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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 : $1\frac{1}{2}$: 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 [HYS
3. Steel wire fabrics.
4. Structural steel.

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

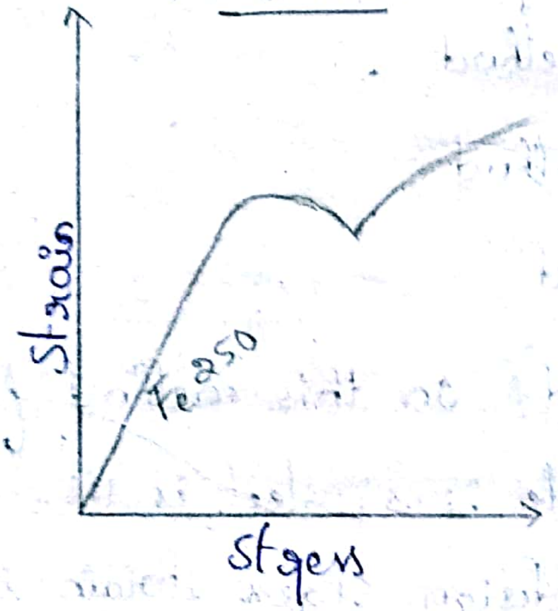
Fe → Ferrous

250 → yield strength (or) yield stress

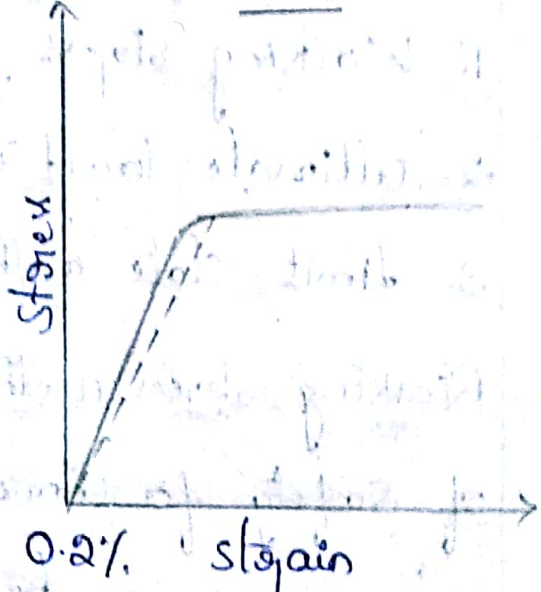
HYSD bars:

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/mm².

→ Unit wt. of steel is 78.5 KN/m³

Types of loads:

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

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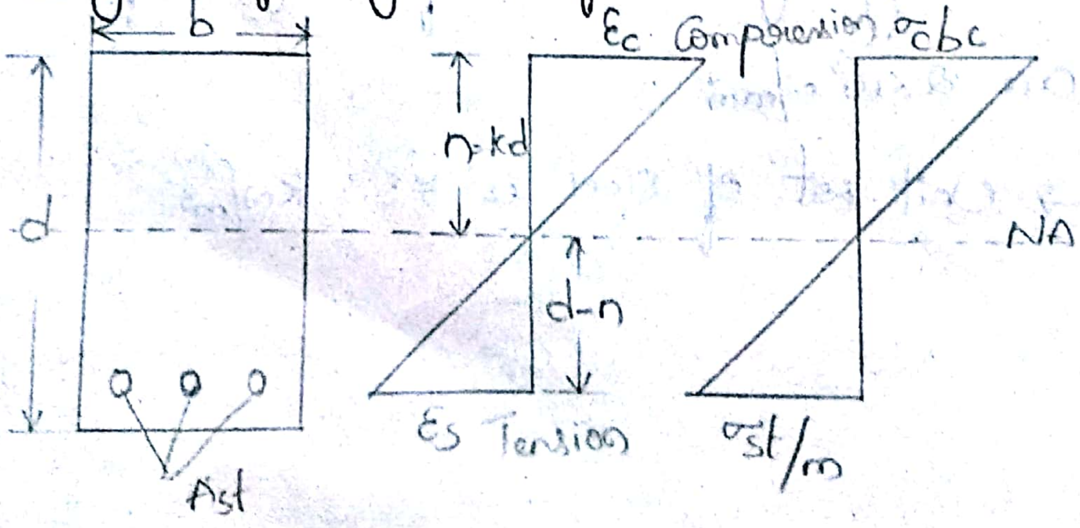
Methods of design of reinforced Concrete:

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

Working stress method: In this method facto 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,

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

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

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

$$\left[\because \varepsilon_s = \frac{\sigma_{st}}{E_s}, \quad \varepsilon_c = \frac{\sigma_{cbc}}{E_c}\right]$$

$$= \frac{\sigma_{st}}{\sigma_{cbc}} \cdot \frac{1}{m}$$

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

$$\Downarrow$$

modular ratio

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

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

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

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

$$n = k \cdot d$$

where $k = \dfrac{1}{1+\dfrac{\sigma_{st}}{m \cdot \sigma_{cbc}}}$

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 x area of beam in compression

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

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

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

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

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

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

~~$\Rightarrow \frac{bn^2}{2} = m \cdot A_{st}(d-n) = 0$~~

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

By solving the above quadratic equation,

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

Lever arm:

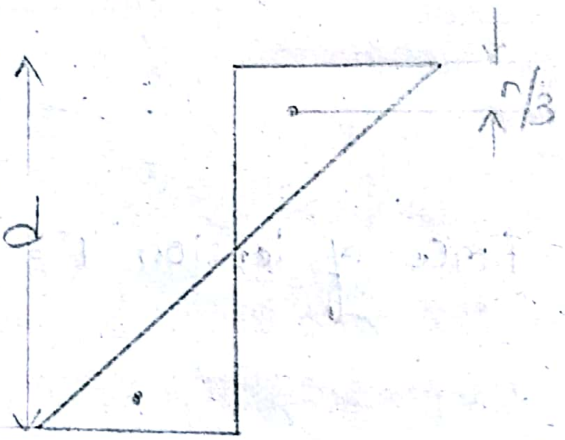
The forces of compression & tension form a couple. The distance between the lines of action of compression & tension forces is called as lever arm.

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

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

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

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



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

Moment of resistance:

Moment of resistance = Total compression or tension × 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) $= C \cdot z$

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

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

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

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

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 φ 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):-

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

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

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

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

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

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 50 mm

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

$= 1256.6mm^2$

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

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

Design Constant for M_{20} & Fe 415

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

Critical depth of neutral axis,

$$n_c = k \cdot d$$

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

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

$k = 0.289$

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

$= 0.289 \times 500$

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

Depth of actual neutral axis (n_a):

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

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

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

$n_a < n_c$ → under ~~a~~reinforced

$n_a > n_c$ → Over reinforced

$$\boxed{n_a > n_c}$$

∴ The section is over reinforced.

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

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

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

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3. A reinforced concrete beam 300mm × 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 excludi 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;

$n_c = k.d$

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

$K = 0.28$

$n_c = 0.28 \times 550$

$\boxed{n_c = 158.76mm}$

Depth of actual neutral axis (n_a):

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

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

$\boxed{n_a = 183.59mm}$

$n_a > n_c$

∴ The section is over reinforced section.

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

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

$\boxed{MR = 114.42 \text{ KN-m}}$

Maximum load:

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

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

MR = max.BM

114.42 = 3.125 w

w = 36.51 [Including self wt]

Self wt. of the beam = 0.3×0.6×1×25

= 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-m

σ_{st} = 230 N/mm^2

σ_{cbc} = 7 N/mm^2

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

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times7} = 13.33$$

$$k = \frac{1}{1+\frac{\sigma_{st}}{m.\sigma_{cbc}}} = \frac{1}{1+\frac{230}{13.33\times7}} = 0.289$$

$$\boxed{k = 0.289}$$

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

$$= 0.904$$

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

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

$$= \underline{0.914}$$

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

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

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

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

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

$d = 377.8 \approx 380mm$

$D = 380 + 40$

$= 420mm$ [∵ Assume 40mm]

Let,

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

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

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

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

$= 3.35 \approx 4 \text{ bars}$

∴ 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

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

$Q = 0.91$

$j = 0.90$

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

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

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

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

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

$$\boxed{k = 0.289}$$

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

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

$j = 0.904$

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

$= 0.914$

~~Depth required~~

~~MRu~~

<u>Loads:</u>

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

$W = 2.5 \text{ kN/m}$

Dead load $= 2.5$

Live load $= 20.00$

total load $W = 25 \text{ kN/m}$

Adopt an effective cover of 50mm

Effective depth $= 400 - 50$

$d = 350\text{mm}$

Bending moments & shear force

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

Depth required.

$MR = Qbd^2$

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

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

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 diametre

Compression reinforcement.

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

$n_c = 0.289 d$

$= 0.289 \times 350$

$= \cancel{99.05}\ 101.15$ 1.22×10^6 1.263×10^5

$$A_{sc} = \frac{13.33 \times 322 \left(350 - \overset{101.15}{\cancel{99.05}}\right)}{(1.5 \times 13.33 - 1)\left(\overset{101.15}{\cancel{99.05}} - 50\right)}$$ 51.15

$= 1096.3 mm^2$

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

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

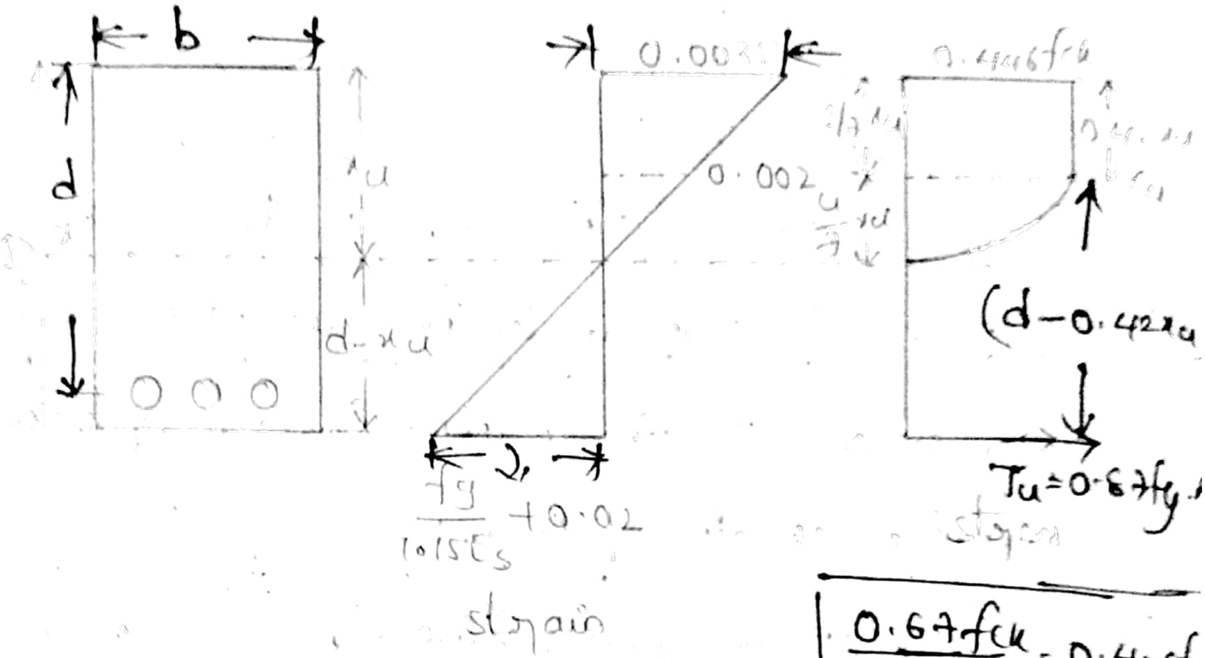
$1096.3 = \text{No. of bars} \times 201.06$

$\text{No. of bars} = 5.46 \approx 6$ bars

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

UNIT-II

Limit state Method



$$\boxed{\frac{0.67fck}{1.5} = 0.446fck}$$

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

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

Area of Concrete = parabolic + rectangle

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

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

$$+ \frac{3}{7} x_u \times 0.446 fck^{0.1911}$$

$$= 0.36 fck \cdot x_u$$

Distance of Centroid of stress block from the top fibre:

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

$$+\left(\frac{2}{3}\times\frac{4}{7}x_u\times0.446f_{ck}\right)\left(\frac{3}{8}\times\frac{4}{7}x_u+\frac{3}{7}x_u\right)$$

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

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

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

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

1) 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 Sections,

$$x_u < x_{umax}$$

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

Steel fails in this section

(iii) for over reinforced Section ($x_u > x_{u\max}$)

In this section Concrete fails.

So, the moment of resistance is,

$$\boxed{MR = 0.36 fck \cdot x_{u\max} \times b (d - 0.42 x_{u\max})}$$

x_u = Actual depth of Neutral axis,

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

⇒ From pg: 70 in Code book :

fy	$x_{u\max}/d$
250	0.53
415	0.48
500	0.46

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

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

for Fe 500 steel, $x_{u\max} = 0.46d$.

Limiting value of MR: (Mu,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]

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

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

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

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

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

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

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

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

$$= 0.133 f_{ck} bd^2$$

1. Find the moment Carrying a singly reinforced beam of 230mm × 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.

Moment of resistance:

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

$= 0.87 \times 415 \times 942.47 (480 - 0.42 (205.48$

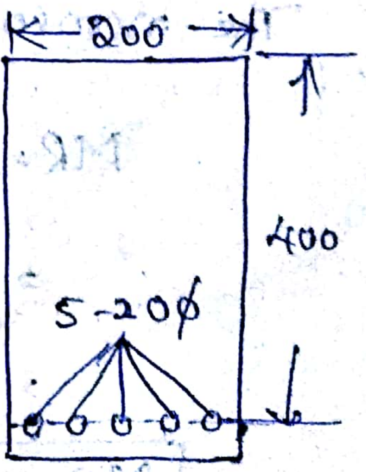
$\boxed{MR = 183.967 \times 10^6\ N/mm^2}$

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

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

2. Find the ultimate moment of resistance of Singly reinforced beam 200mm x400mm effective depth reinforced with 5 bars of 20mm ϕ. Concrete is of M_{20} grade & steel Fe 250.

Sol Given data,



$b = 200mm$

$d = 400mm$

$fck = 20 N/mm^2$

$fy = \cancel{415}\ 250\ N/mm^2$

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

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

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:

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

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

$$= 94.92 \times 10^6\ N\text{-}mm$$

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

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

To determine the Ast when dimensions of the beam & MR is given

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

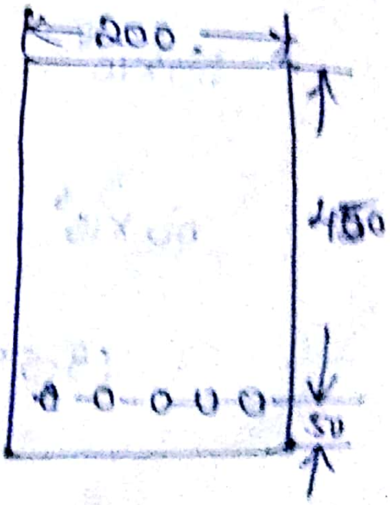
$$= 0.87 f_y . A_{st} \left(d - 0.42 \left(\frac{0.87 f_y . A_{st}}{0.36 f_{ck} . b}\right)\right)$$

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

3. 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 = 80kN-m

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

$M_u > M_u$ limit [Doubly reinforced beam]

$f_y = 250 \, N/mm^2$

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

$$M_{u,limit} = 0.148 f_{ck} b d^2$$

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

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

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

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

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

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

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

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

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

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

Provide 20mm ϕ for finding no. of bars

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

$940.2 = No. of bars \times 314.15$

$314.15 \times No. of bars = 940.2$

$No. of bars = 2.99 \approx 3$

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

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

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:

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

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

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

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

$= 0.48 \times 410$

$= 196.8mm$

$$\boxed{x_u max = 196.8mm}$$

$$\therefore \boxed{x_u > x_u max}$$

∴ So, the Section is over reinforced.

Moment of resistance: (MR)

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

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

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

$= 92.76 kN\text{-}m$

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

Safe load:

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

Equate $BM = M_u$

$2w = 92.76$

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

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

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

∴ 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, fck, fy, 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 (Mu = Mulim)

for Fe 415, $Mulim = 0.138 fck\,b d^2$

$$d = \sqrt{\frac{Mulimit}{0.138 fck \cdot b}}$$

5. Calculated area of reinforcement

$$Mu = 0.87 fy \cdot Ast \times d \left(1 - \frac{fy \cdot Ast}{fck \cdot b \times d}\right)$$

$Ast = ?$

$Ast_{min} < Ast < Ast_{max}$

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 $L = 4m \Rightarrow 4000mm$

$fck = 20 N/mm^2$

$fy = 415 N/mm^2$

$b = 300mm$

Super imposed load = 20kN/m.

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

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

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

$12d = 4000$

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

$d = 333.33$ mm ≃ 350mm

$\boxed{d = 350mm}$

(ii) Effective Cover assuming = 50mm

Overall depth = 50 + 350

$\boxed{D = 400mm}$

(ii) Effective Span:

It is the least of the following

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

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

$= 4300mm \Rightarrow 4.3m$

(ii) clear span + d = 4000 + 350

= 4350mm

= 4.35m

∴ Hence effective span $l = 4.3m$

(iii) Calculation of Loads & BM

Self wt = $0.3 \times 0.4 \times 1 \times 25$

= ~~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

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

= $0.138 f_{ck} bd^2$

= $0.138 \times 20 \times 300 \times$ [illegible]^2

<u>step 4:</u> check of depth required,

$M_u = M_{ulimit}$

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

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

$$\sqrt{\frac{79.74\times10^{6}}{0.138\times20\times300}}$$

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

Hence provided depth is adequate

17 Step 5: Area of reinforcement.

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

$$79.74\times10^{6} = 0.87\times415\times A_{st}\times350\left[1 - \frac{415 A_{st}}{20\times300\times350}\right]$$

$$79.46\times10^{6} = \cancel{12}\ 126.36\times10^{3} A_{st}\left[1 - 1.976\times10^{-4} A_{st}\right]$$

$$79.46\times10^{6} = \boxed{126.36\times10^{3} A_{st} - 24.97 A_{st}^{2}}$$

$$24.97 A_{st}^{2} - 126.36\times10^{3} A_{st} + 79.46\times10^{6} = 0$$

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

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

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

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

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

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

$= \boxed{3.65 \approx 4 \text{ bars}}$

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

Minimum reinforcement: (pg : 47)

$\Rightarrow \left(\dfrac{Ast_{min}}{bd} = \dfrac{0.85}{fy}\right)$

$Ast_{min} = \dfrac{0.85\,bd}{fy}$

$= \dfrac{0.85 \times 300 \times 3500}{415}$

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

Maximum reinforcement:

$Ast_{max} = 0.04 \times bD$

$= 0.04 \times 300 \times 400$

$= 4800\,mm^2$

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

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 contact 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 ~~Clear~~ span l = 4m ⇒ 4000mm

f_{ck} = 15N/mm²

f_y = 250N/mm²

b = 250mm

Dead load = 15kN/m

Live load = 12kN/m

(i) Assuming $\frac{l}{d} = \overset{10}{\cancel{9}}$ based on

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

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

Assuming effective cover = 50mm.

Overall depth = 50+400

= 450mm

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

(ii) Effective span l = 4000mm

(iii) Calculation of loads & BM

Self wt = 0.25×0.45×1×25

= 2.8125 kN/m

~~BM~~

Dead load = 15kN/m

Live load = 12kN/m

Total load = 15+12+2.8125

= 29.81kN/m

∴ Total load = 29.81 kN/m

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

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

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 d,

$$M_u = M_u \, limit$$

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

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

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

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

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

$d = 444.4mm \simeq 450mm$

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

Assuming effective Cover = 50mm

$D = 450 + 50$

$\Rightarrow 500mm$

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

$M_u < M_{u\,limit}$ – singly

$M_u > M_{u\,limit}$ – 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 = $15\ KN/m$

Live load = $12\ KN/m$

Total load = $3.125 + 15 + 12$

$= 30.125\ KN/m$

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

$$\boxed{BM = 60.25\ KN\,m}$$

Factored Bending moment,

$M_u = 1.5 \times 60.25$

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

Step 4: Depth required,

$M_u = M_u\,limit$

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

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

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

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

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,

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

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

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

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

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

$$= 1.072\%$$

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

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

$= 30$

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

$= 10 < 30$

∴ 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:

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

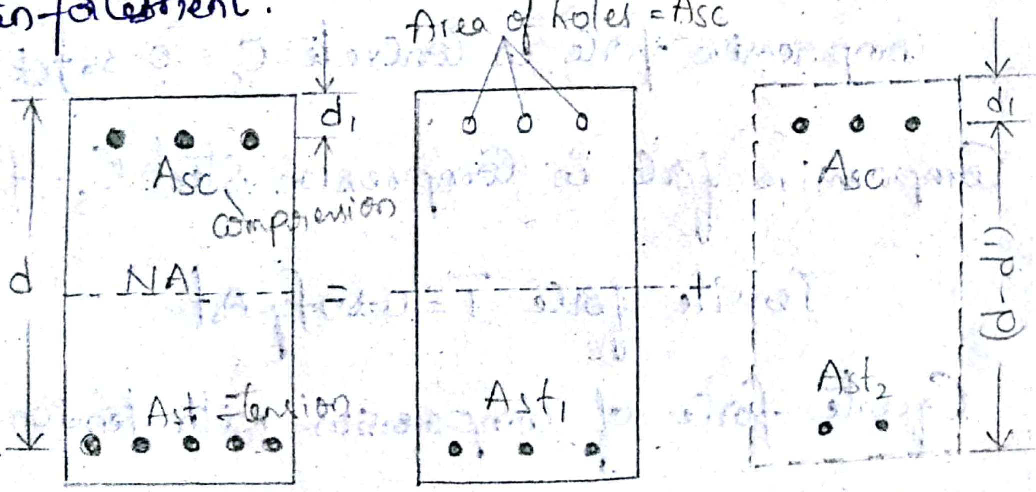
$$= 0.04 \times 250 \times 450$$

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

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 consid

to be composed of two sections as given b

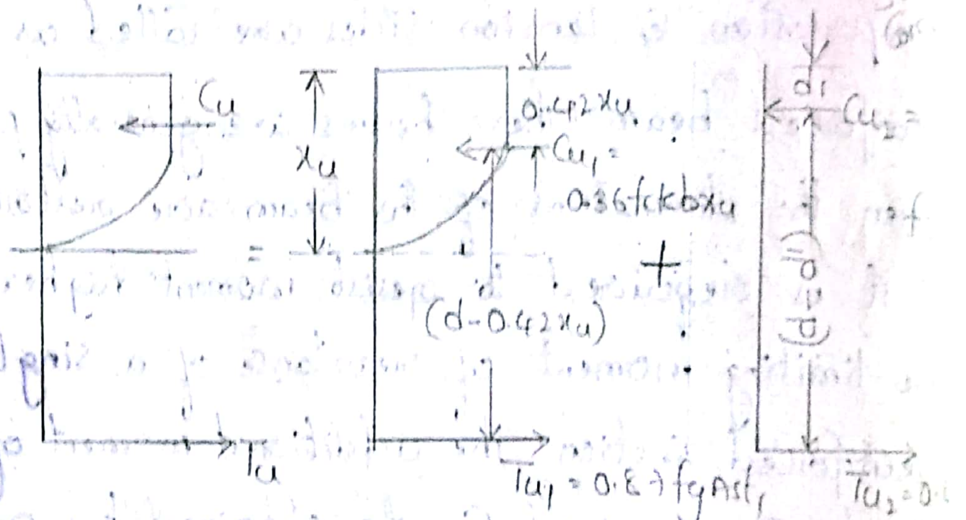
(a). A singly reinforced section with $M_{u,lim}$

(b) A section with compression steel & addit

tension steel to resist additional moment

$M_{u_2} = M_u - M_{u,lim}$ 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 $C_s = f_{sc} \cdot A_s$

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,

$$M_u = \overset{\text{concrete}}{M_{u_1}} + \overset{\text{steel}}{M_{u_2}}.$$

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

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 reinfo

Shall not exceed 0.04bD i.e., 4% of gross

area.

4. **Area of Tension steel**: The limiting mom

of resistance of singly reinforced section is

by,

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

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

Additional area of tensile steel (A_{st_2}) can

Calculated by equating the Compressive force in Compression steel & tensile force in additional tension steel.

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

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

A_{st_2} Can also be Calculated by using.

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

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

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 $0.87f_y$.

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,

$b = 300mm$

$d = 400 - 35$

$= 365mm$

$d' = 35mm$

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

$f_y = 415N/mm^2$

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

$A_{sc} = 2 \times \frac{\pi}{4}(18)^2 = 5089mm^2$

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 − ?

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

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

2. Depth of Neutral Axis:

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

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

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

Limiting depth of Neutral axis:-

$x_{umax} = 0.48d$

$= 0.48 \times 365$

$= 175.2 \text{mm}$

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

So, the section is under reinforced.

3. Moment of resistance:-

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

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

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

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

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

$= 167.62 \text{kN-mm}$

19/7/17

8. A rectangular beam is 200mm wide &

deep it is reinforced beam 6 bars of

In Compression with an effective Cover of

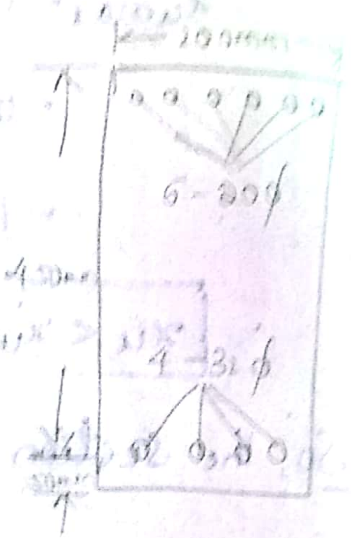
Determine the area of tension reinforcement

to make the beam section fully effective

than would be the moment of resistance

of Concrete & Fe 415 steel.

Ans



Given data,

$b = 200mm$

$d = 500 - 50$

$= 450mm$

$D = 500mm$

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

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

$f_y = 415 N/mm^2$

$d' = 50mm$

A_{st_1} is provided based on Mulim :-

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

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

$$= 111.78 \times 10^6 \text{ N-mm}$$

$$= 111.78 \text{ kN-m}$$

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

~~111.78~~

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

$$= 0.48 \times 450$$

$$= 216 mm$$

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

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

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

for f_{sc} value,

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

from table,

0.10 - 353

0.15 - 342

$$f_{sc} = 353 - \left(\frac{353 - 342}{0.15 - 0.10}\right)(0.11 - 0.10)$$

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

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

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

$$= 1831.43 mm^2$$

$$\boxed{A_{st_2} = 1831.43 mm^2}$$

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

$$= 861.71 + 1831.43$$

$$= 2693.14 mm^2.$$

$$\therefore \boxed{A_{st} = 2693.14 mm^2}$$

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

1804.24 no. of bars

$= 2693.1$

No. of bars $= 4$

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

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

$$x_u = 216.05 mm$$

$$\therefore x_u = x_{u max}$$

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

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

$$= 0.36 \times 20 \times 200 \times 216 (450 - 0.42 (216))$$
$$+ (350.8 \times 1684.95)(450 - 50)$$

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

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

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

2. Concrete grade = M20

3. Fy - 415 steel

4. Factored moment = 200 kN-m

5. Effective Cover = 50mm

Sol

Given data,

$b = 250$mm

$d = 500$mm

$D = 550$mm

$d' = 50$mm

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

$f_y = 415 N/mm^2$

$M_u = 200 kN-m$

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

$M_{ulim} = 0.138 f_{ck} bd^2$

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

$= 172.5 \times 10^6 N-mm$

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

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

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

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

$$A_{st_1} = \frac{0.36\ M_u lim}{0.87 f_y (d - 0.42 x_{umax})}$$

$x_{umax} = 0.48 d$

$= 0.48 \times 500$

$= 240 mm$

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

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

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

$= 200 - 172.5$

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

$Mu_2 = fsc \cdot Asc (d - d')$

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

from table 3.5

0.10 — 353

So, $fsc = 353 N/mm^2$

$Mu_2 = 353 \times Asc (500 - 50)$

$27.5 \times 10^6 = 158.85 \times 10^3 Asc$

$158.85 \times 10^3 Asc = 27.5 \times 10^6$

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

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

4. Additional tensile stress (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} = No. of bars $\times \frac{\pi}{4}(20)^2$

1366.06 = No. of bars × 314.15

314.15 No. of bars = 1366.06

No. of bars = 4.3 ≈ 5 bars

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

$= 1570.7 \, mm^2$

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

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

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

$173.11 = 113.09\ \text{No. of bars}$

No. of bars $= 1.53 \simeq 2$

∴ No. of bars = 2

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

$= 226.18\,\text{mm}^2$

∴ Provided $A_{sc} = 226.18\,\text{mm}^2$

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

τ_v = nominal shear stress

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

$\tau_{c\,max}$ = Max. shear stress [Table no: 20

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

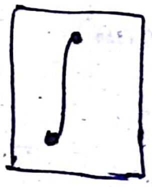
$\tau_v > \tau_c$ – provided shear reinforcement shear reinforcement has to be provided against diagonal tensile stresses caused by th shear force. The longitudinal bars do not pre the diagonal tension failure.

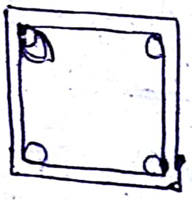
The inclined shear crack starts at th bottom near the support and extend toward compression zone. The shear reinforcement can provided in any of the following forms.

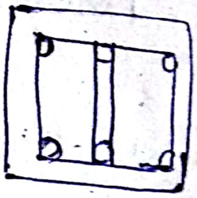
(a). Vertical stirrups.

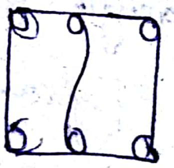
(b). Bent up bars along with stirrups.

(c) Inclined stirrups.


One legged stirrup


Two legged stirrup


Four legged stirrup


Three-leg 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 ~~stirr~~ stirrups

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

$$= \cancel{153}157.07\,mm^2$$

Spacing of stirrups:

1. ~~0.75 X (450~~ +. 0.75 X 450 = 337.5 mm

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

3. For vertical stirrups:

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

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

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

$$S_v = 168.56 \times 10^3\ m$$

$$\boxed{S_v = 153.56\,mm} \simeq 165mm$$

from code book pg: 48.

26.5.1.6 – min shear reinforcement

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

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

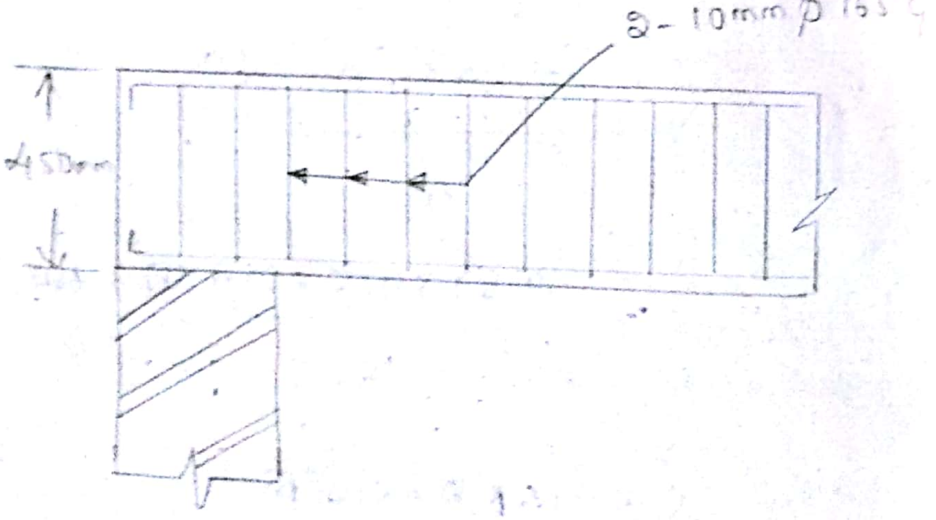
$$S_v = \frac{0.87 f_y \cdot A_{sv}}{0.4 \times b}$$

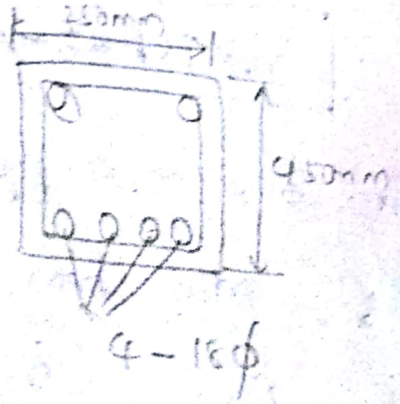
$$= \frac{0.87 \times 415 \times 157.07}{0.4 \times 250}$$

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

Provide 10mm ϕ of 2-legged stirrups with a Spacin

of 165mm





24/7/17

11. A S.S R.C.C beam of 200mm x 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

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

$f_y = 415 N/mm^2$

D = 400+50

= 450mm

Self wt = 25 x 0.45 x 0.2

= 2.25 kN/m

Total load = 2.25+10

= 12.25 kN/m

Factored load W_u = 12.25 x 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 → 0.75

2.00 → 0.79

1.9 → ?

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

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

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

- 0.024

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

Shear reinforcement has to be designed

(i) Design of shear reinforcement:

$V_{uc} = \tau_c . bd \Rightarrow$ ~~0.726~~

$= 0.774 \times 200 \times 400$

$=$ ~~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. $0.75d \Rightarrow 0.75 \times 400$

$$= 300.0mm$$

2. 300mm which ever is less

3. For vertical stirrups,

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

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

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

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

$$= 1691.57mm \approx 1700mm$$

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 stirrups:

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

$= 300.0mm$.

2. 300mm which ever is less

3. For vertical stirrups,

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

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

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

$= 1691.57 \times 10^3 m$

$= 1691.57mm \approx 1700mm$

from Code book pg: 48

26.5.1.6 – Minimum shear reinforcement.

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

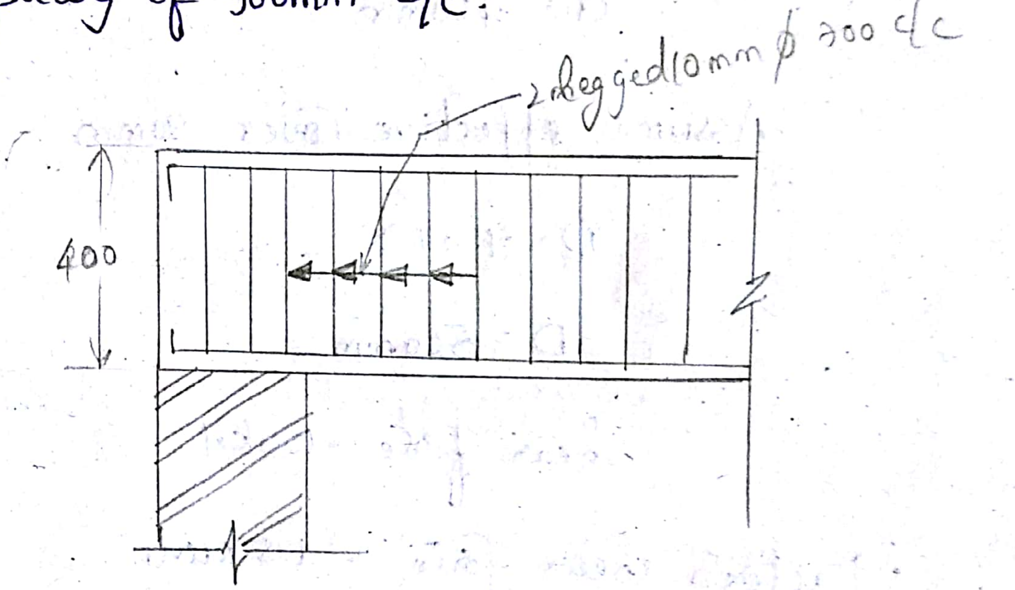
$$S_v = \frac{0.87 fy . Asv}{0.4 \times b}$$

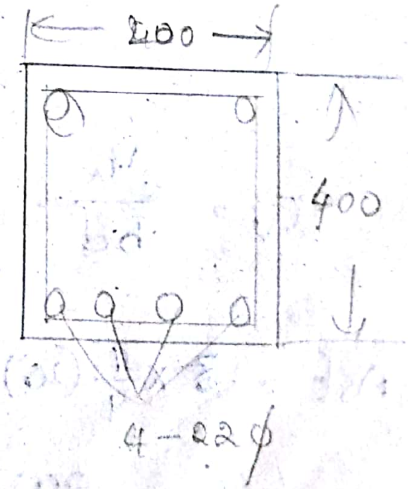
$$= \frac{0.87 \times 415 \times 157.07}{0.4 \times 200}$$

$$= 708.88mm \approx 700mm$$

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

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 s 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 = 150 kN

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

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

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

= 0.698%.

$0.50 \rightarrow 0.48$ $\Big\}$ from table no: 19

$0.75 \rightarrow 0.56$

$0.698 \rightarrow ?$

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

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

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

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

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

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87f_y}$$

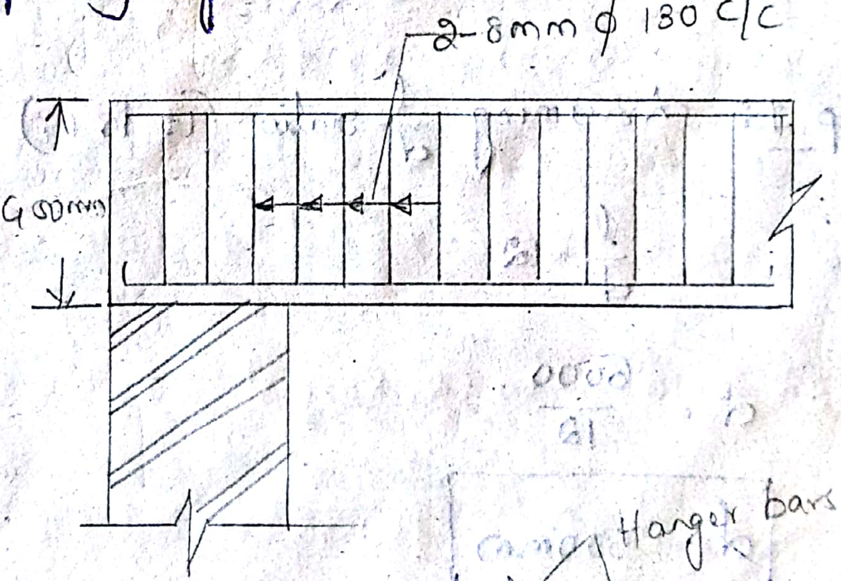
$$\frac{A_{sv}}{b \times s_v} = \frac{0.4}{0.87f_y}$$

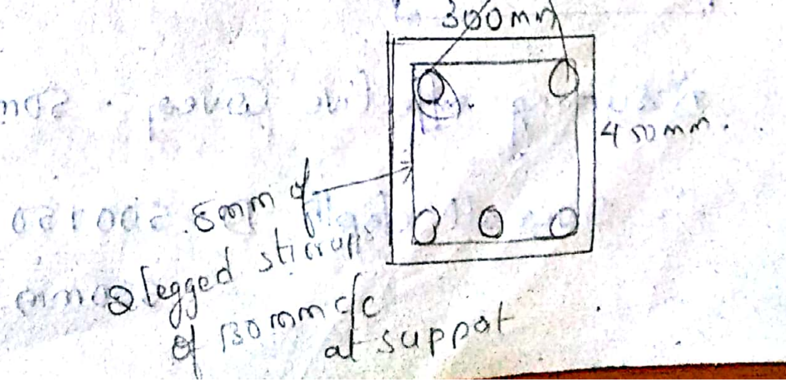
$$S_v = \frac{0.87f_y . A_{sv}}{0.4 \times b}$$

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

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

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, 6mm 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 $= 15kN/m$

$f_y = 415 N/mm^2$

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

$l = 6m$ (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

$$\text{Self wt} = 0.23 \times 0.55 \times 1 \times 25$$

$$= 3.1625 kN/m$$

Imposed load = 15 kN/m

$$\text{Total load} = 15 + 3.1625$$

$$= 18.1625 kN/m$$

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

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

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 fe415 steel, $M_{u limit} = 0.138 f_{ck} bd^2$

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

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

Hence provided depth is adequate.

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

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

$= 198.37 kN\text{-}m$

$M_u < M_{ulim}$

∴ So, the section is singly reinforced.

Step 5:- Area of reinforcement,

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

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

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

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

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

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

Provide 16mm φ for finding no. of bars

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

201.06

$763.21 = 113.09$ No. of bars

201.06

113.09 No. of bars $= 763.21$

18

16

20

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

∴ No. of bars $= 4$

Ast required $= 763.21\ mm^2$

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

$= 804.24\ mm^2$

Min. reinforcement:

$$\rightarrow \frac{Ast\,min}{bd} = \frac{0.85}{fy}$$

$$Ast\,min = \frac{0.85bd}{fy}$$

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

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

Max. reinforcement:

$Ast\,max = 0.04 \times bD$

$= 0.04 \times 230 \times 550$

$= 5060$

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

~~step 05:- check for deflection (stiffness)~~

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

& $\tau_v < \tau_{c\,max}$

$\tau_v < \tau_c$ – min. reinforcement

$\tau_v > \tau_c$ & $\tau_v < \tau_{c\,max}$ – ~~degig~~ design shear reinforcement

$\tau_v > \tau_c$ & $\tau_v > \tau_{c\,max}$ – redesign t beam.

Step 6:- Check for shear reinforcement

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

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

$$\boxed{\tau_{c\,max} = 3.1 N/mm^2}$$

% 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</u> of <u>shear</u> <u>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

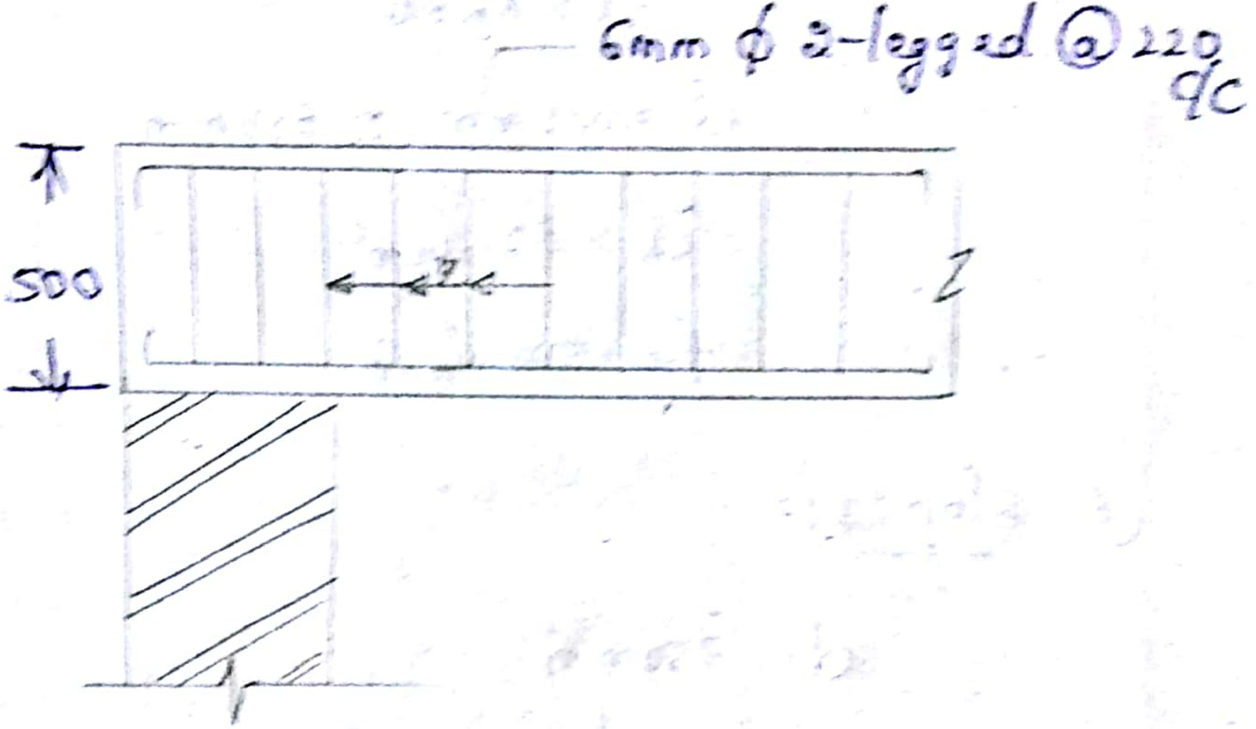
Max. permitted $\frac{l}{d}$ ratio = $\overset{1.2}{1.25} \times 20$

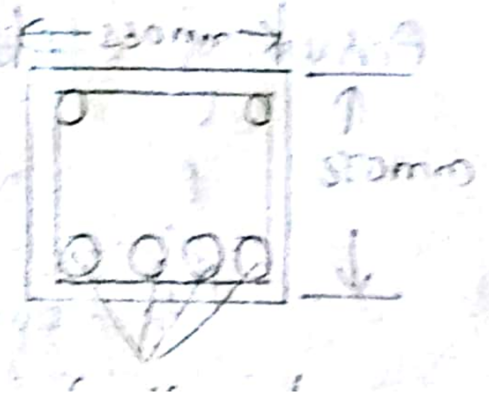
= 24

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

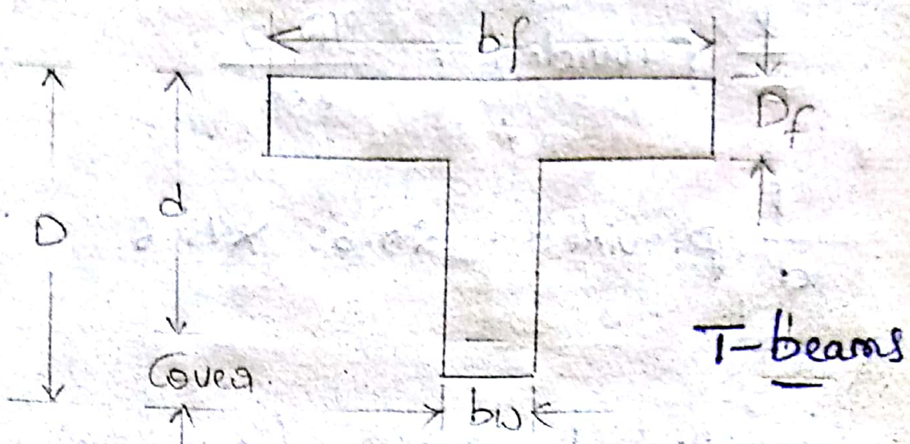
$\frac{l}{d}$ provided = 12 < 24

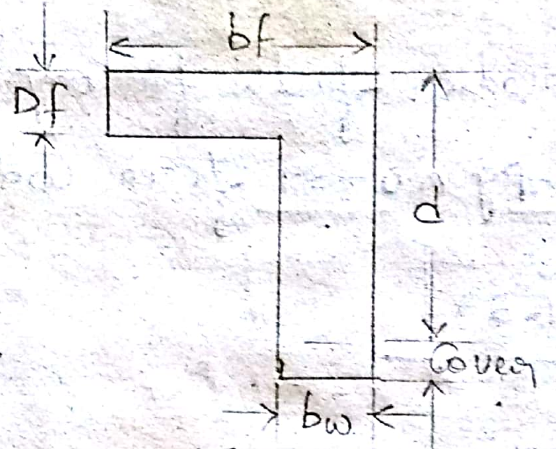
∴ Hence deflection control is safe.





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.

→ 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 $b_f = \dfrac{l_0}{\left(\dfrac{l_0}{b} + 4\right)} + b_w$

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

where,

b_f = effective width of flange

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

b_w = breadth of web.

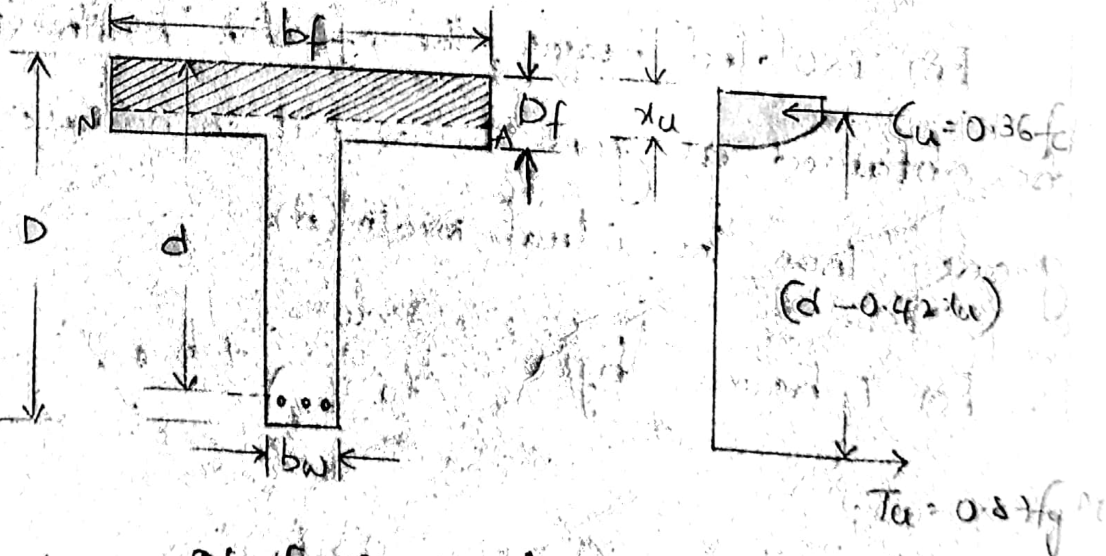
D_f = thickness of flange..

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 ($x_u \le D_f$)



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 = ~~6~~ 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

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

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

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

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

$= 264\text{ mm}$

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

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-mm

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

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

sd

Given data,

$b_w = 300$mm

$b_f = 1650$mm

$D_f = 120mm$

$d = 510mm$

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

$f_y = 415 N/mm^2$

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

Step 1:-

Assuming depth of N.A is in the flange

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

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

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

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

$\Rightarrow 244.8mm$

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

So, the beam is under reinforced section.

Moment of resistance:

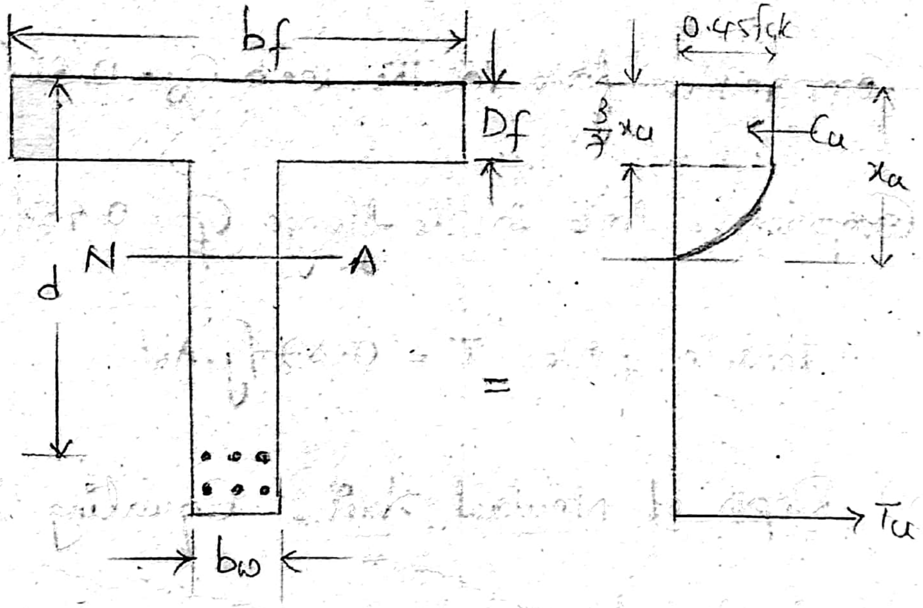
$$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)$$

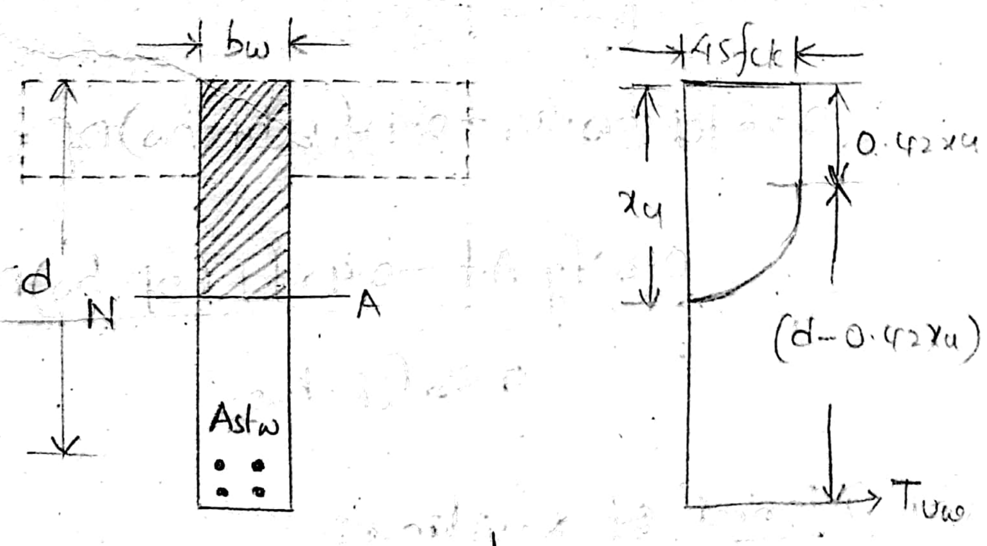
$$= 0.87 \times 415 \times 1963.49 (510 - (0.42 \times 59.67))$$

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

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

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





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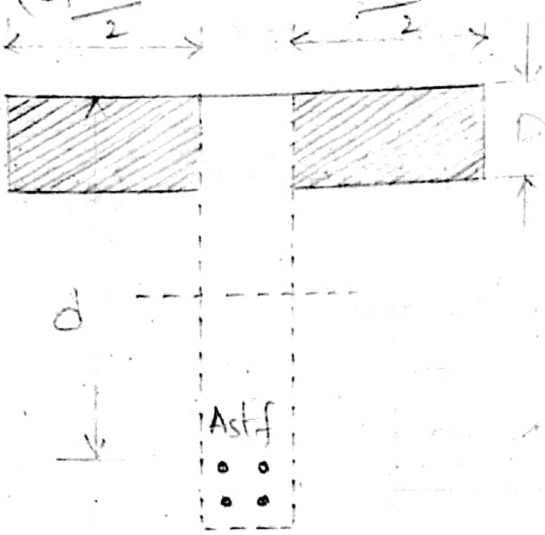
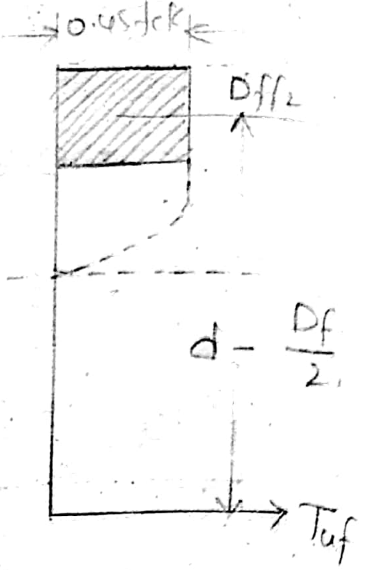




Neutral Axis is below the flange & $\frac{D_f}{d} \le 2$

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

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

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

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

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

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

(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 = Ast

characteristic strength of concrete = 20N/mm^2 = fck

characteristic strength of steel = 415N/mm^2 = fy

Step 1: Assuming x_u is with in the flange:

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

$$= \frac{0.87 \times 415 \times 2000}{0.36 \times 20 \times 800}$$

$x_u = 156.7mm > Df$

∴ Our assumption is wrong. So, the N.A lies in the web.

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

So, it is Case - 2

Step 2:

Actual depth of N.A

$$x_u = \frac{0.87 fy . Ast - 0.45 fck (bf - bw) Df}{0.36 \times fck \times bw}$$

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

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

495000

$x_u max = 0.48d$

$= 0.48 \times 600$

$= 288$

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

∴ The section is under reinforced section.

3. step : Moment of resistance:

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

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

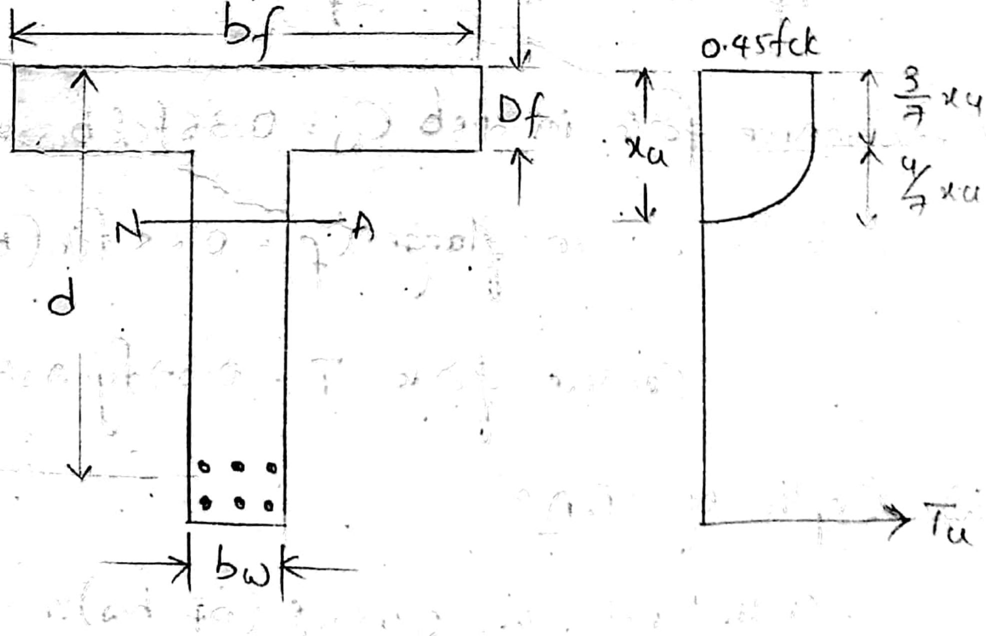
$$+ 0.45 \times 20 (800 - 300) 110 \times \left(\overset{545}{600 - \frac{110}{2}}\right)$$

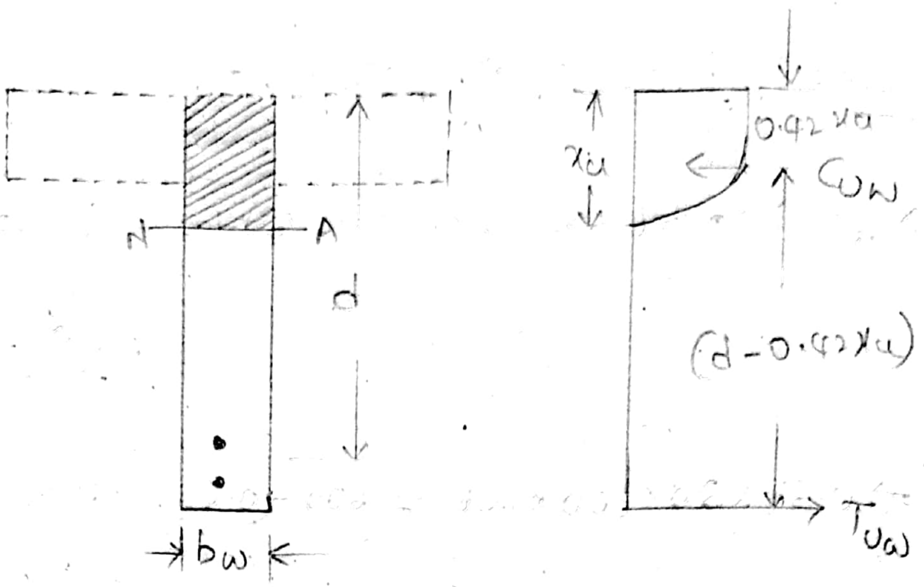
$$\Rightarrow 212.27 \times 10^6 + 269.7 \times 10^6$$

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

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

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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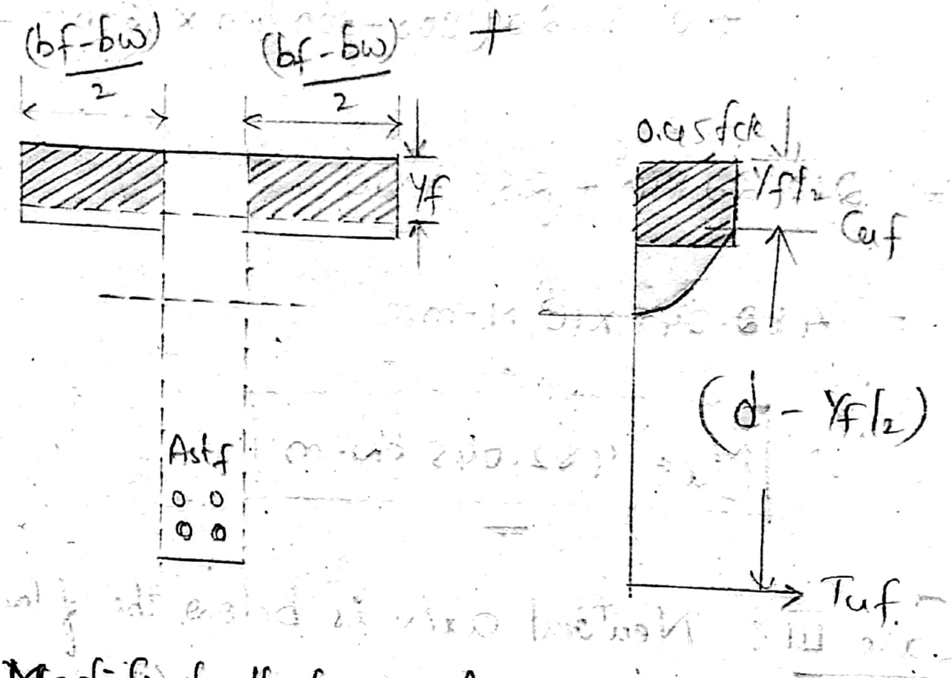


Modified thickness of flange = y_f

$y_f = 0.15 x_u + 0.65 D_f$, but nt 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:

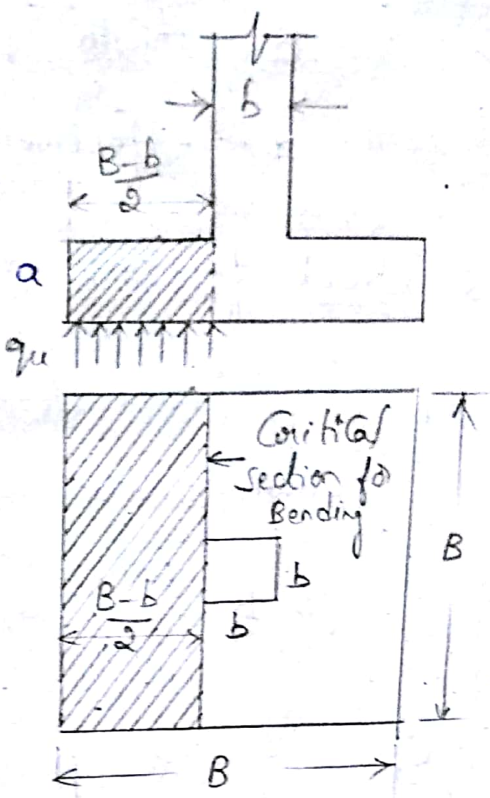
$$q_u = \frac{P_u}{A}$$

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

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

Projection of the footing

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

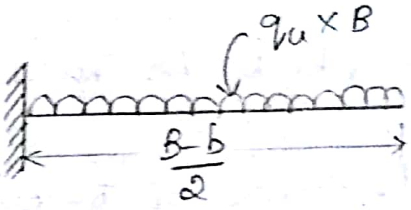


The B.M about x–x is (as a Cantilever slab, $\frac{wl^2}{2}$).

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

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

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,

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

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

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

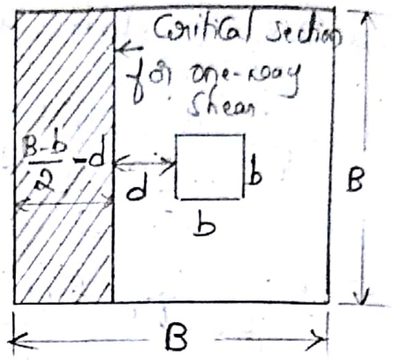
where, a_{st} = area of bar used

A_{st} = total area of steel required.

B = width of the footing

d = effective depth.

5. check for one way shear:

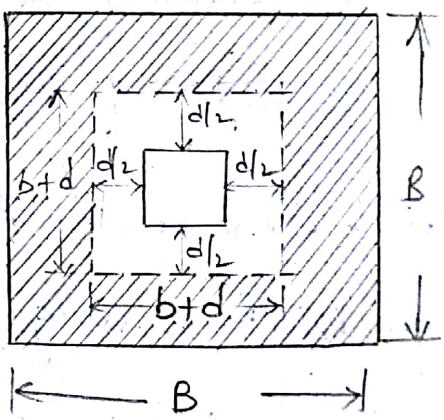


V_u = Soil pressure from the shaded area.

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

$\tau_v = \frac{V_u}{Bd} < \tau_c$, 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 x 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.

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

Permissible value of punching shear stress is,

$$\tau_p = 0.25\sqrt{fck}$$

* **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/m²

if n't " - 3 kN/m²

Dead load:

Thickness of waist slab = D

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

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

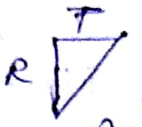
$$= D\sqrt{1+\left(\frac{R}{T}\right)^2} \times 25$$

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

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

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

$\frac{1}{2} bh$



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.

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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,

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

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

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

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 perpendicu to the span direction at 0.12% (for HYSD bars) of gross c/s area & find the spacing of these bars. If 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 & tread of each step are 150mm & 225mm respectively. The stair hall is 2.5m × 4.5m. Live load may be taken as 3kN/m². Use M20 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

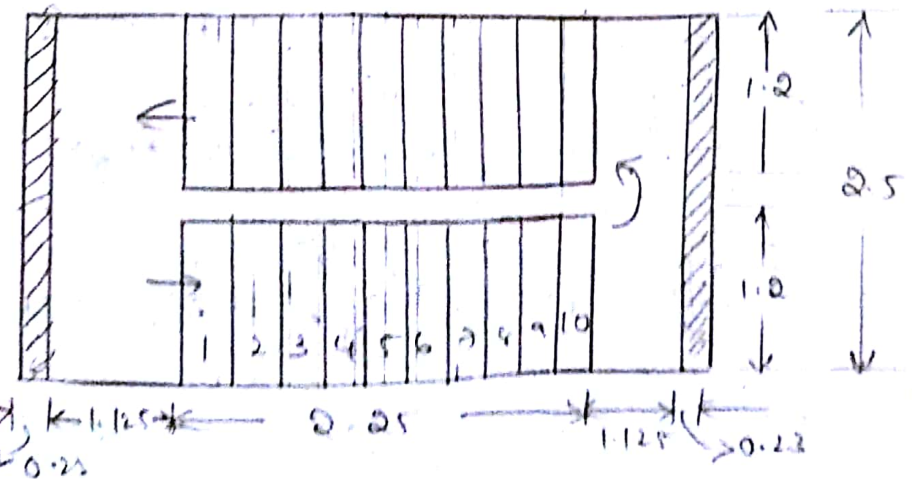
Hence, no. of treads = 11-1 ⇒ 10

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.

$= 4.5 + 0.23$

$$\boxed{l = 4.73m}$$

3. **Thickness of slab:**

Assume effective depth,

$d = \frac{\text{Span}}{25} \Rightarrow \frac{4730}{25} = 189.2mm$

$d = 190mm$

$D = 220mm.$

4. **Loads:** Loads 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 $= \frac{\left(\frac{1}{2}R\cancel{T}\right)}{\cancel{T}} \times 25 \Rightarrow R \times \frac{25}{2}$

$\Rightarrow 0.15 \times \frac{25}{2}$

$\Rightarrow 1.875 \text{ kN/m}^2$

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

$= 18.15 \text{ kN/m}^2$

5. Factored BM:

$$M_u = \frac{w_u l^2}{8} \Rightarrow \frac{18.15 \times (4.73)^2}{8}$$

$M_u = 50.76 \text{ kN-m}$

$= 50.76 \times 10^6 \text{ N-mm}$

6. Min. depth required:

$M_u = 0.138 f_{ck} b d^2$

$50.76 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$$d = \sqrt{\frac{50.76 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d = 135.6 \text{mm} < 190 \text{mm}$, provided depth

Hence provided depth is adequate

7. Tension reinforcement:

$$Mu = 0.87 fy . Ast . d \left[1 - \frac{fy . Ast}{fck\ bd} \right]$$

$$50.76 \times 10^6 = 0.87 \times 415 \times Ast \times 190 \left[1 - \frac{415 Ast}{20 \times 1000 \times 190} \right]$$

$$50.76 \times 10^6 = 68.6 \times 10^3 Ast - 7.49 Ast^2$$

$$7.49 Ast^2 - 68.6 \times 10^3 Ast + 50.76 \times 10^6 = 0$$

$$\boxed{Ast = 812 mm^2}$$

using 12mm ϕ bars, spacing of bars,

$$S = \frac{ast}{Ast} \times 1000$$

$$\Rightarrow \frac{\pi}{4} \times 12^2 \times \frac{1000}{812}$$

$$= 139.3 \text{ mm}$$

Provide 12mm bars @ 130mm c/c.

8. Distribution reinforcement:

$$Ast = 0.12\% \text{ of gross area}$$

$$= 0.12 \times 1000 \times \frac{220}{100}$$

$$\boxed{Ast = 264 mm^2}$$

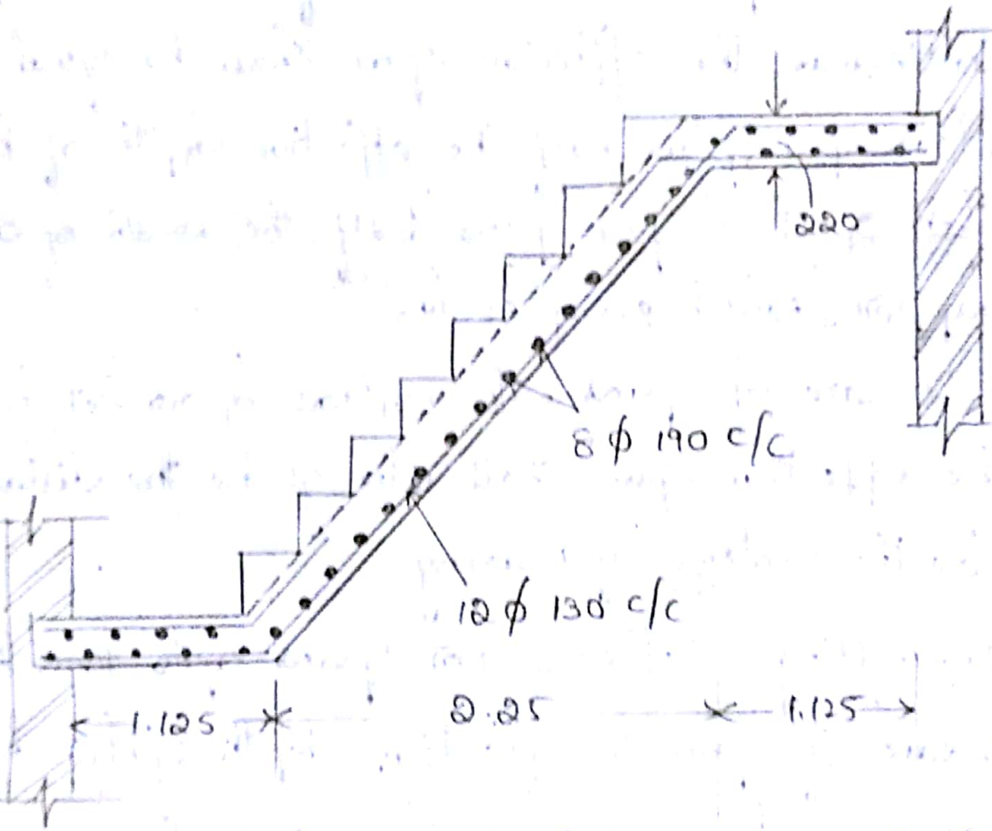
using 8mm bars, spacing

$$S = \frac{\pi}{4} \times 8^2 \times \frac{1000}{264}$$

$$= 190.4 mm$$

Hence, provide 8mm bars @ 190 mm c/c

The details of reinforcement are shown in fig 4.21



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Continuous Beams :-

1. Effective span : If the width (or) Bearing of the Support 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 shalle 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 M_{20} 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/m^2

Imposed load (fixed) = 7.5 kN/m^2 (excluding self wt).

Partial fixity may be expected at the discontinuous edge.

Given data,

$fck = 20N/mm^2$

$fy = 415N/mm^2$ $b = 300mm$

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 $+ \frac{b}{2} + \frac{b}{2}$

$= 3.6 + \frac{0.\cancel{2}3}{2} + \frac{0.\cancel{2}3}{2}$

$=$ ~~3.83m~~ 3.9m

Effective span $l =$ ~~3.9m~~ 3.85m

3. **Loads:-**

Self wt. of the beam $=$ ~~0.3×0×25~~ $0.3 \times 0.3 \times 1 \times 25$

$= 2.25\ kN/m^2$

Imposed load, fixed $= 7.5\ kN/m^2$

Total load fixed $= 9.75\ kN/m^2$

Imposed load, not fixed $= 5\ kN/m^2$

Factored loads:

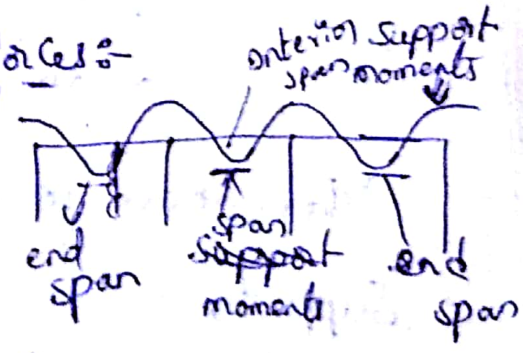
factored fixed load $w_{ud} = 1.5 \times 9.75$

$= 14.63\ kN/m$

not fixed $w_{ul} = 1.5 \times 5$

$= 7.5\ kN/m$

4. **Bending moments & shear forces:-**



Span moments: (At the middle of end span).

$M_u = \left(\frac{1}{12}\right) \times 14.63 \times 3.85^2 + \left(\frac{1}{10}\right) \times 7.5 \times 3.85^2$

$= 18.07 + 11.12$

$= 29.2$ kN-m

At the middle of interior Span.

$M_u = \left(\frac{1}{16}\right) \times 14.63 \times 3.85^2 + \left(\frac{1}{12}\right) \times 7.5 \times 3.85^2$

$= 13.55 + 9.26$

$= 22.81$ kN-m

Support moments :- (At support next to the end Support).

~~Mu =~~

$M_u = \left(\frac{1}{10}\right)\{14.63 \times 3.85^2 + \left(\frac{1}{9}\right) \times 7.5 \times 3.85^2$

$= 21.68 + 12.35$

$= 34.03$ kN-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-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 $= 4 \times \frac{\pi}{4} (12)^2$

$= 452.4 mm^2$

6. Reinforcement at mid spans:-

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

$$29.2 \times 10^6 = 0.87 \times 415 \times Ast \times 250 \left[1 - \frac{415 Ast}{20 \times 300 \times 250}\right]$$

$$29.2 \times 10^6 = 90.26 \times 10^3 - 24.97 Ast^2$$

$$24.97 Ast^2 - 90.26 \times 10^3 + 29.2 \times 10^6 = 0$$

$$\boxed{Ast = 359.2 mm^2}$$

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 $= 3 \times \frac{\pi}{4} (12)^2$

$= 339.3 mm^2$

7. Design of shear reinforcement:

$$\tau_v = \frac{V_u}{bd} = \frac{63.27 \times 10^3}{300 \times 250} = 0.84 N/mm^2$$

percentage of steel $P_t = \frac{Ast}{bd} \times 100$

$= \frac{628.3 \times 100}{300 \times 200} = 1.04\%$

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 $V_{uc} = \tau_c . bd$

$= 0.67 \times 300 \times 200$

$= 40200 N$

$= 40.2 kN$

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

$$A_{sv} = 2 \times \frac{\pi}{4}(6)^2 \Rightarrow 56.55 mm^2$$

$$\text{Spacing } S_v = \frac{0.87 f_y . A_{sv} . d}{V_{us}} \Rightarrow \frac{0.87 \times 415 \times 56.55 \times 250}{8950}$$

$$= 456.25 mm$$

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)

$$\frac{l}{d} = 26 \text{ (for Continuous)}$$

$$\text{\% of steel @ mid span} = \frac{603.18 \times 100}{300 \times 200} = 1.00$$

$$f_s = 0.58 \times 415 \times \frac{456.68}{603.18} \Rightarrow 181.99 N/mm^2$$

from fig. 4 of IS : 456

modification factor = 1.3

$$\text{max. permitted } \frac{l}{d} \text{ ratio} = 1.3 \times 26$$

$$= 33.8$$

$$\frac{l}{d} \text{ provided} = \frac{3800}{200}$$

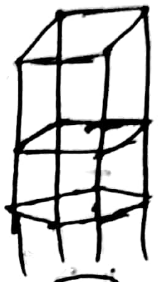
$$= 19 < 33.8$$

∴ 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.

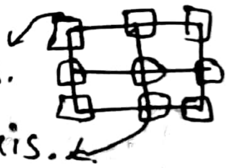
Based on the loading.

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. → $\frac{l}{b} > 12.$

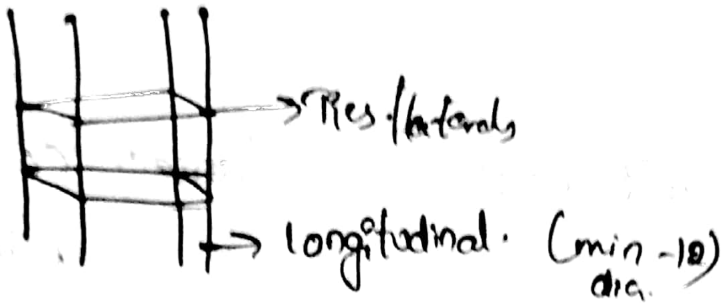
Colomns

Form

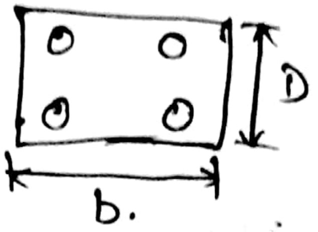
1) Circular

2) Rectangular

3) Square.



$P = \frac{Ast}{bD} \times 100$



D = gross area. (Total)

$\boxed{l_c < 3b}$

25.12 it is coloum : 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 circula - min 6 bars

Oler - min 4 bars.

PgNO-49 :- c) Pitch & diameter of the lateral ties.
↓
spacing

Smax i) b
ii) 16 ϕ times the small diameter of long st
iii) 300mm.

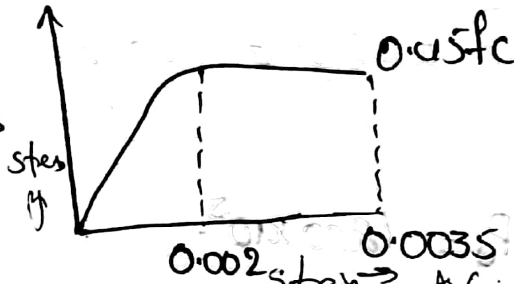
which ever is less.

2) diameter:-

$d = \frac{1}{4} \times \phi \rightarrow$ largest longitudinal bar dia.
$< 16mm.$

PgNO-70

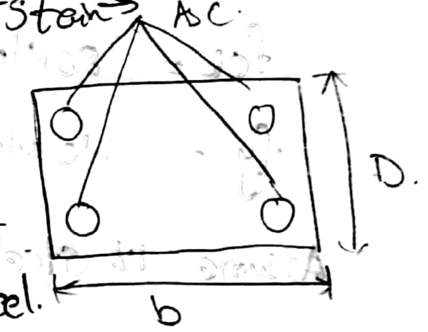
axial compression is 0.002.



C/s of the coloomn.

$A_c = (bD)$ ~~Asc~~ $- A_{sc}$.

P_u = load carried by concrete + load carried by steel.

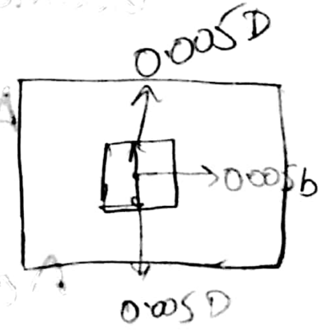


$= A_c(0.45f_{ck}) + A_{sc}(0.75f_y)$

$= 0.9(0.45f_{ck}A_c + 0.75f_yA_{sc})$

$= 0.405f_{ck}A_g + 0.675f_yA_{sc}$.

$\boxed{P_u = 0.4f_{ck}A_c + 0.67f_yA_{sc}}$



$P_u = 0.4 f_{ck} \cancel{*} A_c + 0.67 f_y A_{sc}$.

$A_{sc} = 4 \times \frac{\pi}{4} \times 12^2 = 452.38$

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$.

$A_c = (250 \times 300) - 452.38\cancel{x}$

$= 74547.62$.

$P_u = 0.4 \times 20 \times 74547.62 + 0.67 \times 415 \times 452.8$

$= 596\ kN + 125\ kN$

$= 722.165\ kN$.

Design a Square coloumn for the design load of 1200 ~~kN~~ Use M_{20} 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.

$= 0.01 \times A_c$.

$1200 \times 10^3 = 0.4 \times 20 \times A_c + 0.67 \times 415 \times 0.01 A_c$

$$A_c = \frac{1200 \times 10^3}{10.78} = 111312\ mm^2$$

$A_c = 111312 mm^2$.

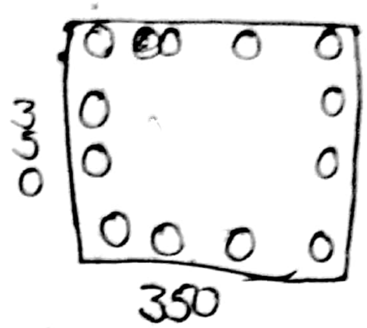
$b = \sqrt{111312}$ for square.

$= 333.6$

∴ Provide 350 x 350 mm.

$$A_{sc} = 0.01 \times (350 \times 350)^2$$

$$= 1225 \, mm^2$$

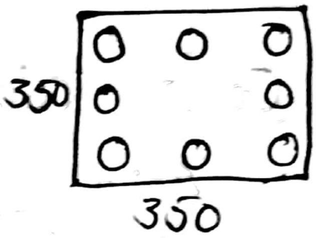


12 ϕ - 10.83.

16 ϕ = 6.09

Provide 12 - 12 ϕ as longitodinal reinforcement

Provide 8 - 16 ϕ 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 6 mm ϕ (ms) bar.

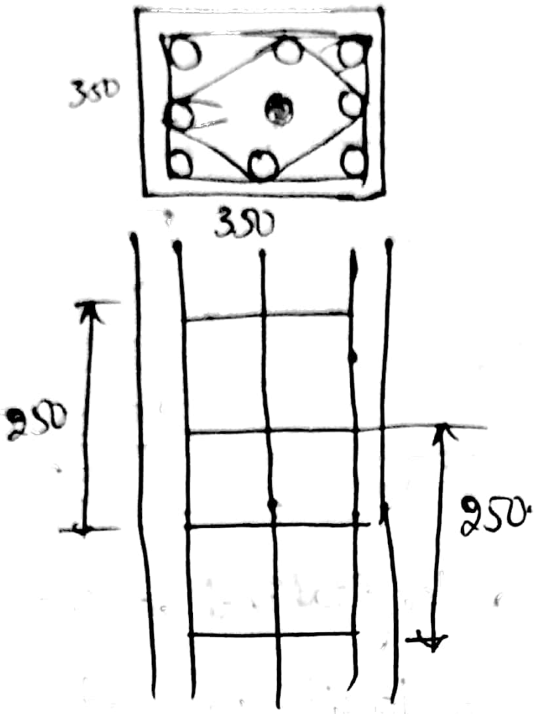
Spacing. least of

350 mm

16 ϕ = 16 x 16 = 256 mm

300 mm.

Provide 6 mm ϕ @ 250 mm c/c.



Center Coloumns :- P_u.. 

Axial :- P_u, M_u 

Biaxial :- P_u, M_x, M_y 

Design ~~an~~ short axially loaded tied coloumn pinned at both ends with an unsupporte length of 3.5 mts. for carrying a characteristic load of 1500 kN. Use M20, Fe 415 steel.

Given data;

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

$P = 1500$ kN.

$f_y = 415 \text{ N/mm}^2$.

$l = 3500$ mm

Pinned - pinned.

Design load $P_u = 1.5 \times 1500 = 2250$ kN.

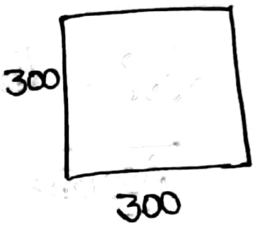
Eff length $l_e = l = 3500$ mm.

For short Coloum $\frac{l_e}{D} = 12.$

$$\frac{3500}{12} = D.$$

$$D = 291.67mm.$$

Assume Col. section 300×300 mm².



$$e_{min} = \frac{l}{500} + \frac{D}{30}.$$

$$= \frac{3500}{500} + \frac{300}{30}$$

$$= 17 < 20mm.$$

$$\frac{l}{D} = \frac{3500}{300}$$

$$= 11.67 < 12.$$

$A_g = 300 \times 300 = 90000 \text{ mm}^2.$

Area of steel $A_{sc} = ?$

$$A_c = A_g - A_{sc} = 90000 - A_{sc}.$$

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}.$

$2250 \times 10^3 = 0.4 \times 20 \times (90000 - A_{sc}) + 0.67$

$2250 \times 10^3 = 720000 - 8A_{sc} + 278.05 A_{sc}.$

$= 720000 - 270.05 A_{sc}.$

$A_{sc} = 5665.61 mm^2$

$$\% \text{ steel} = \frac{A_{sc}}{bd} \times 100 = \frac{5665.61}{300 \times 300} \times 100$$

$$= 6.29\%$$

$A_{sc} = 5665.61\,mm^2.$

25 Φ.

$$A\,n = \frac{5665.61}{490.8}$$

$= 11.54$ bar $\simeq 12$ bas.

12/9/015

Revise the section

Assume 375×375mm

140

~~140625~~.

$$\frac{le}{D} = \frac{3500}{375} = 9.33 < 12$$

$A_g = 375 \times 375 = 140625\,mm^2$

$A_c = A_g - A_{sc} = 140625 - A_{sc}.$

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

$2250 \times 10^3 = 0.4 \times \overset{20}{\cancel{40}} \times ((140625 - A_{sc}) + 0.67 \times 415 \times A_{sc}.$

$A_{sc} = 4165.89\,mm^2.$

$$\% \text{ of steel } p = \frac{A_{sc}}{bd} \times 100$$

$$= \frac{4165.89}{375 \times 375} \times 100 = 2.96\%$$

$A_{sc} = 4165.89\,mm^2.$

Ascume 25Φ $A\phi = \frac{\pi}{4} \times 25^2 = 49087 \text{ mm}^2$

$n = \frac{416589}{490.87}$

$= 8.48$

Provide $10 \times 25\Phi$

% steel provided $= \frac{10 \times \frac{\pi}{4} \times 25^2}{375 \times 375} \times 100 = 3.49\%$

Lateral ties :

$d = \frac{\phi}{4} = \frac{25}{4} = 6.25 \text{mm}.$

Provide 8mm Φ.

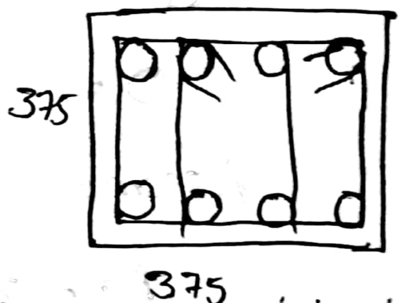
Spacing:

i– 375mm.

ii) $16\phi = 16 \times 25 = 400$mm

iii) 300mm.

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.

$P = 1000$kN.

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

$f_y = 415 \text{N/mm}^2$.

Design load $P_u = 1.5 \times P = 1.5 \times 1000$

$= 1500$ kN.

Asc = 1452.20 mm^2

Assume $16\overline{R}$ $A\phi = 201.06$

$$n = \frac{1452.20}{201.06}$$

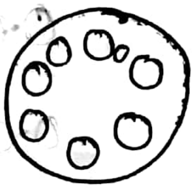
= 7.22 bars.

provide 8 bars $16\overline{R}$.

Asc provided = 1608.49.

$$\text{Asc provided} = \frac{1608.49}{\frac{\pi}{4} \times D^2} \times 100$$

= 1.14



lateral ties;

$$\text{Dia} \quad d = \frac{\phi}{4} = \frac{16}{4} = 4mm.$$

Provide 6mm ϕ (M.S).

Spacing

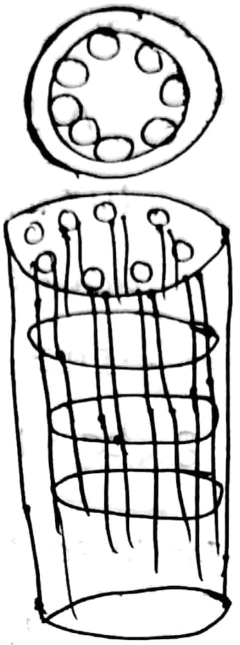
i) 430 mm

ii) 16×16 = 256mm

iii) 300mm.

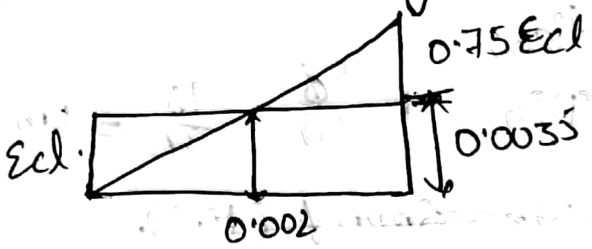
Provide 6 mm ϕ @ 3250 mm c/c.

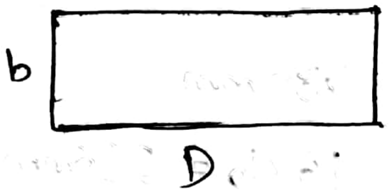
④



0·002 for axial load.

0·0035 for bending.





$\varepsilon_{cl} = 0{\cdot}0035 - 0{\cdot}75\,\varepsilon_{cl}.$

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 cosly

P_u & M_u are present ~~po~~ uniaxial coloumn.

Design a coloumn for the following data.

Axial load $P_u = 1200$ kN.

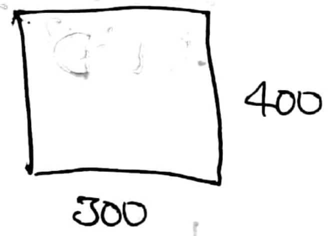
B.M $M_u = 250$ kN-m.

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.

Effecte 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.

$\frac{P_u}{f_{ck} bD} = \frac{1200 \times 10^3}{20 \times 300 \times 400}$

$= 0.5$

$\frac{M_u}{f_{ck} bD^2} = \frac{250 \times 10^6}{20 \times 300 \times 400}$

$=$ ~~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.

$P = 4.8\%$

$$A_{sc} = \frac{4.8}{100} \times 300 \times 400$$

$$= 5760\ mm^2$$

$20\phi \quad A\phi = 314 mm^2$

$$= \frac{5760}{314} = 18.34 \text{ bars}$$

Provide. 20 bars

25ϕ.

$$= \frac{5760}{25}$$

$= 12$ bars.

$$= 300 - 40 - 40 - \frac{25}{2} - \frac{25}{2}$$

$= 195$

$$= \frac{195}{5} = 39 mm.$$

$$\Rightarrow 39 - \frac{25}{2} - \frac{25}{2}$$

$= 14mm < 25mm$.

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

$\frac{P}{f_{ck}} = 0.135$

$P = 2.7\%$

$A_{sc} = \frac{2.7}{100} \times 350 \times 450$

$= 4252.5 \text{ mm}^2$

Assume $25\Phi = A\phi = 490.87 \text{mm}^2$

$n = 8.66$

$10 \times 25\Phi$.

Detailing:-

c/c of outer bars.

$350 - 40 - 40 - \frac{25}{2} - \frac{25}{2}$

$= 245$

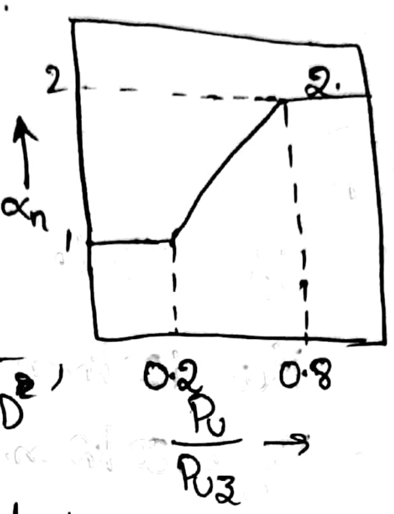
c/c of I.B. $\frac{245}{4} = 61.25\text{mm}$.

c/s of b $\Rightarrow 61.25 - \frac{25}{2} - \frac{25}{2}$

$\Rightarrow 36.25\text{mm}$.

Biaxial colamns

Procedure:- P_u, M_{ux}, M_{uy}.



1) Assume crossSection. (b,D)

Assume % of steel (p).

step-2 Find p/f_{ck}, $\frac{P_u}{f_{ck}bD^2}$,

$\frac{M_{ux_1}}{f_{ck}bD^2} \longrightarrow$ & find M_{ux_1}

step-3 find $\frac{p}{f_{ck}}$; $\frac{M_{uy_1}}{f_{ck}bD^2} \rightarrow M_{uy_1}$.

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{yu_1}}\right)^{\alpha_n} \leq 1.$$

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 400 x 500 mm.

Concrete mix M20.

characteristic strength of rft 415 N/mm^2

Factored load = 1200 kN.

Factored moment acting || to the larger dimension 100 kN-m.

Factored moment acting // to the shorter dimension
= 80 kN-m

Clear cover = 40mm.

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 = 400×500 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.

$$\frac{M_{ux_1}}{f_{ck}bD^2} = \frac{M_{ux_1}}{\cancel{20 \times 400 \times 500^2}} = \cancel{0}\ 0.105$$

0.12

$$= \cancel{5 \times 10^{10} M_{ux_1}}.$$

M_{ux_1} = 240 kN-m. 0.9!

$$\frac{d'}{b} = \frac{52.5}{400} = 0.13125$$

$$\frac{M_{uy_1}}{f_{ck} D b^2} = \frac{M_{uy_1}}{20 \times 500 \times 400^2}$$

chart 45

$$\frac{M_{uy_1}}{f_{ck} b^2 D} = 0.11$$

$$M_{uy_1} = 176 \text{ kN-m.}$$

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha n} \leq 1$$

$$\alpha n = \frac{P_u}{P_z}$$

$$P_z = 0.45 f_{ck} \cancel{4}. A_c + 0.75 f_y A_{sc}.$$

$$A_{sc} = \frac{1.6}{100} \times 400 \times 500$$

$$= 3200 \text{ mm}^2$$

$$A_g = 400 \times 500 = 200000 \text{ mm}^2$$

$$A_c = A_g - A_{sc}$$

$$= 200000 - 3200$$

$$= 196800.$$

$$P_{uz} = 0.45 \times 20 \cancel{+} \times 196800 + 0.75 \times 415 \times 3200$$

$$= 2767200.$$

$$P_u / P_{uz} = \frac{1200 \times 10^3}{2767200}$$

$$= 0.4336$$

$$\left(\frac{100\times10^6}{214\times10^6}\right)^{1.28} + \left(\frac{80\times10^6}{116\times10^6}\right)^{1.38}$$

$= 0.\overset{298}{\cancel{334}} + 0.336$

$=$ ~~0.707 < 1~~ $0.634 \lesssim 1.$

Hence ok.

Assume 25 $\overline{\phi}$.

Ast% of steel provided $= 1.96\%$

$$= \frac{3900}{490.87}$$

$= 6.519$

Provide $8 \times 25\,\overline{\phi}$

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) 300 mm

Provide $8\overline{\phi}$ @ 300 mm c/c

Circular coloumn with

Design a reinforced coloumn 400mm square to carry an ultimate load of 1000kN ad an eccentricity of 160mm. Use M20, Fe250

Sol

$P_u = 1000kN.$

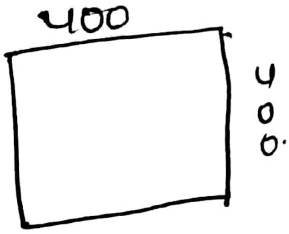
$E = 160 mm.$

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

$f_y = 250 N/mm^2.$

$b = 400mm$

$D = 400mm.$



$M_u = P_u \times E = . 160 kN\text{-}m.$

Assume % of steel = 1.6%.

$$\frac{p}{f_{ck}} = \frac{1.6}{20} = 0.08.$$

$$\frac{P_u}{f_{ck} bD} = \frac{1000 \times 10^3}{20 \times 400 \times 400} = 0.3125.$$

$$\frac{d'}{d} = \frac{50}{400} = 0.125$$

$$\frac{M_u}{f_{ck} bD^2} = \frac{160 \times 10^6}{20 \times 400 \times 400^2}$$

$$= 0.125$$

Design a short circular coloumn 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)

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

$$\frac{P_u}{f_{ck}D^2} = \frac{800\times10^3}{20\times500^2} = 0.16.$$

$$\frac{M_u}{f_{ck}D^3} = \frac{162.5\times10^3}{20\times500^3} = 0.065.$$

Chart-56 $\frac{p}{f_{ck}} = 0.05.$

$P = 0.05\times20 = 1\%$

$P = 1\%.$

$$A_{sc} = \frac{1}{100}\times\frac{\pi\times500^2}{4}$$

$$= 1963\,mm^2.$$

20ϕ = $A\phi = 314 mm^2$

$n = 6.25$

$7 \times 20\phi$.

lateral ties

$d = \frac{20}{4} = 5mm$

Provide 6ϕ

spacing i) 500mm

ii) $16 \times 20 = 400mm$

iii) 300mm.

Provide 6mm⌀ @ 300 mm c/c.

17/015

$b = 250mm$

$P_u = 600 kN\text{-}m.$

$M_{ux} = 60 kN\text{-}m.$

$M_{uy} = 40 kN\text{-}m.$

$f_y = 415 N/mm^2$

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

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 $P = 4.2.$

$\frac{P}{f_{ck}} = \frac{4.2}{30} = 0.14.$

$\frac{M_{ua_1}}{f_{ck} b D^2} = 0.14.$

$M_{ua_1} = 65.625 \text{ kN-m}.$

$M_{uy_1} = 65.625 \text{ kN-m}.$

$A_g = 250 \times 250 = 62500 \cdot mm^2.$

$A_{sc} = \frac{PbD}{100} = 2625 mm^2$

$A_c = 59875 mm^2$

$P_{uz} = 0.45 f_{ck} A_c + 0.75 \times f_y \times A_{sc}.$

$= 1625343.75.$

$\frac{P_u}{P_z} = \frac{600 \times 10^3}{1625343.75}$

$= 0.369.$

$0.2 \rightarrow 1$

$0.8 \rightarrow 2.$

$0.369 \rightarrow ?$

$1 + \frac{(2-1)}{(0.8-0.2)}(0.369 - 0.2)$

$= 1.281$

$$= \left(\frac{60}{65.625}\right)^{1.28} + \left(\frac{40}{65.695}\right)^{1.28}$$

$= 0.891 + 0.53$

$= 1.42.$ $\qquad\qquad P = 4.8$

$M_{ua_1} = 0.16 \times 30 \times 250^3$

$= 75kN\text{-}m.$

$A_g = 3000$

$A_c = 59500$

$A_g = 62500.$

$P_{uz} = 1737000$

~~P~~α_n-0.345.

0.34

$$1 + \frac{(P-1)}{(0.8-0.2)}(0.34-0.2)$$

$= 1.23.$

$= 0.75 + 0.46.$

1.21.

Scanned by CamScanner

5·4

$\frac{P}{P_{cr}} = 0{\cdot}018$

0·17

79·6887500

3375

59125

$P_{uz} = 1848656{\cdot}25$

$P_u = 0{\cdot}324$

14 (0·

~~1·015~~ 1·206

$$\left(\frac{60}{79{\cdot}688}\right)^{\overset{1{\cdot}206}{\cancel{1{\cdot}015}}} + \left(\frac{40}{79{\cdot}688}\right)^{1{\cdot}206}$$

~~0·749+0·49~~ 0·711 + 0·435

= ~~1·039~~ 1·14

6y = 02

0·19 x 89062500

= 3750
58750

$P_z = 1960312{\cdot}5$

= 0·306.

[illegible] 1·16.

$$= \left(\frac{60}{89.0625}\right)^{1.16} + \left(\frac{40}{89.0625}\right)^{1.16}.$$

= 0·6[illegible] + 0·3[illegible]

= 0·9.

Spiral coloumn:-

Design a short circular coloumn 500 dia

factored load = 800kN

factored moment = 162·5 kN-m

Provide $\overset{\text{Helical}}{\text{Hoop}}$ reinforcement. Take M20, Fe415 steel.

P_U = 800 kN

M_U = 162·5 kN-m.

f_{ck} = ~~415~~ 20N/mm².

f_y = 415N/mm².

$$\left(\frac{P_U}{f_{ck}D^2}\right)_{Hoop} = 0.16.$$

$$\left(\frac{P_U}{f_{ck}D^2}\right)_{Helical} = 0.152.$$

$\left(\frac{M_U}{f_{ck} D^3}\right)_{Hoop} =$ 0.065

$\left(\frac{M_U}{(f_{ck} D^3)}\right)_{Helical} = 0.061.$

$\frac{P}{f_{ck}} = 0.04 \Rightarrow P = 0.04 \times 20$

$= 0.8\%.$

∴ Area of steel $= \cancel{0}\ \frac{P}{100} \times \frac{\pi}{4} \times D^2$

$= 1570 \cdot mm^2.$

Assume 20Φ.

∴ provide 6 x 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.

∴ Core dia; $D_c = 500 - 2 \times 40 + 2 \times 8$

$= 438$ mm.

500-2x40+2x8

∴ Core area $= \frac{\pi}{4} \times D_c^2$

$V_c = A_c = 149301 \cdot mm^2$

∴ $\frac{A_g}{A_c} = \frac{\frac{\pi}{4} \times 500^2}{436^2} = 1.3151$

$0.36\left(\frac{A_g}{A_c}-1\right) f_{ck}/f_{yh}$

$0.36\left(1.315-1\right)\frac{20}{250}$

$= 0.00907$

Dia. of core upto center of helics.

$500 - 2\times40 + 8 = 428$ mm.

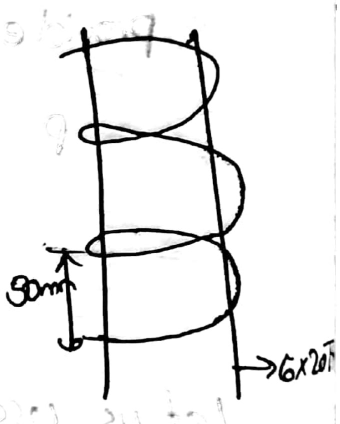
Let the pitch of the spiral be ~~5mm~~ 'S' mm

volume of spiral for 1mm length of coloumn.

$V_H = \frac{\pi d}{s}\times\frac{\pi}{4}\times\phi^2$

$= \frac{\pi\times428}{s}\times\frac{\pi}{4}\times8^2$

$\frac{V_H}{V_C} \quad = \frac{67587}{s}$



$\frac{V_H}{V_{AC}} = \frac{\frac{67587}{s}}{149301}$

$= \frac{0.452}{s}$

$\frac{V_H}{V_C} = 0.36 \xrightarrow{} 0.36\left(\frac{A_g}{A_c}-1\right)\frac{f_{ck}}{f_{yh}} = 0.00907.$

$\frac{0.452}{s} = 0.00907. = 49.9$ mm.

$s = 50$ mm

pitch should not more than

i) 75mm.

ii) $\frac{1}{6}D_c. = \frac{1}{6}\times436$

$= 72.67$

Pitch should not be less than = $3 \times \phi_s$

$= 3 \times 8 = 24mm$

Hence pitch 50mm is 'ok'

Long coloumns:-

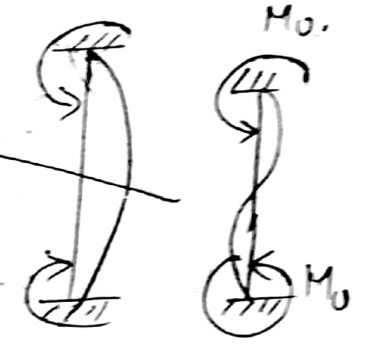
$$M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

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.

M_{20} concrete

Fe 415 steel.

Effective length 6mts, unsupported length 7mts.

factored load $P_u = 1200$ kN.

a 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

$$e_{ax} = \frac{D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

$$= \frac{400}{2000}(15)^2$$

$$= 45mm.$$

$$M_{ax} = P_u \times e_{ax}$$

$$= 1800 \times \frac{45}{1000}$$

$$= 54 kN\text{-}m.$$

Calc: of k_a

$$k_a = \frac{P_{uz} - P_u}{P_{uz} - P_b}.$$

$$P_b = \left(k_1 + k_2 \frac{p}{f_{ck}}\right) f_{ck} D^2.$$

P_2 2.5%.

$$\frac{d'}{D} = \frac{50}{400} = 0.125.$$

Table 60; $k_1 = 0.155$

$k_2 = 0.266.$

$$= \left(0.155 + 0.266 \times \frac{2.5}{20}\right) 20 \times 400^2$$

= ~~1747800~~ = 601 kN.

2/9/01.

$$K = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

$A_g = \frac{\pi}{4} \times D^2 = \frac{\pi}{4} \times 400^2 = 125663.7 \, mm^2.$

$A_s = \frac{P}{100} \times \frac{\pi}{4} \times D^2 = \frac{2.5}{100} \times \frac{\pi}{4} \times 400^2.$

$3141.59 \, mm^2$

$A_c = 122522.1 \, mm^2$

$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}$

$= 0.45 \times 20 \times 122522.1 + 0.75 \times 415 \times 3141.59.$

$= 2080518.854 \, N\text{-}m.$

$$\underline{K =} \frac{2080518854 - 1200 \times 10^3}{2080518.854 - 60\cancel{2} \times 10^3.}$$

$\sim 0.596 < 1.$

Assume $20\overline{\phi}$.

$= \frac{3141.59}{314} = 10.005$ bar

provide $10 \times 20\overline{\phi}$.

∴ Reduced Additional moment.

$\underline{M'_a = k M_a.}$

$= 0.59 \times 54.$

$= 31.86$ kN-m

Slabs

Slabs :- Flexure, shear.

$$M = \frac{wl_e^2}{8} = \frac{Wl_e}{8} \qquad W = wl_e$$

$$V = \frac{wl_e}{2} = \frac{W}{2}$$

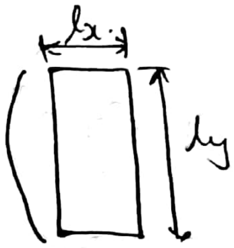
l_{eff} is the least of

i) clearspan + d.

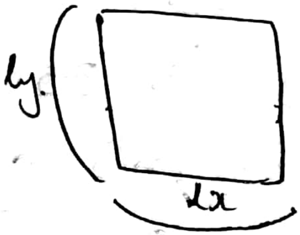
ii) c/c of supports.

Slab types:-

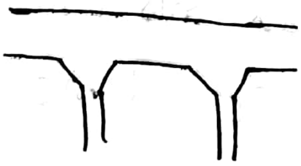
1) One-way slab → $\frac{l_y}{l_x} > 2$.



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 without access.

For floors:-

2~~3~~ 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~~ rft 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 haule 3.5 mts x 8mts (inside dimensions) with 25 mm walls all around. Assume a live load of 4kN/m² and finishing load 1kN/mm². Use M20 & Fe41 steel.

Step-1 Given data:-

Hall = 3.5m x 8m

width of masonary = 250mm

live load = 4kN/m².

Finishing = 1 kN/m²

f_{ck} = 20N/mm²

f_y = 415 N/mm²

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 = $1 kN/m^2$

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.

leff 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.

Factored BM $(M_u) = \frac{w l_{eff}^2}{8}$

$= \frac{13.5 \times 3.64^2}{8}$

$= 22.35$ kN-m

Factored SF $(V_u) = + \frac{W_u l_{eff}}{2}$

$= 24.5$ kN.

Step-4 Check for depth.

$M_u = 0.138 f_{ck} b d^2$

$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$

$= \sqrt{\frac{22.5 \times 10^6}{0.138 \times 20 \times 1000}}$

$= 90mm < 140mm.$

Step-5 Calc. of steel area.

$\frac{x}{d} = 1.2 - \sqrt{1.44 - \frac{6.6 M_u}{f_{ck} b d^2}}$

$\frac{6.6 M_u}{f_{ck} b d^2} = \frac{6.6 \times 22.35 \times 106}{20 \times 1000 \times 140^2}$

$= 0.376$

$\frac{x}{d} = 0.168$

$\frac{x}{d} = 0.168 < \left(\frac{x_{umax}}{d} = 0.48\right)$

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 $A_{st} = \frac{M_u}{0.87 f_y z}$

$= 475.7 \text{ mm}^2$.

Assume 10TMT $A\phi = \frac{\pi}{4} \times 10^2 = 78.53 \text{mm}^2$.

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 × 140 = 420mm.

ii 300m

$S_{max} = 300$mm

Provide 10TMT @ 160 mm c/c.

Step-7 Check for deflection

For simply supported case basic value $\frac{l}{d}$ = 20.

% of tensile steel at midspan.

$$A_{st}\ \text{provide} = \frac{1000}{s} \cdot \frac{\pi}{4} d^2$$

$$= \frac{1000}{160} \times \frac{\pi}{4} \times 10^2.$$

490.8.

$= \text{~~490.8 mm²~~}$. ~~523~~

$A_{st\,req} = 475\ mm^2$.

$$f_s = 0.58 f_y \frac{A_{st\,req}}{A_{st\,provided}}$$

$$= 0.58 \times 415 \times \frac{475}{490.8}$$

$= 232.9\ mm^2$.

~~Modi~~ % of steel $= \frac{5490.8}{1000 \times 140} \times 100 = 0.35$.

$\frac{l}{d} = 20 \times 1.$

$\frac{l}{d} = \frac{3500}{140} = 25$

Hence deflection limit exceeds. So change the spacing of the main steel. ie, Assume S = 150 mm instead of 160 mm.

$$\text{~~fs~~} = A_{st}\ \text{provided} = \frac{1000}{150} \times \frac{\pi}{4} \times 10^2$$

$= 523.5\ mm^2$.

$$0.58 \times 415 \times \frac{475}{523.5} = 1.55$$

$\frac{l}{d} = 20 \times 1.55$

$= 31.$

$\left(\frac{l}{d}\right)_{provided} = \frac{3500}{140} = 25$

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.

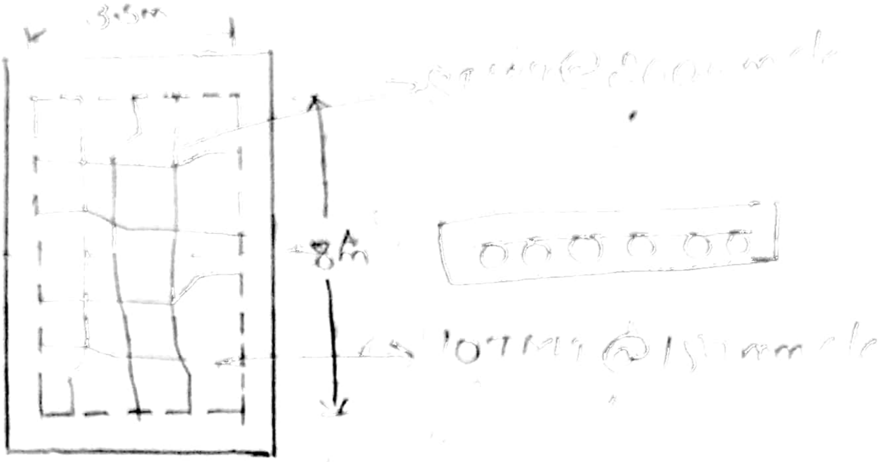
~~M20~~ Concrete = M20.

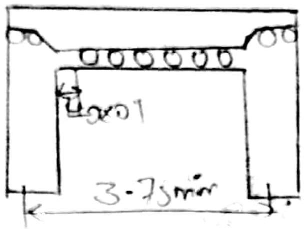
Steel Fe415.

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

$$\Delta_{max.} = \frac{5}{384}\,\frac{q_y l_y^4}{EI.}$$

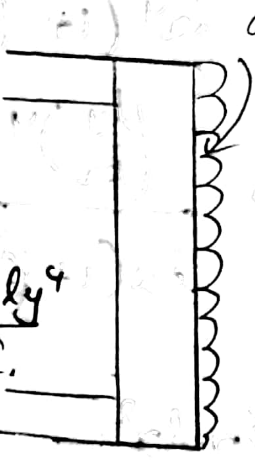
$$\frac{5}{384} \times \frac{q_x l_x^4}{EI.} = \frac{5}{384}\,\frac{q_y l_y^4}{EI.}$$

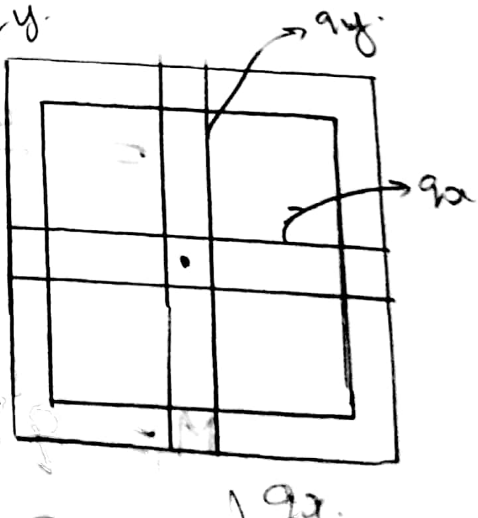
$$q_x = q_y\left(\frac{l_y}{l_x}\right)^4.$$

$$q_y = q_x\left(\frac{l_x}{l_y}\right)^4.$$

$$q = q_x + q_y.$$

$$q = q_x + q_x\left(\frac{l_x}{l_y}\right)^4 + q_x.$$





$$q_x = \frac{q}{\left(\frac{l_x}{l_y}\right)^4 + 1.}$$

$$q_x = \frac{q \times l_y^4}{l_x^4 + l_y^4.}$$

$$q_y = \frac{q\, l_x^4}{l_x^4 + l_y^4.}$$

$$M_x = \frac{q_x l_x^2}{8}$$

$$= \frac{q\, l_y^4}{8(l_x^4 + l_y^4)} \times l_x^2.$$

$$= \frac{q}{8} \frac{l_y^4}{l_x^4\left(1 + \left(\frac{l_y}{l_x}\right)^4\right)} l_x^2$$

$$= \frac{q}{8} \frac{\left(\frac{l_y}{l_x}\right)^4}{\cancel{l_x}\left(1 + \left(\frac{l_y}{l_x}\right)^4\right)}.$$

$$M_x = q\, B_x \cdot l_x^2.$$

$$B_x = \frac{\left(\frac{l_y}{l_x}\right)^4}{8\left(1 + \left(\frac{l_y}{l_x}\right)^4\right.}$$

$$M_y = \frac{q_y l_y^2}{8}$$

$$= \frac{q l_x^4}{8 l_x^4 + l_y^4} l_y^2.$$

$$= q \cdot \frac{l_x^4}{8 (l_x^4 (1+ \left(\frac{l_y}{l_x}\right)^4)} l_y^2$$

$$= \frac{q \cdot}{8(1+ \left(\frac{l_y}{l_x}\right)^4)} l_y^2.$$

$$= \frac{q}{8(1+ \left(\frac{l_y}{l_x}\right)^4} \cdot \frac{l_y^2}{l_x^2} \times l_x^2.$$

$$M_y = q \beta_y l_x^2.$$

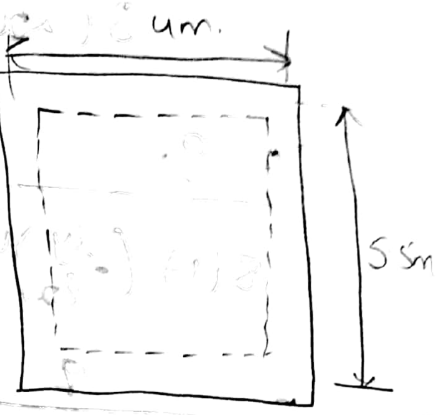
$$\beta_y = \frac{\left(\frac{l_y}{l_x}\right)^2}{8 \left(1+\left(\frac{l_y}{l_x}\right)^4\right)}.$$

$$d = \frac{\text{span}}{25 \text{ to } 30}$$

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 car a characteristic live load of $8 kN/m^2$ M25, Fe415 exposure condition mild.

Given data.

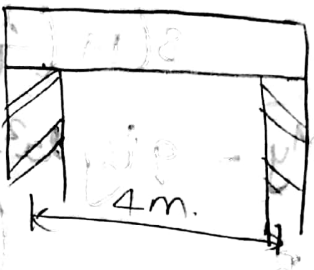


Room Dimensions : 4m x 5.5m.

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.

$l_x = 4000mm.$

$$\text{Total depth } D = \frac{l_x}{24} = \frac{4000}{24.}$$

$= 166.67.$

$= 166$ mm.

Take D = 170 mm.

Effective depth $d = D - CC - \frac{\phi}{2}$

$= 170 - 20 - \frac{10}{2}$ (10TMT 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 = 8kN/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;

$\frac{l_y}{l_x} = 1.3$ $\alpha_x = 0.093$.

$\frac{l_y}{l_x} = 1.4$ $\alpha_x = 0.099$

$$0.093 + \frac{(0.099 - 0.093)}{(1.4 - 1.375)}(1.375 - 1.3)$$

$= 0.0975$.

$\frac{l_y}{l_x} = 1.3$ 0.055

$\frac{l_y}{l_x} = 1.4$ 0.051.

$\alpha_y = 0.052$.

$M_x = \alpha_x w l_x^2$

$= 0.0975 \times 19.341 \times 4^2$

$= 30.17$ kN-m.

$M_y = \alpha_y w l_x^2 = 0.052 \times 19.341 \times 4^2$

$= 16.09$ kN-m.

Step-4 Check for depth.

$M_{max} = 30.17$ kN-m.

For Fe415 $M_u = 0.138 f_{ck} bd^2$.

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{30.17 \times 10^6}{0.138 \times 25 \times 100}}$$

$= 93$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) $3d = 3\times145 = 435$mm.

ii) 300mm.

$S_{max} = 300$mm.

10 TMT @ 120mm c/c.

$$A_{st} \text{ provided} = \frac{1000}{S} \cdot A\phi$$

$$= \frac{1000}{120} \times \frac{\pi}{4} \times 10^2 = 654\text{mm}^2.$$

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

$$= \frac{654}{1000\times145} \times 100 = 0.45\ \%.$$

Step-6 Secondary steel.

$$M_{\cancel{u}}\ 1.2 - \sqrt{1.44 - \frac{6.6M_u}{f_{ck} b d^2}}$$

$$\frac{a}{d} = 0.08 < 0.48.$$

Hence. Ok.

lever arm $z = d\left(1 - 0.416 \times \frac{a}{d}\right)$

$= 318.3$mm.

Assume 10 TMT.

$A\phi = 78.5$

$$n = \frac{A_{st}}{A\phi} = 4.03.$$

$S = \frac{1000}{n} = 246mm$

(d = 135mm)

Smax i) 3d = 3×145

= 435mm.

ii) 450 mm.

Provide 10TMT @ 220mm c/c.

$Ast\ provide = \frac{1000}{S} A\phi$

$= \frac{1000}{220} \times 78.5$

$= 356.9mm^2$

$\left(\frac{356.9}{1000 \times 135} \times 100\right)$

$= 0.26$

% of steel = $\frac{356.9}{1000 \times 145} \times 100 = 0.24\%$

Check for deflectin.

For simply supported Case $\frac{l}{d} = 20$.

M25 fs = ~~0.58fy x~~ $0.58 \times 415 \times \frac{620}{654}$

$= 228.18 N/mm^2$

Modification fectr = 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.

Summay :-

Depthof slab = 170mm

C.C = 20mm.

Primay 10TMT @ 120 mm c/c.

secondary 10TMT @ 220mm c/c.