 0.36 \times 20 \times 212 \times 200 (400 - (0.42 \times 212))$

$= 94.92 \times 10^6 N-mm$

$\boxed{MR = 94.92 kN-m}$

$Mu = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)$

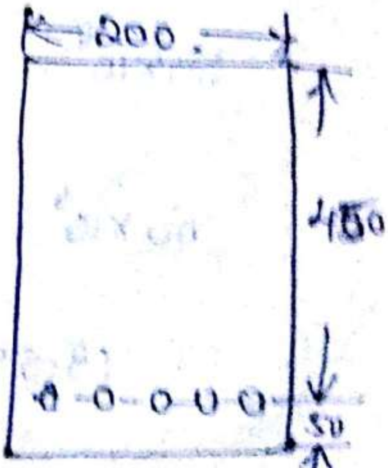
To determine the A_{st} dimensions of the beam & MR is given

$$M_u = 0.87 f_y . A_{st} (d - 0.42 x_u)$$

$$= 0.87 f_y . A_{st} \left(d - 0.42 \left(\frac{0.87 f_y . A_{st}}{0.36 f_{ck} . b}\right)\right)$$

$$\therefore \boxed{M_u = 0.87 f_y . A_{st} . d \left[1 - \frac{f_y . A_{st}}{f_{ck} \times bd}\right]}$$

2. A reinforced Concrete beam has a section of 200 x 500mm overall. It is subjected to a factored moment of 80 kN-m. Design the reinforcement using Fe 250 steel & M_{20} grade Concrete. use effective cover of 50mm



Sol

Given data,

$b = 200mm$

$d = 500 - 50$

$= 450mm$

$M_u = 80 kN-m$

$M_u < M_{u\,limit} \rightarrow$ Single reinforced beam

$M_u > M_u$ limit [Doubly reinforced beam]

$f_y = 250\ N/mm^2$

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

$M_u, \text{limit} = 0.148 f_{ck} b d^2$

$= 0.148 \times 20 \times 200 \times 450^2$

$= 119.88 \times 10^6\ N\text{-}mm$

$= 119.88\ kN\text{-}m$

$\therefore M_u < M_u$ limit (single reinforced)

$$\therefore \quad M_u = 0.87 f_y . A_{st} \times d \left[1 - \frac{f_y . A_{st}}{f_{ck} b d}\right]$$

$$80 \times 10^6 = 0.87 \times 250 \times A_{st} \times 450 \left[1 - \frac{250 \times A_{st}}{20 \times 200 \times 450}\right]$$

$$80 \times 10^6 = 97875 A_{st} \left[1 - 1.388 \times 10^{-4} A_{st}\right]$$

$$80 \times 10^6 = 97875 A_{st} - 13.593 A_{st}^2$$

$$13.593 A_{st}^2 - 97875 A_{st} + 80 \times 10^6 = 0$$

$$\boxed{A_{st} = 940.2 mm^2}$$

Provide 20mm ϕ for finding no. of bars

$$A_{st} = \text{No. of bars} \times \frac{\pi}{4}(20)^2$$

$$940.2 = \text{No. of bars} \times 314.15$$

$$314.15 \times \text{No. of bars} = 940.2$$

$$\text{No. of bars} = 2.99 \approx 3$$

$$\therefore \boxed{\text{No. of bars} = 3}$$

General Design Considerations for beam:

1. Effective span: The effective span of S.S beam shall be taken as clear span plus effective depth of the beam (or) Centre to Centre distance b/w the supports which ever is less

2. Limiting stiffness: $\left(\frac{l}{d}\right)$ ratio

   Cantilever – 7

   Simply supported – 20

   Continuous – 26.

   l = Effective span

   d = Effective depth

3. Minimum reinforcement = $\frac{A_{st\,min}}{bd} = \frac{0.85}{f_y}$

4. Maximum reinforcement = $0.04\,bD$

4. A Singly reinforced Concrete beam section 200x450mm is reinforced with 4 bars of 20mm ϕ with an effective Cover of 40mm. The beam is S.S over a span of 4m. Find the Safe uniform distributed load the beam Can Carry. Use M_{20} grade Concrete & Fe 415 steel

Sol

Given data,

$b = 200\,mm$

$d = 450 - 40$

$d = 410\,mm$

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

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

$A_{st} = 4 \times \frac{\pi}{4}(20)^2$

$= 1256.6\,mm^2$

Depth of Neutral axis:

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

$$= \frac{0.87 \times 415 \times 1256.6}{0.36 \times 20 \times 200}$$

$$\boxed{x_u = 315.1\,mm}$$

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$x_{u\,max} = 0.48d$ [for Fe 415 steel]

$= 0.48 \times 410$

$= 196.8\,mm$

$$\boxed{x_{u\,max} = 196.8\,mm}$$

$$\therefore \boxed{x_u > x_{u\,max}}$$

$\therefore$ So, the Section is over reinforced.

<u>Moment of resistance:</u> (MR)

$MR = 0.36 f_{ck} \cdot x_{u\,max} \times b (d - 0.42\, x_{u\,max})$

$= 0.36 \times 20 \times 196.8 \times 200 (410 - 0.42 (196.8))$

$= 92.76 \times 10^6\, N\text{-}mm$

$= 92.76\, kN\text{-}m$

$$\boxed{M_u = 92.76\, kN\text{-}m}$$

<u>Safe load:</u>

$BM = \frac{wl^2}{8} = \frac{w(4)^2}{8} = 2w$

Equate $BM = M_u$

$2w = 92.76$

$$\omega = \frac{92.76}{2}$$

$$\boxed{\omega = 46.38\ kN/m}$$

Safe working load $\omega = \frac{\text{Bending moment } (\omega_u)}{\text{load factor}}$

$$= \frac{46.38}{1.5}$$

$= 30.92\ kN/m$ (including self wt)

Self wt of the beam $= 0.2 \times 0.45 \times 25$

$= 2.25\ kN/m$

$\therefore$ Net Super imposed load $= 30.92 - 2.25$

$= 28.67\ kN/m$

7/17

General Design requirements for beams:

5. ~~Cover~~ Spacing of bars: The horizontal distance between two parallel main reinforcement bars shall usually be not less than the greatest of the following.

(a). Diametre of the bar if the diameters are equal.

(b) Diameter of the largest bar if the bars are unequal.

(c) 5mm more than the normal max. size of the aggregate.

Design procedure: (l, f_{ck}, f_y, b)

1. Assuming $\frac{l}{d}$ ratio (10 to 15)

Assume $b = 230mm$

2. Effective span:

a) clear span + d

b) clear span + $\frac{b}{2} + \frac{b}{2}$ } ~~max~~ min

3. Calculation of loads & BM

Self weight = volume of beam x density of Concrete

$= b \times d \times 1 \times 25$

$BM = \frac{wl^2}{8}$

factored BM, $M_u = 1.5 \times BM$

$RCC = 25 kN/m^3$

$PCC = 24 kN/m^3$

4. Check of depth required,

Mulimit is equating to Mu ($M_u = M_{u\,lim}$)

for Fe 415, $M_{u\,lim} = 0.138\, f_{ck}\, b\, d^2$

$$d = \sqrt{\frac{M_{u\,limit}}{0.138\, f_{ck} \cdot b}}$$

5. Calculated area of reinforcement

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

$$A_{st} = ?$$

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

6. check for deflection,

5. Design a rectangular S.S reinforced concrete beam over a clear span of 4m. If the super imposed load is 20kN/m and support width is 300mm each. Use M_{20} and Fe 415 steel. Check for deflection also.

Sol Given data,

clear span $L = 4m \Rightarrow 4000mm$

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

$f_y = 415 N/mm^2$

$b = 300mm$

Super imposed load = 20 kN/m.

(i) Assuming $\frac{l}{d} = 12$ based on

Stiffness $d = \frac{4000}{12}$ $\qquad \frac{4000}{d} = 12$

$12d = 4000$

$d = 333.33\,mm \approx 350\,mm$ $\qquad d = \frac{4000}{a}$

$$\boxed{d = 350\,mm}$$

(ii) Effective Cover assuming = 50 mm

Overall depth = 50 + 350

$$\boxed{D = 400\,mm}$$

(ii) <u>Effective Span:</u>

It is the least of the following

(i) Centre to Centre = clear span + $\frac{b}{2}$ + $\frac{b}{2}$

$= 4000 + \frac{300}{2} + \frac{300}{2}$

$= 4300\,mm \Rightarrow 4.3\,m$

(ii) clear span + d = 4000 + 350

= 4350 mm

= 4.35 m

∴ Hence effective span $l = 4.3\,m$

(iii) Calculation of Loads & BM

Self wt = ~~0.3~~ 0.3 × 0.4 × 1 × 25

= ~~2.625 kN/m~~ 3 kN/m

$$BM = \frac{wl^2}{8}$$

Super imposed load = 20 kN/m

∴ Total load = 20 + 3

= 23 kN/m

∴ Total load = 23 kN/m

$$BM = \frac{23 \times 4.3^2}{8}$$

∴ $\boxed{BM = 53.15 \text{ kN-m}}$

factored BM, = 1.5 × B

M_u = 1.5 × BM

= 1.5 × 53.15

$\boxed{M_u = 79.74 \text{ kN-m}}$

= 0.138 fck bd²

= 0.138 × 20 × 300 × 50²

<u>Step 4:</u> Check of depth required.

$M_u = M_{u limit}$

for Fe 415, $M_{u lim} = 0.138 f_{ck} b d^2$

$$d = \sqrt{\frac{M_{u limit}}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{79.74 \times 10^6}{0.138 \times 20 \times 300}}$$

$\boxed{d = 310.32\,mm}$ < d provided ($d = 350mm$)

Hence provided depth is adequate

17 Step 5: Area of reinforcement.

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

$$79.74 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \left[1 - \frac{415 A_{st}}{20 \times 300 \times 350}\right]$$

$$79.46 \times 10^6 = 126.36 \times 10^3 A_{st} \left[1 - 1.976 \times 10^{-4} A_{st}\right]$$

$$79.46 \times 10^6 = 126.36 \times 10^3 A_{st} - 24.97 A_{st}^2$$

$$24.97 A_{st}^2 - 126.36 \times 10^3 A_{st} + 79.46 \times 10^6 = 0$$

$$\boxed{A_{st} = 735.8\,mm^2}$$

Provide ~~4 bars of~~ 16mm ϕ for finding No. of bars

$$A_{st} = \text{No. of bars} \times \frac{\pi}{4}(16)^2$$

$$735.8 = \text{No. of bars} \times 201.06$$

$$201.06 \text{ No. of bars} = 735.8$$

$\therefore$ No. of bars = $\frac{735.8}{201.06}$

$= 3.65 \approx 4$ bars.

$\therefore$ No. of bars = 4 bars.

<u>Minimum</u> reinforcement: (pg: 47)

$\Rightarrow \left(\frac{A_{st\,min}}{bd}\right) = \frac{0.85}{f_y}$

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

$= \frac{0.85 \times 300 \times 350}{415}$

$\therefore \boxed{A_{st\,min} = 215.06\,mm^2}$

<u>Maximum</u> reinforcement:

$A_{st\,max} = 0.04 \times bD$

$= 0.04 \times 300 \times 400$

$= 4800\,mm^2$

$\therefore \boxed{A_{st\,max} = 4800\,mm^2}$

Ast ~~provided~~ required = 735.8 mm^2

Ast provided = $\frac{\pi}{4}(16)^2 \times 4$

= 804.24 mm^2

6. Check for deflection: (stiffness)

For S.S beam basic value $\left(\frac{l}{d} = 20\right)$

In Code book, pg: no – 38

Modification factor for tension reinforcement.

$$f_s = 0.58 f_y \times \frac{\text{Area of c/s of Ast required}}{\text{Area of c/s of Ast provided}}$$

$$= 0.58 \times 415 \times \frac{735.8}{804.24}$$

$$\boxed{f_s = 220.22 \, N/mm^2}$$

$$\% \text{ of steel} = \frac{\text{Ast provided}}{bd} \times 100$$

$$= \frac{804.24}{300 \times 350} \times 100$$

$$= 0.76\%$$

From Fig. 4 of IS : 456, modification factor = 1.15

Max. permitted $\frac{l}{d}$ ratio = 1.15 × 20
= 23

$\frac{l}{d}$ provided = $\frac{4300}{350}$

= 12.28 < 23

∴ Hence deflection control is safe.

6. Design a rectangular beam for effective span 4m. which is subjected to dead load of 15kN/m and a live load of 12kN/m. use M15 & Fe 250 grade steel. Assume b = 250mm

Sol Given data,

Effective span l = 4m ⇒ 4000mm

f_{ck} = 15N/mm^2

f_y = 250 N/mm^2

b = 250mm

Dead load = 15kN/m

Live load = 12kN/m

(i) Assuming $\frac{l}{d}$ = 10 based on

Stiffness $d = \frac{4000}{10}$

$$\boxed{d = 400mm}$$

Assuming effective cover = 50mm.

Overall depth = 50 + 400

= 450mm

$$\boxed{D = 450mm}$$

(ii) Effective span $l = 4000mm$

(iii) Calculation of loads & BM

Self wt = $0.25 \times 0.45 \times 1 \times 25$

= 2.8125 kN/m

~~BM~~

Dead load = 15 kN/m

Live load = 12 kN/m

Total load = 15 + 12 + 2.8125

= 29.81 kN/m

∴ Total load = 29.81 kN/m

$$BM = \frac{29.81 \times 4^2}{8}$$

$$\boxed{BM = 59.62 \text{ kN/m}}$$

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Factored Bending moment,

$$M_u = 1.5 \times BM$$
$$= 1.5 \times 59.62$$
$$\therefore \boxed{M_U = 89.43 \text{ kN-m}}$$

Step 4: Depth required,

$$M_u = M_u \text{limit}$$

for F_e 250 steel, M_u limit = $0.148 f_{ck} b d^2$

$$d = \sqrt{\frac{M_u \cancel{\text{limit}}}{0.148 f_{ck} b}}$$

$$= \sqrt{\frac{89.43 \times 10^6}{0.148 \times 15 \times 250}}$$

$$\boxed{d = 401.4 \text{mm}} > \boxed{d = 400}$$

from Code book procedure $\frac{l}{d}$ ratio (10 to ...)

Take, $\frac{l}{d}$ ratio = 9

So, procedure will be redesign

(i) Assuming $\frac{l}{d} = 9$ based on

stiffness $d = \frac{4000}{9}$

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$d = 444.4mm \simeq 450mm$

$$\boxed{d = 450mm}$$

Assuming effective Cover = 50mm

$D = 450 + 50$

$= 500mm$ · $M_u < M_{u limit}$ – Singly

$$\boxed{D = 500mm}$$

$M_u > M_{u limit}$ – Doubly reinforced beam

(ii) Effective span $l = 4000mm$

(iii) Calculation of loads & BM:

Self wt = $0.25 \times 0.5 \times 1 \times 25$.

$= 3.125 kN/m$

Dead load = $15kN/m$.

Live load = $12kN/m$

Total load = $3.125 + 15 + 12$

$= 30.125 kN/m$

$$BM = \frac{30.125 \times 4^2}{8}$$

$$\boxed{BM = 60.25kNm}$$

Factored Bending moment,

$M_u = 1.5 \times 60.25$

$$\therefore \boxed{M_u = 90.375 \text{ kN-m}}$$

Step 4: Depth required,

$$M_u = M_{u\,limit}$$

for Fe 250 steel, $M_{u\,limit} = 0.148 f_{ck} b d^2$

$$d = \sqrt{\frac{M_{u\,limit}}{f_{ck} \times 0.148 \times b}}$$

$$= \sqrt{\frac{90.375 \times 10^6}{15 \times 0.148 \times 250}}$$

$$\boxed{d = 403.53 \text{ mm}} < 450 \text{ mm}$$

Hence provided depth is adequate.

Step 5: Area of reinforcement,

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

$$= 0.87 \times 250 \times A_{st} \times 450 \left[1 - \frac{250 A_{st}}{15 \times 250 \times 450}\right]$$

$$= 97.875 \times 10^3 A_{st} - 14.5 A_{st}^2 = 0$$

$$14.5 A_{st}^2 - 97.875 \times 10^3 A_{st} + 90.375 \times 10^6$$

$\boxed{A_{st} = 1103.9\,mm^2}$ required.

Provide 16mm ϕ for finding no. of bars

$$A_{st} = \text{No. of bars} \times \frac{\pi}{4}(16)^2$$

$$201.06\ \text{No. of bars} = 1103.9$$

$$\text{No. of bars} = \frac{1103.9}{201.06}$$

$$= 5.49 \approx 6\ \text{bars}$$

$$\therefore \boxed{\text{No. of bars} = 6}$$

$$A_{st}\ \text{provided} = \frac{\pi}{4}(16)^2 \times 6$$

$$= 201.06 \times 6$$

$$= 1206.37\,mm^2$$

$$\boxed{A_{st}\ \text{provided} = 1206.37\,mm^2}$$

6. Check for deflection: (Stiffness)

For s.s beam basic value $\left(\frac{l}{d} = 20\right)$.

In Code book pg: no-38

Modification for tension reinforcement,

$$f_s = 0.58 f_y \times \frac{\text{Area of c/s of } A_{st} \text{ required}}{\text{Area of c/s of } A_{st} \text{ provided}}$$

$$f_s = 0.58 \times 250 \times \frac{1103.9}{1206.37}$$

$$\boxed{f_s = 132.68\, N/mm^2}$$

$$\%\text{ of steel} = \frac{Ast\ provided}{bd} \times 100$$

$$= \frac{1206.37}{250 \times 450} \times 100$$

$$= 1.072\%$$

From fig.4 of IS: 456, modification factor = 1.5

Max. permitted $\frac{l}{d}$ ratio = 1.5×20

$= 30$

$\frac{l}{d}$ provided = $\frac{4000}{400}$

$= 10 < 30$

∴ Hence deflection control is safe.

Min reinforcement: Step 5:-

$$\frac{Ast_{min}}{bd} = \frac{0.85}{fy}$$

$$Ast_{min} = \frac{0.85 \times 250 \times 400}{250}$$

$$\boxed{Ast_{min} = 340 mm^2}$$

Max. reinforcement:

$$A_{st\,max} = 0.04 \times bD$$

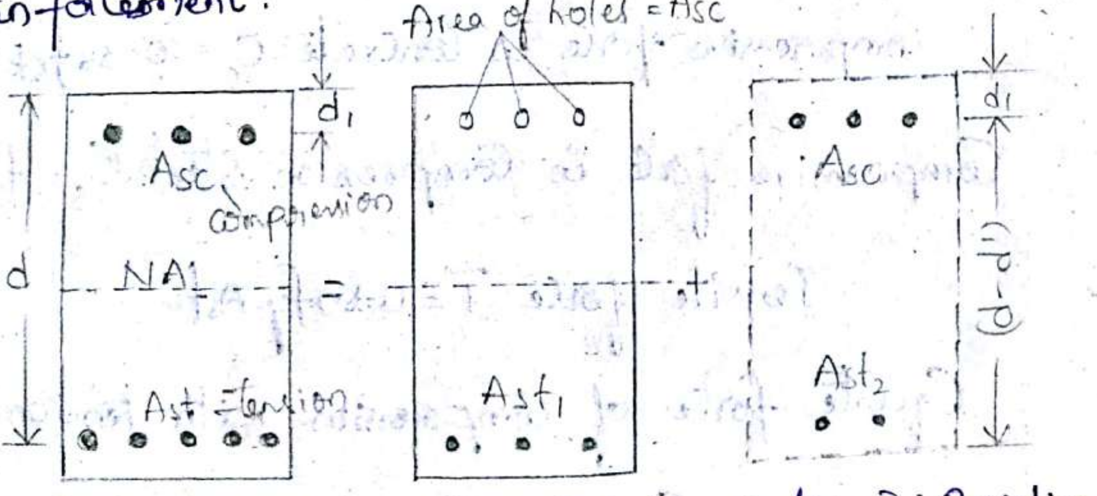
$$= 0.04 \times 250 \times 450$$

$$\boxed{A_{st\,max} = 4500\,mm^2}$$

15/3/17

Doubly reinforced beams:

Beams which are reinforced in both compression & tension sides are called as doubly reinforced beam. These beams are generally provided when the dimensions of the beam are restricted & it is required to resist moment higher than the limiting moment of resistance of a singly reinforced section. The additional moment of resistance required can be obtained by providing compression reinforcement & additional tension reinforcement.



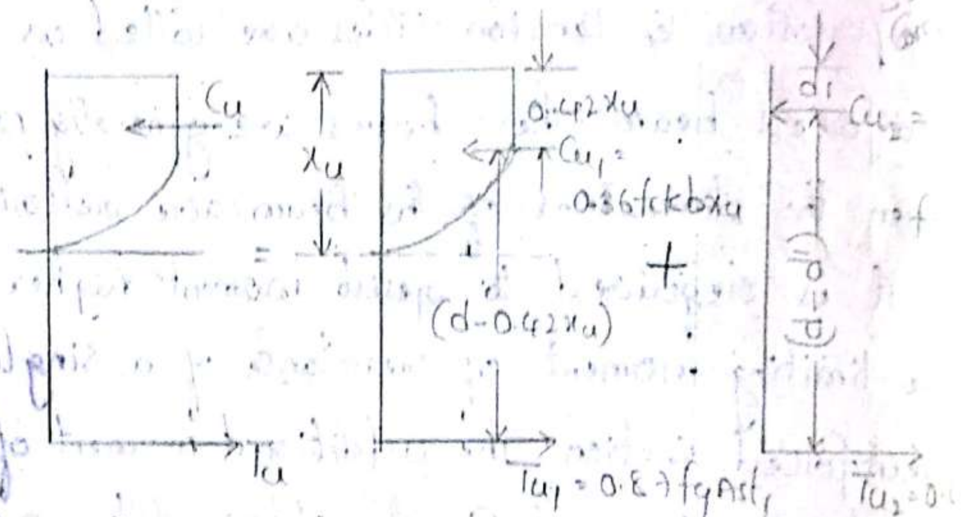
Section subjected to moment M_u | Section-I: Resisting moment $M_{u_1} = M_{ulim}$ | Section-II: Resisting balance moment $M_{u_2} = M_u - M_{ulim}$

This doubly reinforced section can be considered to be composed of two sections as given below

(a). A singly reinforced section with $M_{u,lim}$

(b) A section with compression steel & additional tension steel to resist additional moment

$M_{u2} = M_u - M_{u,lim}$ i.e, a steel beam without concrete.



1. Depth of Neutral axis :

Compressive force in concrete $C_c = 0.36 f_{ck} \cdot b \cdot x_u$

Compressive force in compression steel $C_s = f_{sc} \cdot A_{sc}$

Tensile force $T = 0.87 f_y \cdot A_{st}$

Equate force of compression with tension

$C_c + C_s = T$

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

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

17/7/17

2. ultimate moment of resistance:

The ultimate moment of resistance of doubly reinforced is given by,

$$M_u = \underbrace{M_{u_1}}_{\text{concrete}} + \underbrace{M_{u_2}}_{\text{steel}}$$

$$= 0.36 f_{ck} \cdot b \cdot x_u (d - 0.42 x_u) + f_{sc} \cdot A_{sc} (d - d')$$

when,

$x_u > x_{u,max}$, x_u is limited to $x_{u,max}$.

$$\boxed{M_u = 0.36 f_{ck} \cdot b \cdot x_{umax} (d - 0.42 x_{umax}) + f_{sc} \cdot A_{sc} (d - d')}$$

3. Area of Compression steel:

Additional moment of resistance M_{u_2}

$$M_{u_2} = f_{sc} \cdot A_{sc} (d - d')$$

$$\therefore \boxed{A_{sc} = \frac{M_{u_2}}{f_{sc}(d - d')}}$$

The maximum area of compression reinf. shall not exceed 0.04bD i.e., 4% of gross area.

4. Area of Tension steel: The limiting moment of resistance of singly reinforced section is given by,

$$M_{u,lim} = 0.87 f_y \cdot A_{st_1} (d - 0.42 x_{u,max})$$

$$A_{st_1} = \frac{M_{u,lim}}{0.87 f_y (d - 0.42 x_{u,max})}$$

Additional area of tensile steel (A_{st_2}) can be calculated by equating the compressive force in compression steel & tensile force in additional tension steel.

$$\therefore 0.87 f_y \cdot A_{st_2} = f_{sc} \cdot A_{sc}$$

$$\boxed{A_{st_2} = \frac{f_{sc} \cdot A_{sc}}{0.87 f_y}}$$

A_{st_2} can also be calculated by using,

$$M_{u_2} = 0.87 f_y \cdot A_{st_2} (d - d').$$

$$\therefore \boxed{A_{st_2} = \frac{M_{u_2}}{0.87 f_y (d - d')}}$$

Total area of tension steel $A_{st} = A_{st_1} + A_{st_2}$

Stress in Compression steel (f_{sc}) based on d'/d

Table 3.5:

stress in compression steel f_{sc}, N/mm^2 in doubly reinforced beams with cold worked bars.

(Table-F in SP16) when $d'/d < 0.2$

Grade of steel	d'/d			
	0.05	0.10	0.15	0.20
Fe 415	355	353	342	329.
Fe 500	424	412	395	370

For $d'/d < 0.2$, f_{sc} for mild steel is $0.87 f_y$.

7. Calculate the ultimate moment of resistance of an R.C beam of rectangular section 300mm wide & 400mm deep. Area of steel consists of 6 Nos 18ϕ in tension side & 2 Nos 18ϕ in compression side. Assume steel of grade Fe 415 & concrete of grade M_{20} & an effective cover 35mm on both sides.

Sol Given data,

$b = 300mm$

$d = 400 - 35$

$= 365mm$

$d' = 35mm$

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

$f_y = 415 N/mm^2$

$A_{st} = 6 \times \frac{\pi}{4}(18)^2 = 1526.8 mm^2$

$A_{sc} = 2 \times \frac{\pi}{4}(18)^2 = 508.91 mm^2$

1. Stress in Compression steel:

$$\frac{d'}{d} = \frac{35}{365} = 0.09$$

$$\frac{d'}{d} = 0.09 < 0.2$$

from ~~Code~~ Text book table 3.5.

0.05 - 355

0.10 - 353

0.09 - ?

$$\Rightarrow 355 - \left(\frac{355 - 353}{0.10 - 0.05}\right)(0.09 - 0.05)$$

$$\boxed{f_{sc} = 353.4 N/mm^2}$$

2. Depth of Neutral Axis:

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

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$$= \frac{0.87 \times 415 \times 1526.8 - (353.4 \times 508.91)}{0.36 \times 20 \times 300}$$

$$\boxed{x_u = 171.94\,mm}$$

Limiting depth of neutral axis:-

$x_{umax} = 0.48d$

$= 0.48 \times 365$

$= 175.2\,mm$

$\therefore \boxed{x_u < x_{umax}}$

So, the section is under reinforced.

3. Moment of resistance:-

$M_u = 0.36 f_{ck} \cdot b \cdot x_u (d - 0.42 x_u) + f_{sc} \cdot A_{sc} (d - d')$

$\Rightarrow 0.36 \times 20 \times 300 \times 171.94 (365 - (0.42 \times 171.94))$

$+ (353.4 \times 508.91)(365 - 35)$

$\Rightarrow 167.62 \times 10^6\,N\text{-}mm$

$M_u = 167.62 \times 10^6\,N\text{-}mm$

$= 167.62\,kN\text{-}m$

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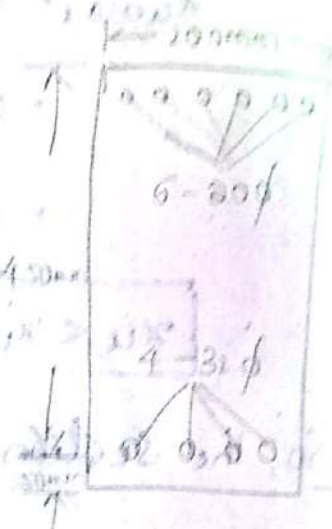
19/7/17

8. A rectangular beam is 200mm wide & deep it is reinforced beam 6 bars of 20 In compression with an effective cover of Determine the area of tension reinforcement to make the beam section fully effective than would be the moment of resistance of concrete & Fe 415 steel.

Sol



Given data,

$b = 200mm$

$d = 500 - 50$

$= 450mm$

$D = 500mm$

$A_{sc} = 6 \times \frac{\pi}{4}(20)^2 \Rightarrow 1884.95mm^2$

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

$f_y = 415 N/mm^2$

$d' = 50mm$

A_{st_1} is provided based on $M_{u lim}$:-

$M_{u.lim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 200 \times 450^2$

$= 111.78 \times 10^6$ N-mm

$= 111.78$ kN-m

$$A_{st_1} = \frac{M_{ulim}}{0.87 f_y (d - 0.42 x_{u,max})}$$

$x_{u}max = 0.48d$

$= 0.48 \times 450$

$= 216mm$

$$\therefore A_{st_1} = \frac{111.78 \times 10^6}{0.87 \times 415 (450 - 0.42(216))}$$

$$\boxed{A_{st_1} = 861.71 mm^2}$$

$$A_{st_2} = \frac{f_{sc} \cdot A_{sc}}{0.87 f_y}$$

for fsc value,

$$\frac{d'}{d} = \frac{50}{450} \simeq 0.11$$

from table,

0.10 - 353

0.15 - 342

$$f_{sc} = 353 - \left(\frac{353 - 342}{0.15 - 0.10}\right)(0.1\cancel{1} - 0.10)$$

$$\boxed{f_{sc} = 350.8 N/mm^2}$$

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$$Ast_2 = \frac{fsc \cdot Asc}{0.87fy}$$

$$= \frac{350.8 \times 1889.95}{0.87 \times 415}$$

$$= 1831.43mm^2$$

$$\boxed{Ast_2 = 1831.43mm^2}$$

$$Ast = Ast_1 + Ast_2$$

$$= 861.71 + 1831.43$$

$$= 2693.14mm^2$$

$$\therefore \boxed{Ast = 2693.14\ mm^2}$$

= no. of bars $\times \frac{\pi}{4}$(

1804.24 no. of bars

= 2693.14

No. of bars = 4

$$x_u = \frac{0.87fy \cdot Ast - fsc \cdot Asc}{0.36fck \times b}$$

$$= \frac{0.87 \times 415 \times 2693.14 - (350.8 \times 1889.95)}{0.36 \times 20 \times 200}$$

$$x_u = 216.05mm$$

$$\therefore x_u = x_{umax}$$

$\therefore$ So, the section is balanced section.

$$M_u = 0.36 f_{ck} \times b \times x_{u\,max}(d - 0.42 x_u) + f_{sc} \cdot A_{sc}(d - d')$$

$$= 0.36 \times 20 \times 200 \times 216 \left(450 - 0.42(216)\right) + (350.8 \times 1884.95)(450 - 50)$$

$$= 376.24 \times 10^6 \text{ N-mm}$$

$$\boxed{M_u = 376.24 \text{ kN-m}}$$

9. Determine the main tensile & Compression reinforcement required for a rectangular beam with a following data. 1. Overall size of the beam = 250 × 550mm

2. Concrete grade = M20
3. Fy - 415 steel
4. Factored moment = 200kN-m
5. Effective Cover = 50mm

Sol Given data,

$b = 250$mm

$d = 500$mm

$D = 550$mm

$d' = 50$mm

$f_{ck} = 20$N/mm²

Scanned by CamScanner

$f_y = 415 N/mm^2$

$M_u = 200 kN\text{-}m$

1. Limiting moment of resistance of the given Section as singly reinforced section:

$M_{ulim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 250 \times 500^2$

$= 172.5 \times 10^6 N\text{-}mm$

$\boxed{M_{u,lim} = 172.5 \ kN\text{-}m}$

$\therefore \boxed{M_u > M_{u,lim}}$

So, the section should be designed as doubly reinforced section.

2. Area of tension steel Corresponding to $M_{u,lim}$

$$Ast_1 = \frac{0.85\ M_{ulim}}{0.87 f_y (d - 0.42 x_{umax})}$$

$x_{umax} = 0.48 d$

$= 0.48 \times 500$

$= 240 mm$

$$Ast_1 = \frac{172.5 \times 10^6}{0.87 \times 415 (500 - (0.42 \times 240))}$$

$$\boxed{Ast_1 = 1196.82 mm^2}$$

3. $Mu_2 = Mu - Mu_{lim}$

$= 200 - 172.5$

$$\boxed{Mu_2 = 27.5 kN\text{-}m}$$

$Mu_2 = f_{sc} \cdot A_{sc} (d - d')$

$\frac{d'}{d} = \frac{50}{500} = 0.1$

from table 3.5

$0.10 - 353$

So, $f_{sc} = 353 N/mm^2$

$Mu_2 = 353 \times A_{sc} (500 - 50)$

$27.5 \times 10^6 = 158.85 \times 10^3 A_{sc}$

$158.85 \times 10^3 A_{sc} = 27.5 \times 10^6$

$$A_{sc} = \frac{27.5 \times 10^6}{158.85 \times 10^3}$$

$\boxed{A_{sc} = 173.11 mm^2}$ required.

4. Additional tensile steel (A_{st_2}):

$$A_{st_2} = \frac{f_{sc} \cdot A_{sc}}{0.87 f_y}$$

$$= \frac{353 \times 173.1}{0.87 \times 415}$$

$$\boxed{A_{st_2} = 169.24\, mm^2}$$

∴ Total tension steel,

$$A_{st} = A_{st_1} + A_{st_2}$$

$$= 1196.82 + 169.24$$

$$\boxed{A_{st} = 1366.06\, mm^2}$$ required

Assume 20mm ϕ to finding no. of bars

$$A_{st} = \text{No. of bars} \times \frac{\pi}{4}(20)^2$$

$$1366.06 = \text{No. of bars} \times 314.15$$

$$314.15\ \text{No. of bars} = 1366.06$$

$$\text{No. of bars} = 4.3 \approx 5\ \text{bars}$$

$$A_{st} = 5 \times \frac{\pi}{4}(20)^2$$

$$= 1570.7\, mm^2$$

$\boxed{Ast\ provided = 1570.7\ mm^2}$

Provide 12mm ϕ to find out no. of bars in Compression

$$A_{sc} = No.\ of\ bars \times \frac{\pi}{4}(12)^2$$

$$173.11 = 113.09\ No.\ of\ bars$$

$$No.\ of\ bars = 1.53 \simeq 2$$

$$\therefore No.\ of\ bars = 2$$

$$A_{sc} = 2 \times \frac{\pi}{4}(12)^2$$

$$= 226.18\ mm^2$$

$\therefore$ Provided $A_{sc} = 226.18\ mm^2$

Design of shear: [Pg: No - 72, 73]

τ_v = nominal shear stress

τ_c = shear strength resistance by Concrete [from table No: 19]

τ_{cmax} = Max. shear stress [Table no: 20

$$\tau_v = \frac{V}{bd} \ ; \ V = shear\ force$$

$\tau_v > \tau_c$ - provided shear reinforcement

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Shear reinforcement has to be provided against diagonal tensile stresses caused by th shear force. The longitudinal bars do not pr the diagonal tension failure.

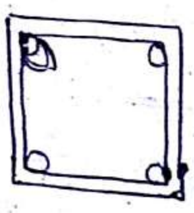
The inclined shear crack starts at th bottom near the support and extend toward compression zone. The shear reinforcement can provided in any of the following forms.

(a). Vertical stirrups.

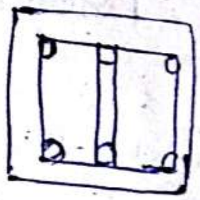
(b). Bent up bars along with stirrups.

(c) Inclined stirrups.

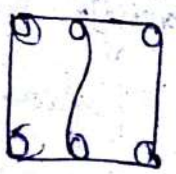
One legged stirrup



Two legged stirrup



Four legged stirrup



Three legged stirrup.

22/7/17 $\tau_c < \tau_v \rightarrow$ minimum reinforcement is to be provided.

from Code book - Table no:19 - τ_c values.

$\tau_{c\,max}$ - Table no:20.

10. A simply supported R.C.C beam 250mm x 450mm (effective) is reinforced with 4 No.s of 18mm ϕ bars. Design the shear reinforcement if M_{20} grade concrete & Fe 415 steel is used & beam is subjected to a force of 150kN at service load

Sol Given data,

$b = 250mm$

$d = 450mm$

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

$f_y = 415\ N/mm^2$

Shear force = 150kN

Factored load shear force $V_u = 1.5 \times 150$

$= 225kN$

$$\tau_v = \frac{V_u}{bd} = \frac{225 \times 10^3}{250 \times 450} = 2N/mm^2$$

from Code book table no:19

$Ast = 4 \times \frac{\pi}{4}(18)^2$

$= 1017.8 mm^2$

$\% \text{ of steel} = \frac{1017.8}{250 \times 450} \times 100$

$= 0.9047$

from table no:19

0.75 − 0.56

1.00 − 0.62

0.9047 − ?

$$\tau_c = 0.56 + \left(\frac{0.56 - 0.62}{1.00 - 0.75}\right)(0.9047 - 0.75)$$

$\tau_c = 0.52 N/mm^2$, $\tau_{cmax} = 2.8 N/mm^2$

$\boxed{\tau_v > \tau_c}$ Shear reinforcement has to be designed.

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

(ii) Design of shear reinforcement:

$V_{uc} = \tau_c . bd \Rightarrow 0.52 \times (250 \times 450)$

$= 58500 N. \Rightarrow 58.5 kN$

$V_{us} = V_u - V_{uc}$

$= 225 - 58.5 \Rightarrow 166.5 kN$

Design of vertical stirrups:

Spacing of 2-legged 10mm ~~stirrup~~ stirrups

$$A_{sv} = 2 \times \frac{\pi}{4}(10)^2$$

$$= \cancel{1532} 157.07 \, mm^2$$

Spacing of stirrups:

1. ~~0.75 × 450~~ 1. $0.75 \times 450 = 337.5$ mm

~~2. 300~~ 2. 300mm which ever is less.

3. For vertical stirrups:

$$V_{us} = \frac{0.87 f_y . A_{sv} . d}{S_V}$$

$$S_V = \frac{0.87 f_y . A_{sv} . d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times 157.07 \times 450}{166.5}$$

$$S_V = 153.56 \times 10^3 \, m$$

$$\boxed{S_V = 153.56 \, mm} \approx 165mm$$

from Code book pg: 48.

26.5.1.6 – Min shear reinforcement

$$\frac{A_{sv}}{b S_V} \geq \frac{0.4}{0.87 f_y}$$

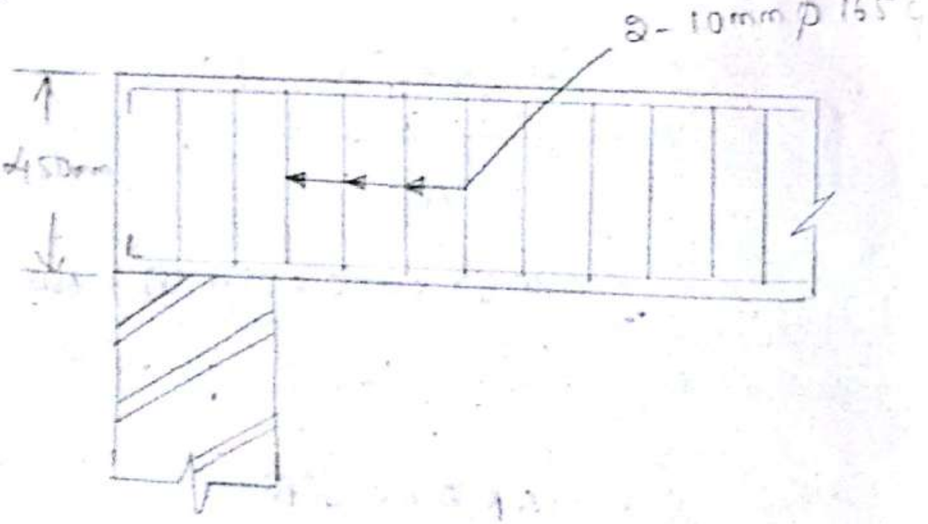
$$\frac{A_{sv}}{b \cdot s_v} = \frac{0.4}{0.87 f_y}$$

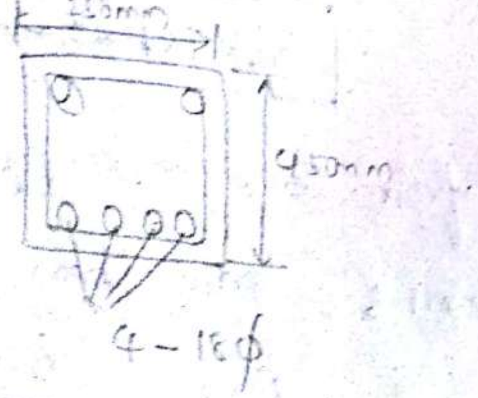
$$S_v = \frac{0.87 f_y \cdot A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times 157.07}{0.4 \times 250}$$

$$\boxed{S_v = 567.10\,mm}$$

Provide 10mm ϕ of 2-legged stirrups with a spacing of 165mm





24/7/17

11. A S.S R.C.C beam of 200mm × 400mm (effective) is reinforced with 4 bars of 22mm ϕ on tension side the beam is carrying a udl of 10kN/m over a clear span of 8m. Design the shear reinforcement by using M_{20} grade concrete & Fe 415 steel.

Sol

Given data,

$b = 200mm$

$d = 400mm$

$load = 10kN/m$

$l = 8m$

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

$f_y = 415N/mm^2$

$D = 400+50$

$= 450mm$

Self wt = $25 \times 0.45 \times 0.2$

$= 2.25 kN/m$

Total load = $2.25 + 10$

$= 12.25 kN/m$

Factored load $W_u = 12.25 \times 1.5$

$= 18.375 kN/m$

Factored shear force $= \frac{W_u l}{2}$

$$= \frac{18.375 \times 8.2}{2}$$

$$\boxed{V_u = 75.33 kN}$$

Effective span $= 8 + \frac{0.2}{2} + \frac{0.2}{2}$

$l = 8.2m$

1. Nominal shear stress:

$$\tau_v = \frac{V_u}{bd} \Rightarrow \frac{75.33 \times 10^3}{200 \times 400}$$

$$\boxed{\tau_v = 0.942 N/mm^2}$$

$Ast = 4 \times \frac{\pi}{4}(22)^2$

$Ast = 1520.5 mm^2$

% of steel $= \frac{1520.5}{200 \times 400} \times 100$

$= 1.9\%$

from table no: 19

$1.75 \rightarrow 0.75$

$2.00 \rightarrow 0.79$

$1.9 \rightarrow ?$

$$\tau_c = 0.75 + \left(\frac{0.79 - 0.75}{2.00 - 1.75}\right)(1.9 - 1.75)$$

$$\boxed{\tau_c = 0.774\ N/mm^2}$$

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

- 0.024

$$\boxed{\tau_v > \tau_c}$$

Shear reinforcement has to be designed

(i) Design of shear reinforcement:

$$V_{uc} = \tau_c \cdot bd \Rightarrow \cancel{0.726}$$

$$= 0.774 \times 200 \times 400$$

$$= \cancel{16920N}\ 61920\ N$$

$$\boxed{V_{uc} = 61.92\ kN}$$

$$V_{us} = V_u - V_{uc}$$

$$= 75.33 - 61.92$$

$$= 13.41\ kN$$

6, 8, 10, 12

$$\boxed{V_{us} = 13.41\ kN}$$

(ii) Design of vertical stirrups,

Spacing of 2-legged comm stirrups.

$$A_{sv} = 2 \times \frac{\pi}{4}(10)^2$$

$$= 157.07 \, mm^2.$$

Spacing of stirrups:

1. $0.75d \Rightarrow 0.75 \times 400$

$$= 300.0 \, mm$$

2. 300mm which ever is less

3. For Vertical stirrups,

$$V_{us} = \frac{0.87 f_y . A_{sv} . d}{S_V}$$

$$S_V = \frac{0.87 \times 415 \times 157.07 \times 400}{V_{us}}$$

$$= \frac{0.87 \times 415 \times 157.07 \times 400}{13.41}$$

$$= 1691.57 \times 10^3 m$$

$$= 1691.57 mm \approx 1700 mm$$

from Code book pg: 48

26.5.1.6 – Minimum shear reinforcement.

$$\frac{A_{sv}}{b s_v} = \frac{0.4}{0.87 f_y}$$

$A_{sv} = 2 \times \frac{\pi}{4}(10)^2$

$= 157.07 mm^2.$

Spacing of stirraps:

1. $0.75d \Rightarrow 0.75 \times 400$

$= 300.0 mm.$

2. 300mm which ever is less

3. For Vertical stirraps,

$$V_{us} = \frac{0.87 f_y . A_{sv} . d}{S_V}$$

$$S_V = \frac{0.87 \times 415 \times 157.07 \times 400}{V_{us}}$$

$$= \frac{0.87 \times 415 \times 157.07 \times 400}{13.41}$$

$= 1691.57 \times 10^3 m$

$= 1691.57 mm \approx 1700 mm$

from Code book pg: 48

26.5.1.6 - Minimum shear reinforcement.

$$\frac{A_{sv}}{b s_v} = \frac{0.4}{0.87 f_y}$$

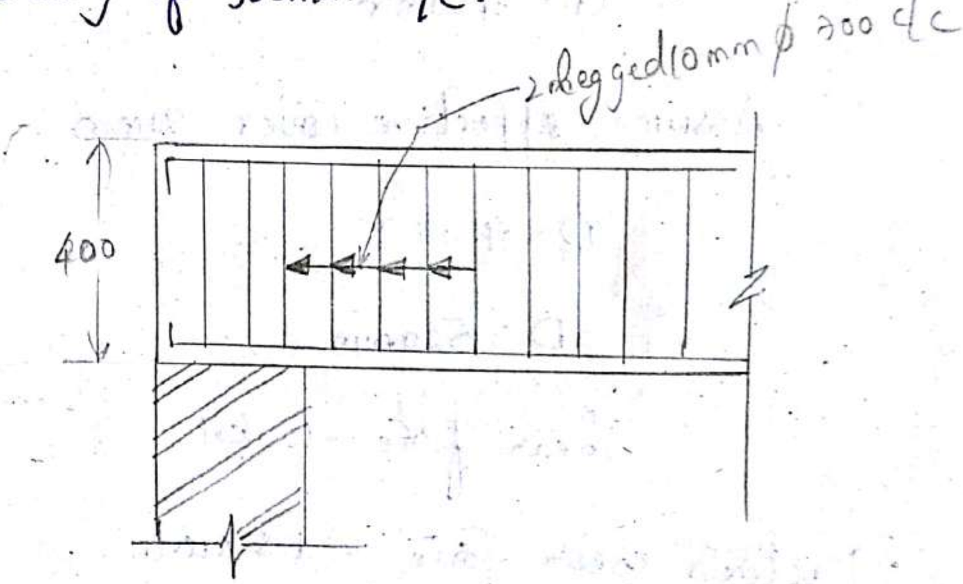
$$S_v = \frac{0.87 f_y A_{sv}}{0.4 \times b}$$

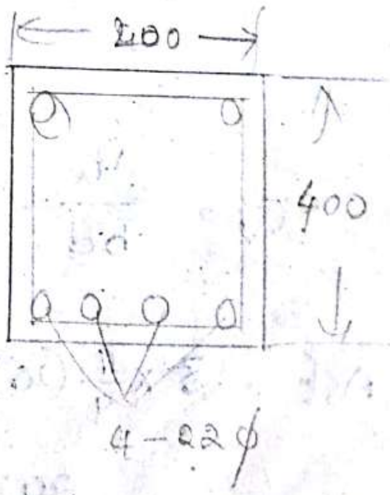
$$= \frac{0.87 \times 415 \times 157.07}{0.4 \times 200}$$

$$= 708.88mm \simeq 700mm$$

$$\boxed{S_v = 700mm}$$

Hence provide 2-legged -10mm ϕ stirrups with a spacing of 700mm c/c.





12. A R.C.C beam has an effective depth 450mm & breadth of 300mm it contains of 20mm ϕ bars. Mild steel out of wh 2 bars are curtailed at a section where force is 100kN at service load. Design reinforcement if the concrete is M_{20}

Sol

Given data,

$b = 300mm$

$d = 450mm$

Assume effective cover = 50mm

$D = 450 + 50$

$D = 500mm$

Shear force = 100kN

Factored shear force = 1.5×100

$V_u = 150kN$

$$\tau_v = \frac{V_u}{bd} = \frac{150 \times 10^3}{300 \times 450} = 1.11 N/mm^2$$

$Ast = 3 \times \frac{\pi}{4}(20)^2 \rightarrow$ ~~1570.79mm²~~ 942.47mm²

$$\% \text{ of steel} = \frac{942.47}{300 \times 450} \times 100$$

$= 0.698\%$

$0.50 \rightarrow 0.48$

$0.75 \rightarrow 0.56$ } from table no:19

$0.698 \rightarrow ?$

$$\tau_c = 0.48 + \left(\frac{0.56-0.48}{0.75-0.50}\right)(0.698-0.50)$$

$$\tau_c = 0.543\, N/mm^2$$

$$\tau_{cmax} = 2.8\, N/mm^2$$

$$\boxed{\tau_v > \tau_c}$$

Shear reinforcement has to be designed.

(ii) Design of shear reinforcement:

$$V_{uc} = \tau_c \times bd \Rightarrow 0.543 \times 300 \times 450$$

$$V_{uc} = 73305\, kN$$

$$\boxed{V_{uc} = 73.305\, kN}$$

$$V_{us} = V_u - V_{uc}$$

$$= 150 - 73.305$$

$$\boxed{V_{us} = 76.695\, kN}$$

(iii) Design of vertical stirrups:

spacing of 2-legged 8mm stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4}(8)^2 \Rightarrow 100.53mm^2$$

Spacing of stirrups:

(i) $0.75 \times d \Rightarrow 0.75 \times 450 = 337.5mm$

(ii) 300mm which ever is less

(iii) for vertical stirrups:

$$V_{us} = \frac{0.87 f_y . A_{sv} . d}{S_v}$$

$$S_v = \frac{0.87 f_y A_{sv} . d}{V_{us}}$$

$$= \frac{0.87 \times 250 \times 100.53 \times 450}{76.695}$$

$$= 128.29 \times 10^3 m$$

$$S_v = 128.29mm$$

$$\boxed{S_v = 130mm}$$

from Code book pg:48

26.5.1.6 - Min. shear reinforcement

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

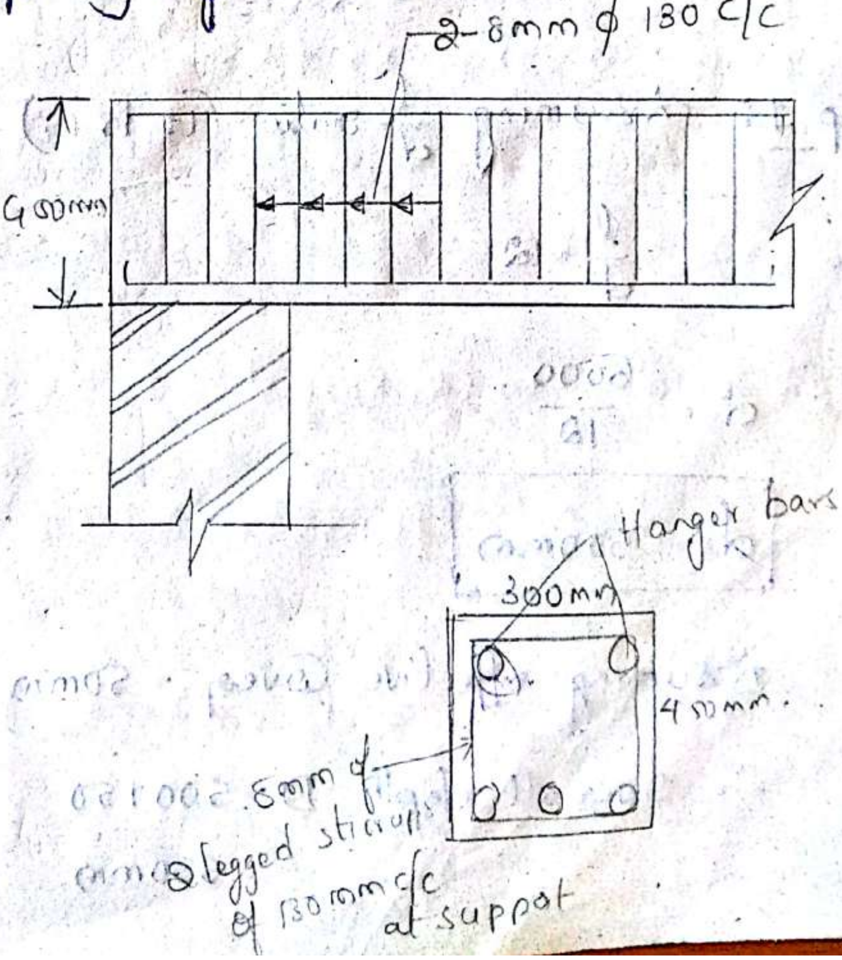
$$\frac{A_{sv}}{b \times s_v} = \frac{0.4}{0.87 f_y}$$

$$S_v = \frac{0.87 f_y \cdot A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 250 \times 100.53}{0.4 \times 300}$$

$$\boxed{S_v = 182.21 \text{ mm}}$$

Provide 8mm ϕ of 2-legged stirrups with of spacing of 130mm.



26/7/17

13. A rectangular Concrete beam is S.S on two masonary walls 230mm thick, 6m apart (Centre Centre) the beam is Carrying an imposed load 15kN/m. Design the beam with all neccessary checks. use M_{25} Concrete & Fe 415 steel. sketch the details of reinforcement

Sol

Given data,

$b = 230mm$

imposed load $= 15 kN/m$

$f_y = 415 N/mm^2$

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

$l = 6m$ (Centre to Centre)

Step 1: Assuming $\frac{l}{d}$ ratio (10 to 15)

$$\frac{l}{d} = 12$$

$$d = \frac{6000}{12}$$

$$\boxed{d = 500mm}$$

Assuming effective Cover - 50mm

∴ Overall depth $D = 500+50$

$= 550mm$

(ii) Effective span $l = 6000mm$

(iii) Calculation of loads & BM

Self wt = $0.23 \times 0.55 \times 1 \times 25$

$= 3.1625 kN/m$

Imposed load = $15 kN/m$

Total load = $15 + 3.1625$

$= 18.1625 kN/m$

$BM = \frac{wl^2}{8} \Rightarrow \frac{18.1625 \times 6^2}{8}$

$\boxed{BM = 81.73 kN/m}$

factored bending moment,

$M_u = 1.5 \times BM$

$= 1.5 \times 81.73$

$\boxed{M_u = 122.6 kN/m}$

Step 4:- Depth required,

$M_u = M_{u limit}$

for fe 415 steel, $M_u limit = 0.138 f_{ck} b d^2$

$d = \sqrt{\frac{M_{u limit}}{0.138 f_{ck} \cdot b}} = \sqrt{\frac{122.6 \times 10^6}{0.138 \times 25 \times 230}}$

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$d = 393.07mm \approx 400mm < 500mm$

Hence provided depth is adequate.

$M_{ulim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 25 \times 230 \times 500^2$

$= 198.37 kN\text{-}m$

$M_u < M_{ulim}$

$\therefore$ So, the section is Singly reinforced.

Step 5:- Area of reinforcement,

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

$$122.6 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500 \left[1 - \frac{415 A_{st}}{25 \times 230 \times 500}\right]$$

$$122.6 \times 10^6 = 180.525 \times 10^3 A_{st} \left[1 - 1.443 \times 10^{-4} A_{st}\right]$$

$$122.6 \times 10^6 = 180.525 \times 10^3 A_{st} - 26.058 A_{st}^2$$

$$26.058 A_{st}^2 - 180.525 \times 10^3 A_{st} + 122.6 \times 10^6 = 0$$

$$\boxed{A_{st} = 763.21 mm^2}$$

Scanned by CamScanner

Provide 16mm ϕ for finding no. of bars

A_{st} = No. of bars $\times \frac{\pi}{4}(16)^2$

763.21 = ~~113.09~~ 201.06 No. of bars

~~113.09~~ 201.06 No. of bars = 763.21

18
16
20

No. of bars = 3.79 $\approx$ 4 bars

$\therefore$ No. of bars = 4

A_{st} required = 763.21 mm^2

A_{st} provided = $4 \times \frac{\pi}{4}(16)^2$

= 804.24 mm^2

Min. reinforcement:

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

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

$= \frac{0.85 \times 230 \times 500}{415}$

$\therefore$ $\boxed{A_{st\,min} = 235.5\,mm^2}$

Max. reinforcement:

$A_{st\,max} = 0.04 \times bD$

Scanned by CamScanner

$= 0.04 \times 230 \times 550$

$= 5060$

$\boxed{A_{st}\,max = 5060 mm^2}$

~~step 5~~:- ~~check for deflection (stiffness)~~

~~For s.s beam basic value ($\frac{l}{d} = 20$)~~

& $\tau_v < \tau_c max$

$\tau_v < \tau_c$ - min. reinforcement

$\tau_v > \tau_c$ & $\tau_v < \tau_c max$ - ~~desig~~ design shear reinforcement

$\tau_v > \tau_c$ & $\tau_v > \tau_c max$ - redesign the beam.

Step 6:- Check for shear reinforcement

$$\tau_v = \frac{V_u}{bd} \Rightarrow \frac{81.73}{230 \times 500} \Rightarrow 7.106 \times 10^{-4}$$

$$\tau_v = 7.106 \times 10^{-4} N/mm^2$$

$$\boxed{\tau_c max = 3.1 N/mm^2}$$

% of steel = $\frac{A_{st}}{bd} \times 100$

$$= \frac{804.24}{230 \times 500} \times 100$$

$$= 0.699$$

from Code book table no:19, τ_c value

for 0.699

0.50 – 0.49

0.75 – 0.57

0.699 – ?

$$\tau_c = 0.49 + \left(\frac{0.57 - 0.49}{0.75 - 0.50}\right)(0.699 - 0.50)$$

$$= 0.55$$

$$\boxed{\tau_c = 0.55 N/mm^2}$$

$\boxed{\tau_{c_{max}} = 3.1 N/mm^2}$ from table no:20

$$\tau_c < \tau_{cmax}$$

$\tau_v > \tau_c$ & $\tau_v < \tau_{cmax}$ → So ~~degin~~ design shear rein forcement

<u>Design of shear rein-forcement:</u>

Hence provided 6mm ϕ of 2 legged stirrups 220 mm c/c.

Check for deflection:-

for s.s beam basic value $\left(\frac{l}{d} = 20\right)$

In Code book, pg no - 38

modification factor for tension reinforcement

$$f_s = 0.58 f_y \times \frac{\text{Area of c/s of Ast required}}{\text{Area of c/s of Ast provided}}$$

$$= 0.58 \times 415 \times \frac{763.21}{804.29}$$

$$\boxed{f_s = 228.42 \, N/mm^2}$$

$$\% \text{ of steel} = \frac{\text{Ast provided}}{bd} \times 100$$

$$= \frac{804.29}{230 \times 500} \times 100$$

$$= 0.699\%$$

from fig. 4 of IS:456, modification factor = 1.2

Max. permitted $\frac{l}{d}$ ratio = $\frac{1.2}{1.25} \times 20$

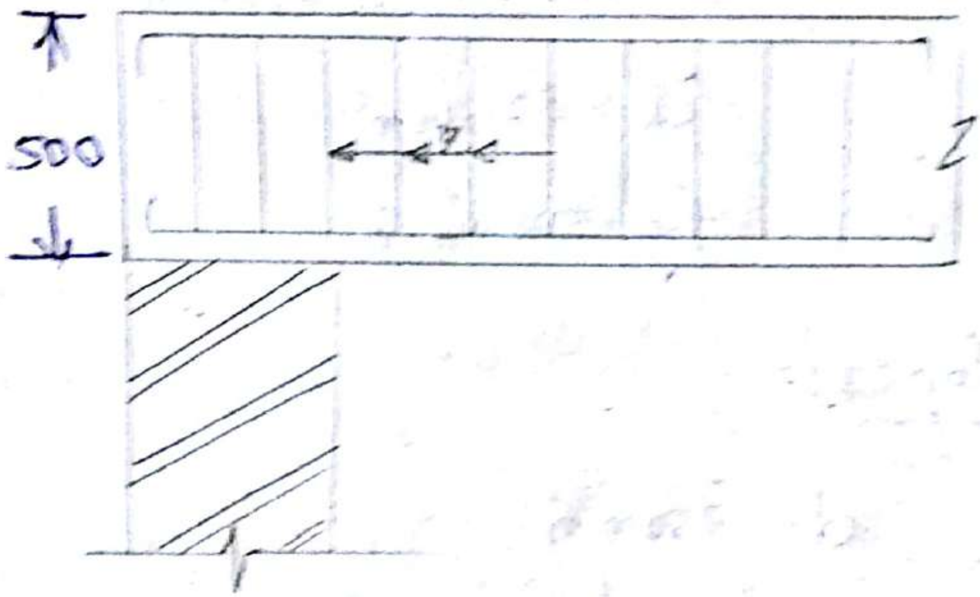
$= 24$

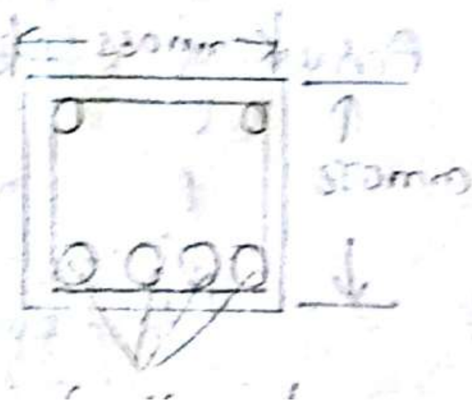
$\frac{l}{d}$ provided = $\frac{6000}{500}$

$\frac{l}{d}$ provided = $12 < 24$

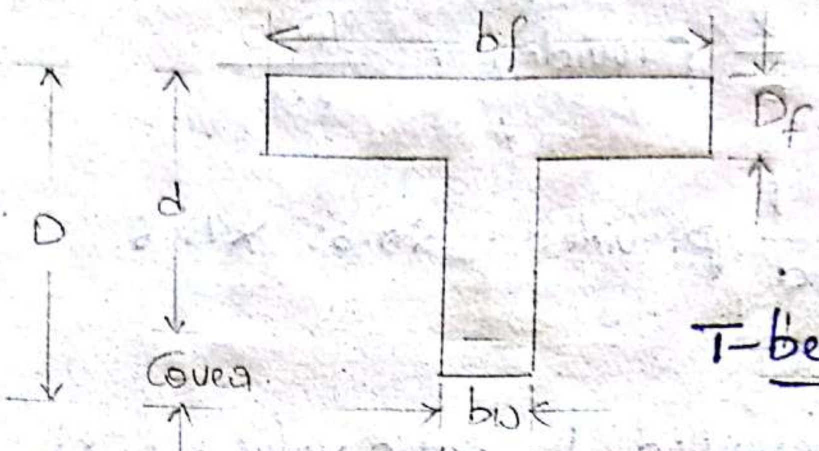
$\therefore$ Hence deflection Control is Safe.

— 6mm ϕ 2-legged @ 220 c/c

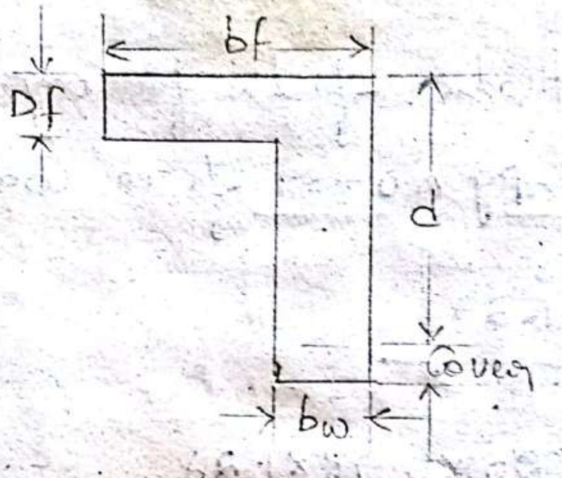




T-Beams:



T-beams



L-Beams

For T-beams:

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

For L-beams:

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

→ When the slab occurs on both the sides of the beam (intermediate beams), the beam is known as T-beam.

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→ When the slab is only on one side of the beam (end beams), the beam is known as "L-beam"

Advantages of T-Beams:

1. As the slab being monolithic with the beam is also compressed & shares the compressive force with the beam, which significantly increases the moment of resistance of the beam.
2. As most of the compressive force is shared by the flange, the depth of the beam required is less and hence the maximum deflections are also less.

For isolated beams, the effective width shall be obtained as given below but in no case greater than the actual width (b)

For T-beams $b_f = \dfrac{l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w$

For L-beams $b_f = \dfrac{0.5 l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w$

where,

b_f = effective width of flange

l_o = distance b/w points of zero moments in the beam.

b_w = breadth of web.

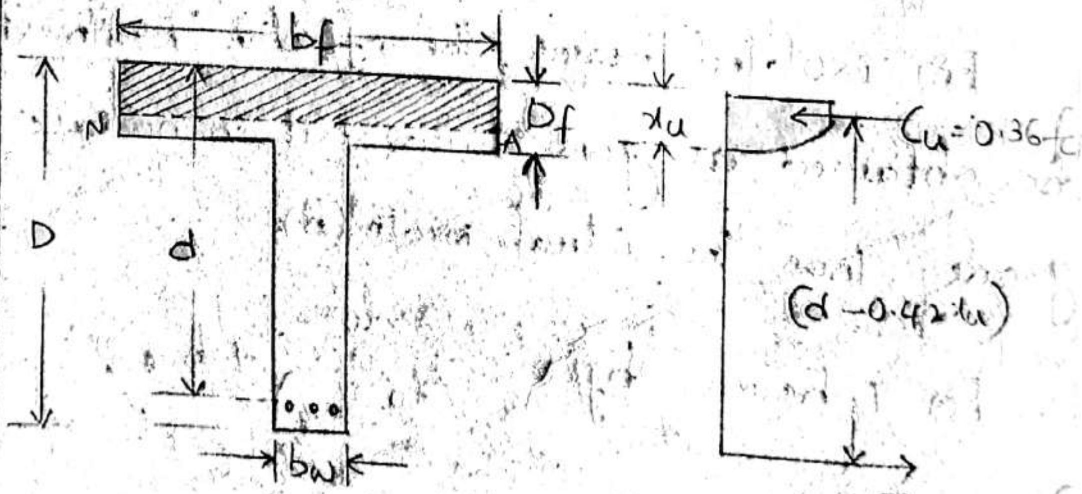
D_f = thickness of flange.

b = Actual width of the flange.

x_1, x_2 = half the clear distance b/w two adjacent beams.

Analysis of T-beams:-

Case 1: Neutral axis is with in the flange ($x_u \leq D_f$)



Neutral axis lying inside the flange.

When the NA falls with in the flange as shown, the T-beam can be treated as a normal rectangular beam of width bf & depth d. The moment of resistance of the section can be calculated by the same procedure as that of rectangular section of width bf & depth 'd'.

$$C = 0.36 f_{ck} \cdot b_f \cdot x_u$$

$$T = 0.87 f_y \cdot A_{st}$$

1. Depth of Neutral axis:

$$C = T$$

$$0.36 f_{ck} \cdot b_f \cdot x_u = 0.87 f_y \cdot A_{st}$$

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

2. Moment of resistance:

$$M_u = 0.36 f_{ck}\, b_f \cdot x_u (d - 0.42 x_u) \text{ — Concrete}$$

$$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 x_u) \text{ — Steel}$$

Case II: Neutral axis is below the flange ($x_u > D_f$) i.e., in the web and $D_f / d \leq 0.2$

1. Find the moment of resistance of T-beam section having $b_w = 300$, $b_f = 1650mm$, $D_f = 150mm$, $d = 550mm$. The reinforcement consists of 6 bars of 20mm. Use M_{20} concrete & Fe 415 steel.

Sol Given data,

$b_w = 300mm$

$b_f = 1650mm$

$D_f = 150mm$

$d = 550mm$

No of bars = 6

Dia = 20mm

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

$f_y = 415 N/mm^2$

$A_{st} = \frac{\pi}{4} \times (20)^2 \times 6 = 1884.9mm^2$

Step 1:-

Assuming depth of N.A is in the flange

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

$$= \frac{0.87 \times 415 \times 1884.9}{0.36 \times 20 \times 1650}$$

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$\boxed{x_u = 57.28\,mm} \leq D_f$

$x_{u,max} = 0.48d \Rightarrow 0.48 \times 550$

$= 264\,mm$

$\boxed{x_u < x_{u\,max}}$

So, the beam is under reinforced section

Moment of resistance:

$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)$

$= 0.87 \times 415 \times 1884.9 (550 - 0.42(57.28))$

$= 357.92 \times 10^6\ N\text{-}mm$

$\boxed{M_u = 357.92\ kN\text{-}m}$

2/8/17

2. Find the moment of resistance of a T-beam having $b_w = 300mm$, $b_f = 1650mm$, $D_f = 120mm$, $d = 510mm$. The reinforcement consists of 4 bars of 25mm ϕ. use M_{20} concrete & Fe 415 steel.

Sol Given data,

$b_w = 300mm$

$b_f = 1650mm$

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$D_f = 120mm$

$d = 510mm$

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

$f_y = 415N/mm^2$

$A_{st} = 4 \times \frac{\pi}{4}(25)^2 \Rightarrow 1963.49mm^2$

Step 1:-

Assuming depth of N.A is in the flange

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

$$= \frac{0.87 \times 415 \times 1963.49}{0.36 \times 20 \times 1650}$$

$\boxed{x_u = 59.67mm} \leq D_f$

$x_{u\,max} = 0.48d \Rightarrow 0.48(510)$

$\Rightarrow 244.8mm$

$\boxed{x_u < x_{u\,max}}$

So, the beam is under reinforced section.

Moment of resistance:

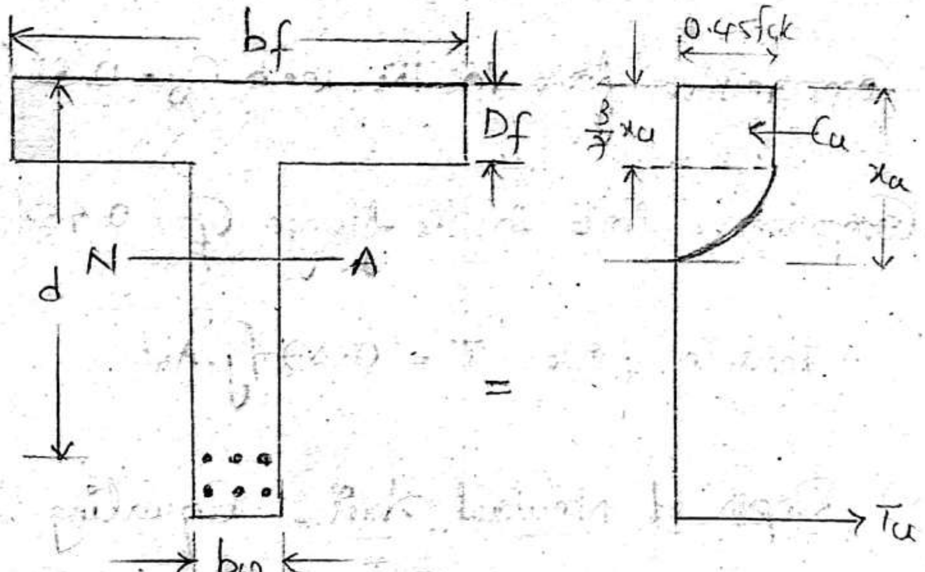
$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)$

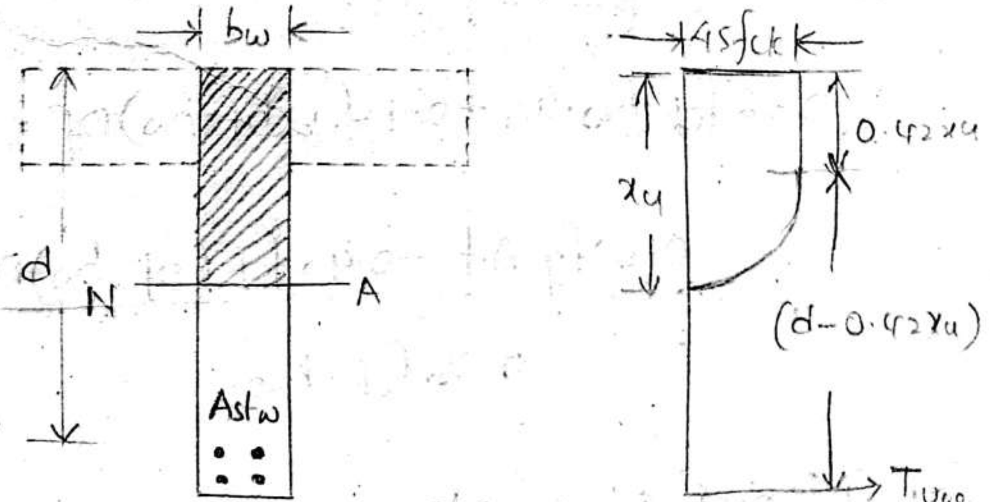
$= 0.87 \times 415 \times 1963.49 (510 - (0.42 \times 59.67))$

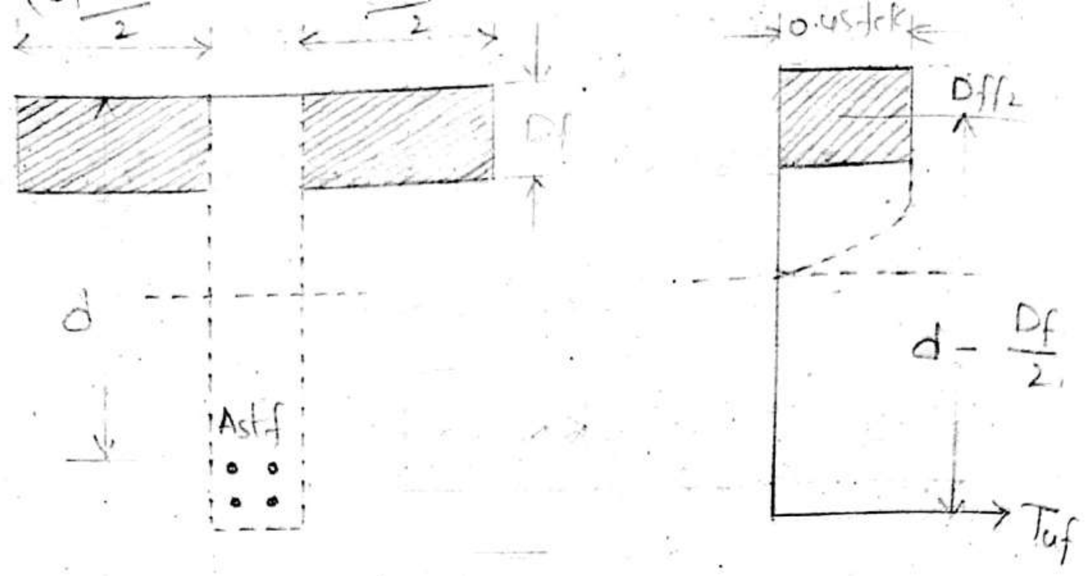
$= 343.78 \times 10^6 \ N\text{-}mm$

$$\boxed{M_u = 343.78 \ kN\text{-}m}$$

Case 2: Neutral Axis is below the flange ($x_u > D_f$) i.e., in the web & $D_f/d \leq 0.2$







Neutral Axis is below the flange & $\frac{D_f}{d} \leq 2$

Compressive force in the web $C_w = 0.36 f_{ck} \cdot b_w \cdot x_u$

Compressive force in the flange $C_f = 0.45 f_{ck} (b_f - b_w) D_f$

Tensile force $T = 0.87 f_y \cdot A_{st}$

(a). Depth of Neutral Axis: Equating Compression & tension force.

$$0.36 f_{ck} \cdot b_w \cdot x_u + 0.45 f_{ck}(b_f - b_w) D_f = 0.87 f_y \cdot A_{st}$$

$$x_u = \frac{0.87 f_y \cdot A_{st} - 0.45 f_{ck}(b_f - b_w) D_f}{0.36 f_{ck} \cdot b_w}$$

(b). Moment of Resistance:

$$M_u = M_{u,web} + M_{u,flange}$$

$$M_u = C_w(d - 0.42x_u) + C_f\left(d - \frac{D_f}{2}\right)$$

Substituting the values of C_w & C_f

$$M_u = 0.36 f_{ck} \cdot b_w \cdot x_u (d - 0.42x_u) + 0.45 f_{ck}(b_f - b_w)D_f \left[d - \frac{D_f}{2}\right]$$

3. Calculate the moment of resistance of the T-beam with the following data.

Width of the flange = 800mm = b_f

Thickness of slab = 110mm = D_f

width of rib = 300mm = b_w

Effective depth = 600mm = d

Area of tension steel = $2500mm^2$ = A_{st}

characteristic strength of concrete = $20 N/mm^2$ = f_{ck}

characteristic strength of steel = $415 N/mm^2$ = f_y

Step 1: Assuming x_u is with in the flange:

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

$$= \frac{0.87 \times 415 \times 2500}{0.36 \times 20 \times 800}$$

$x_u = 156.7 mm > D_f$

$\therefore$ Our assumption is wrong. So, the N.A lies in the web.

$$\frac{D_f}{d} = \frac{110}{600} = 0.18 < 0.2$$

So, it is Case - 2

Step 2:

Actual depth of N.A

$$x_u = \frac{0.87 f_y . A_{st} - 0.45 f_{ck} (b_f - b_w) D_f}{0.36 \times f_{ck} \times b_w}$$

$$= \frac{0.87 \times 415 \times 2500 - 0.45 \times 20 (800 - 300) 110}{0.36 \times 20 \times 300}$$

$$\boxed{x_u = 188.72 mm}$$

495000

$x_u max = 0.48 d$

$= 0.48 \times 600$

$= 288$

$$\boxed{x_u < x_u max}$$

$\therefore$ The section is under reinforced section.

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3. step : Moment of resistance:

$$M_u = 0.36 f_{ck} \cdot b_w \cdot x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

$$\Rightarrow 0.36 \times 20 \times 300 \times 188.72 \left(600 - (0.42 \times 188.72)\right)$$

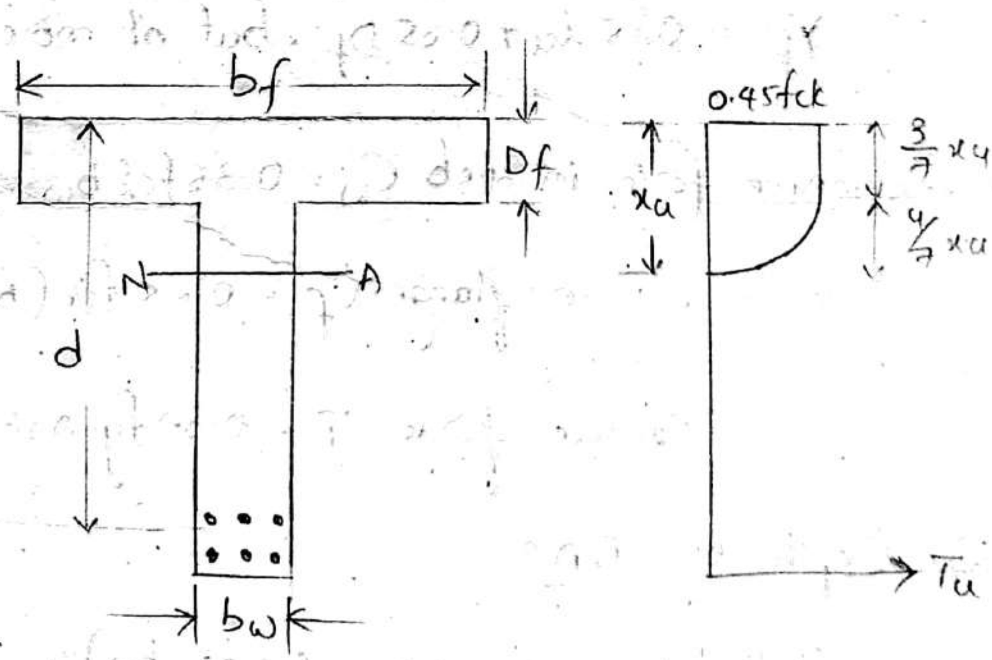
$$+ 0.45 \times 20 (800 - 300) 110 \times \left(\overset{545}{600 - \frac{110}{2}}\right)$$

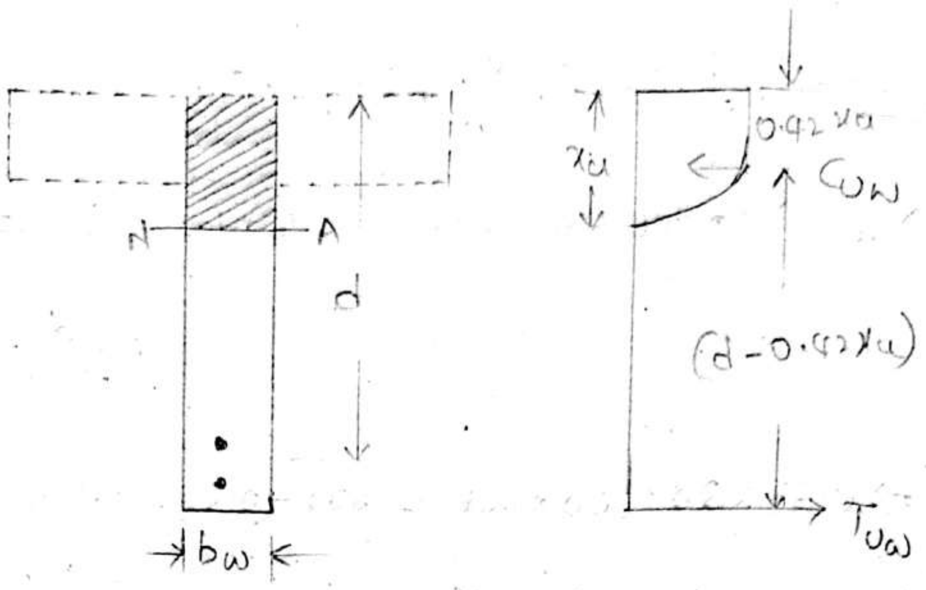
$$\Rightarrow 212.27 \times 10^6 + 269.7 \times 10^6$$

$$= 482.045 \times 10^6 \text{ N-mm}$$

$$\therefore \boxed{M_u = 482.045 \text{ kN-m}}$$

Case III: Neutral axis is below the flange ($x_u > D_f$) i.e., in the web & $D_f/d > 0.2$





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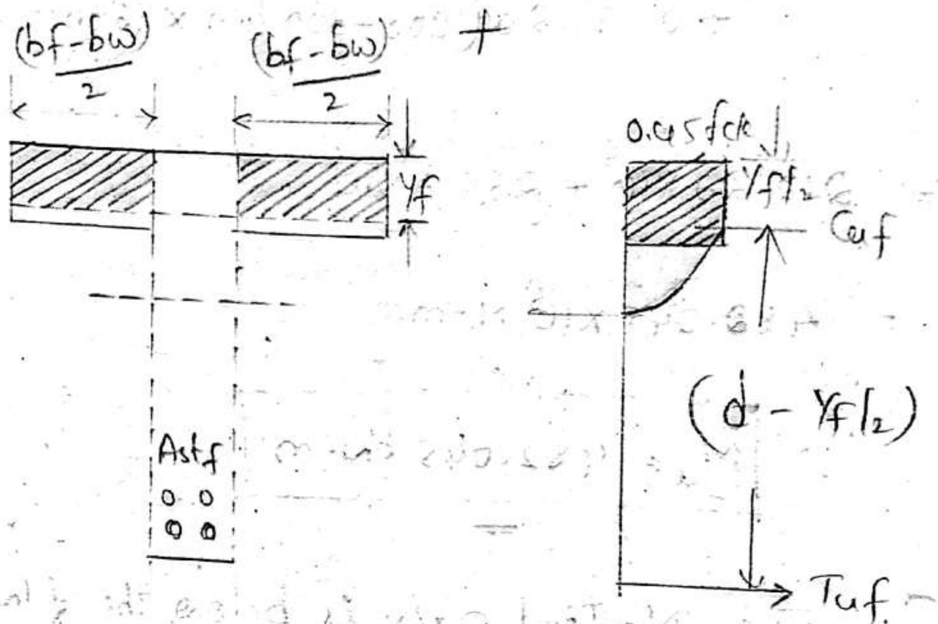


Modified thickness of flange $= y_f$

$y_f = 0.15x_u + 0.65D_f$, but not more than D_f

Compressive force in web $C_w = 0.36 f_{ck} b_w \cdot x_u$

" " in flange $C_f = 0.45 f_{ck} (b_f - b_w) y_f$

Tensile force $T = 0.87 f_y \cdot A_{st}$

(a). Depth of N-A:

$0.36 f_{ck} \cdot b_w \cdot x_u + 0.45 f_{ck} (b_f - b_w) y_f = 0.87 f_y \cdot A_{st}$

$$x_u = \frac{0.87 f_y \cdot A_{st} - 0.45 f_{ck} (b_f - b_w) y_f}{0.36 f_{ck} b_w}$$

(b) Moment of resistance:

$$M_u = C_w (d - 0.42 x_u) + C_f \left(d - \frac{y_f}{2}\right)$$

$$M_u = 0.36 f_{ck} \cdot b_w \cdot x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

Footings.

Types of footings:-

1. Isolated footing
2. Combined footing.
3. Strap footing.
4. Raft or mat footing

Design procedure:-

1. Size of the footings:

Area of the footing required,

$$A = \frac{1.1P}{SBC \text{ of soil}}$$

where P = working load

SBC = Safe bearing Capacity.

2. Determine the upward soil reaction for the factored load:

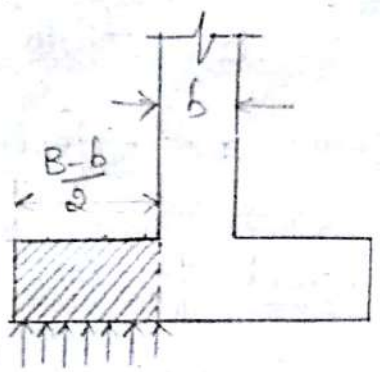
$$q_u = \frac{P_u}{A}$$

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

3. Determine the min. depth required to resist Bending moment:

Projection of the footing

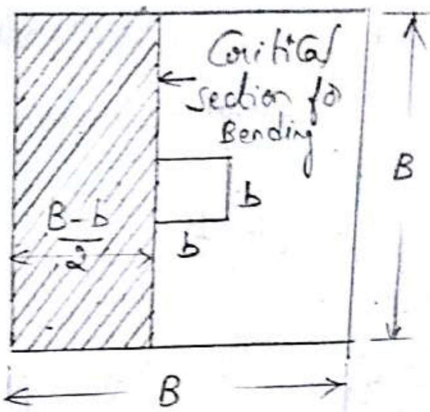
$$= \frac{(B-b)}{2}$$



The B.M about x–x is (as a Cantilever slab, $\frac{wl^2}{2}$).

$$M_u = \frac{q_u \cdot B\left(\frac{B-b}{2}\right)^2}{2}$$

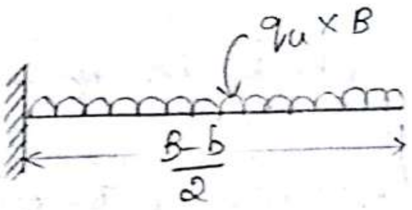
$$= q_u \cdot \frac{B(B-b)^2}{8}$$



where q_u = upward soil Pressure

B = width of footing.

b = width of Column.



4. Determine the Area of reinforcement required in width B using,

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

using the bars of diameter not less than 10mm, find the spacing of bars

$$\text{Spacing} = \frac{B \cdot a_{st}}{A_{st}}$$

where, a_{st} = area of bar used

A_{st} = total area of steel required.

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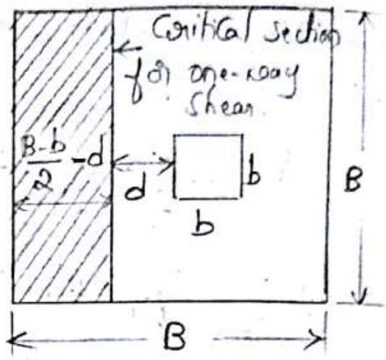
B = width of the footing

d = effective depth.

5. Check for one way shear:

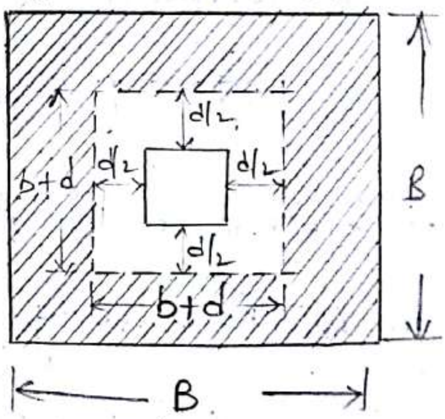


V_u = Soil pressure from the shaded area.

$$= q_u \cdot B\left[\frac{B-b}{2} - d\right]$$

$\tau_v = \frac{V_u}{Bd} < \tau_c$, permissible shear stress in Concrete.

6. Check for Two way shear: (Punching shear).



Perimetre of the punching area = $4(b+d)$

Area of Concrete resisting Punching ~~area~~ force = perimetre of Punching × depth

$$A = 4(b+d)d$$

Force of punching $S = q_u \times$ shaded Area

$$= q_u\left[B^2 - (b+d)^2\right]$$

Punching shear stress.

$$\tau_p = \frac{S}{A} < \text{permissible Value.}$$

Permissible value of punching shear stress is,

$$\tau_p = 0.25\sqrt{f_{ck}}$$

* Check for Bond Length:

$$L_d = \frac{0.87 f_y \phi}{4 \cdot \tau_{bd}}$$

Design of staircase :

(1) Assuming $\frac{l}{d}$ ratio (20 to 25).

2. Effective Span:

x	y	span in m
< 1m	< 1m	G+x+y
< 1m	> 1m	G+x+1
> 1m	< 1m	G+y+1
> 1m	> 1m	G+1+1

3. Loads on stairs:

Live load if crowded - $5 kN/m^2$

if n't " - $3 kN/m^2$

Dead load :

Thickness of waist slab = D

(a). weight of waist slab per unit horizontal area,

$$w_1 = \frac{D\sqrt{R^2+T^2}}{T} \times 25$$

$$= D\sqrt{1+\left(\frac{R}{T}\right)^2} \times 25$$

(b) wt. of steps per unit horizontal Area,

$\frac{1}{2} bh$

$$= \frac{\frac{1}{2} \times R \times \cancel{T} \times 25}{\cancel{T}}$$

$$= \frac{1}{2} R \times 25$$

Area of $\Delta^{le} = \frac{1}{2} bh$

where R in metre

Finishing load (0.5 to 1 kN/m²) may be added to the above values.

18/9/17

4. Determine the max. bending moment,

$$M_u = \frac{w_u l^2}{8}$$

5. Determine the min. depth required to resist the bending moment by equating,

$$M_u = M_{ulim} = k.f_{ck}.bd^2$$

b = 1000mm, k = 0.138 for Fe 415 steel & 0.148 for mild steel.

Provided depth should be more than this value. otherwise increase the depth.

6. Calculate the area of steel per metre width of slab by using,

6. Calcu

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

7. Find the spacing of bars using,

$$S = \frac{1000 \times a_{st}}{A_{st}}$$

where,

a_{st} = area of bar used.

A_{st} = total area of steel required.

Spacing should n't be more than 3d or 300mm which ever is less.

8. Providing distribution reinforcement perpendicular to the span direction at 0.12% (for HYSD bars) of gross c/s area & find the spacing of these bars. of mild steel bars are used, provide 0.15% of gross area as distribution steel.

4. Design a doglegged staircase for a building in which the height of floor is 3.3m. Adopt rise & thread of each step are 150mm & 225mm respectively. The stair hall is 2.5m x 4.5m. Live load may be taken as $3kN/m^2$. Use M_{20} grade Concrete & Fe 415 grade steel. Assume the stairs are supported on 230mm walls at the ends of

Outer edges of landing slabs.

1. proportioning of stairs:

Dimensions of stair hall = 2.5m × 4.5m

Height of the floor = 3.3m

Height of one flight = $\frac{3.3}{2}$ ⇒ 1.65m

⇒ 1650mm

Rise, R = 150m

Tread, T = 225mm

No. of risers = $\frac{\text{Height of one flight}}{\text{Rise}}$

⇒ $\frac{1650}{150}$

= 11

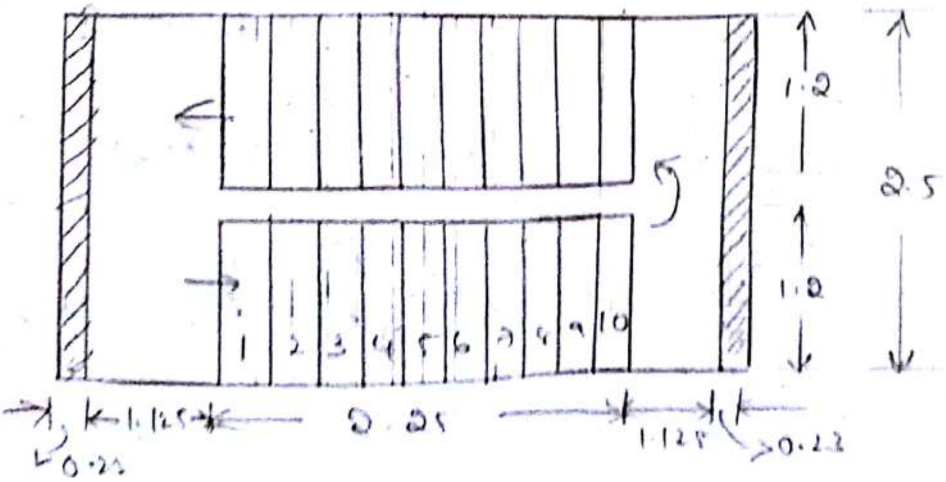
Hence, no. of treads = 11 − 1 ⇒ 10

Adopt width of stair = 1.2m

For 10 threads, the length required = 10 × 0.025

= 2.25m

width of landing = $\left(\frac{4.5 - 2.25}{2}\right)$ ⇒ 1.125m



2. Effective span:

As the stair slab is spanning longitudinally,

Effective span = Centre to centre distance of walls.

$= 4.5 + 0.23$

$$\boxed{l = 4.73 m}$$

3. Thickness of slab:

Assume effective depth,

$$d = \frac{Span}{25} \Rightarrow \frac{4730}{25} = 189.2 mm$$

$d = 190 mm$

$D = 220 mm$.

4. Loads: Load per metre horizontal width of stairs are as follows.

$$wt.\ of\ waist\ slab = D\sqrt{1+\left(\frac{R}{T}\right)^2} \times 25$$

$$= 0.22\sqrt{1+\left(\frac{0.15}{0.225}\right)^2} \times 25$$

$$= 6.61 kN/m^2$$

$$wt.\ of\ steps = \frac{\left(\frac{1}{2} R\cancel{T}\right)}{\cancel{T}} \times 25 \Rightarrow R \times \frac{25}{2}$$

$\Rightarrow 0.15 \times \frac{25}{2}$

$\Rightarrow 1.875\ kN/m^2$

live load = $3 kN/m^2$

Floor finish = $0.6 kN/m^2$

Total load = $12.1 kN/m^2$

Factored load w_u = 1.5×12.1

$= 18.15 kN/m^2$

5. Factored BM:

$$M_u = \frac{w_u l^2}{8} \Rightarrow \frac{18.15 \times (4.73)^2}{8}$$

$M_u = 50.76 kN\text{-}m$

$= 50.76 \times 10^6 N\text{-}mm$

6. Min. depth required:

$M_u = 0.138 f_{ck} b d^2$

$50.76 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$$d = \sqrt{\frac{50.76 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d = 135.6mm < 190mm$, provided depth

Hence provided depth is adequate

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7. Tension reinforcement:

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

$$50.76 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 190}\right]$$

$$50.76 \times 10^6 = 68.6 \times 10^3 A_{st} - 7.49 A_{st}^2$$

$$7.49 A_{st}^2 - 68.6 \times 10^3 A_{st} + 50.76 \times 10^6 = 0$$

$$\boxed{A_{st} = 812 mm^2}$$

using 12mm ϕ bars, spacing of bars,

$$S = \frac{a_{st}}{A_{st}} \times 1000$$

$$\Rightarrow \frac{\pi}{4} \times 12^2 \times \frac{1000}{812}$$

$$= 139.3 \text{ mm}$$

Provide 12mm bars @ 130mm c/c.

8. Distribution reinforcement:

$$A_{st} = 0.12\% \text{ of gross area}$$

$$= 0.12 \times 1000 \times \frac{220}{100}$$

$$\boxed{A_{st} = 264 mm^2}$$

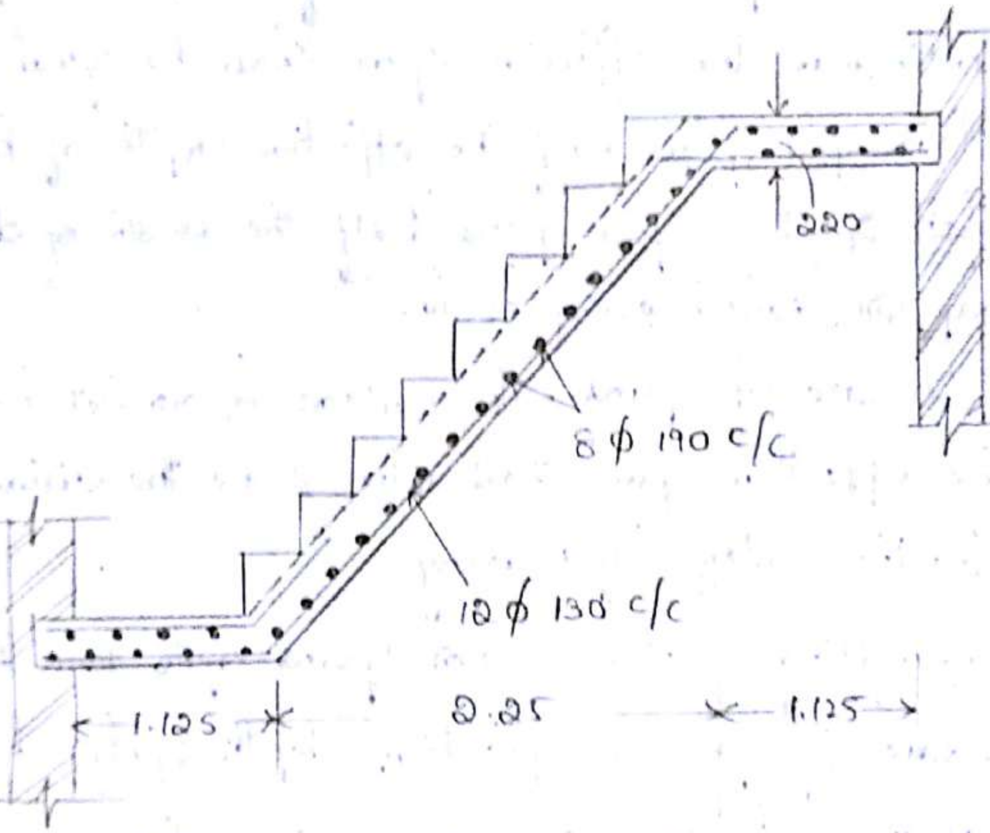
using 8mm bars, spacing

$$S = \frac{\pi}{4} \times 8^2 \times \frac{1000}{264}$$

$$= 190.4 mm$$

Hence, provide 8mm bars @ 190 mm c/c

The details of reinforcement are shown in fig 4.21



19/9/17

Continuous Beams :-

1. Effective span: If the width of the support (or) (barrier) is less than $\frac{1}{12}$ of clear span, the effective span shall be as per simply supported beam

(i) width of support < $\frac{1}{12}$ of clear span

(ii) width of support > $\frac{1}{12}$ of clear span (or) 600mm which ever is less, the effective span shall be as follows.

- For end spans with one end free & the other continuous or for intermediate spans, the effective span shall be the clear span b/w the supports.
- For end span with one end free & the other continuous, the effective span shall be equal to the clear span plus half the effective depth of beam/slab or clear span plus half the width of discontinuous support, which ever is less.
- In case of spans with rollers or rocket bearings, the effective span shall always be the distance b/w the centers of bearing.

2. Limiting stiffness: For spans upto 10m, the basic value of span to effective depth ratio should not exceed 26. Depending up on the area & type of tension steel the span to depth ratio may be multiplied by the modification factor.

In general, continuous beams carry heavy loads & consequently the span/depth ratio recommended in practical designs is normally in between 15 to 20

22/9/17

3. Bending moments & shear forces: It needs rigorous structural analysis to get the design

moments & shear forces. However IS: 456-2000 permits use of design coefficients shown in table 4.1 & 4.2 (Table 12 & 13 of IS: 456-2000) subjected to the following conditions.

- there are three or more spans.
- spans don't differ by 15% of the longest.
- Loads are predominantly uniformly distributed loads.

4. Reinforcement:

Same rules apply as for simply supported beams/slabs.

1. Mid span reinforcement
2. Support reinforcement.

5. Design a singly reinforced continuous RC rectangular beam for flexure for the following conditions. use M20 grade concrete & Fe 415 steel.

No. of spans = 3

Clear distance b/w supports = 3600mm

width of the support = 300mm

Imposed load (not fixed) = 5 kN/m²

Imposed load (fixed) = 7.5 kN/m² (excluding self wt).

Partial fixity may be expected at the dis continuous edge.

Given data,

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

$f_y = 415N/mm^2$ $b = 300mm$

width = 300mm

No. of spans = 3

Clear span $L = 3.6m$

1. Depth of the beam:

Selecting the depth in range of $\frac{l}{15}$ to $\frac{1}{20}$ based on stiffness

Assume $\frac{l}{d}$ ratio as 15

$$\frac{l}{d} = 15$$

$$d = \frac{3600}{15}$$

$$= 240mm$$

Adopt $d = 250mm$

Assume effective cover = 50mm

$D = 250 + 50$

$$\boxed{D = 300mm}$$

2. Effective Span:

(i) clear span + d ⇒ $3.6 + 0.25$

$= 3.85m$

(ii) clear span $+ \frac{b}{2} + \frac{b}{2}$

$= 3.6 + \frac{0.23}{2} + \frac{0.23}{2}$

$= \cancel{3.84m}\ 3.9m$

Effective span $l = \cancel{3.9m}\ 3.85m$

3. <u>Loads</u>:-

Self wt. of the beam $= \cancel{0.3\times0.3\times}\ 0.3\times0.3\times1\times25$

$= 2.25\ kN/m^2$

Imposed load, fixed $= 7.5\ kN/m^2$

Total load fixed $= 9.75\ kN/m^2$

Imposed load, not fixed $= 5\ kN/m^2$

<u>Factored loads</u>:

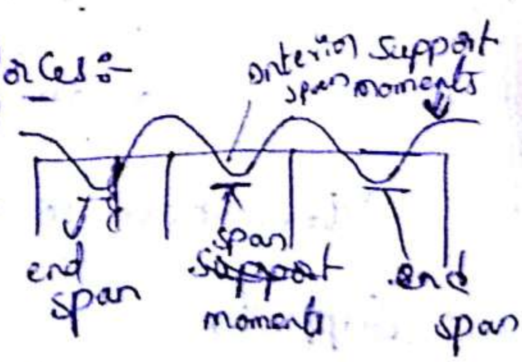
factored fixed load $w_{ud} = 1.5\times9.75$

$= 14.63\ kN/m$

not fixed $w_{ul} = 1.5\times5$

$= 7.5\ kN/m$

4. <u>Bending moments & shear forces</u>:-



<u>Span moments</u>: (At the middle of end spans).

$M_u = \left(\frac{1}{12}\right)\times14.63\times3.85^2 + \left(\frac{1}{10}\right)\times7.5\times3.85^2$

$= 18.07 + 11.12$

$= 29.2\ kN\text{-}m$

At the middle of interior spans.

$M_u = \left(\frac{1}{16}\right) \times 14.63 \times 3.85^2 + \left(\frac{1}{12}\right) \times 7.5 \times 3.85^2$

$= 13.55 + 9.26$

$= 22.81\ kN\text{-}m$

Support moments :- (At support next to the end support).

$M_u = \left(\frac{1}{10}\right) \times 14.63 \times 3.85^2 + \left(\frac{1}{9}\right) \times 7.5 \times 3.85^2$

$= 21.68 + 12.35$

$= 34.03\ kN\text{-}m$

At other interior supports,

$M_u = \left(\frac{1}{12}\right) \times 14.63 \times 3.85^2 + \left(\frac{1}{9}\right) \times 7.5 \times 3.85^2$

$= 18.07 + 12.35$

$= 30.42\ kN\text{-}m$

Max. shear force at support next to the end support,

$V_{u,max} = (0.6 \times 14.63 \times 3.85) + (0.6 \times 7.5 \times 3.85)$

$= 63.27\ kN$

4. Depth required:-

$$M_u = 0.138 \cdot f_{ck} b d^2$$

$$34.03 \times 10^6 = 0.138 \times 20 \times 300 \times d^2$$

$$d = \sqrt{\frac{34.03 \times 10^6}{0.138 \times 20 \times 300}}$$

$$d = 202.72 < 250mm$$

Hence provided depth is adequate.

5. Reinforcement at supports:

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

$$34.03 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left[1 - \frac{415 A_{st}}{20 \times 300 \times 250}\right]$$

$$34.03 \times 10^6 = 90.26 \times 10^3 A_{st} - 24.97 A_{st}^2$$

$$24.97 A_{st}^2 - 90.26 \times 10^3 A_{st} + 34.03 \times 10^6 = 0$$

$$A_{st} = 427.6 mm^2$$

Provide, 12mm ϕ to finding no. of bars

$$\text{No. of bars} = \frac{A_{st}}{\frac{\pi}{4}(12)^2}$$

$$= \frac{427.6}{\frac{\pi}{4}(12)^2}$$

$$= 3.78 \simeq 4 \text{ bars}$$

Hence provide 4 bars of 12mm ϕ

Ast provided $= 4 \times \frac{\pi}{4}(12)^2$

$= 452.4mm^2$

6. Reinforcement at mid span:-

$$M_u = 0.87 \cdot f_y \times Ast \times d\left[1 - \frac{f_y \cdot Ast}{f_{ck} \cdot bd}\right]$$

$$29.2\times10^6 = 0.87\times415\times Ast\times250\left[1 - \frac{415 Ast}{20\times300\times250}\right]$$

$$29.2\times10^6 = 90.26\times10^3 - 24.97Ast^2$$

$$24.97Ast^2 - 90.26\times10^3 + 29.2\times10^6 = 0$$

$$\boxed{Ast = 359.2mm^2}$$

Assume 12mm ϕ bars,

No. of bars $= \frac{359.2}{\frac{\pi}{4}(12)^2} \Rightarrow 3.17 \approx 3$ bars.

Hence provide 3 bars of 12mm ϕ,

Ast provided $= 3\times\frac{\pi}{4}(12)^2$

$= 339.3mm^2$

7. Design of shear reinforcement:

$$\tau_v = \frac{V_u}{bd} = \frac{63.27\times10^3}{300\times250} = 0.84N/mm^2$$

percentage of steel $p_t = \frac{A_{st}}{bd} \times 100$

$= \frac{628.3 \times 100}{300 \times 200} = 1.04\%$

Referring to the table -19 of IS: 456, shear strength of concrete is

1.00 — 0.62

1.25 — 0.67

1.04 — ?

$\tau_c = 0.67 + \left(\frac{0.67 - 0.62}{1.25 - 1.00}\right)(1.04 - 1.00) \Rightarrow 0.67 N/mm^2$

max. shear stress in concrete τ_c max from table 20 of IS 456 (Page NO: 73)

$\tau_c max = 2.8 N/mm^2$

As $\tau_v > \tau_c$, shear reinforcement has to be designed

shear resistance of concrete $V_{uc} = \tau_c \cdot bd$

$= 0.67 \times 300 \times 200$

$= 40200 N$

$= 40.2 kN$

shear to be resisted by shear reinforcement (vertical stirrups)

$V_{us} = V_u - V_{uc}$

$= 49.15 - 40.2$

$= 8.95 kN$

using 6mm, 2 legged Fe 415 steel

$A_{sv} = 2 \times \frac{\pi}{4}(6)^2 \Rightarrow 56.55mm^2$

Spacing $S_v = \frac{0.87 f_y . A_{sv} . d}{V_{us}} \Rightarrow \frac{0.87 \times 415 \times 56.55 \times 250}{8950}$

$= 456.25mm$

max. allowed spacing $0.75d \Rightarrow 0.75 \times 200 \Rightarrow 150mm$

a 300mm which ever is less.

Hence, provide 2 legged 6mm stirrups @ 150 mm c/c

Check for deflection: (Ref Stiffness)

$\frac{l}{d} = 26$ (for continuous)

% of steel @ mid span $= \frac{603.18 \times 100}{300 \times 200} = 1.00$

$f_s = 0.58 \times 415 \times \frac{456.68}{603.18} \Rightarrow 181.99 N/mm^2$

from fig. 4 of IS: 456

modification factor $= 1.3$

max. permitted $\frac{l}{d}$ ratio $= 1.3 \times 26$
$= 33.8$

$\frac{l}{d}$ provided $= \frac{3800}{200}$

$= 19 < 33.8$

∴ Hence deflection control is safe.

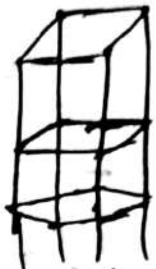
Coloumns   Clause - 25

Limit state of collapse :- compression.

Axial moment

Biaxial moment

Based on reinforcement



Tied coloumn.

Spiral coloumn.

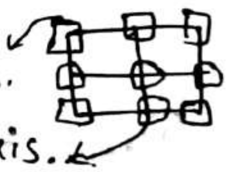
Composite coloumn.

Based on the loading.

i) Axially loaded.

ii) Eccentrically loaded

i) Ecc. " " in one axis.



" " " " double axis.

iii) Based on slenderness ratio

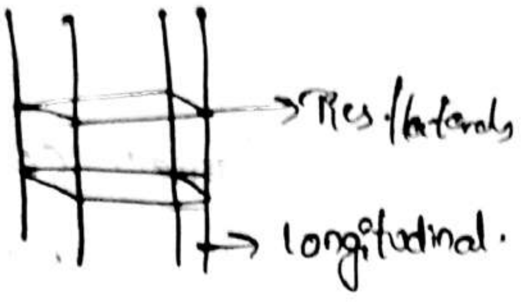
i) Short coloumn. $\frac{l}{b} \leq 12$.

ii) long or slender coloumn. $\rightarrow \frac{l}{b} > 12$.

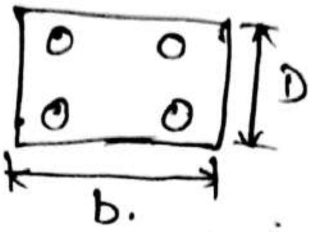
<u>Coloumns</u>

<u>Form</u>

1) Circular
2) Rectangular
3) Square.



$P = \frac{Ast}{bD} \times 100$



D = gross area. (Total)

$\boxed{l_c < 3b}$

25.12

it is coloumn : if not pedestal.

Pg.41.

$$\boxed{e = \frac{l}{500} + \frac{b}{30}}$$

25.4 subjected to min of 20mm

$\boxed{e_{min} = 20mm}$

Pg.No-48 , 26.5.3. , 26.5.3.1

↓ Coloums ↓ longitudinal reinforcement

↓

*** 0.8% to 6%

for circular - min 6 bars

Dler - min 4 bars.

PgNO-49 : c) Pitch & diameter of the lateral ties.
↓
spacing

S_{max} i) b
ii) 16 ϕ times the small diameter of long st
iii) 300 mm.

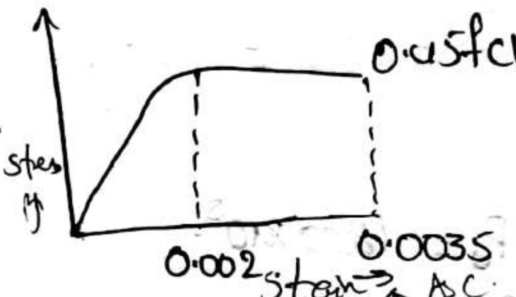
which ever is less.

2) diameter :-

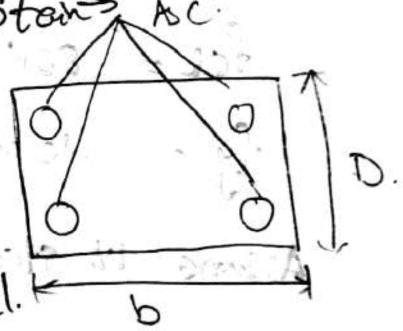
$d = \frac{1}{4} \times \phi \rightarrow$ largest longitudinal bar dia.
< 16mm.

PgNO- 70

axial compression is
0.002.



C/s of the coloumn.



$A_c = (bD) \cancel{Asc} - Asc.$

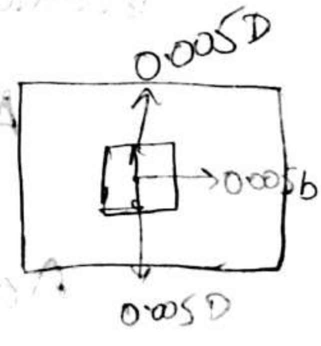
P_u = load carried by concrete + load carried by steel.

$= A_c (0.45 f_{ck}) + Asc (0.75 f_y)$

$= 0.9 (0.45 f_{ck} A_c + 0.75 f_y A_{sc})$

$= 0.405 f_{ck} A_c + 0.675 f_y Asc.$

$\boxed{P_u = 0.4 f_{ck} A_c + 0.67 f_y Asc}$



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

$A_{sc} = 4 \times \frac{\pi}{4} \times 12^2 = 452.38$

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$.

$A_c = (250 \times 300) - 452.38$

$= 74547.62$.

$P_u = 0.4 \times 20 \times 74547.62 + 0.67 \times 415 \times 452.38$

$= 596\ kN + 125\ kN$

$= 722.165\ kN$.

Design a square coloumn for the design load of 1200 kN Use M20 Concrete, Fe415 steel.

$P_u = 1200 \times 10^3$.

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

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

Assume 1% steel $A_{sc} = 1\%$ steel.

$= 0.01 \times A_c$.

$1200 \times 10^3 = 0.4 \times 20 \times A_c + 0.67 \times 415 \times 0.01 A_c$

$$A_c = \frac{1200 \times 10^3}{10.78} = 111312\ mm^2$$

$A_c = 111312 mm^2$.

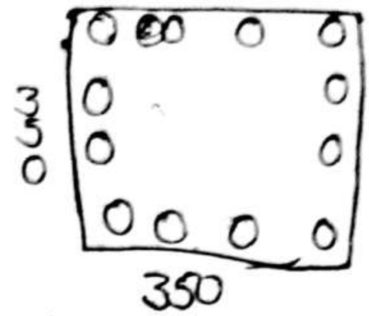
$b = \sqrt{111312}$ for square.

$= 333.6$

∴ Provide 350 x 350 mm.

$$A_{sc} = 0.01 \times (350 \times 350)^2$$

$$= 1225\,mm^2$$

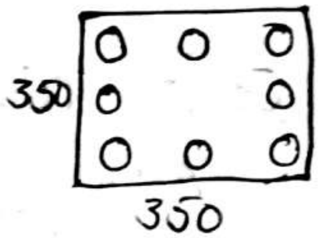


12 Φ - 10.83.

16 Φ = 6.09

Provide 12 - 12Φ as longitodinal reinforcement

Provide 8 - 16Φ as longitodinal rft.



$$P = \frac{A_{sc}}{bd} \times 100.$$

$$= \frac{8 \times \frac{\pi}{4} \times 16^2}{350 \times 350} \times 100$$

$$= 1.3\ \%.$$

Hence Ok.

Lateral ties:-

Diameter $d = \frac{1}{4} \times 16$

$= 4mm.$

Hence provide 6mm φ (m.s) bar.

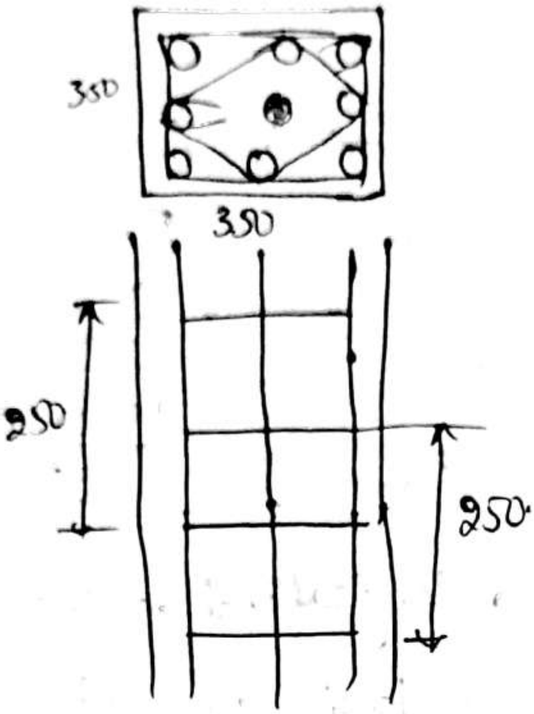
spacing. least of

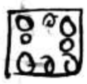
350 mm

16 φ = 16×16 = 256mm

300 mm.

Provide 6mm φ @ 250mm c/c.



Center coloumns :- P_U 

Axial :- P_U, M_U 

Biaxial :- P_U, M_x, M_y 

Design a short axially loaded tied coloumn pinned at both ends with an unsupported length of 3.5 mts. for carrying a characteristic load of 1500 kN. Use M20, Fe 415 steel.

Given data;

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

$P = 1500\ kN.$

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

$l = 3500\ mm$

Pinned - pinned.

Scanned by CamScanner

Design load $P_u = 1.5 \times 1500 = 2250$ kN.

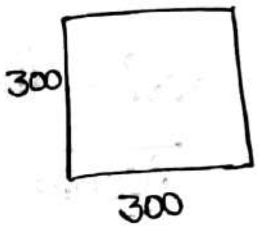
Eff length $l_e = l = 3500$ mm.

For short Coloum $\frac{l_e}{D} = 12$.

$$\frac{3500}{12} = D.$$

$$D = 291.67 \text{mm}.$$

Assume Col. section 300×300 mm².



$$e_{min} = \frac{l}{500} + \frac{D}{30}.$$

$$= \frac{3500}{500} + \frac{300}{30}$$

$$= 17 < 20 \text{mm}.$$

$$\frac{l}{D} = \frac{3500}{300}$$

$$= 11.67 < 12.$$

$A_g = 300 \times 300 = 90000$ mm².

Area of steel $A_{sc} = ?$

$$A_c = A_g - A_{sc} = 90000 - A_{sc}.$$

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

$$2250 \times 10^3 = 0.4 \times 20 \times (90000 - A_{sc}) + 0.67$$

$$2250 \times 10^3 = 720000 - 8A_{sc} + 278.05 A_{sc}.$$

$$= 720000 - 270.05 A_{sc}.$$

$$A_{sc} = 5665.61 \text{mm}^2$$

$$\% \text{ steel} = \frac{A_{sc}}{bd} \times 100 = \frac{5665.61}{300 \times 300} \times 100$$

$$= 6.29\%$$

$A_{sc} = 5665.61 \, mm^2$.

25 ϕ.

$$A\,n = \frac{5665.61}{490.8}$$

$$= 11.54 \text{ bar} \simeq 12 \text{ bars.}$$

12/9/015

Revise the section

Assume 375×375mm

140

$$\frac{le}{D} = \frac{3500}{375} = 9.33 < 12$$

~~140625~~.

$$A_g = 375 \times 375 = 140625 \, mm^2$$

$$A_c = A_g - A_{sc} = 140625 - A_{sc}.$$

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

$$2250 \times 10^3 = 0.4 \times 20 \times ((140625 - A_{sc}) + 0.67 \times 415 \times A_{sc}.$$

$$A_{sc} = 4165.89 \, mm^2.$$

$$\% \text{ of steel } p = \frac{A_{sc}}{bd} \times 100$$

$$= \frac{4165.89}{375 \times 375} \times 100 = 2.96\%$$

$$A_{sc} = 4165.89 \, mm^2.$$

Assume 25ϕ $A\phi = \frac{\pi}{4} \times 25^2 = 490.87\ mm^2$

$$n = \frac{416589}{490.87}$$

$$= 8.48$$

Provide $10 \times 25\phi$

$$\%\ \text{steel provided} = \frac{10 \times \frac{\pi}{4} \times 25^2}{375 \times 375} \times 100 = 3.49\%$$

Lateral ties :

$$d = \frac{\phi}{4} = \frac{25}{4} = 6.25mm.$$

Provide 8mm ϕ.

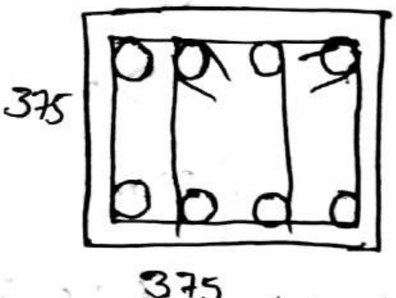
Spacing:

i– 375mm.

ii) $16\phi = 16 \times 25 = 400mm$

iii) 300 mm.

Ties 8ϕ @ 300 c/c.



① Design a circular coloumn to carry an axial load of 1000kN using lateral ties. Use M_{20}, Fe415 steel.

$P = 1000kN.$

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

$f_y = 415 N/mm^2.$

Design load $P_u = 1.5 \times P = 1.5 \times 1000$

$= 1500\ kN.$

$A_{sc} = 1452.20 \text{ mm}^2$

Assume 16 ϕ $\quad A\phi = 201.06$

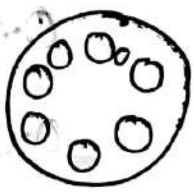
$$n = \frac{1452.20}{201.06}$$

$= 7.22$ bars.

provide 8 bars 16 ϕ.

A_{sc} provided $= 1608.49$.

$$A_{sc} \text{ provided} = \frac{1608.49}{\frac{\pi}{4} \times D^2} \times 100$$



$= 1.44$

lateral ties;

Dia $\quad d = \frac{\phi}{4} = \frac{16}{4} = 4\text{mm}$.

provide 6mm ϕ (M.S).

<u>spacing</u>

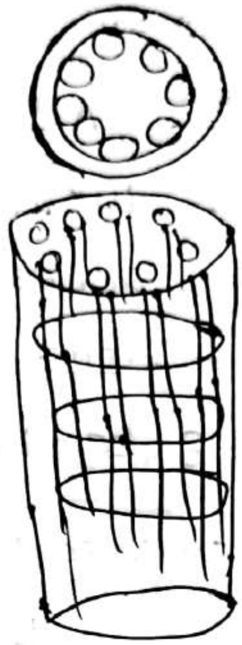
i) 430 mm

ii) $16 \times 16 = 256$mm

iii) 300mm.

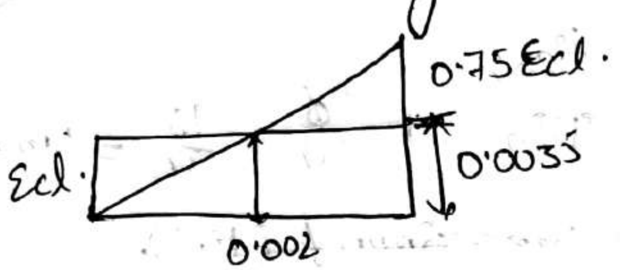
Provide 6 mm ϕ @ ~~3~~250 mm c/c.

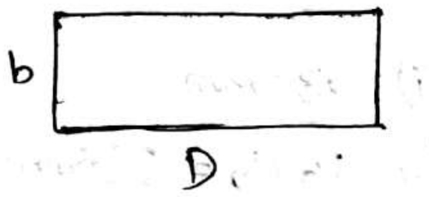
(4)



0.002 for axial load.

0.0035 for bending.





$\varepsilon_{cl} = 0.0035 - 0.75\,\varepsilon_{cl}$.

Design of columns having axial load & moment (P_U, M_U)

$M_U = 0$ then it is a axially loaded coloumn.

$P_U = 0$ then pure bending casing

P_U & M_U are present ~~po~~ uniaxial coloumn.

Design a coloumn for the following data.

Axial load $P_U = 1200$ kN.

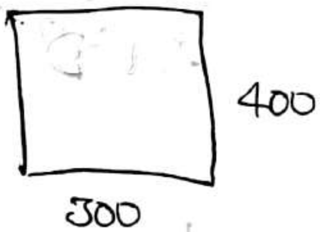
B.M $M_U = 250$ kN-m.

Unsupported length $l = 3200$ mm.

Clear cover 40mm.

pinned-pinned case.

Given data.



$P_U = 1200$ kN.

$M_U = 250$ kN-m.

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

$f_y = 415 N/mm^2$.

C.C = 40mm

$l = 3200$ mm.

Pinned-pinned.

Effective length $l_e = l = 3200$ mm.

$$e = \frac{l}{500} + \frac{D}{30}.$$

$= \frac{3200}{500} + \frac{400}{30}$

$= 19.73 < 20mm.$

$e_{min} = 20mm.$

$M_U = P_U \times e \Rightarrow e = \frac{M_U}{P_U} = \frac{250 \times 10^6}{1200 \times 10^3}$

$= 208.33$

$e > e_{min}$

11/9/015

Design for bending.

$\frac{P_U}{f_{ck} bD} = \frac{1200 \times 10^3}{20 \times 300 \times 400}$

$= 0.5$

$\frac{M_U}{f_{ck} bD^2} = \frac{250 \times 10^6}{20 \times 300 \times 400}$

$= \cancel{0.5}\ 0.26$

$d' = 40mm + \frac{\phi}{2}$

$= 50mm$

$\frac{d'}{D} = \frac{50}{400} = 0.125$

From SP-16, chart 33, $f_y = 415 N/mm^2$

$\frac{P}{f_{ck}} = \frac{0.24}{\cancel{20}} \times 20$

$= 4.8$

Detailing.

$P = 4.8\%$

$$A_{sc} = \frac{4.8}{100} \times 300 \times 400$$

$$= 5760\ mm^2$$

$20\ \phi \quad A\phi = 314\ mm^2$

$$= \frac{5760}{314} = 18.34\ bars$$

Provide. 20 bars

$25\ \phi$.

$$= \frac{5760}{25}$$

$$= 12\ bars.$$

$$= 300 - 40 - 40 - \frac{25}{2} - \frac{25}{2}$$

$$= 195$$

$$= \frac{195}{5} = 39\ mm.$$

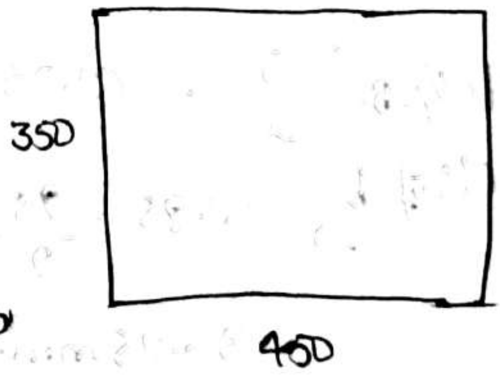
$$\Rightarrow 39 - \frac{25}{2} - \frac{25}{2}$$

$$= 14\ mm < 25\ mm.$$

Revise the section;

$$\frac{P_u}{f_{ck} b d} = \frac{1200 \times 10^3}{20 \times 350 \times 450} = 0.38$$

$$\frac{M_u}{f_{ck} b D^2} = \frac{250 \times 10^6}{20 \times 350 \times 450^2} = 0.176$$



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

$= 52.5$

$\frac{d'}{D} = 0.15$

Chart-33, SP-16

$\frac{P}{f_{ck}} = 0.135$

$P = 2.7\%$

$A_{sc} = \frac{2.7}{100} \times 350 \times 450$

$= 4252.5 \; mm^2$

Assume 25 Φ = $A\phi$ = $490.87 \; mm^2$

$n = 8.66$

$10 \times 25 \; \Phi$.

Detailing:-

c/c of outer bars.

$350 - 40 - 40 - \frac{25}{2} - \frac{25}{2}$

$= 245$

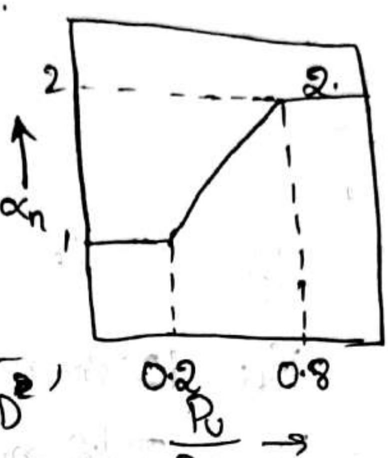
c/c of I.B. $\frac{245}{4} = 61.25mm$.

c/s of b $\Rightarrow$ $61.25 - \frac{25}{2} - \frac{25}{2}$

$= 36.25mm$.

Biaxial colamns

Procedure:- P_u, M_{ux}, M_{uy}.



1) Assume crossSection. (b,D)

Assume % of steel (p).

step-2 find p/f_{ck}, $\frac{P_u}{f_{ck}bD^2}$,

$\frac{M_{ux_1}}{f_{ck}bD^2} \rightarrow$ & find M_{ux_1}

step-3 find $\frac{p}{f_{ck}}$; $\frac{M_{uy_1}}{f_{ck}bD^2} \rightarrow M_{uy_1}$.

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{yy}}{M_{yu_1}}\right)^{\alpha_n} \leq 1.$$

Rectangular coloumn with biaxial bending.

Determine the rft to be provided in a coloumn subjected to bioxial bending with the following data

Size of coloumn 400X500 mm.

Concrete mix M20.

chaecteristic strength of rft 415 N/mm^2

Factored load = 1200kN.

Factored moment acting ll to the larger dimension 100 kN-m.

Factored moment acting // to the shorter direction
= 80 kN-m

Clear cover = 40 mm.

Given;

$P_U = 1200$ kN.

$M_{ux} = 100$ kN-m.

$M_{uy} = 80$ kN-m.

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

$f_y = 415 N/mm^2$

Sized coloumn = 400x500 mm.

Assume % of steel as P = 1.6%.

Moment carrying capacity

$$\frac{P}{f_{ck}} = \frac{1.6}{20} = 0.08.$$

$$\frac{P_U}{f_{ck}bD} = \frac{1200 \times 10^3}{20 \times 400 \times 500}$$

chart

$= 0.3.$

$$\frac{d'}{D} = \frac{52.5}{500} = 0.105$$

chart - 44.

$$\frac{M_{ux_1}}{f_{ck}bD^2} = \frac{M_{ux}}{\cancel{20 \times 400 \times 500^2}} = \cancel{0.105}$$

0.12

$= \cancel{5 \times 10^{10} M_{ux_1}}.$

$M_{ux_1} = 240$ kN-m.

0.9

$$\frac{d'}{D} = \frac{52.5}{400} = 0.13125$$

$$\frac{M_{uy_1}}{f_{ck}Db^2} = \frac{M_{uy_1}}{20 \times 500 \times 400^2}$$

Chart 45

$$\frac{M_{uy_1}}{f_{ck}b^2D} = 0.11$$

$$M_{uy_1} = 176 \text{ kN-m.}$$

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha n} \leq 1$$

$$\alpha n = \frac{P_u}{P_z}$$

$$P_z = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}.$$

$$A_{sc} = \frac{1.6}{100} \times 400 \times 500$$

$$= 3200 \text{ mm}^2$$

$$A_g = 400 \times 500 = 200000 \text{ mm}^2$$

$$A_c = A_g - A_{sc}$$

$$= 200000 - 3200$$

$$= 196800$$

$$P_{uz} = 0.45 \times 20 \times 196800 + 0.75 \times 415 \times 3200$$

$$= 2767200$$

$$P_u / P_{uz} = \frac{1200 \times 10^3}{2767200}$$

$$= 0.4336$$

$$\left(\frac{100\times10^6}{240\times10^6}\right)^{1.28} + \left(\frac{80\times10^6}{116\times10^6}\right)^{1.38}$$

$= 0.\cancel{334}\,298 + 0.336$

$= \cancel{0.707} < 1$ $0.634 < 1.$
Hence ok.

Assume 25 Φ.

As % of steel provided $= 1.96\%$

$= \dfrac{3200}{490.87}$

$= 6.519$

Provide 8x 25 Φ.

Lateral ties - $\frac{\phi}{4} = \frac{25}{4} = 6.25$

Provide 8 mm bars.

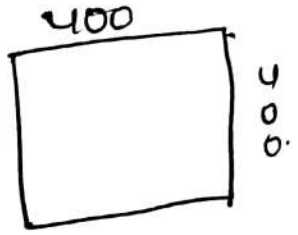
Spacing / Pitch.
i) 400 mm.
ii) $16\phi = 16\times25 = 400$
iii) 300mm

Provide 8Φ @ 300 mm c/c

Circular coloumn with

Design a reinforced coloumn 400mm square to carry an ultimate load of 1000kN at an eccentricity of 160mm. Use M20, Fe250

Sol $P_U = 1000kN$.



$E = 160mm$.

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

$f_y = 250N/mm^2$.

$b = 400mm$

$D = 400mm$.

$M_U = P_U \times E = 160kN\text{-}m$.

Assume % of steel $= 1.6\%$.

$$\frac{P}{f_{ck}} = \frac{1.6}{20} = 0.08$$

$$\frac{P_U}{f_{ck}bD} = \frac{1000\times10^3}{20\times400\times400} = 0.3125$$

$$\frac{d'}{d} = \frac{50}{400} = 0.125$$

$$\frac{M_U}{f_{ck}bD^2} = \frac{160\times10^6}{20\times400\times400^2}$$

$= 0.125$

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Design a short circular column of 500mm dia with the following data.

Factored load = 800kN.

Factored moment = 162.5 kN-m.

Provide Hoop reinforcement. Take M20, Fe415 steel.

$D = 500mm.$

$P_u = 800kN.$

$M_u = 162.5 \, kN\text{-}m.$

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

$f_y = 415N/mm^2.$

$d' = 40 + \frac{20}{2} = 50\,mm$ (Assume 20Φ)

$$\frac{d'}{D} = \frac{50}{500} = 0.1$$

$$\frac{P_u}{f_{ck}D^2} = \frac{800 \times 10^3}{20 \times 500^2} = 0.16.$$

$$\frac{M_u}{f_{ck}D^3} = \frac{162.5 \times 10^3}{20 \times 500^3} = 0.065.$$

Chart-56 $\frac{P}{f_{ck}} = 0.05.$

$P = 0.05 \times 20 = 1\%$

$P = 1\%.$

$$A_{sc} = \frac{1}{100} \times \frac{\pi \times 500^2}{4}$$

$= 1963\,mm^2.$

$20\,\bar{\Phi} = A\phi = 314\,mm^2$

$n = 6.25$

$7 \times 20\,\bar{\Phi}$.

lateral ties

$d = \frac{20}{4} = 5\,mm$

Provide 6ϕ

spacing i) 500 mm

ii) $16 \times 20 = 400\,mm$

iii) 300 mm.

Provide 6 mmϕ @ 300 mm c/c.

17/015

$b = 250\,mm$

$P_U = 600\,kN\text{-}m.$

$M_{ux} = 60\,kN\text{-}m.$

$M_{uy} = 40\,kN\text{-}m.$

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

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

Assume $P = 3\%$

$$\frac{P}{f_{ck}} = \frac{3}{30} = 0.1$$

$$\frac{P_U}{f_{ck}\,bD} = \frac{600 \times 10^3}{30 \times 250 \times 250}$$

$$= 0.32.$$

$\frac{M_{ua_1}}{f_{ck} b D^2} = ? = 0.11$

$40 + \frac{20}{2} = 50$

$\frac{d'}{D} = \frac{50}{250} = 0.2$

$M_{ua_1} = 51.5625 \text{ kN m}$

Assume $P = 4.2$

$\frac{P}{f_{ck}} = \frac{4.2}{30} = 0.14$

$\frac{M_{ua_1}}{f_{ck} b D^2} = 0.14$

$M_{ua_1} = 65.625 \text{ kN-m}$

$M_{uy_1} = 65.625 \text{ kN-m}$

$A_g = 250 \times 250 = 62500 \text{ mm}^2$

$A_{sc} = \frac{PbD}{100} = 2625 \text{ mm}^2$

$A_c = 59875 \text{ mm}^2$

$P_{uz} = 0.45 f_{ck} A_c + 0.75 \times f_y \times A_{sc}$

$= 1625343.75$

$\frac{P_u}{P_z} = \frac{600 \times 10^3}{1625343.75}$

$= 0.369$

$0.2 \rightarrow 1$

$0.8 \rightarrow 2$

$0.369 \rightarrow ?$

$1 + \frac{(2-1)}{(0.8-0.2)}(0.369-0.2)$

$= 1.281$

$$= \left(\frac{60}{65.625}\right)^{1.28} + \left(\frac{40}{65.695}\right)^{1.28}$$

$= 0.891 + 0.53$

$= 1.42.$ $\qquad P = 1.8$

$M_{ua} = 0.16 \times 30 \times 250^3$

$= 75 kN\text{-}m.$

$A_g = 3000$

$A_c = 59500$

$A_g = 62500.$

$P_{uz} = 1737000$

$P_{\alpha n} - 0.345$

0.34

$$1 + \frac{(2-1)}{(0.8-0.2)}(0.34-0.2)$$

$= 1.23.$

$= 0.75 + 0.46.$

$1.21.$

5.4

$\frac{P}{P_{ct}} = 0.018$

0.17

79.6887500

3375

59125

$P_{UZ} = 1848656.25$

$P_{U} = 0.324$

14 (0.

~~1.015~~ 1.206

$$\left(\frac{60}{79.688}\right)^{\cancel{1.015}\ 1.206} + \left(\frac{40}{79.688}\right)^{1.206}$$

~~0.744 + 0.49~~ 0.711 + 0.435

= ~~1.239~~ 1.14

6) = 0.2

0.19 x 89062500

= 3750 / 58750

$P_{z} = 1960312.5$

$= 0.306.$

1.16.

$$= \left(\frac{60}{89.0625}\right)^{1.16} + \left(\frac{40}{89.0625}\right)^{1.16}.$$

$$= 0.6\cancel{82} + 0.3$$

$$= 0.9.$$

<u>Spiral coloumn:-</u>

Design a short circular coloumn 500 dia

factored load = 800kN

factored moment = 162.5 kN-m

provide Helical Hoop reinforcement. Take M20, Fe415 steel

$P_u = 800$ kN

$M_u = 162.5$ kN-m.

$f_{ck} = \cancel{415}\ 20 N/mm^2$.

$f_y = 415 N/mm^2$.

$$\left(\frac{P_u}{f_{ck}D^2}\right)_{Hoop} = 0.16.$$

$$\left(\frac{P_u}{f_{ck}D^2}\right)_{Helical} = 0.152.$$

$\left(\frac{M_U}{f_{ck} D^3}\right)_{Hoop} = 0.063$

$\left(\frac{M_U}{(f_{ck} D^3)}\right)_{Helical} = 0.061.$

$\frac{P}{f_{ck}} = 0.04 \Rightarrow P = 0.04 \times 20$

$= 0.8\%.$

$\therefore$ Area of steel $= \frac{P}{100} \times \frac{\pi}{4} \times D^2$

$= 1570 \text{ mm}^2.$

Assume 20ϕ.

$\therefore$ provide 6×20 TMT.

$P = \frac{6 \times \frac{\pi}{4} \times 20^2}{\frac{\pi}{4} \times 500^2} \times 100$

$= 0.96\%$

Let us use 8mm dia mild steel bar for helical reinforcement.

$\therefore$ Core dia; $D_c = 500 - 2\times40 + 2\times8$

$= 438$ mm.

500-2x40+2x8

$\therefore$ Core area $= \frac{\pi}{4} \times D_c^2$

$V_c = A_c = 149301 \text{ mm}^2$

$\therefore \frac{A_g}{A_c} = \frac{\frac{\pi}{4} \times 500^2}{436^2} = 1.3151$

$0.36\left(\frac{A_g}{A_c}-1\right) f_{ck} / f_{yh}$

$0.36\left(1.315-1\right)\frac{20}{250}$

$= 0.00907$

Dia. of core upto center of helics.

$500 - 2\times40 + 8 = 428$ mm.

Let the pitch of the spiral be 'S' mm

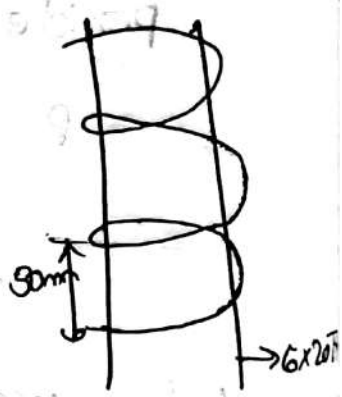
volume of spiral for 1mm length of coloumn.

$V_H = \frac{\pi d}{s} \times \frac{\pi}{4} \times 8^2$

$= \frac{\pi \times 428}{s} \times \frac{\pi}{4} \times 8^2$

$\frac{V_H}{V_C} \quad = \frac{67587}{s}$

$\frac{V_H}{V_{AC}} = \frac{\frac{67587}{s}}{149301}$



$= \frac{0.452}{s}$

$\frac{V_H}{V_C} = 0.36\left(\frac{A_g}{A_c}-1\right)\frac{f_{ck}}{f_{yh}} = 0.00907$

$\frac{0.452}{s} = 0.00907 \cdot = 49.9$ mm.

$S = 50$ mm

pitch should not more than

i) 75mm.

ii) $\frac{1}{6} D_c . = \frac{1}{6} \times 436$

$= 72.67$

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Pitch should not be less than $= 3 \times \phi_s$

$= 3 \times 8 = 24mm$

Hence pitch 50mm is 'ok'

Long coloumns:-

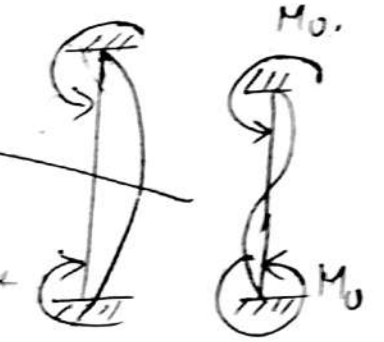
$$M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

we have

$$M_{ax} = P_u e_{ax}$$

$$\therefore e_{ax} = \frac{D}{2000}\left(\frac{l_{ex}}{D}\right)^2.$$

$$M_{ay} = \frac{P_u b}{2000}\left(\frac{l_{ey}}{b}\right)^2.$$



$$M_{ux} = 0.6M_2 + 0.4M_1.$$

$$P_b = \left(k_1 + k_2 + \frac{p}{f_{ck}}\right) f_{ck} bD.$$

Design a slender braced circular coloumn under uniaxial bending with the following data.

Size of coloumn – 400mm.

M20 concrete

Fe 415 steel.

Effective length 6mts, unsupported length 7mts.

factored load $P_u = 1200$ kN.

moment $M_{ux} = 75$ kN-m at top & 50kN-m at bottom. Assume coloumn is bent in single curvature d

$$\frac{l_{ex}}{D} = \frac{6000}{400} = 15 > 12.$$

Hence it is long coloumn.

Additional moments

$$e_{ax} = \frac{D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

$$= \frac{400}{2000}(15)^2$$

$$= 45\text{mm.}$$

$$M_{ax} = P_u \times e_{ax}$$

$$= 1800 \times \frac{45}{1000}$$

$$= 54\text{ kN-m.}$$

Calc: of ka:

$$k_a = \frac{P_{uz} - P_u}{P_{uz} - P_b}.$$

$$P_b = \left(k_1 + k_2 \frac{p}{f_{ck}}\right) f_{ck} D^2.$$

$P =$ 2.5%.

$$\frac{d'}{D} = \frac{50}{400} = 0.125.$$

Table 60; $k_1 = 0.155$

$k_2 = 0.266.$

$$= \left(0.155 + 0.266 \times \frac{2.5}{20}\right) 20 \times 400^2$$

$$= \cancel{1744800} = 601\text{ kN.}$$

2/9/01.

$$K = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

$A_g = \frac{\pi}{4} \times D^2 = \frac{\pi}{4} \times 400^2 = 125663.7 \ mm^2$.

$A_s = \frac{P}{100} \times \frac{\pi}{4} \times D^2 = \frac{2.5}{100} \times \frac{\pi}{4} \times 400^2$.

$3141.59 \ mm^2$

$A_c = 122522.1 \ mm^2$

$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}$

$= 0.45 \times 20 \times 122522.1 + 0.75 \times 415 \times 3141.59$.

$= 2080518.854 \ N\text{-}m$.

$$K = \frac{2080518.854 - 1200 \times 10^3}{2080518.854 - 60 \times 10^3}$$

$= 0.596 < 1$.

Assume 20 ϕ.

$$= \frac{3141.59}{314} = 10.005 \text{ bar}$$

provide $10 \times 20 \phi$.

$\therefore$ Reduced Additional moment.

$M'_a = k M_a$.

$= 0.59 \times 54$.

$= 31.86 \ kN\text{-}m$

Slabs

Slabs :- Flexure, shear.

$$M = \frac{wl_e^2}{8} = \frac{W l_e}{8} \qquad W = wl_e$$

$$V = \frac{wl_e}{2} = \frac{W}{2}.$$

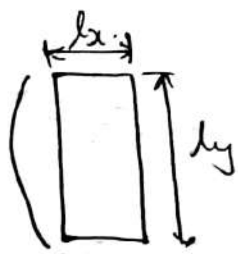
l_{eff} is the least of

i) Clearspan + d.

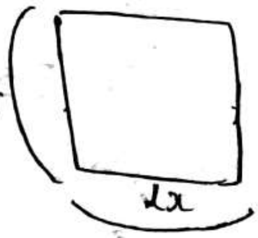
ii) c/c of supports.

Slab types:-

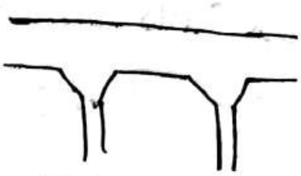
1) One-way slab $\rightarrow \frac{l_y}{l_x} > 2$.



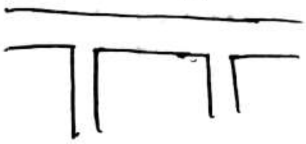
2) Two-way slab.



3) Flat slab



4 Flat plate.



IS 875 (Part 1 & 2).

Finishes and Partitions 1.5 kN/m².

For roofs:-

1.5 kN/m² with access.

0.75 kN/m² with out access

for floors:-

2 kN/m² for residential buildings.

3 kN/m² for office building.

Eeffective span:-

l_{eff} small of

i) c/c of supports.

ii) c.s + d.

minimum reft for slabls - 0.12%

Minimum % of steel:-

0.12% for HYSD

0.15% for mild steel.

Design of One-way slab:-

Design a simply supported R.C.C-slab for a roof a hale 3.5 mts x 8mts (inside dimensions) with 250 mm walls all around. Assume a live load of $4 kN/m^2$ and finishing load $1 kN/mm^2$. Use M20 & Fe415 steel.

Step-1 Given data:-

Hall = 3.5m x 8m

width of masonry = 250mm

live load = $4 kN/m^2$.

Finishing = $1 kN/m^2$

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

$f_y = 415 N/mm^2$

Step 1 Cal. of factored loads.

$d = \frac{l}{d} = 25$

$d = \frac{l}{25}$

Assume $\frac{l}{d} = \frac{3500}{25} = 140mm$.

Overall depth $D = d + c.c + \frac{\phi}{2}$

$= 140 + 15 + \frac{10}{2}$

$D = 160mm$.

Dead load;

i) slab $= 25 \times 0.16 = 4 kN/m^2$.

ii) finishish $= \underline{\quad 1 kN/m^2}$

Total dead load $= 5 kN/m^2$.

Live load $= 4 kN/m^2$.

Total load $W = D.L + L.L = 5 + 4 = 9 kN/m^2$.

Factored load $W_U = 1.5 \times W$

$= 1.5 \times 9 = 13.5 kN/m^2$

Step-2 Calc. of effective span.

l_{eff} is the least of

i) c/c of supports.

$= 3.5 + 0.25$

$= 3.75m$.

ii) $c.s + d = 3.5 + 0.14$

$= 3.64 mts$.

Step 3

Factored B.M & S.F.

$$\text{Factored BM } (M_u) = \frac{w l_{eff}^2}{8}$$

$$= \frac{13.5 \times 3.64^2}{8}$$

$$= 22.35 \text{ kN-m}$$

$$\text{Factored SF } (V_u) = \pm \frac{w_u l_{eff}}{2}$$

$$= 24.5 \text{ kN}.$$

Step-4 Check for depth.

$$M_u = 0.138 f_{ck} b d^2$$

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{22.5 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 90\text{mm} < 140\text{mm}.$$

Step-5 Calc. of steel area.

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - \frac{6.6 M_u}{f_{ck} b d^2}}$$

$$\frac{6.6 M_u}{f_{ck} b d^2} = \frac{6.6 \times 22.35 \times 10^6}{20 \times 1000 \times 140^2}$$

$$= 0.376$$

$$\frac{x}{d} = 0.168$$

$$\frac{x}{d} = 0.168 < \left(\frac{x_{umax}}{d} = 0.48\right)$$

Hence it is under reinforced section

lever arm $z = d\left(1 - 0.42\frac{x}{d}\right)$

$= 140(1 - 0.42 \times 0.168)$

$= 130.12$ mm.

Area of steel $A_{st} = \frac{M_u}{0.87 f_y z}$

$= 475.7 \, mm^2$.

Assume 10TMT $A\phi = \frac{\pi}{4} \times 10^2 = 78.53 \, mm^2$.

No of bars $n = \frac{A_{st}}{A\phi}$.

$= \frac{475.7}{78.53}$

$= 6.05$.

Spacing $s = \frac{1000}{n}$.

$= \frac{1000}{6.05}$

$= 165.28$ mm.

Max. spacing (S_{max}) ~~3d~~

i = $3d = 3 \times 140 = 420$mm.

ii' 300m

$S_{max} = 300$mm

Provide 10TMT @ 160 mm c/c.

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Step-7 Check for deflection

For simply supported case basic value $\frac{l}{d} = 20$.

% of tensile steel at midspan.

$$Ast\ provide = \frac{1000}{S} \cdot \frac{\pi}{4} d^2$$

$$= \frac{1000}{160} \times \frac{\pi}{4} \times 10^2$$

$$= 490.8$$

$= \cancel{490.8\ mm^2}$. $\cancel{583}$

$Ast\ req = 475\ mm^2$.

$$f_s = 0.58 f_y \frac{Ast\ req}{Ast\ provided}$$

$$= 0.58 \times 415 \times \frac{475}{490.8}$$

$$= 232.9\ mm^2.$$

$$\cancel{Modi}\ \%\ of\ steel = \frac{490.8}{1000 \times 140} \times 100 = 0.35.$$

$$\frac{l}{d} = 20 \times 1.$$

$$\frac{l}{d} = \frac{3500}{140} = 25$$

Hence deflection limit exceeds. So change the spacing of the main steel. ie, Assume $S = 150$ mm instead of 160mm.

$$Ast\ provided = \frac{1000}{150} \times \frac{\pi}{4} \times 10^2$$

$$= 523.5\ mm^2.$$

$$0.58 \times 415 \times \frac{475}{523.5} = 1.55$$

$\frac{l}{d} = 20 \times 1.55$

$= 31.$

$\left(\frac{l}{d}\right)_{provided} = \frac{3500}{140} = 25$

Hence ok.

Step-8 Check for shear (optional).

$V_U = 24.5\,kN.$

$\tau_v = \frac{V_U}{bd} = \frac{24.5 \times 10^3}{1000 \times} = 0.175\,N/mm^2.$

Calc. of τ_c.

50%. $P_t = \frac{0.37}{2} = 0.185\%$. (at supports)

from Table 19 $\tau_c = 0.285\,N/mm$

$\tau_c > \tau_v$ Hence ok.

Step-9 Summary of design.

Depth of slab $D = 160\,mm.$

Cover = 15 mm.

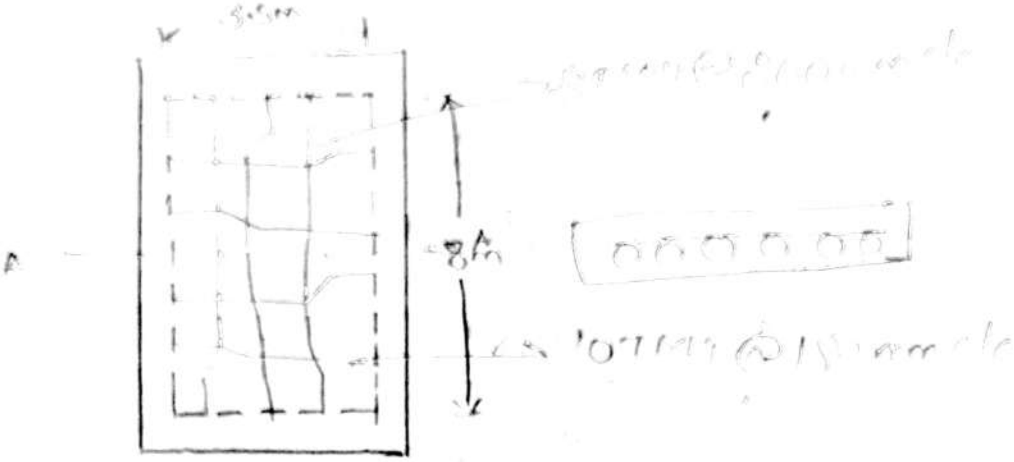
Concrete = M_{20}.

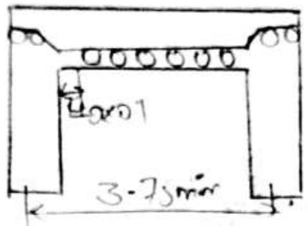
Steel Fe415.

Primary 10TMT @ 150 mm c/c

8 TMT @ 260 mm c/c

Step-10 Detailing:-

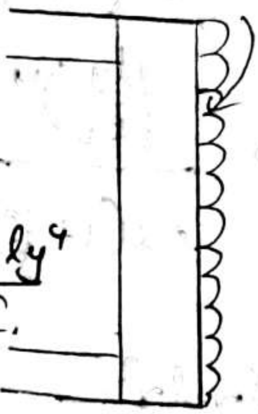


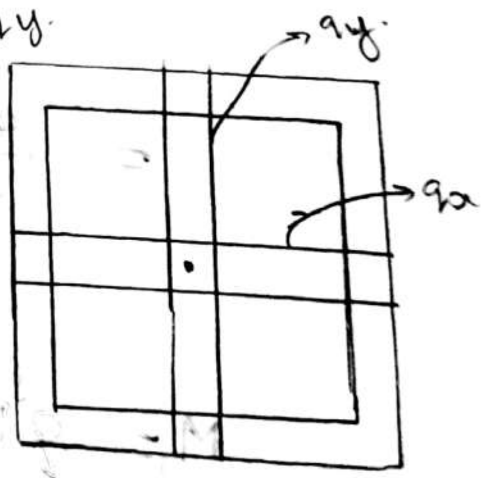


Moments developed in Two way slabs (simply supports case).

shorter. $\Delta_{max} = \frac{5}{384} \frac{q_x l_x^4}{EI}$.

longer $\Delta_{max.} = \frac{5}{384} \frac{q_y l_y^4}{EI}$.

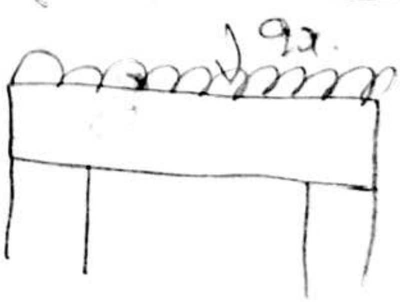




$\frac{5}{384} \frac{q_x l_x^4}{EI} = \frac{5}{384} \frac{q_y l_y^4}{EI}$.

$q_x = q_y \left(\frac{l_y}{l_x}\right)^4$.

$q_y = q_x \left(\frac{l_x}{l_y}\right)^4$.



$q = q_x + q_y$.

$q = q_x + q_x\left(\frac{l_x}{l_y}\right)^4 + q_x$.

Scanned by CamScanner

$$q_x = \frac{q}{\left(\frac{l_x}{l_y}\right)^4 + 1}.$$

$$q_x = \frac{q \times l_y^4}{l_x^4 + l_y^4}.$$

$$q_y = \frac{q\, l_x^4}{l_x^4 + l_y^4}.$$

$$M_x = \frac{q_x l_x^2}{8}$$

$$= \frac{q\, l_y^4}{8(l_x^4 + l_y^4)} \times l_x^2$$

$$= \frac{q}{8} \frac{l_y^4}{l_x^4\left(1 + \left(\frac{l_y}{l_x}\right)^4\right)} l_x^2$$

$$= \frac{q}{8} \frac{\left(\frac{l_y}{l_x}\right)^4}{\cancel{l_x}\left(1 + \left(\frac{l_y}{l_x}\right)^4\right)}$$

$$M_x = q\, \beta_x \cdot l_x^2.$$

$$\beta_x = \frac{\left(\frac{l_y}{l_x}\right)^4}{8\left(1 + \left(\frac{l_y}{l_x}\right)^4\right)}$$

$$M_y = \frac{q_y l_y^2}{8}$$

$$= \frac{q l_x^4}{8 l_x^4 + l_y^4} l_y^2$$

$$= q \cdot \frac{l_x^4}{8\left(l_x^4\left(1+\left(\frac{l_y}{l_x}\right)^4\right)\right)} l_y^2$$

$$= \frac{q}{8\left(1+\left(\frac{l_y}{l_x}\right)^4\right)} l_y^2$$

$$= \frac{q}{8\left(1+\left(\frac{l_y}{l_x}\right)^4\right)} \cdot \frac{l_y^2}{l_x^2} \times l_x^2$$

$$M_y = q \beta_y l_x^2$$

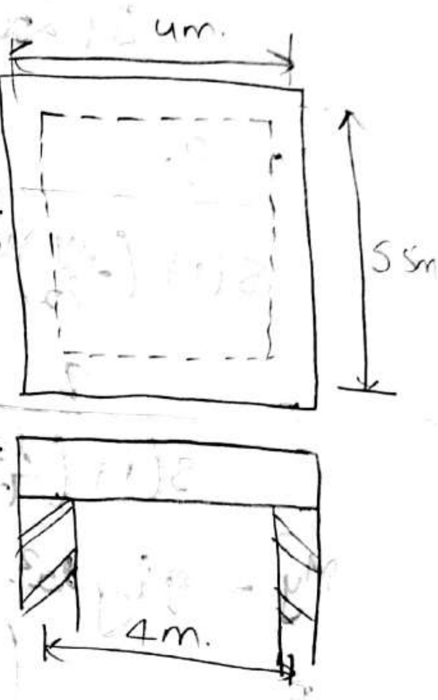
$$\beta_y = \frac{\left(\frac{l_y}{l_x}\right)^2}{8\left(1+\left(\frac{l_y}{l_x}\right)^4\right)}$$

$$d = \frac{\text{span}}{25 \text{ to } 30}$$

Design of Simply supported two-way slab:

Design a reinforced concrete slab 5.5 mts x 4mts. Simply supported on all the four sides it has to carry a characteristic live load of $8 kN/m^2$ M25, Fe415 exposure condition mild.

Given data.



Room Dimensions : 4m x 5.5m.

Concrete f_{ck} = 25 N/m

Steel f_y = $415 N/mm^2$.

Table 16 mild exposure = 20mm.

Step
Calc. of Depth of the slab.

$l_y = 5.5 m$

$l_x = 4m.$

$$\frac{l_y}{l_x} = \frac{5.5}{4} = 1.375 < 2.$$

Hence it is two way slab.

$l_x = 4000 mm.$

$$\text{Total depth } D = \frac{l_x}{24} = \frac{4000}{24}.$$

$= 166.67.$

$= 166 mm.$

Take D = 170 mm.

Effective depth $d = D - CC - \frac{\phi}{2}$

$= 170 - 20 - \frac{10}{2}$ (10 TMT bars)

$= 145$ mm.

Step-2 Design load.

i, Dead load =

1) Self weight of slab = $0.17 \times 25 = 4.25$ kN/m^2

2) Finishing load (25mm thick) $= 0.5$ kN/m^2

$\frac{25}{1000} \times 20$

3) Plastering load (6mm) $= \frac{6}{1000} \times 4 = 0.144$ kN/m^2

Total dead load = 4.894 kN/m^2

ii) Live load = 8 kN/m^2.

Total load = DL + LL

$= 4.89 + 8$

$w = 12.89$ kN/m^2

Design load = $1.5w$ $= 1.5 \times 12.89$

$= 19.341$ kN/m^2

Step-3 Design moment and S.F.

$w_u = 19.341$ kN/m^2

~~Moment~~ = $l_x = 4000$

$l_y = 5500$ mm.

$\frac{l_y}{l_x} = 1.375$

Table 27;

$$\frac{l_y}{l_x} = 1.3 \qquad \alpha_x = 0.093.$$

$$\frac{l_y}{l_x} = 1.4 \qquad \alpha_x = 0.099$$

$$0.093 + \frac{(0.099-0.093)}{(1.4-1.375)}(1.375-1.3)$$

$$= 0.0975.$$

$$\frac{l_y}{l_x} = 1.3 \qquad 0.055$$

$$\frac{l_y}{l_x} = 1.4 \qquad 0.051.$$

$$\alpha_y = 0.052.$$

$$M_x = \alpha_x w l_x^2$$

$$= 0.0975 \times 19.341 \times 4^2$$

$$= 30.17 \text{ kN-m}.$$

$$M_y = \alpha_y w l_x^2 = 0.052 \times 19.341 \times 4^2$$

$$= 16.09 \text{ kN-m}.$$

Step-4 Check for depth.

$$M_{max} = 30.17 \text{ kN-m}.$$

For Fe415 $M_u = 0.138 f_{ck} b d^2$.

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{30.17 \times 10^6}{0.138 \times 25 \times 100}}$$

$$= 93\text{mm}.$$

$d_{cal} = 93 < (d_{max} = 145\,mm)$.

Hence ok.

step-5 Primary steel.

$M_u = 30.17\,kN\text{-}m.$

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

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - \frac{6.6 M_u}{f_{ck} b d^2}}$$

$$= 1.2 \sqrt{1.44 - \frac{6.6 M_u}{f_{ck} b d^2}}$$

$$\frac{6.6 M_u}{f_{ck} b d^2} = 0.378.$$

$$\frac{x}{d} = 0.169 < 0.48$$

It is a under reinforced.

lever arm $= 145 (1 - 0.416 \times 0.169)$

$= 134.8\,mm.$

Area of steel.

$$\frac{M_u}{0.87 f_y z} = \frac{30.17 \times 10^6}{0.87 \times 415 \times 134.8}$$

$= 620\,mm^2.$

10TMT bar

$$A\phi = \frac{\pi}{4} \times 10^2 = 78.5\,mm^2$$

$$n = \frac{A_{st}}{A\phi} = \frac{620}{78.5} = 7.89$$

Spacing $s = \frac{1000}{n} = \frac{1000}{7.89} = 126\,mm$

max. spacing is the least of

i) $3d = 3 \times 145 = 435mm.$

ii) $300mm.$

$S_{max} = 300mm.$

10 TMT @ 120mm c/c.

$$A_{st} \text{ provided} = \frac{1000}{S} A\phi$$

$$= \frac{1000}{120} \times \frac{\pi}{4} \times 10^2 = 654mm^2.$$

$$\% \text{ of steel} = \frac{A_{st}}{bt} \times 100$$

$$= \frac{654}{1000 \times 145} \times 100 = 0.45\ \%$$

Step 6 Secondary steel.

$$M_u = 1.2 - \sqrt{1.44 - \frac{6.6 M_u}{f_{ck} b d^2}}$$

$$\frac{a}{d} = 0.08 < 0.48.$$

Hence. Ok.

lever arm $z = d\left(1 - 0.416 \times \frac{a}{d}\right)$

$= 318.3mm.$

Assume 10 TMT.

$A\phi = 78.5$

$$n = \frac{A_{st}}{A\phi} = 4.05.$$

$S = \frac{1000}{n} = 246mm$

$(d = 135mm)$

Smaa i) $3d = 3 \times 145$

$= 435 mm.$

ii) 450 mm.

Provide 10 TMT @ 220 mm c/c.

Ast provide $= \frac{1000}{S} A\phi$

$= \frac{1000}{220} \times 78.5.$

$= 356.9mm^2.$

$\%$ of steel $= \frac{356.9}{1000 \times 145} \times 100 = 0.24\%$

$\left(\frac{356.9}{1000 \times 135} \times 100\right)$
$= 0.26$

Check for deflection.

For simply supported case $\frac{l}{d} = 20.$

M25 $f_s = \cancel{0.58 f_y} \times 0.58 \times 415 \times \frac{620}{654}.$

$= 228.18 N/mm^2.$

Modification factor $= 1.5.$

$\left(\frac{l}{d}\right)_{max} = 20 \times 1.5 = 30$

$\left(\frac{l}{d}\right)$ provide $= \frac{4000}{145} = 27.6 < 30.$

Hence Ok.

Summary :-

Depth of slab = 170 mm.

C.C = 20 mm.

Primary 10TMT @ 120 mm c/c.

Secondary 10TMT @ 220mm c/c.