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Chapter-2 Working stress Method

Assumptions

1. Plane section before bending remains plane after bending.
2. All tensile forces are taken by reinforcement alone unless otherwise stated.
3. Stress strain curve is always linear.
4. Bond between steel and concrete is uniform.
5. Modular ratio (m) is taken as $\frac{280}{3\sigma_{cbc}}$

where, σ_{cbc} = Permissible compressive bending stress in concrete

Permissible stress or allowable stress

It is a fraction of yield stress which is allowed in material.

$\sigma_{cbc} = \frac{f_{ck}}{FOS}$, FOS = 3 for concrete

$\sigma_{st} = \frac{f_y}{FOS}$, FOS = 1.78

permissible stress in steel in tension.

$\sigma_{sc} = \frac{f_y}{FOS}$

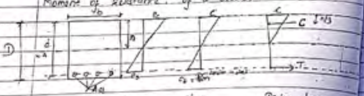
permissible stress in steel in compression

Types of steel

1. Mild steel (Fe 250) : $f_y = 250\ N/mm^2$
2. Torsteel (Torkari) (Fe 415) ; $f_y = 415\ N/mm^2$
3. TMT (Fe 500) $f_y = 500\ N/mm^2$

Thermo mechanically treated

Moment of resistance of a section



strain diagram | stress diagram | Design stress diagram

Stress in steel (t) $= E_s \cdot \varepsilon_s$ — (1)

Stress in concrete around steel

$$c_1 = E_c \cdot \varepsilon_s$$

$$= E_c \cdot \frac{t}{E_s} \qquad \boxed{\frac{E_s}{E_c} = m}$$

$$= \frac{t}{m}$$

Depth of N.A (n) $= n_1 \cdot d$

$$n_1 = \frac{m}{m+r}$$

$$r = \frac{t}{c}$$

$$MOR_{concrete} = (b \cdot n) \cdot \left(\frac{1}{2} \cdot c\right) \cdot \left(d - n/3\right)$$

$$= \frac{1}{2} \cdot b \cdot n \cdot c \left(d - \frac{n}{3}\right)$$

$$= \frac{1}{2} \cdot b \cdot n_1 \cdot d \cdot c \left(d - \frac{n_1 d}{3}\right)$$

$$= \frac{1}{2} \cdot c \cdot n_1 \cdot \left(1 - \frac{n_1}{3}\right) \cdot bd^2$$

$$MOR = Qbd^2$$

$$Q = \frac{1}{2} \cdot c \cdot n_1 \cdot \left(1 - \frac{n_1}{3}\right)$$

$$MOR_{steel} = T \cdot \left(d - \frac{n}{3}\right)$$

$$MOR = t \cdot A_{st} \cdot \left(d - \frac{n}{3}\right)$$

Types of section

1. Balanced or critical section

If steel and concrete at a section fails at the same time, such section is known as balanced section.

If max stresses in steel and concrete ~~reaches~~ reaches simultaneously such section is balanced section. In such case, moment of resistance is calculated as:

$$MOR = \frac{1}{2} b \cdot n \cdot c \left(d - n/3\right) \quad \text{or}$$

$$MOR = T \cdot \left(d - n/3\right)$$

2. Under reinforced section

If steel of a section fails before concrete, such section is known as under reinforced section.

In such case MOR is calculated as:

$$MOR = T \cdot \left(d - n/3\right)$$

Depth of neutral axis in such case is less than that of balanced case.

3. Over reinforced section

concrete fails before steel at a section

In such case, MOR is calculated as

$$MOR = \frac{1}{2} \cdot b \cdot n \cdot c \left(d - n/3\right)$$

Depth of neutral axis in such case is more than that of balanced case.

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Permissible stress or allowable stress

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$$\sigma_{cbc} = \frac{f_{ck}}{FOS}, \quad FOS = 3 \text{ for concrete}$$

$$\sigma_{st} = \frac{f_y}{FOS}, \quad FOS = 1.78$$

σ_{st} = permissible stress in steel in tension,

$$\sigma_{sc} = \frac{f_y}{FOS}$$

σ_{sc} = permissible stress in steel in compression

C $\frac{1}{2}$ bnd * c

Types of steel

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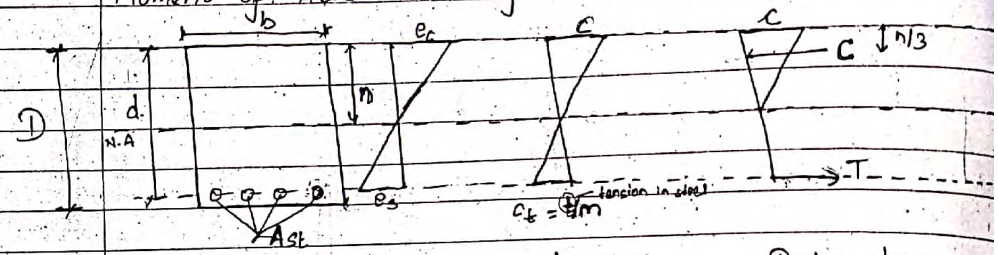
Moment of Resistance of a section:



strain diagram — stress diagram — Design stress diagram

Stress in steel (t) = $E_s * e_s$ — (i)

Stress in concrete around steel

$$c_t = E_c * e_s$$
$$= E_c * \frac{t}{E_s}$$
$$= \frac{t}{m}$$

$$\boxed{\frac{E_s}{E_c} = m}$$

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Depth of N.A (n) = $n_1 * d$

$$n_1 = \frac{m}{m+r}$$

$$r = \frac{t}{c}$$

$$MOR_{concrete} = (b*n) * \left(\frac{1}{2} * c\right) * \left(d - n/3\right)$$

$$= \frac{1}{2} * b * n * c \left(d - \frac{n}{3}\right)$$

$$= \frac{1}{2} * b * n_1 * d * c \left(d - \frac{n_1 d}{3}\right)$$

$$= \frac{1}{2} * c * n_1 * \left(1 - \frac{n_1}{3}\right) * bd^2$$

$$MOR = Qbd^2$$

$$Q = \frac{1}{2} * c * n_1 * \left(1 - \frac{n_1}{3}\right)$$

$$MOR_{steel} = T * \left(d - \frac{n}{3}\right)$$

$$MOR = t * A_{st} * \left(d - \frac{n}{3}\right)$$

Types of section

1. Balanced or critical section

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$$MOR = T * \left(d - n/3\right)$$

2. Under reinforced section

If steel at a section fails before concrete, such section is known as underreinforced section.

In such case, MOR is calculated as:

$$MOR = T * \left(d - n/3\right)$$

Depth of neutral axis in such case is less than that of balanced case.

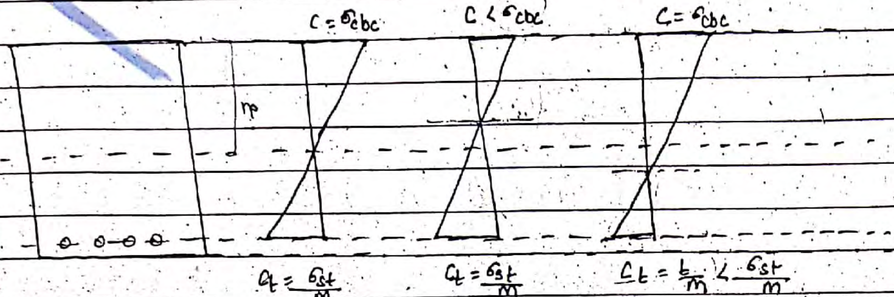
3. Over reinforced section

concrete fails before steel at a section

In such case, MOR is calculated as

$$MOR = \frac{1}{2} * b * n * c \left(d - n/3\right)$$

Depth of neutral axis in such case is more than that of balanced case.



Balanced section | under-reinforced section | over-reinforced section

Which section would you prefer and why?

→ We prefer under-reinforced section because

i. Steel being expensive component.

ii. Under-reinforced section are stiffer than other section.

iii. It minimizes long term and short term deflection.

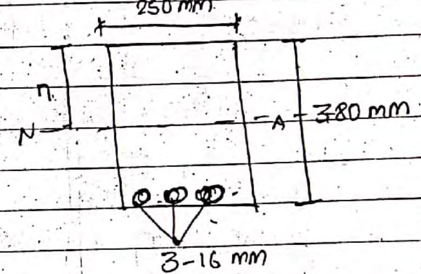
iv) Over-reinforced → brittle failure

under-reinforced → ductile failure

Numericals.

Q.1. Calculate depth of Neutral axis for a singly reinforced b 250 × 380 mm to the centre of reinforcement. Also calc maximum stress in ~~steel and~~ concrete if it is reinfo with 3-16 mm dia bar with permissible stress in ste 140 N/mm² is achieved. Take m = 13.33

Solution:



$b = 250 mm$

$d = 380 mm$ (eff. depth)

$A_{st} = 3 * \frac{\pi}{4} * 16^2$

$= 603.18 \ mm^2$

$m = 13.33$

$\sigma_{st} = 140 \ N/mm^2$

Taking moment about N.A

$$250 * n * \frac{n}{2} = (m \ A_{st}) * (380 - n)$$

$$250 * \frac{n^2}{2} = 13.33 * 603.18 * (380 - n)$$

$$\therefore n = 127.45 \ mm$$

$$\frac{c}{c_t} = \frac{n}{d-n}$$

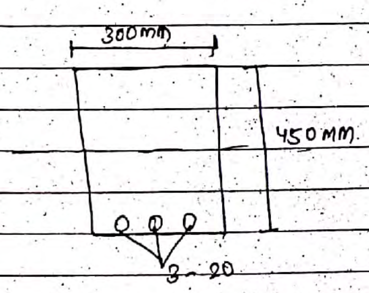
or, $$\frac{c}{(t/m)} = \frac{n}{(d-n)}$$

or, $$\frac{c}{(\sigma_{st}/m)} = \frac{127.45}{380-127.45}$$

$\therefore c = 5.3\ N/mm^2$

$t = \sigma_{st} = 140\ N/mm^2$

Q.2. Calculate moment of resistance of a singly reinforced RC beam having dimension 300 * 450 mm (effective) and it is reinforced with 320 mm dia bar. Max. permissible stresses in concrete and steel are 7 N/mm² and 140 N/mm².



$b = 300mm$

$d = 450\ mm$

$A_{st} = 3 \times \frac{\pi}{4} * 20^2 = 942.47\ mm^2$

$\sigma_{st} = 140\ N/mm^2$

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

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

Taking moment about N.A

$$300 * n * \frac{n}{2} = m A_{st} * (450-n)$$

$\therefore n = 156.73\ mm$

Now,

$n_c = n_1 * d$ $\qquad r = t/c$ [for balanced case

$$n_1 = \frac{m}{m+r}$$

$$r = \frac{t}{c} = \frac{\sigma_{st}}{\sigma_{cbc}} = \frac{140}{7} = 20$$

$$n_1 = \frac{13.33}{13.33+20} = 0.4$$

$n_c = 0.4 * 450 = 180\ mm$

Since, $n < n_c$, it is under-reinforced section.

$MOR = A_{st} \cdot \sigma_{st} \cdot \left(d - \frac{n}{3}\right)$

$= 942.47 * 140 * \left(450 - \frac{156.73}{3}\right)$

$= 52.48 \text{ KNm}$

Q.3. Design a singly reinforced RC beam of eff. span 4m carrying an UDL of 15KN/m over the span. Beam is 250mm wide and the material used are M20 concrete and Fe415 bar.

Soln,

$l = 4m$

$w = 15 \text{ KN/m}$

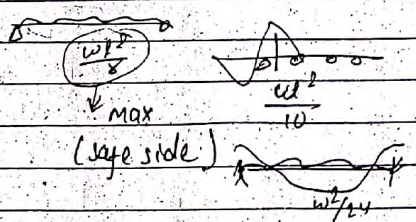
$\sigma_{st} = \frac{415}{1.78} = 233.15 = 230 \text{ N/mm}^2$ (Annex-B Table-21, 22)

$\sigma_{cbc} = \frac{20}{3} = 6.67 \approx 7 \text{ N/mm}^2$

$b = 250 \text{ mm}$

$M_{max} = \frac{wl^2}{8} = \frac{15 * 4^2}{8}$

$= 30 \text{ KN-m}$



Now, for balanced section

$n_c = n_1 * d$

$n_1 = \frac{m}{m+r}$

$r = \frac{t}{c} = \frac{\sigma_{st}}{\sigma_{cbc}} = \frac{230}{7} = 32.86$

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

$n_1 = \frac{m}{m+r}$

$= \frac{13.33}{13.33+32.86}$

$= 0.288$

$MOR = \frac{\sigma_{cbc}}{2} * b * n * \left(d - \frac{n}{3}\right)$

$30 \times 10^6 = \frac{7}{2} * 250 * 0.288 * d \left(d - \frac{0.288d}{3}\right)$

$d = 362.89 \approx 400mm$

$MOR = A_{st} * \sigma_{st} * (d - n/3)$

$30 \times 10^6 = n\frac{\pi d^2}{4} * 230 * \left(\cancel{365}\ 400 - \frac{0.288 * \cancel{365}\ 400}{3}\right)$

$n =$

$\frac{D}{b} = 1.5$ / 1.103

Since

$C = T$

$\frac{\sigma_{cbc}}{2} * b * n = A_{st} * \sigma_{st}$

$A_{st} = \frac{\frac{7}{2} * 250 * 0.288 * 400}{230}$

$= 438.26 \ mm^2$

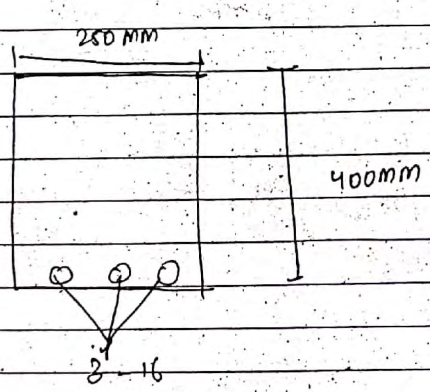
Adopt $\phi = 16 mm$

$a = A_{st} = \frac{\pi}{4} * 16^2 = 201.06$

No. of bars $= \frac{438.26}{201.06}$

$= 2.18$

≈ 3 nos.



Moment of resistance of a RC beam 'b' mm wide and 'D' mm deep is $0.85 \ bd^2$. Calculate the ratio of depth of N.A to depth of beam to the centre of reinforcement if m = 18.33. Permissible stresses in concrete and steel are $7 N/mm^2$ and $230 N/mm^2$.

Solution:

$\frac{n}{d}$

Moment of resistance $= 0.85bd^2$

$m = 18.33$

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

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

For balanced section,

$r = \frac{t}{c} = \frac{230}{7} = 32.86$

$n_1 = \frac{m}{m+r} = \frac{18.33}{18.33+32.86} = 0.358$

$n_c = n_1 * d$

$= 0.358 * d$

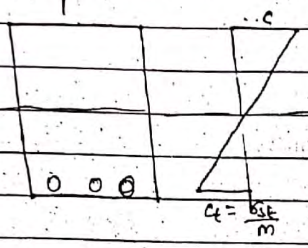
$MOR = \frac{\sigma_{cbc}}{2} * b * n * \left(d - \frac{n}{3}\right)$

$\cancel{0.85bd^2} = \frac{7}{2} * b * \cancel{0.358} * d * \left(d - \frac{\cancel{0.358} * d}{3}\right)$

$\cancel{1.9125 \ b^3} \quad \cancel{\frac{n}{d} = 0.2 \ ? \ 2.73 \ 0.266}$

$= 1.103bd$

Since, $(MOR)_1 < (MOR)_{critical}$, it is assumed as under-reinforced section.



let, $n = n_1 * d$

$$\frac{c}{c_t} = \frac{n}{d-n}$$

$$C = \frac{c}{2} * b * n$$

$$\frac{c}{\frac{\sigma_{st}}{m}} = \frac{n_1 * d}{d - n_1 * d}$$

$$T = t * A_{st}$$

$$\therefore c = \frac{\sigma_{st}}{m} * \frac{n_1 * d}{d - n_1 d}$$

$$= \frac{\sigma_{st}}{m} * \frac{n_1}{1-n_1}$$

$$= \frac{230}{18.23} \left(\frac{n_1}{1-n_1}\right) \quad \text{--- (i)}$$

$$MOR = \frac{c}{2} * b * n * \left(d - \frac{n}{3}\right)$$

$$0.85 = \frac{1}{2} * \frac{230}{18.33}\left(\frac{n_1}{1-n_1}\right) * n_1 * \left(1 - \frac{n_1}{3}\right)$$

$n_1 = 0.321$

2.1)

$b = 300mm$

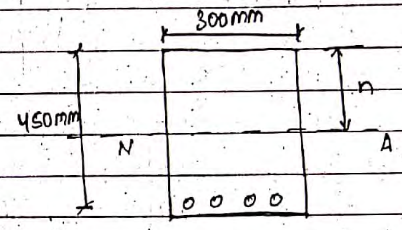
$d = 450mm$

No. of bars = 4

dia. of bars = 20mm

$m = 9$

Permissible stress in concrete, $\sigma_{cbc} = 7 N/mm^2$

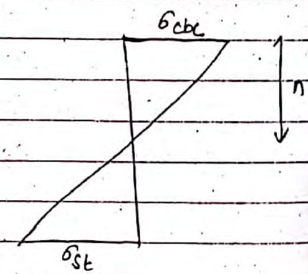


Taking moment about N.A,

$$300 * n * \frac{n}{2} = m \; A_{st} * (450 - n)$$

$$300 * \frac{n^2}{2} = 9 * 4 * \frac{\pi}{4} * 20^2 * (450 - n)$$

$\therefore n = 150.32mm$



From similar triangles,

$$\frac{\sigma_{cbc}}{n} = \frac{\sigma_{st}/m}{450-n}$$

or, $$\frac{7}{150.32} = \frac{\sigma_{st}/9}{450-150.32}$$

$\therefore \sigma_{st} = 125.6 \; N/mm^2$

2.2

$b = 300\,mm$

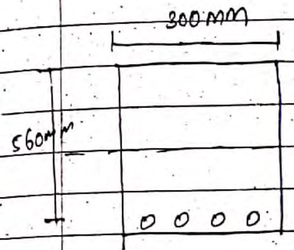
$D = 600\,mm$

$d = 600 - 40 = 560\,mm$

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

$\sigma_{cbc} = \frac{20}{\cancel{1.783}} = 6.67 \approx 7\,N/mm^2$

$\sigma_{st} = \frac{415}{1.78} = 233.15 \approx 230\,N/mm^2$



For balanced section,

$r = \frac{t}{c} = \frac{230}{7} = 32.85$

$n_1 = \frac{m}{m+r} = \frac{13.33}{13.33+32.85} = 0.288$

$n_c = n_1 * d$

$= 0.288 * 560$

$= 161.28\,mm$

Taking moment about N.A,

$300 * \frac{n^2}{2} = m * A_{st} * (560 - n)$

or, $150 * n^2 = 13.33 * 4 * \frac{\pi}{4} * 16^2 * (560 - n)$

$n = 167.49\,mm$

Since $n > n_c$, it is over-reinforced section.

$MOR = A_{st} * t * \left(d - \frac{n}{3}\right)$

$= 4 * \frac{\pi}{4} * 16^2 * 230 * \left(560 - \frac{167.49}{3}\right)$

$= 93.25\,kNm$

$MOR = \frac{1}{2} * c * n_1 * \left(1 - \frac{n_1}{3}\right)$

$= \frac{1}{2} * 7 * 0.288 ($

$= \frac{1}{2} * b * n * c \left(d - \frac{n}{3}\right)$

$= \frac{1}{2} * 300 * 167.49 * 7 * \left(560 - \frac{167.49}{3}\right)$

$= 88.66\,kNm$

2.3) b = 250 mm

d = 500 mm

MOR = 45 KNm

$\sigma_{st} = \frac{415}{1.78} = 233.15 \approx 230 \ N/mm^2$

$\sigma_{cbc} = \frac{20}{3} = 7 \ N/mm^2$

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

$MOR = \frac{1}{2} * b*n*C*\left(d-\frac{n}{3}\right)$

$45*10^6 = \frac{1}{2} * 250*n*7*\left(500-\frac{n}{3}\right)$

$n = 111.08 \ mm$

$n_c = n_1 * d$ $\quad r = \frac{t}{c} = \frac{230}{7} = 32.86$

$n_1 = \frac{m}{m+r} = \frac{13.33}{13.33+32.86} = 0.288$

$n_c = 0.288 * 500 = 144.3 \ mm$

Since, $n < n_c$, it is under-reinforced section.

$MOR = A_{st} * t * (d - n/3)$

$45*10^6 = A_{st} * 230 * \left(500 - \frac{111.08}{3}\right)$

$A_{st} = 422.6 \ mm^2$

$\cancel{\frac{\pi}{4} d^2}$

Adopt $\phi = 16$ mm

$A_{st} = 201.06$

No. of bars ≈ 3

2.4) $l = 4m$

Total load = 30 KN/m

$\sigma_{cbc} = \frac{20}{3} \approx 7 \ N/mm^2$

$\sigma_{st} = \frac{415}{1.78} \approx 230 \ N/mm^2$

$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3*7} = 13.33$ $\quad r = \frac{t}{c} = 32.86$

~~MO~~ For balanced section,

$n = n_1 * d$

$n_1 = \frac{m}{m+r} = \frac{13.33}{13.33+32.86} = 0.288$

$n =$

$Moment_{max} = \frac{wl^2}{8}$

$= \frac{30 * 4^2}{8}$

$= 60$ KN-m

$MOR = \frac{1}{2} b * n * c \left(d - \frac{n}{3}\right)$

$60 * 10^6 = \frac{1}{2} *$

Q.4)

Dead load = 10 KN/m

live load = 20 KN/m

Total load = 30 KN/m

clear span = 4m

As per clause 22.2(a) IS 456:2000

eff. span shall be smaller of

i) Centre to centre distance between bearings

$= 4000 + 300$

$= 4300$ mm.

ii) Clear span plus eff depth

$= 4000 + 550$ (say)

$= 4550$ mm

$\therefore$ eff span = 4.3 m

Max. bending moment $= \frac{wl^2}{8} = \frac{80 * 4.3^2}{8}$

$= 69.33$ KN m

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

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

$m = 13.33$

$r = \frac{t}{c} = 32.86$

For balanced section,

$n = n_1 * d$

$n_1 = \frac{m}{m + r} = \frac{13.33}{13.33 + 32.86} = 0.$

$MOR = \frac{1}{2} b * n * c \left(d - \frac{n}{3}\right)$

Assume, $d/b = 1.5$

$d = 1.5b$

or, $69.33 * 10^6 = \frac{1}{2} * b * 0.288 * 1.5b * 7 \left(1.5b - \frac{0.288 * 1.5b}{3}\right)$

$\therefore b =$ ~~618.58~~ 323.37 mm

≈ 350 mm

$d = 1.5 * 350 = 525$ mm

$n = 0.288 * 525$

$= 151.2$ mm

$MOR = A_{st} * t * (d - n/3)$

$69.33 * 10^6 = A_{st} * 230 * \left(525 - \frac{151.2}{3}\right)$

$A_{st} = 635.1$ mm^2

Adopt ϕ 16mm bar

$\therefore$ A of 1 bar $= \frac{\pi}{4} * 16^2 = 201.06$ mm^2

No. of bar $= \frac{635.1}{201.06} = 3.15 \approx 4$

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Provide 4-16 mm dia bar

From clause 26.5.1.1

$\frac{A_s}{bd} = \frac{0.85}{f_y}$

or, $A_s = \frac{0.85\,bd}{f_y}$

$= \frac{0.85 * 350 * 525}{415}$

$= 376.33\ mm^2 < 804\ mm^2$ OK.

Max. area of tension reinforcement $= 0.04 * \cancel{525}^{350} * 550$

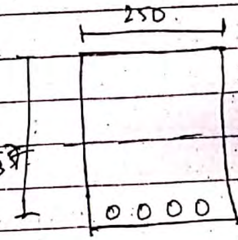
$= 7700\ mm^2$

$> 804\ mm^2$ OK

2.5) $b = 250mm$

$D = 500mm$

$A_{st} = 4 * \frac{\pi}{4} * 16^2 = 804\ mm^2$



$d = 500 - 35 - 8 = \cancel{465\ mm}\ 457mm$

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

$\sigma_{cbc} = \cancel{20}\ 7\ N/mm^2$

$r = \frac{t}{c} = \frac{230}{7} = 32.85$

$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3*7} = 13$

$n = n_1 * d$

$n_1 = \frac{m}{m+r}$

$= \frac{13.33}{13.33 + 32.85}$

$= 0.288$

$n_c = 0.288 * 457 = \cancel{134.22\ mm}\ 131.616\ mm$

~~MOR:~~

Taking moment about N.A,

$250 * n * \frac{n}{2} = 13.33 * 4 * \frac{\pi}{4} * 16^2 * (457 - n)$

$n = 159.66\ mm$

$\because n > n_c$, it is over-reinforced section

$$MOR = \frac{1}{2} * b * n * c * \left(d - \frac{n}{3}\right)$$

$$= \frac{1}{2} * 250 * 159.66 * 7 * \left(457 - \frac{159.66}{3}\right)$$

$$= 56.41 \ KN/m$$

$$\frac{wl^2}{8} = 56.41$$

$$\frac{w * 4^2}{8} = 56.41$$

$$w = 28.205 \ KN/m$$

Self wt of beam = $0.25 * 0.5 * 25 = 3.125 \ KN/m$

Chapter-3 Limit state Method

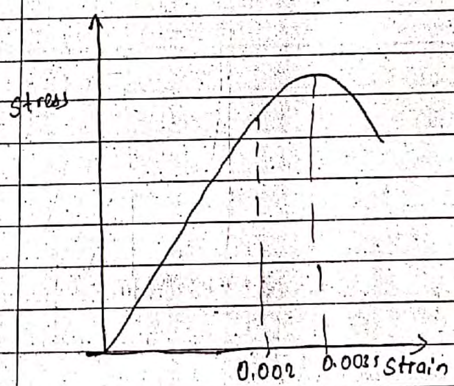
Limit state

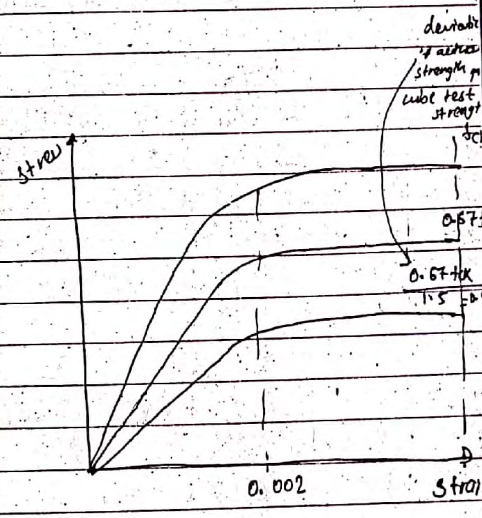
beyond which structure is considered unusable.

i) Limit state of safety / strength

ii) Limit state of serviceability

Property of concrete





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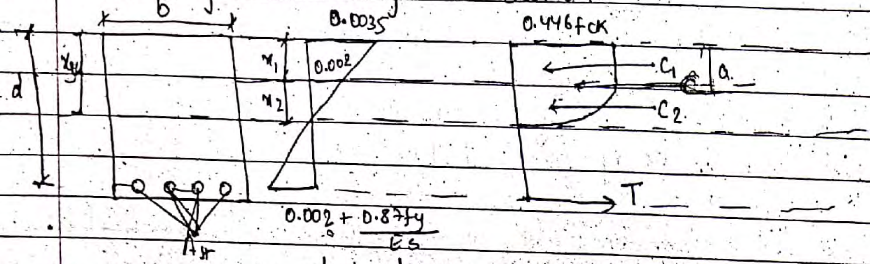
Moment of resistance of the section.



strain diagram   stress diagram

$$\frac{x_2}{x_u} = \frac{0.002}{0.0035} \qquad T = 0.87 f_y * A_{st}$$

$$x_2 = \frac{4}{7} x_u$$

$$x_1 = \frac{3}{7} x_u$$

$$C_1 = b * x_1 * 0.446 f_{ck} = b * \frac{3}{7} * x_u * 0.446 f_{ck} = 0.191 f_{ck} b * x_u$$

$$C_2 = b * x_2 * \frac{2}{3} * (0.446 f_{ck})$$

$$= b * \frac{4}{7} x_u * \frac{2}{3} * (0.446 f_{ck})$$

$$= 0.169 f_{ck} b * x_u$$

Total compressive force $(C) = C_1 + C_2$

$$= 0.191 f_{ck} b * x_u + 0.169 f_{ck} * b * x_u$$

$$= 0.36 f_{ck} b * x_u$$

Here,

$$C = T$$

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

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

$$C_1 * \frac{x_1}{2} + C_2 * \left(x_1 + \frac{3}{8} x_2\right) = C * a$$

$$0.191 f_{ck} b * x_u * \frac{1}{2} \cdot \frac{3}{7} x_u + 0.169 f_{ck} b * x_u \left(\frac{3}{7} x_u + \frac{3}{8} * \frac{4}{7} x_u\right)$$

$$= 0.36 f_{ck} b * x_u * a$$

$$0.0818 x_u + 0.1086 x_u = 0.36 a$$

$$a = \cancel{0.529} x_u$$

$$a = 0.416 x_u \approx 0.42 x_u$$

lever arm $(z) = (d - 0.42 x_u)$

$(MOR)_{concrete} = C * z$

$$\boxed{MOR = 0.36 f_{ck} b * x_u (d - 0.42 x_u)}$$

$(MOR)_{rebar} = T * z$

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

$(MOR)_{concrete} = C * Z$

Limiting depth of N.A

$$\frac{x_{u,lim}}{d - x_{u,lim}} = \frac{0.0035}{0.002 + \frac{0.87 f_y}{E_s}}$$

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

i) for mild-steel

$f_y = 250\ N/mm^2$ Page-70

$$\frac{x_{u,lim}}{(d - x_{u,lim})} = 1.1336$$

$x_{u,lim} = 0.53\, d$

ii) Torsteel (Fe415)

$f_y = 415$

$$\frac{x_{u,lim}}{d} = 0.48$$

iii) TMT (Fe500)

$f_y = 500\ N/mm^2$

$$\frac{x_{u,lim}}{d} = 0.46$$

For Fe250

$MOR = 0.36\ f_{ck} * b * (0.53d) * (d - 0.42 * 0.53 * d)$

$= 0.148\ f_{ck}\, bd^2$

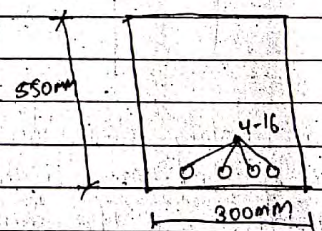
For Fe 415

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

For Fe 500

$MOR = 0.133\ f_{ck}\, bd^2$

Q. Calculate MOR of a singly reinforced RC beam 300*550 mm (eff. depth) reinforced with 4-16 mm dia. bar. material used are M20 concrete and Torsteel bar.



$b = 300mm$

$d = 550mm$

$A_{st} = 4 * \frac{\pi}{4} * 16^2 = 804.25\ mm^2$

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

$f_y = 415\ N/mm^2$

Then,

Depth of N.A

$T = C$

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

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

$$= \frac{0.87 * 415 * 804.25}{0.36 * 20 * 300}$$

$= 134.4\ mm$

Now,

Depth of N.A for balanced section

$$\frac{x_{u,lim}}{d} = 0.48$$

$$\Rightarrow x_{u,lim} = 0.48 * 550 = \cancel{28}\ 264\,mm$$

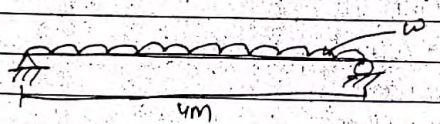
Since,

$x_u \le x_{u,lim}$, it is under-reinforced section.

Hence,

$$MOR = 0.87\, f_y\, A_{st} * (d - 0.42 x_u)$$
$$= 0.87 * 415 * 804.25 * (550 - 0.42 * 134.4)$$
$$= 143.314\ KN\text{-}m$$

Q. Design a simply supported beam having clear span of 4m and carrying an imposed load of 20 KN/m over the span and having 300 mm wide section. Use M20 concrete and TMT bar.

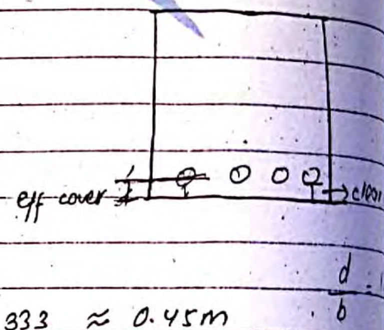


$b = 300mm$

$l = 4m$

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

$f_y = 500\,N/mm^2$



For depth

$$\frac{l}{d} = 10 - 15 \approx 12$$

$$d = \frac{l}{12} = \frac{4}{12} = 0.333 \approx 0.45m$$

RCC

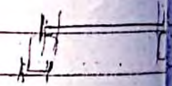
$d = 450\,mm$

eff ~~clear~~ cover = 50mm (Page 47)

Overal depth = 450 + 50 = 500 mm

Effective length

Assume support width = 230 mm



From clause 22.2 a)

i) Clear span + eff depth = 4 + 0.45 = 4.45m

ii) $c/c = 4 + \frac{0.23}{2} + \frac{0.23}{2} = 4.23$

$l_{eff} = 4.23m$

load calculation

i) Imposed load = 20 kN/m

ii) Floor finish = 1 kN/m

iii) self-weight = $\frac{\gamma * b * d * \cancel{l}}{\cancel{l}}$

$= 25 * 0.3 * 0.5$

$= 3.75$ kN/m

Total load = 24.75 kN/m.

Factored load = $1.5 * 24.75$

$= 37.125$ kN/m.

$M_{max} = \frac{wl^2}{8}$

$= \frac{37.125 * 4.23^2}{8}$

$= 83.03$ kN.m

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

$= 0.133 * 20 * 300 * 450^2$

$= 161.6$ kN-m

Since, $M_{max} \le MOR$, it is designed as singly reinforced section.

$A_{st} = ?$ Assume under-reinforced sec

Annex-G 1.1(b), page-96.

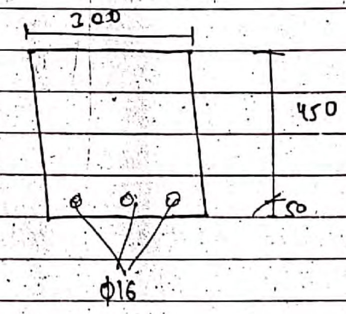
$M_{max} = 0.87\ f_y\ A_{st}\ d * \left(1 - \frac{A_{st} f_y}{bd\ f_{ck}}\right)$

$83.03 * 10^6 = 0.87 * 500 * A_{st} * 450 * \left(1 - \frac{500 * A_{st}}{20 * 300 * 450}\right)$

$A_{st} = 464.04\ mm^2$

Adopt, ϕ 16 mm

$n = \frac{464.04}{201} \approx 3$



Cl 26.5.1.1

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

$(A_{st})_{min} = \frac{0.85 * 300 * 550}{500} = 280.5\ mm^2$

$(A_{st})_{max} = 0.04$ of bD

$= 0.04 * 300 * 500$

$= 7200\ mm^2$

Since, $(A_{st})_{min} < 603.18 > A_{st\,max}$ OK

Q. Calculate MOR of a RC beam 300 * 500 mm overall depth with 5 - 25mm dia bar with having 50mm effective cover. Use M20 concrete and Torsteel bar.

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

$f_y = 415 N/mm^2$

over-reinforced - brittle failure
under- " ductile "

achieve करता है

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

$$= \frac{0.87 * 415 * 5 * \frac{\pi}{4} * 25^2}{0.36 * 20 * 300 * 450}$$

$$= 0.912$$

$x_u = 0.912 * d$
$= 410.25$

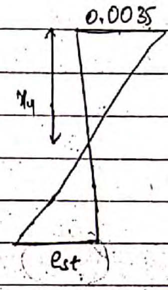
$$\frac{x_{u,lim}}{d} = 0.48 \Rightarrow x_{u,lim} = 0.48 * d$$
$= 216 mm$

Since $x_u > x_{u,lim}$, it is over-reinforced

Trial-I
let us consider,

$$x_u = \frac{216 + 410.25}{2}$$

$= 313.125 mm$



$$e_{st} = \frac{(d - x_u)}{x_u} * 0.0035$$

$= 0.00153$

By interpolation (Table A SP-16, Page - 6)

$f_{st} = 297.23 N/mm^2$

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

$= 337.73 mm$

Trial-II

$$x_u = \frac{313.125 + 337.73}{2} = 325.427 mm$$

$$e_{st} = \frac{(450 - 325.427)}{325.427} * 0.0035$$

$= 0.00134$

$f_{st} = 288.7 N/mm^2$

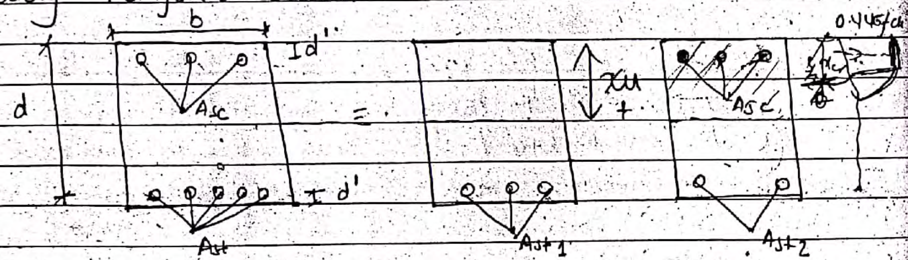
$x_u = 328.04 mm$

$$MOR = 0.36\frac{x_u}{d}\left(1-0.42\frac{x_u}{d}\right)bd^2 f_{ck}$$

$$= 0.36 * \frac{328.04}{450}\left(1-0.42 * \frac{328.04}{450}\right) 300 * 450^2 * 20$$

$$= 221.28 \, KN\,m$$

Doubly - Reinforced section



$T = 0.87 f_y A_{st}$

$C = 0.36 f_{ck}\, b * x_u + (f_{sc} - f_{cc}) A_{sc}$

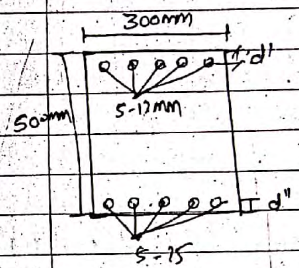
f_{sc} = compressive strength of re bar

f_{cc} = " " concrete $= 0.446 f_{ck}$

$$MOR = 0.36 f_{ck}\, b * x_u * (d - 0.42 x_u) + (f_{sc} - f_{cc}) A_{sc} * (d-d')$$

$$e_{sc} = 0.0035 * \left(\frac{x_u - d'}{x_u}\right)$$

Q. Calculate MOR of a RC beam reinforced with 5-25 mm ϕ bar as tensile reinforcement and 5-12mm ϕ bar as compression reinforcement. The beam is 300 x 500 mm with 25 mm clear cover on both sides. Use M20 concrete and Fe 415 steel.



$b = 300mm$

$D = 500mm$

$A_{st} = 5 \times \pi/4 \times 25^2 = 2454.37 \, mm^2$

$A_{sc} = 5 * \frac{\pi}{4} * 12^2 = 565.49 \, mm^2$

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

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

$d' = 25 + \frac{12}{2} = 31mm$

$d'' = 25 + \frac{25}{2} = 37.5mm$

$d = D - d''$
$= 500 - 37.5$
$= 462.5mm$

$x_{u,lim} = 0.48 * d$
$= 222 \, mm$

$$x_u = \frac{(0.87 * f_y) A_{st} - (f_{sc} - f_{cc}) A_{sc}}{0.36 f_{ck}\, b}$$

$$= \frac{0.87 * 415 * 2454.37 - (0.87 * 415 - \ldots) * 565.49}{0.36 * 20 * 300}$$

$= 318.06mm$

Trial-I

$x_u = \frac{222 + 318.06}{2} = 270\ mm$

Now,

$e_{sc} = 0.0035\left(\frac{x_u - d'}{x_u}\right)$

$= 0.0031$

$e_{st} = 0.0035\left(\frac{d - x_u}{x_u}\right)$

$= 0.0025$

$\frac{e_t}{0.0035} = \frac{d - x_u}{x_u}$

Now, Table-A

$f_{sc} = \frac{360.9 - 351.8}{0.00380 - 0.00276}(0.0031 - 0.00276) + 351.8$

$= 354.77\ N/mm^2$

$f_{st} = \frac{351.8 - 342.8}{0.00276 - 0.00241}(0.0025 - 0.00241) + 342.8$

$= 345.11\ N/mm^2$

$x_u = \frac{(f_{st})A_{st} - (f_{sc} - f_{cc})A_{sc}}{0.36 f_{ck} b}$

$= \frac{345.11 * 2454.37 - (354.77 - 0.446 * 20) * 565.49}{0.36 * 20 * 300}$

$= 301.6mm$

Trial-II

$x_u = \frac{301.6 + 270}{2}$

$= 285.8\ mm$

$e_{sc} = 0.00312$

$e_{st} = 0.0022$

$f_{sc} = 354.95\ N/mm^2$

$f_{st} = \cancel{337.4}\ 335.08\ N/mm^2$

$x_u = \frac{\cancel{354.95}\ 335.08 * 2454.37 - (354.95 - 0.446 * 20) * 565.49}{0.36 * 20 * 300}$

$= 290.15\ mm$

$MOR = 0.36 f_{ck}\, b * x_u (d - 0.42\, x_u) + (f_{sc} - f_{cc}) A_{sc} (d - d')$

$= 0.36 * 20 * 300 * 290.15 (462.5 - 0.42 * 290.15) + (354.95 - 0.446 * 20) * 565.49 (462.5 - 31)$

$= 297.91\ kNm$

Q. Calculate MOR of RC beam 300 * 600 mm reinforced with 4-20 mm dia bar as tensile reinforcement and 3-16 mm dia bar as compression reinforcement. Use effective cover of 50mm on both reinforcement. Use M25 concrete, Fe 415.

$b = 300\ mm$

$D = 600\ mm$

$A_{st} = 4 * \frac{\pi}{4} * 20^2 = 1256.63\ mm^2$

$A_{sc} = 3 * \frac{\pi}{4} * 16^2 = 603.185\ mm^2$

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

$f_y = 415\ N/mm^2$

~~$d' = 50 + \frac{16}{2} = 58\ mm$~~

~~$d'' = 50 + \frac{20}{2} = 60\ mm$~~

$d = D - d''$

$= 600 - 50 = 550\ mm$

$x_{u,lim} = 0.48 * d$

$= 0.48 * 550$

$= 264\ mm$

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

$= \frac{0.87 * 415 * 1256.63 - (\cancel{603.185} - 0.446 * 20) * 603.185}{0.36 * 25 * 300}$

$= 90\ mm$

$e_{sc} = \frac{0.0035 (90 - 50)}{90}$

$= 0.00155$

From table-A

$f_{sc} = \frac{306.7 - 288.7}{0.00163 - 0.00144} (0.00155 - 0.00144) + 288.7$

$= 299.12\ N/mm^2$

$x_u = \frac{\cancel{f_{st}}\ 0.87 f_y A_{st} - (f_{sc} - f_{cc}) A_{sc}}{0.36 f_{ck}\ b}$

$= \frac{0.87 * 415 * 1256.63 - (299.12 - 0.446 * 25) * 603.1}{0.36 * 25 * 300}$

$= 103.708\ mm$

$e_{sc} = 0.0035 \frac{(\cancel{90}\ \cancel{103.208}\ 96.853 - 50)}{\cancel{103.208}\ 96.853} = \cancel{0.0018}\ 0.0017$

$f_{sc} = \frac{324.8 - 306.7}{0.00192 - 0.00163} (0.0017 - 0.00163) + 306.7$

$= \cancel{317.31\ N/mm^2}\ 311.068\ N/mm^2$

$x_u = \frac{0.87 * 415 * 1256.63 - (\cancel{317.31}\ 311.068 - 0.446 * 25) * 603.1}{0.36 * 25 * 300}$

$= 101.037$ ~~N/mm^2~~ mm

MOR =

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Q. Design an RC beam of clear span 5m carrying UDL of 50 kN/m. Use effective cover of 50 mm on either side and material used are M 20 concrete and TMT bar.

Soln,

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

$d = \frac{5}{12} = 0.416 \approx 450$ mm

Take

$\frac{d}{b} = 1.5$

$b = \frac{450}{1.5} = 300$ mm

Given $d' = 50$

$D = d + d' = 450 + 50 = 500$ mm

Effective length (assume 230 mm support width)

i) Clear span + eff. depth $= 5 + 0.45 = 5.45$ m

ii) c/c $= 5 + \frac{0.23}{2} + \frac{0.23}{2} = 5.23$ m

$\therefore l_{eff} = 5.23$ m

Load calculation

i) UDL = 50 kN/m

ii) Self wt $= 25 * 0.3 * 0.5 = 3.75$ kN/m

Total load = 53.75 kN/m

Factored load = 80.625 kN/m

Now,

$M_{max} = \frac{wl^2}{8} = \frac{80.625 * 5.23^2}{8} = 275.67$ kNm

For TMT

$MOR = 0.133\, f_{ck}\, bd^2$

$= 0.133 * 20 * 300 * 450^2$

$= 161.595$ kNm

Since, $M_{max} > MOR$, it is designed as doubly reinforced section

First calculate area of steel (A_{st1}) to balance the concrete section.

$MOR = 0.87 f_y A_{st1} * d * \left(1 - \frac{f_y A_{st1}}{f_{ck}\, bd}\right)$

or, $161.595 * 10^6 = 0.87 * 500 * A_{st1} * 450 \left(1 - \frac{500 * A_{st1}}{20 * 300 * 450}\right)$

$\therefore A_{st1} = 1017.08$ mm^2

Now, remaining moment

$M_R = M_{max} - MOR$

$= 275.67 - 161.595 = 114.075$ kNm

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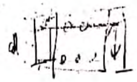
(ii) $0.87 f_y$ A_{st2}



Now, $M_R = 0.87 f_y A_{st2} * (d-d')$

$114.075 * 10^6 = 0.87 * 500 * A_{st2} (450-50)$

$\therefore A_{st2} = 655.6 \ mm^2$

Similarly,

$M_R = (f_{sc} - f_{cc}) A_{sc} (d-d')$

for $\dfrac{d'}{d} = \dfrac{50}{450} = 0.11 \approx 0.1$

$f_{sc} = 412 N/mm^2$ (Page 13 sp 16)

Then,

$114.075 * 10^6 = (412 - 0.446 * 20) A_{sc} (450-50)$

$A_{sc} = 707.52 \ mm^2$

$A_{st} = A_{st1} + A_{st2}$

$= 1672.68 \ mm^2$

$(A_{st})_{min} = \dfrac{0.85 * bd}{f_y}$

$= \dfrac{0.85 * 300 * 450}{500}$

$= 229.5 mm^2$

Available bars = 3, 4, 5, 6, 7, 8, 10, 12, 14, 16, 18, 20, 22, 25, 32, 36, 40

* order

$(A_{st})_{max} = 0.004$ of bD

$= 0.004 * 300 * 550$

$= 6000 \ mm^2$

For tension rebar,

Adopt $\phi = 25mm$

$(A_{st})_{ind} = \dfrac{\pi}{4} 25^2 = 490.87 \ mm^2$

No. of bar $= \dfrac{1672.68}{490.87} = 3.41 \approx 4$

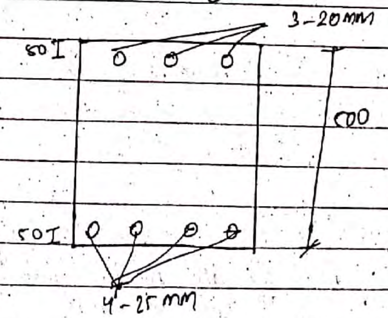
For comp. rebar

$A_{sc} = 707.52 \ mm^2$

Adopt $\phi = 20mm$

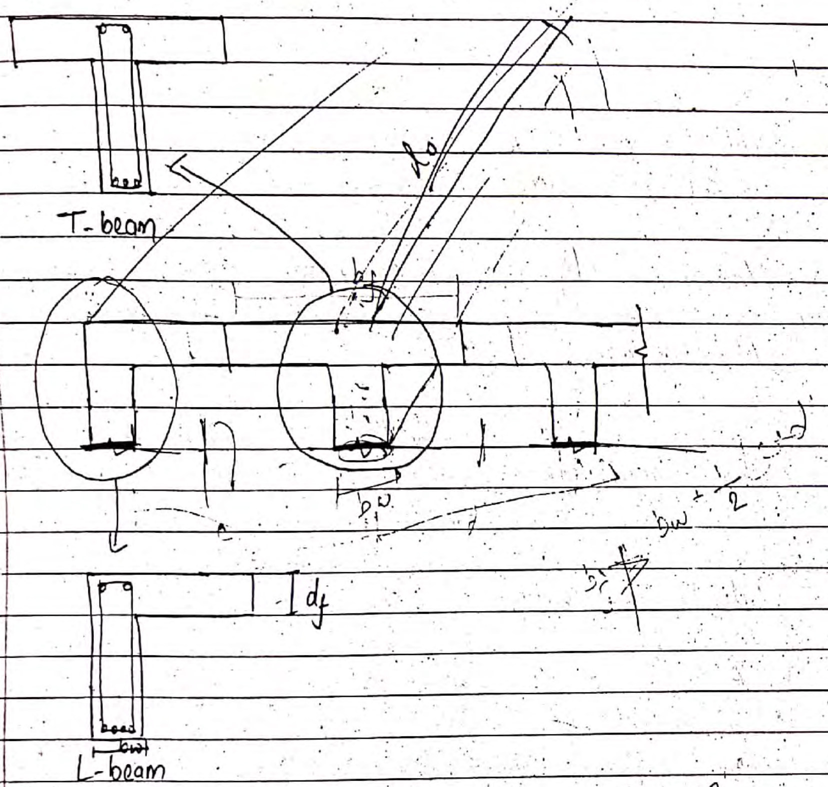
$(A_{sc})_{ind} = \dfrac{\pi}{4} 20^2 = 314.16 \ mm^2$

No. of bars $= \dfrac{707.52}{314.16} = 2.25 \approx 3$



(refer 26.3.2 pg 45)

Flanged beam



pg:- 37

For continuous

$$b_f = \frac{l_o}{6} + b_w + 6D_f \quad (\text{T-beam})$$

$$b_f = \frac{l_o}{12} + b_w + 3D_f \quad (\text{L-beam})$$

b_f = effective width of flange
b_w = Breadth of the web
D_f = Thickness of flange

For isolated,

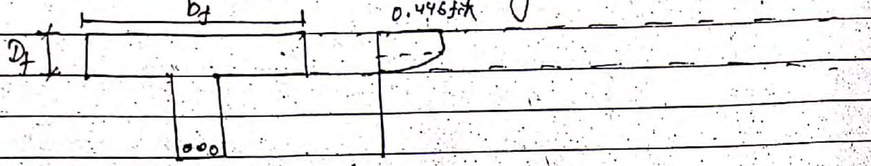
$$b_f = \frac{l_o}{\frac{l_o}{b} + 4} + b_w \quad (\text{T-beam})$$

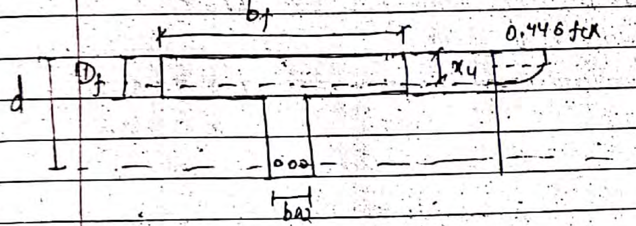
$$b_f = \frac{0.5\, l_o}{\left(\frac{l_o}{b}\right) + 4} + b_w \quad (\text{L-beam})$$

l_o = distance between two points of contraflexure
Taken as centre to centre distance (or effective in case of simply supported beam.
In case of continuous beam, l_o = 0.7 times eff length of beam.

Moment of Resistance (MOR)

Case-I when N.A lies on the flange





Compressive force $(C) = 0.36 f_{ck} b_f * x_u$

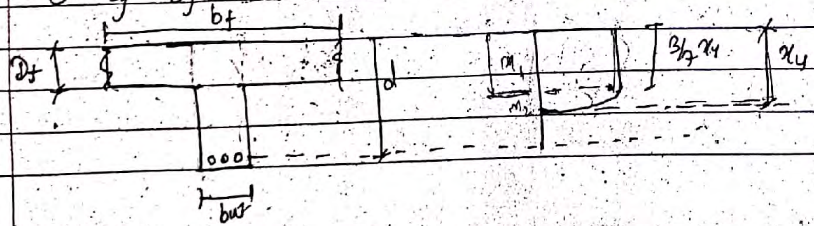
Tensile force $(T) = 0.87 f_y A_{st}$

$MOR = 0.36 f_{ck} b_f * x_u (d - 0.42 x_u)$

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

Case II: when N.A lies outside of flange

ⓐ If $D_f \leq 3/7 x_u$



whitney's stress block

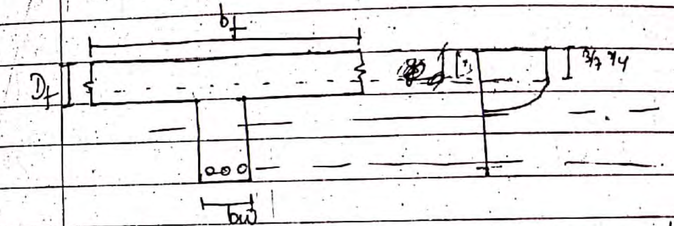
Compressive force $(C) = 0.36 f_{ck} b_w * x_u + 0.446 f_{ck} (b_f - b_w)$

Tensile force $(T) = 0.87 f_y A_{st}$

$MOR = 0.36 f_{ck} b_w * x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) * D_f (d - \frac{D_f}{2})$

$= 0.87 f_y A_{st}$

ⓑ If $D_f > 3/7 x_u$



Depth of rectangular stress block is assumed to be

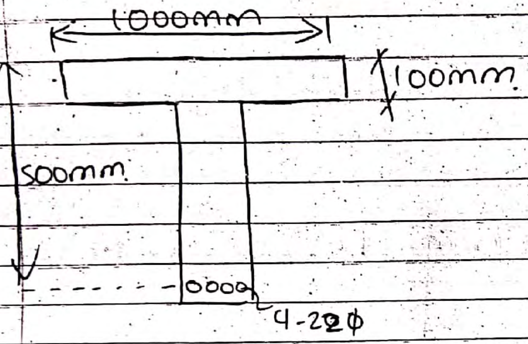
$y_f = 0.15 x_u + 0.65 D_f$

Compressive force $= 0.36 f_{ck} b_w * x_u + 0.446 f_{ck} (b_f - b_w) * y_f$

$T = 0.87 f_y A_{st}$

$MOR = 0.36 f_{ck} b_w * x_u * (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) y_f (d - \frac{y_f}{2})$

Q. A T-beam of flange width 1000mm, flange thickness 100mm, ribbed width 250mm has an effective depth of 500mm. The beam is reinforced with 4 bars of 22mm diameter. Find the ultimate MOR. Use M 20 concrete and Fe 250 steel.



Here,

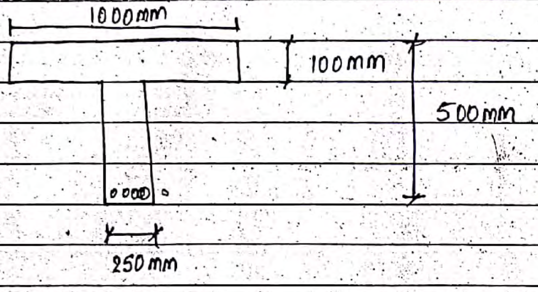
$b_f = 1000mm$ $\quad$ $d = 500mm$

$D_f = 100mm$ $\quad$ $f_{ck} = 20N/mm^2$

$b_w = 250mm$ $\quad$ $f_y = 250N/mm^2$

$A_{st} = 4 \times \frac{\pi}{4} \times 22^2 = 1520.54 mm^2.$

Q. A T-beam of flange width $b_f = 1000mm$, $t_f = 100mm$ ribbed with of 250 mm has an effective depth of 500mm. The beam is reinforced with 4 bars of 22mm ϕ. Find ultimate MOR. Use M20 concrete and Fe 250 steel.



$b_f = 1000mm$

$D_f = 100mm$

$b_w = 250mm$

$d = \cancel{25}\ 500mm$

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

$f_y = 250\ N/mm^2$

$A_{st} = 4 * \frac{\pi}{4} * 22^2 = 1520.53mm^2$

Let us assume $x_u < D_f$

(ie NA lies on flange)

Compressive force $C = 0.36 f_{ck}\ b_f * x_u$

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

$C = T$

or, $0.36 f_{ck}\ b_f * x_u = 0.87 f_y\ A_{st}$

or, $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 * 250 * 1520.53}{0.36 * 20 * 1000} = 45.93\ mm < D_f$ (ie 100mm)

∴ Our assumption is true

Now,

$MOR = 0.36 f_{ck} * b_f * x_u * (d - 0.42 x_u)$

$= 0.36 * 20 * 1000 * 45.93 * (500 - 0.42 * 45.93)$

$= 158.97\ KNm \approx 159\ KNm$

For Fe 250

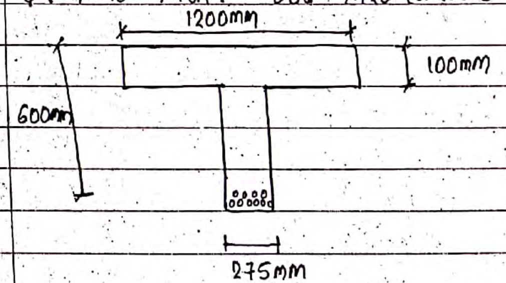
$x_{u\,lim} = 0.53d$

$= 0.53 * 500$

$= 265\ mm$

Since, $x_u < x_{u\,lim}$, it is under reinforced section.

Q. A T-beam with flange width 1200mm, flange thicknes 10 ribbed width of 275mm has an effective depth of 600 mm. an is reinforced with 6 bars with 25 mm ϕ and 4 bars of 16 ϕ. Find MOR. Use M20 concrete.



Here,

$b_f = 1200mm$

$D_f = 100mm$

$d = 600mm$

$b_w = 275mm$

$A_{st} = 6 * \frac{\pi}{4} * 25^2 + 4 * \frac{\pi}{4} * 16^2 = 3749.5mm^2$

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

$f_y = 415 N/mm^2$

let us assume N.A lies on flange (i.e. case I)

$x_u < D_f$

Compressive force, $C = 0.36 f_{ck} b_f * x_u$

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

$T = C$

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

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

$= \frac{0.87 * 415 * 3749.5}{0.36 * 20 * 1200}$

$= 156.68 mm > 100mm$

∴ Our assumption is wrong.

let us assume NA lies on flange and $3/7 x_u \geq D_f$

$C = 0.36 f_{ck} b_w * x_u + 0.446 f_{ck} (b_f - b_w) D_f$

$T = 0.87 f_y A_{st}$

$C = T$

or, $0.36 f_{ck} b_w * x_u + 0.446 f_{ck} (b_f - b_w) D_f = 0.87 f_y A_{st}$

or, $0.36 * 20 * 275 * x_u + 0.446 * 20 (1200 - 275) * 100 = 0.87 * f_y * 415 * 3749.5$

∴ $x_u = 267 mm$

$\frac{3}{7} x_u = 114.43mm > D_f$

∴ Our assumption is correct

$x_{u,lim} = 0.48 d$

$= 0.48 * 600$

$= 288mm > x_u$

∴ Section is under-reinforced.

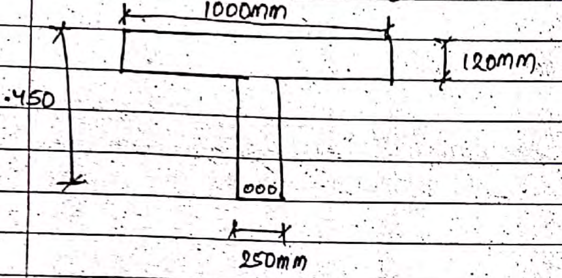
$MOR = 0.36 f_{ck} b_w * x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) * D_f (d - D_f/2)$

$= 711.7 KNm$

(How to check case a and case b)

$\frac{D_f}{d} = 0.167$ ($D_f/d < 0.2$ ~~How~~)

Q. A T-beam of flange width 1000mm, flange thickness 120 mm, ribbed width of 250mm and an effective depth of 450 mm. It is provided with a tension reinforcement of 4909 mm^2. Find ultimate MOR. Use M20 concrete and Fe250 steel.



$f_{ck} = 20\ N/mm^2$
$f_y = 250\ N/mm^2$
$b_f = 1000\ mm$
$b_w = 250mm$
$d = 450\ mm$
$D_f = 120\ mm$

let us assume, N.A lies on flange i.e $x_u < D_f$

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

$C = T$

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

or, $x_u = \dfrac{0.87\ f_y A_{st}}{0.36\ f_{ck}\ b_f}$

$= \dfrac{0.87 * 250 * 4909}{0.36 * 20 * 1000}$

$= 148.3\ mm > D_f$

$\therefore$ Our assumption is wrong.

let us assume N.A lies below flange and $D_f < \frac{3}{7}$

$C = 0.36 f_{ck} b_w x_u + 0.446\ f_{ck}\ (b_f - b_w) * D_f$
$T = 0.87\ f_y\ A_{st}$

$C = T$

or, $0.36\ f_{ck}\ b_w x_u + 0.446 f_{ck} (b_f - b_w) * D_f = 0.87\ f_y A_{st}$

$x_u = 147.17mm$

$\frac{3}{7}\ x_u = 63.07\ mm < D_f$

(assumption is wrong)

$\therefore$ NA lies below flange and $D_f \not> \frac{3}{7}\ x_u$

$y_f = 0.15\ x_u + 0.65\ D_f$

$C = 0.36\ f_{ck}\ b_w\ x_u + 0.446\ f_{ck}\ (b_f - b_w) * y_f$
$T = 0.87\ f_y\ A_{st}$

$C = T$

or, $0.36\ f_{ck}\ b_w\ x_u + 0.446\ f_{ck} (b_f - b_w) * y_f = 0.87\ f_y A_{st}$

$\therefore\ x_u = 194.716\ mm$

$\frac{3}{7}\ x_u = 83\ mm < D_f$

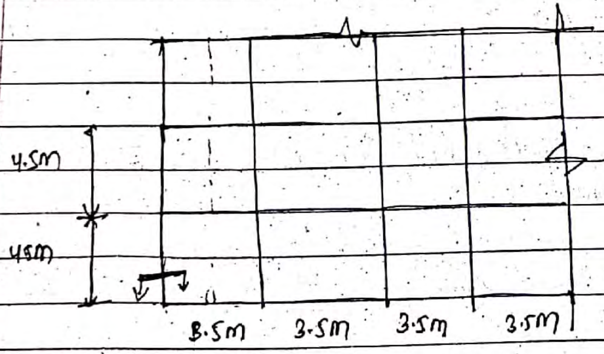
$y_f = 107.207\ mm$

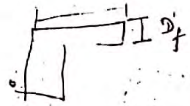
$MOR = 0.36 f_{ck}\, b_w\, x_u (d - 0.42 x_u) + 0.446\, f_{ck} (b_f - b_w) y_f (d - \frac{y_f}{2})$
$= 413.36$ kN m

$\frac{D_f}{d} = 0.267 > 0.2$ ∴ case b
i.e. $D_f > 3/7\, x_u$

$\frac{D_f}{d} < 0.2$	(case II a)
$\frac{D_f}{d} > 0.2$	(case II b)

Q. Design a corner beam of span 4.5m carrying an UDL of 5 kN/m² with adjacent beam spaced at 3.5m c/c. The depth of slab is 150 mm. Use M20 concrete and Fe 500 rod. Assume both ends are fixed.





Assume $\frac{l}{d} = 12 \Rightarrow d = \frac{4.5\,m}{12} = 375$mm

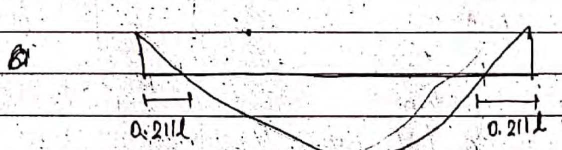
Take $d = 450$mm
$d' = 50$ mm
$D = d + d' = 500$mm

Take $d/b_w = 1.5$
$b_w = \frac{450}{1.5} = 300$mm

$D_f = 150$mm

$l_0 = 4.5 - (0.211 \times l) \times 2$
$= 4.5 - 0.211 \times 4.5 \times 2$
$= 2.601$ m

l_0 = distance between points of zero moments (Page 37)



For fixed beam

(Since it is at edge, we use formula for L beam page-37)

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

$= \frac{2601}{12} + 300 + 3 \times 150 = 966.75$mm $= 967$mm

Load calculation

i) Imposed load = $5 * \frac{3.5}{2} = 8.75$ KN/m

ii) Self wt. of slab = $25 * 0.15 * \frac{3.5}{2} = 6.56$ KN/m

iii) self wt of beam = $25 * 0.3 * (0.5 - 0.15)$
= 2.625 KN/m

Total load = 17.935 KN/m
Factored load = $1.5 * 17.935$ KN/m
= 26.9 KN/m

$M_{max} = \frac{wl^2}{12}$
$= \frac{26.9 * 4.5^2}{12}$
= 45.4 KN.M

Assume N.A lies at flange

$M_{max} = 0.87 f_y A_{st} (d - 0.42 x_u)$

$M_{max} = 0.36 f_{ck} \cdot b * x_u (d - 0.42 x_u)$
$45.4 * 10^6 = 0.36 * 20 * 967 * x_u (450 - 0.42 * x_u)$
$x_u = 14.69$ mm $< D_f$

$45.4 * 10^6 = 0.87 * 500 * A_{st} (450 - 0.42 * 14.69)$
$A_{st} = 235.15$ mm^2
$\frac{\pi}{4} d^2 =$
$d = 17.3 \approx$

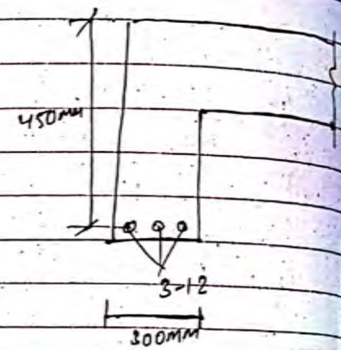
150

3

$(A_{st})_{min} = \frac{0.85\, b_w * d}{f_y}$
$= \frac{0.85 * 300 * 450}{500}$
$= 229.5$ mm^2

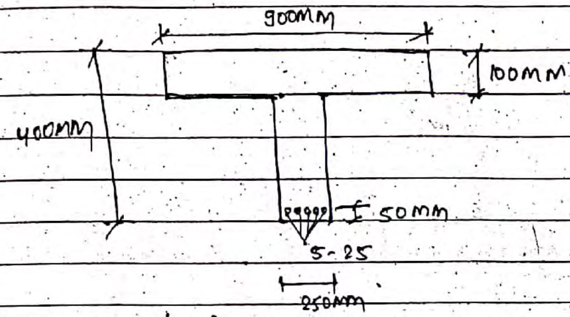
$(A_{st})_{max} = 0.04$ of bD
$= \frac{0.04}{100} * 300 * 500$
$= 6000$ mm^2

Adopt $\phi = 12$ mm
No. of bar = $\frac{235.12}{113.09}$
= 2.1
$\approx$ 3 nos,



NOTE: Incase of over-reinforced section, calculate MOR assuming critical depth

Q. Calculate MOR of the flanged beam as shown



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

$f_y = 415 N/mm^2$

$D = 400 mm$

$d = 350 mm$

$b_w = 250 mm$

$b_f = 900 mm$

$D_f = 100 mm$

$A_{st} = 5 * \frac{\pi}{4} * 25^2 = 2454.37 mm^2$

$x_{u,max} = 0.48 d$

$= 0.48 * 350$

$= 168 mm$

Assuming N.A lies at flange, $x_u < D_f$

$C = 0.36 f_{ck} b_f x_u$

$T = 0.87 f_y A_{st}$

$C = T$

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 * 415 * 2454.37}{0.36 * 20 * 900}$

$= 136.75 mm$

Assumption is wrong.

$\frac{D_f}{d} = \frac{100}{350} = 0.28 > 0.2$

(case II (b))

$y_f = 0.15 x_u + 0.65 D_f$

$\cancel{= 0.15 *}$

$C = T$

$0.36 f_{ck} b_w * x_u + 0.446 f_{ck} (b_f - b_w) y_f = 0.87 f_y A_{st}$

$0.36 * 20 * 250 * x_u + 0.446 * 20 (900 - 250) (0.15 x_u + 0.65 * 100)$

$= 0.87 * 415 * 2454.37$

$x_u = 190.76 \text{mm} > \cancel{x_{u,max}}$ $\cancel{D_f}$

$3/7 x_u < D_f$

$MOR = 0.36 f_{ck} b_w x_{u,max} (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) y_f * (d - \frac{y_f}{2})$

$= 0.36 * 20 * 250 * \overset{168}{\cancel{190.76}} (350 - 0.42 * \overset{168}{\cancel{190.76}}) +$

$0.446 * 20 (900 - 250) * (0.15 * \overset{168}{\cancel{190.76}} + 0.65 * 100)$

$* \left(350 - \frac{0.15 * \overset{168}{\cancel{190.76}} + 0.65 * 100}{2} \right)$

$= 243.96 KNM$

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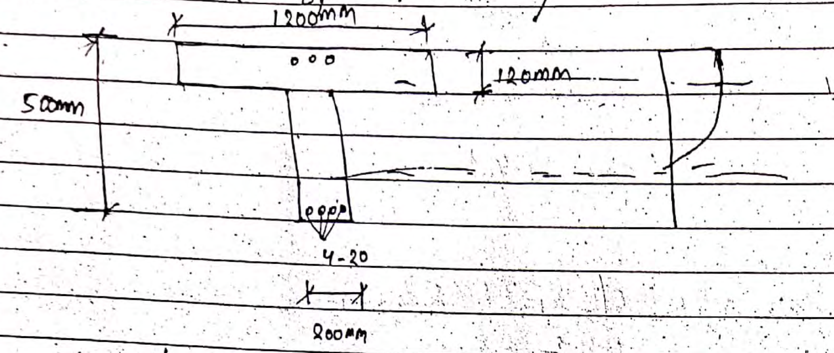
Q. Calculate MOR of following section.



Use $d' = 50mm$ on both sides

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

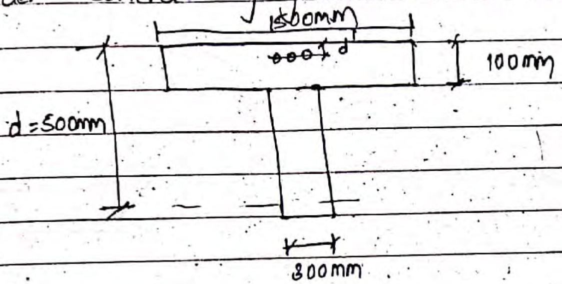
$f_y = 415 N/mm^2$

Calculate the depth of line of action of compressive force.

$$x_m = 0.48 d$$

$$a = 0.36 f_{ck} b_w \cdot x_m \cdot 0.42 \cdot x_m + 0.446 f_{ck} (b_f - b_w) \cdot y_f \cdot \frac{y_f}{2}$$

Q. Design a T-beam section of 1500 mm width of flange 100 mm depth of flange, 300 mm width of web & 500 mm eff. depth which is subjected to a moment of 750 kNm. Consider concrete of grade M20 and steel of grade Fe 415.



$b_f = 1500 mm$ $\quad$ $f_{ck} = 20 N/mm^2$

$b_w = 300 mm$ $\quad$ $f_y = 415 N/mm^2$

$d = 500 mm$

$D_f = 100 mm$

For Fe 415

$x_{u,lim} = 0.48 * d$

$= 0.48 * 500$

$= 240 mm > D_f$

Also,

$3/7 \, x_{u,lim} = 3/7 * 240$

$= 102.857 mm > D_f$

Then,

$MOR = 0.36 f_{ck} * b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) D_f (d - D_f/2)$

or, $= 0.36 * 20 * 300 * 240 (500 - 0.42 * 240) + 0.446 * 20 (1500 - 300) * 100 * (500 - \frac{100}{2})$

$= 688.63 kN.m$

Since, MOR < Mmax, it is designed as doubly reinforced section.

Then,

$MOR = 0.87 f_y A_{st_1} * (d - 0.42 x_u)$

$688.63 \times 10^6 = 0.87 * 415 * A_{st_1} * (500 - 0.42 * 240)$

$A_{st_1} = 4777.77 mm^2$

Remaining moment (M_R)

$= M_{max} - MOR$

$= 750 - 688.63$

$= 61.37 kNm$

let, $d' = 50 mm$

$M_R = 0.87 f_y A_{st_2} (d - d')$

$61.37 * 10^6 = 0.87 * 415 * A_{st_2} * (500 - 50)$

$A_{st_2} = 377.7 mm^2$

Now,

$M_R = (f_{sc} - f_{cc}) A_{sc} (d - d')$ $\longrightarrow$ $61.37 * 10^6 = (351.8 - 0.446 * 20) * A_{sc} * (500 - 50)$

$A_{sc} = 397.7 mm^2$

$e_{sc} = 0.0035 * \left(\frac{x_{u,lim} - d'}{x_{u,lim}}\right)$

$= 0.0035 * \left(\frac{240 - 50}{240}\right)$

$= 0.00277$

From table - 1 (SP-16)

$e_{sc} = 0.00277 \approx 0.00276$

$f_{sc} = \cancel{345}\ 351.8 N/mm^2$

$(A_{st})_{min} = 0.85 \frac{b_w d}{f_y}$

$= \frac{0.85 * 300 * 500}{415}$

$= 307.2 mm^2$

$(A_{st})_{max} = (A_{sc})_{max} = 0.04 * 500 * 300$

$= 6000 mm^2$

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$A_{st} = A_{st1} + A_{st2} = 4777.77 + 377.77$
$= 5155.54\ mm^2$

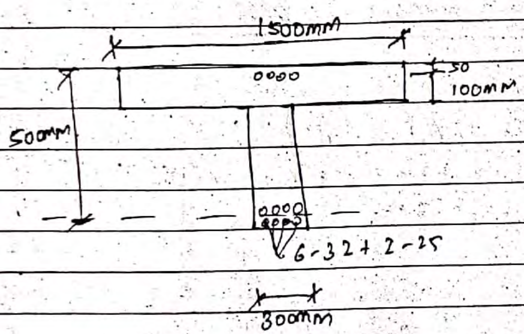
Adopt ϕ = 32mm and 25mm

∴ Provide 6-32 + 2-25 mm on tension zone

for compression

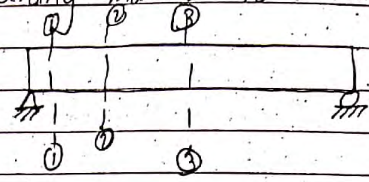
Adopt $\phi = 12mm$

No. of bar $= \frac{397.7}{113.09} = 3.51 \approx 4$ nos.



Shear and Bond

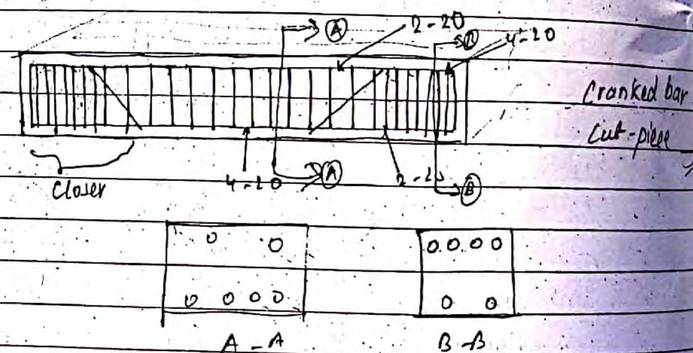
→ At a member, generally shear force is maximum at supp. and Bending moment is maximum at the mid-span



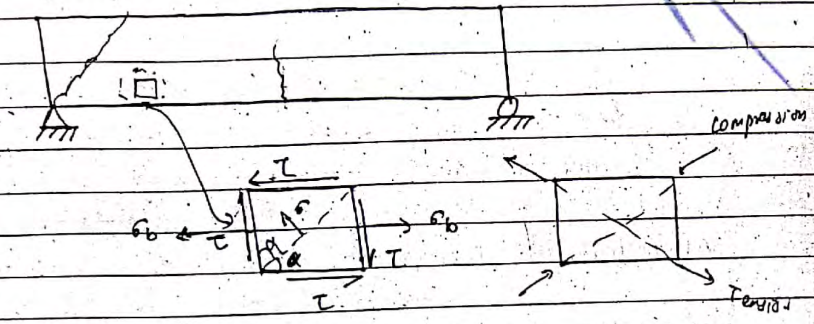
At position ①, shear force is maximum & B.M is negligible

At position ②, SF & B.M co-exist

At position ③, S.F is almost negligible & B.M is max.



Shear failure due to tension



$\tan 2\alpha = \frac{2\tau_v}{\sigma_b}$

At support, $\sigma_b \approx 0$

$\alpha = 45^\circ$

At mid span, $\tau_v \approx 0$

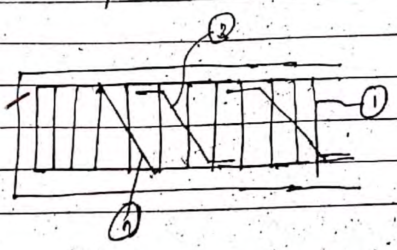
$\alpha \approx 0^\circ$

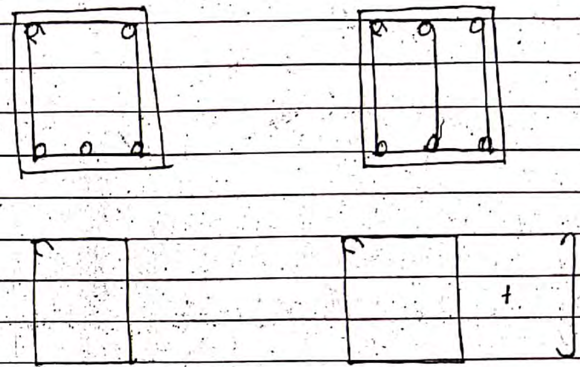
Shear reinforcement

i) Vertical shear bar

ii) Bent-up bar

iii) Inclined stirrups.





2 legged stirrup

2 legged + 1 legged

Nominal Shear Capacity : (τ_c) (40.1) pg. 72

$\tau = \frac{V}{bd}$

τ_c = Nominal design shear strength calculat as per $\frac{A_t}{bd}$ % and concrete grac

τ_c is due to

i) Interlocking of aggregate

ii) Dowel action of ~~ten~~ main reinforcement

iii) Vertical shear carried by section

i) If $\tau \le \tau_c$, provide nominal shear bar.

26.5.1.5 Nominal shear bar represents spacing not ~~less~~ more than
i) $0.75d$ for vertical stirrup and d for inclined stirrup.
ii) 300 mm
& minimum area of steel is given by

$$A_{sv} \ngtr \cancel{\frac{0.85\, f_y A_{sv}}{0.4\, b}} \quad 26.5.1.6$$

$$\frac{A_{sv}}{b\, S_v} \ge \frac{0.4}{0.87 f_y}$$

$$S_v \le \frac{0.87\, f_y A_{sv}}{0.4\, b} \qquad \tau < \tau_c$$

ii) If $\tau_c < \tau \le \tau_{max}$, Design shear bar for

$$V_{us} = (\tau_v - \tau_c)\, bd$$

and spacing is calculated as

$$S_v = \frac{0.87\, f_y A_{sv} d}{V_{us}} \qquad (40.4\ (a)\ \text{page-73}$$

$\le$ Nominal spacing as in (i)

iii) If $\tau > \tau_{max}$, redesign the section
A_{sv} = shear reinforcement area.

ϕ – (M)
Φ – Torsteel

Q. Design shear reinforcement for a section of a beam dimension 300 x 450mm effective depth to carry factored shear of 250 KN. Use M20 concrete and Fe415 provided with 3-20mm bar as tensile reinforcement.

Soln

Given

$b = 300$ mm
$d = 450$ mm
$V_u = 250$ KN
$A_{st} = 3 \times \frac{\pi}{4} \times 20^2$
$= 942.48\ mm^2$
$f_{ck} = 20\ N/mm^2$
$f_y = 415\ N/mm^2$

Shear stress at section $(\tau) = \frac{V_u}{bd} = \frac{250 \times 10^3}{300 \times 450}$

Now,

$$\frac{A_{st}}{bd}\,\% = \frac{942.48}{300 \times 450} \times 100 = 0.698$$

For M20.

$$\tau_c = \frac{0.56 - 0.48}{0.75 - 0.5}(0.698 - 0.5) + 0.48$$
$$= 0.543\ N/mm^2$$

For M20 (Table-20)
$\tau_{max} = 2.8\ N/mm^2$

$\tau_c < \tau \leq \tau_{max}$

$V_{us} = (\tau - \tau_c)\, bd$

$= (1.85 - 0.543) * 300 * 450$

$= 176.445\ KN$

Adopt 8mm dia bar - 2 legged stirrup.

$A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.53\ mm^2$

Then,

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

$= \frac{0.87 * 415 * 100.53 * 450}{176.445 * 10^3}$

$= 92.57\ mm$

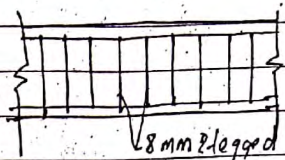
Nominal shear is given by spacing not greater than $\leq 0.75 * d = 0.75 * 450 = 337.5\ mm$

$\leq 300\ mm$

$\leq \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 * 415 * 100.53}{0.4 * 300}$

$= 302.47\ mm$

Provide 8mm 2 legged stirrup @ 90mm c/c



Q.1 A RC beam has an effective depth of 500mm and a breadth of 350mm. It consists of 4-25mm dia-bars. Calculate the shear reinforcement needed for a factored shear force of 350 KN. Adopt $\sigma_{ck} = 20\ N/mm^2$ and $\sigma_{sv} = 415\ N/mm^2$.

$d = 500\ mm$

$b = 350\ mm$

$A_{st} = 4 * \frac{\pi}{4} * 25^2 = 1963.5\ mm^2$

$V_u = 350\ KN$

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

$\sigma_{sv} = 415\ N/mm^2$

Nominal shear stress $(\tau) = \frac{V_u}{bd} = \frac{350 * 10^3}{350 * 500} = 2\ N/mm^2$

Now,

$\frac{A_{st}}{bd} \% = \frac{1963.5}{500 * 350} * 100 = 1.122\ \%$

From Table 19,

$\tau_c = \frac{0.67 - 0.62}{1.25 - 1} * (1.122 - 1) + 0.62 = 0.644\ N/mm^2$

Maximum shear stress in M20, τ_{cmax} = 2.8 N/mm^2

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

$V_{us} = (\tau - \tau_c) b d$

$= (2 - 0.649) * 500 * 350$

$= 238.0$ kN

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

$= \frac{0.87 * 415 * 1963.5 * 500}{238.0}$

=

Adopt 8mm dia bar - 2 legged stirrups,

$A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.5 mm^2$

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

$= \frac{0.87 * 415 * 100.5 * 500}{288 * 10^3}$

$= 76.23$ mm

$= 76$ mm

The code requires that $S_v < 800$mm

$< 0.75 d$ i.e 375mm

Minimum shear reinforcement

$A_{sv} \geq \frac{0.4 b S_v}{0.87 f_y} = \frac{0.4 * 350 * 76}{0.87 * 415} = 29.47 mm^2$

$\therefore 100.51 > 29.47 mm^2$ (OK)

Although all requirement of the code are satisfied by 8mm - 2 legged Fe 415 grade bars @ 76mm c/c, but it suggested that minimum spacing of stirrups be limited to 100mm inorder to permit space for compaction of the concrete.

$\therefore$ Revised area of stirrups

$A_{sv} = \frac{V_{us} \cdot S_v}{0.87 * f_y * d} = \frac{238 * 10^3 * 100}{0.87 * 415 * 500} = 131.8$

Area of one leg $= \frac{131.83}{2} = 65.9 mm^2$

Area of 10mm dia bar $= \frac{\pi}{4} * 10^2 = 78.5 mm^2 > 65.9 mm^2$

$\therefore$ Use 10mm ϕ - 2 legged vertical stirrups @ 100mm

Check:

Min shear reinforcement

$A_{sv} \geq \frac{0.4 b S_v}{0.87 f_y} = \frac{0.4 * 350 * 100}{0.87 * 415} = 88.97 mm^2$

$< 2 * 78.5$

OK

(Page - 73) 40.4 (b) (c)

Shear resistance due to bent-up bar & inclined stirrups

i) If bars are bent-up at one section

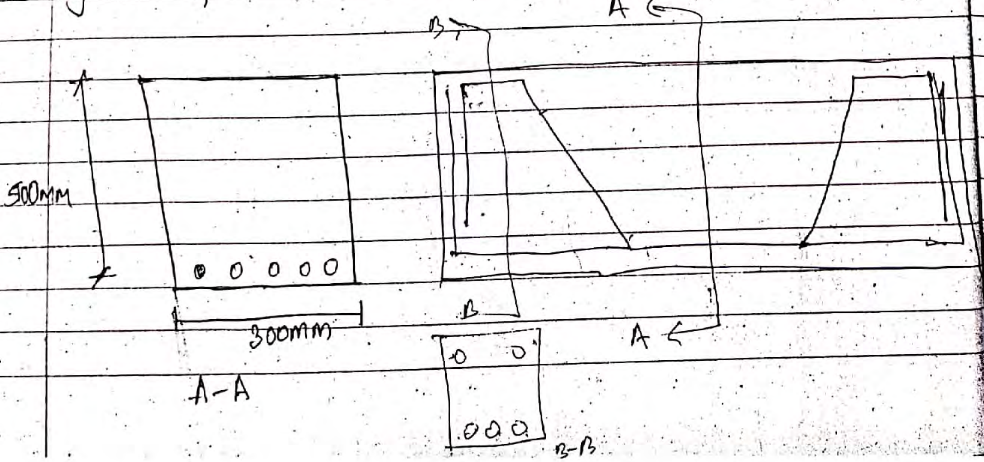
$V_{us} = 0.87 f_y A_{sv} \sin\alpha$

ii) If bars are bent up at different section

$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v} (\sin\alpha + \cos\alpha)$

A_{sv} = area of bent-up bar or inclined stirrup

Q. A rectangular beam section 300 mm wide and 500 mm eff. depth is reinforced with 5 bars of 20mm out of which 2 bars have been bent at 45°. Determine the shear resistance of the bent-up bar and the additional shear reinforcement required if it is subjected to ultimate shear force of 300 KN. Concrete grade of M20 and steel grade of Fe 415.



$V_u = 300 KN$

$A_{st} = 3 * \frac{\pi}{4} * 20^2$

$= 942.47 mm^2$

Then,

$\frac{A_{st}}{bd} * 100\% = 0.628\%$

$\tau_c = \frac{0.56 - 0.48}{0.75 - 0.5}(0.628 - 0.5) + 0.48$

$= 0.521 N/mm^2$

$\tau_c < \tau < \tau_{max}$

$\tau = \frac{V}{bd} = \frac{300 * 10^3}{300 * 500} = 2 N/mm^2$

From Table - 20

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

$\tau - \tau_c = 2 - 0.521 = 1.479 N/mm^2$

For bent-up bar

Shear capacity of bent-up bar = $0.87 f_y A_{sv} \sin\alpha$

$= 0.87 * 415 * 2 * \frac{\pi}{4} * 20^2 * \sin 45°$

$= 160.41 KN$

Also,

$V_{us} = 1.479 * 300 * 500 = 221.85 KN$

Page 73 - (c) bent up bar doesnot carry more than ½ of V_{us}

Hence, Design shear force = $\frac{1}{2} V_{us} = \frac{1}{2} * 221.85 = 110.925 kN$

Adopt 8mm dia bar - 2 legged stirrup

$A_{sv} = 2 * \frac{\pi}{4} 8^2 = 100.53 mm^2$

Then;

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

$= \frac{0.87 * 415 * 100.53 * 500}{110.925 * 10^3}$

$= 163.6 mm$

Nominal shear is given by spacing not greater than

$0.75 * 500$

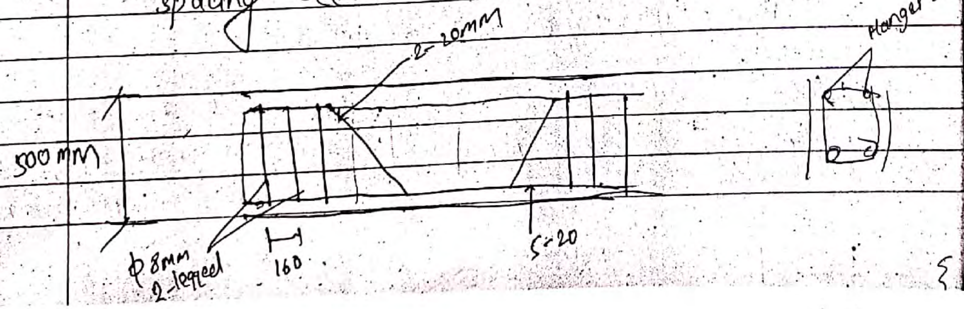
$< = 375 mm$

$\leq \frac{0.87 f_y A_{sv}}{0.4 b}$

$= \frac{0.87 * 415 * 100.53}{0.4 * 300}$

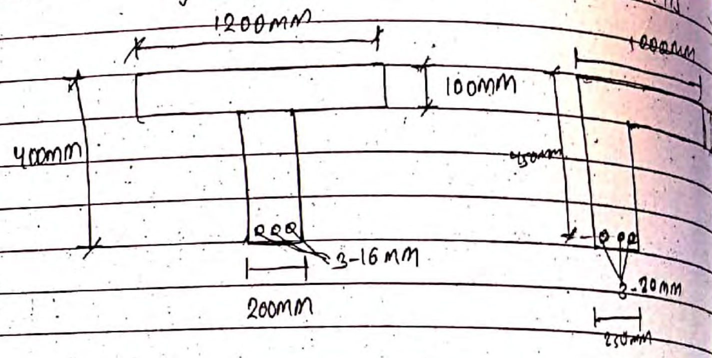
$= 302.47$

∴ Provide 8mm dia bar - 2 legged stirrup @ 160mm spacing c/c.



Q. Design shear reinforcement for following T-beam to factored shear force of 220 KN.

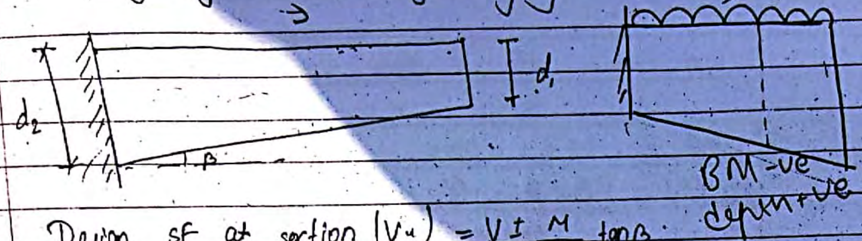
M20 Fe415



Double reinforced → Tension

$V_{max} = 3$

Shear design of the Beam of Varying Depth

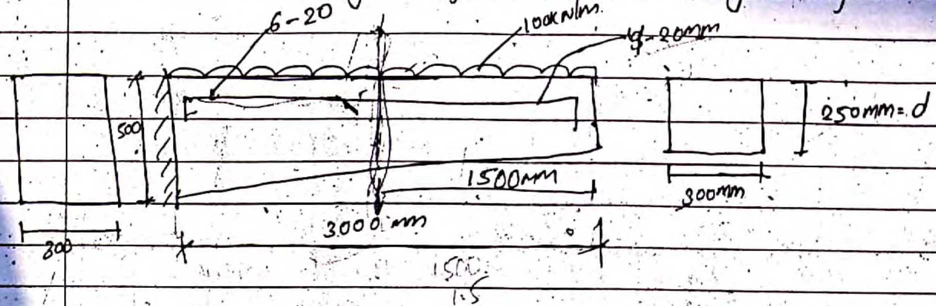


Design SF at section $(V_u) = V \pm \frac{M}{d} \tan\beta$

-ve if BM increases along increase in depth ~~and viceversa~~
+ve BM decreases along increase in depth

V = actual SF at section
M = actual BM at section
d = eff depth

Q. Design shear reinforcement for the RCC beam as shown in fig. Consider concrete grade of 120 and steel grade of Fe415.



Design shear reinforcement for midspan and fixed support.

At mid span

Eff. depth $= \frac{500+250}{2} = 375$ mm

$b = 300$ mm

$A_{st} = 4 * \frac{\pi}{4} * 20^2 = 1256.64 \text{ mm}^2$

$V = 100 * 1.5 = 150$ kN

$\therefore$ Factored $V = 150 * 1.5 = 225$ kN

$M = \left(\frac{100 * 1.5 * 1.5}{2}\right) * \underset{(FOS)}{1.5} = 168.75$ kNM

Design SF at mid-span $(V_{u1}) = V - \frac{M}{d} \tan\beta$

$= \cancel{168.75}$

$= 225 - \frac{168.75}{0.375} \times \frac{250}{3000}$

$= \cancel{224.96\,KN}$ $187.50\,KN$

Adopt 8mm dia bar 2 legged stirrup

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

Then,

$\frac{A_{st}}{bd} \times 100\% = \frac{1256.64}{300 \times 375} \times 100\% = 1.12\%$

$\tau_c = \frac{0.67 - 0.62}{1.25 - 1}(1.12 - 1) + 0.62 = 0.64\,N/mm^2$

For M 20

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

$\tau = \frac{V_u}{bd} = \frac{187.50 \times 10^3}{300 \times 375} = 1.67\,N/mm^2$

$\tau_c < \tau < \tau_{c\,max}$

$V_{us} = (\tau - \tau_c)\,b\,d$

$= (1.67 - 0.64) \times 300 \times$

$= 115.875\,KN$

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

$= \frac{0.87 \times 415 \times 100.53 \times 375}{115875 \times 10^3}$

$= 117.46\,mm$

Nominal shear is given by spacing not greater than

$\leq 0.75 \times d = 0.75 \times 375 = 281.25$; $\leq 300\,mm$

$\leq \frac{0.87 f_y A_{sv}}{0.4 b}$

i.e. $\frac{0.87 \times 415 \times \ldots}{0.4 \times \ldots}$

$= 301.47$

Provide 8mm dia bar 2 legged stirrup @ 100 mm c/c.

https://civinnovate.com/civil-engineering-notes/

$V = 100 * 3 * 1.5 = 450 \text{ KN}$

$M = 100 * 3 * \frac{3}{2} * 1.5 = 675 \text{ KNm}$

At fixed support,

$$V_{us} = V - \frac{M}{d} \tan\beta$$

$$= 450 - \frac{675}{0.500} \left(\frac{250}{3000}\right)$$

$$= 337.5 \text{ KN}$$

$$\frac{A_{st}}{bd} = \frac{6 * \frac{\pi}{4} * 20^2}{500 * 300} \times 100 = 1.26\ \%$$

From Table - 19

$$\tau_c = \frac{0.72 - 0.67}{1.5 - 1.25} * (1.26 - 1.25) + 0.67$$

$$= 0.67 \text{ N/mm}^2$$

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

$$\tau = \frac{V_u}{bd} = \frac{337.5 * 10^3}{500 * 300} = 2.25 \text{ N/mm}^2$$

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

Adopt 8mm dia bar 2 legged stirrup

$A_{sv} = 100.53 \text{ mm}^2$

$$V_{us} = (\tau - \tau_c) b d$$

$$= (2.25 - 0.67) * 500 * 300 = 237 \text{ KN}$$

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

$$= \frac{0.87 * 415 * 100.53 * 500}{237 * 1000}$$

$$= 76.57 \text{ mm}$$

$< 0.75d = 0.75 * 500 = 375 \leq 300 \text{ mm}$

$$\leq \frac{0.87 f_y A_{sv}}{0.4 b}$$

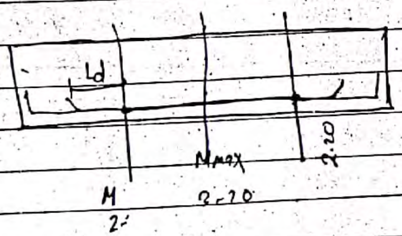
$$\frac{0.87 * 415 * 100.53}{0.4 * 300}$$

302.47 mm

75mm spacing

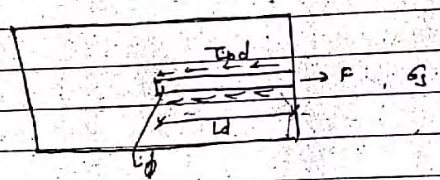
Bond and Development length

When load is applied at the structure, there should be effective transfer of load between used material (such as concrete and steel) in RCC. It is done by ensuring proper bond between concrete and steel. Bond between steel and concrete is due to aggregate inter-locking friction between member and groove on bars.



Development length

It is a length required to be embedded in concrete so that maximum stress of $0.87f_y$ can be achieved at desired section.



Here, let τ_{bd} be the bond stress between concrete and steel.

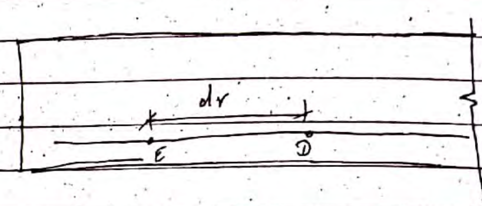
Then,

$$\pi\phi * L_d * \tau_{bd} = \frac{\pi}{4} * \phi^2 * \sigma_s$$

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

Page 42

26.2.1



Here, if T_D and T_E are the tension at D & E then,

$M_D = T_D * Z$ where, Z = lever arm at the section

$M_E = T_E * Z$

Now,

$$M_D - M_E = (T_D - T_E) Z$$

$$\frac{M_D - M_E}{Z} = T_D - T_E$$

$$\frac{dM}{Z} = \pi\phi * dx * \tau_{bd}$$

$$\frac{dM}{dx} * \frac{1}{\pi\phi Z} = \tau_{bd}$$

$$\tau_{bd} = \frac{V}{\pi\phi Z} \quad \text{—— (i)}$$

If there are 'N' no. of bars.

$$\tau_{bd} = \frac{V}{\pi (N\phi) * Z} \quad \text{--- (i)}$$

Also,

$$0.87 f_y A_{st} = (\pi * \phi * l_d * \tau_{bd}) * N$$

$$0.87 f_y A_{st} = \pi * \phi * l_d * \frac{V}{\pi (N\phi) * Z} * N$$

$$l_d = \frac{(0.87 f_y A_{st}) Z}{V}$$

$$\therefore l_d = \frac{M}{V}$$

it is increased by 30% for compression

Hence,

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

finally

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

where, l_0 = anchorage length beyond mid of support

26.2.3.3 (c)

Chapter-5 : Design in Torsion

There are basically two types of Torsion on the section.

1) Primary Torsion :- Designed for this torsion at the section.

ii) Secondary Torsion : Balanced by continuity of structure

IS-code (cl 41) suggest equivalent shear and bending for given torsion rather than torsion itself.

Cl 41.3 suggest equivalent shear due to torsion as

$$V_T = 1.6 \left(\frac{T_u}{b}\right), \quad T_u = \text{factored torsion}, \; b = \text{width of section}$$

and effective shear force $(V_e) = V_u + V_T$

V_u = factored shear force

Cl 41.4 Suggest equivalent B.M for torsion as

$$M_t = T_u \left(\frac{1 + D/b}{1.7}\right)$$

and effective B.M is given by

$$M_e = M_u \pm M_t$$

where,

$$M_{e1} = M_u + M_t$$

and $M_{e2} = M_t - M_u$

If $M_t < M_u$, equivalent longitudinal reinforcement is provided for M_{e1} in tension side.

If $M_t > M_u$, in addition to tension bar for M_{e1}, additional compression bar should be provided for M_{e2}.

for shear

$$\tau_e = \frac{V_e}{bd}$$

i) If $\tau_e < \tau_c$, nominal shear bar is provided and flexural reinforcement for flexural moment is provided.

ii) If $\tau_e > \tau_c$ but $\leq \tau_{max}$, longitudinal reinforcement for equivalent B.M needs to be provided and shear reinforcement is given by (Page no 75)

$$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)}$$

$$S_v = \frac{(0.87 f_y) A_{sv} \; b_1 d_1}{T_u + 0.4 V_u b_1}$$

and area of shear reinforcement should not be less than

$$A_{sv} \geq \frac{(\tau_e - \tau_c) S_v \cdot b}{0.87 f_y}$$

$$S_v \leq \frac{0.87 f_y A_{sv}}{(\tau_e - \tau_c) b}$$

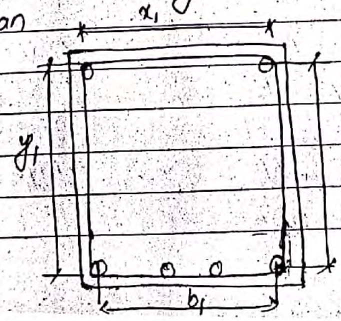
Cl 26.5.1.7 also states that spacing in such case should not be greater than

i) x_1

ii) $\frac{x_1 + y_1}{4}$

iii) 300 mm

iv) 0.75d



b_1 = c/c distance between corner bar along width

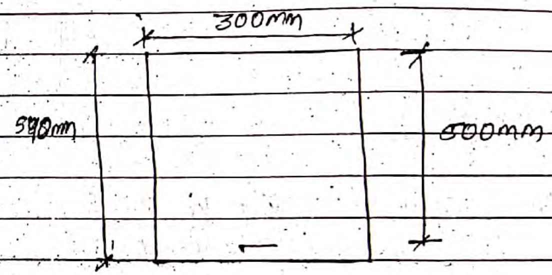
d_1 = " " " depth

x_1 = shorter dimension of stirrup

y_1 = longer dimension of stirrup

Side-face reinforcement (Cl 26.5.1.3)

\# Design a beam with following data.



M_u = 50 KN-m

V_u = 60 KN

T_u = 20 KN-m —— (i)

= 40 KN-m —— (ii)

Use Fe 415 steel and M20 concrete

i) → Soln

Given; M_u = 60 KN-m

V_u = 60 KN

T_u = 20 KN-m

low,

$$V_t = 1.6\left(\frac{T_u}{b}\right)$$

$$= 1.6\left(\frac{20}{0.3}\right)$$

$$= 106.66\,KN$$

$$M_t = T_u\left(\frac{1+D/b}{1.7}\right)$$

$$= 20\left(\frac{1+\frac{0.540}{0.3}}{1.7}\right)$$

$$= \cancel{32.745}\ 32.94\ KN\text{-}m$$

equivalent shear force $(V_e) = V_u + V_t$

$$= 50 + 106.66$$

$$= 156.66\ KN$$

and,

$$\tau_e = \frac{V_e}{bd}$$

$$= \frac{156.66}{300 \times 500}$$

$$= 1.044\ N/mm^2$$

Equivalent moment in the given section

$$M_{e1} = M_t + M_u \quad (\because M_u > M_t)$$

$$= 50 + 32.94$$

$$= 82.94\ KN\text{-}m$$

Since, $M_u > M_t$, provide tension bar for equivalent moment

$$M_{e1} = 0.87\, f_y\, A_{st}\, (d - 0.42\, x_u)$$

Here, for Fe 415

$$x_u = x_{u,lim} = 0.48d$$

$$82.94 = 0.87 \times 415 \times A_{st}\,(500 - 0.42 \times 0.48 \times 500)$$

$$A_{st} = 575.45\,mm^2$$

Adopt $\phi = 16\,mm$

No. of bars $= \frac{575.45}{201.06} = 2.86 \approx 3$ nos.

Also, provide 2-16mm bar as hanger bar.

$$\frac{A_{st}}{bd}\,\% = \frac{3 \times 201.06}{300 \times 500} \times 100 = 0.402\,\%$$

From table-19

$$\tau_c = \frac{0.48 - 0.36}{0.5 - 0.25}\,(0.402 - 0.25) + 0.36$$

$$\tau_c = 0.43\ N/mm^2$$

Since, $\tau_e > \tau_c$

let us assume 8 mm 2-legged stirrup

$$A_{sv} = 100.53\ mm^2$$

$$S_v = \frac{0.87 f_y\ A_{sv}\ b_1\, d_1}{T_u + 0.4\ V_u\, b_1}$$

Let clear cover of 30 mm

effective cover $(d') = 30 + 8 + \frac{16}{2}$

stirrup   longitudinal bar

Effective depth $(d') = 40$ mm

Clear cover for main bar $= 40 - \frac{16}{2} - 8 = 24$ mm

$b_1 = 300 - 2*40 = 220$ mm

$d_1 = 540 - 2*40 = 460$ mm

$x_1 = 300 - 2*24 = 252$ mm

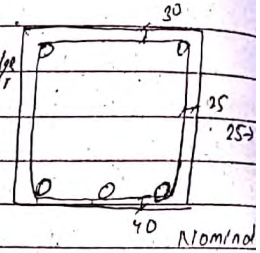
$y_1 = 540 - 2*24 = 492$ mm

$$S_v = \frac{0.87*415*100.53*220*460}{20 + 0.4*50*220}$$

=

Chapter-6 Reinforcement Detailing

Types of cover



1. Clear cover

It is a minimum value of cover provided at all section of member.

2. Nominal cover

It is a minimum value of cover to the edge of reb for major bar (stirrup)

3. Effective cover

It is a cover provided to the centre of reinforcement.

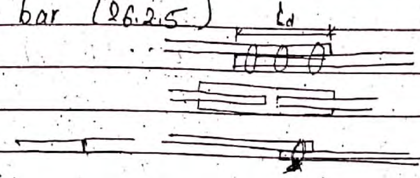
Curtailment of bar (26.2.3 & 26.2.4)

Splicing of bar (26.2.5)

i) Lapping

ii) Conductor

iii) Welding



$$l_d = \frac{\phi \sigma_s}{4 \tau_{bd}}$$

$$= \frac{\phi(0.87*415)}{4*(1.2*1.6)} = 47.01\,\phi \quad (Fe415)$$

$$= \frac{\phi(0.87*500)}{4*1.2*1.6} = 56.64\,\phi$$

Chapter-7 (cl 35.3)

There are basically two aspects of limit state of serviceability.

i) Deflection control

ii) Crack control

Deflection control

Factors affecting deflection

i) Magnitude of load and their distribution

ii) Span and types of support

iii) Cross sectional characteristics of structural member

iv) Type of concrete

v) Stress in steel reinforcement

vi) Amount and extent of cracking

Deflection control is done by two method:

i) Theoretical method

$$\delta \leq \delta_{span}$$

ii) Emperical method (Method of sufficient stiffness) (23.2.1, pg-37)

$$\frac{l}{d} \leq \alpha \beta \gamma \lambda \delta$$

l = effective length

d = " depth

α = Basic value (Boundary Condition)

β = depends upon length of member

γ = depends upon tensile reinforcement %

λ = depends upon compression "

δ = " " shape of member

ii) Crack control

Causes of cracking

i) Due to settlement of plastic concrete

ii) Due to Uneven volumetric change of concrete

iii) due to external load

i) Theoretical method (Annex-F)

$$\Delta_{cr} \leq \Delta_{per}$$

Δ_{cr} = calculated crack width

Δ_{per} = permissible crack

Allowable crack as per ACI

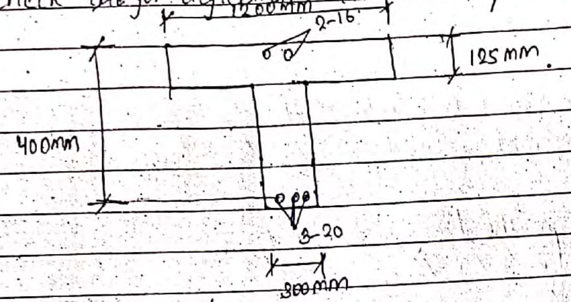
Exposure condition	Permissible crack (mm)
Dry air	0.41
Humid & moist	0.3
Chemically affected	0.18
Sea water	0.15
Water retaining structure	0.1

ii) Emperical method (Rule of proper Detailing)

Crack controlling is done by proper detailing.

slab

Q. Check ~~the for~~ deflection criteria of the following beam.



Length of beam = 5m

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

$f_y = 500 \ N/mm^2$ It is a continous beam.

Here,

$$\frac{l}{d} = \frac{5000}{400} = 12.5$$

Cl 23.2.1 pg - 37

$\alpha = 26$

$\beta = 1$

for γ

$$A_{st} = 3 * \frac{\pi}{4} * 20^2 = 942.47 \ mm^2$$

$$\frac{A_{st}}{bD} \% = \frac{942.47}{300 * 450} = 0.00698 * 100 = 0.698 \%$$

$$f_s = 0.58 f_y * \frac{\text{Area required}}{\text{Area provided}} = 0.58 * 500 * 1 = 290 \ N/mm^2$$

$\therefore \gamma = 0.9$ (From fig:- 4) pg - 38

for λ,

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

$$\frac{A_{sc}}{bD} * 100 = \frac{402.12}{300 * 450} * 100 = 0.298 \%$$

fig - 5 $\lambda = 1.1$

For δ,

$$\frac{b_w}{b_f} = \frac{300}{1200} = 0.25$$

$\therefore \delta = 0.8$

$$\alpha \beta \gamma \lambda \delta = 26 * 1 * 0.9 * 1.1 * 0.8 = 20.6$$

$\therefore \frac{l}{d} < \alpha \beta \gamma \lambda \delta$, OK

Chapter-8

slab

It is a thin sheet / membrane that carries load on it's surface and bends on it's own plane. It is critical under bending moment and depth of slab is governed by shear criteria.

Types of slab

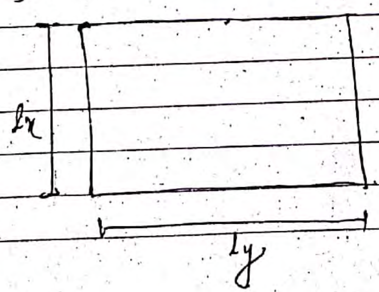
A) As per section
i) Solid slab
ii) Hollow slab
iii) Ribbed slab
iv) Grid slab

B) As per Boundary condition
i) simply supported
ii) Continous
iii) Fixed

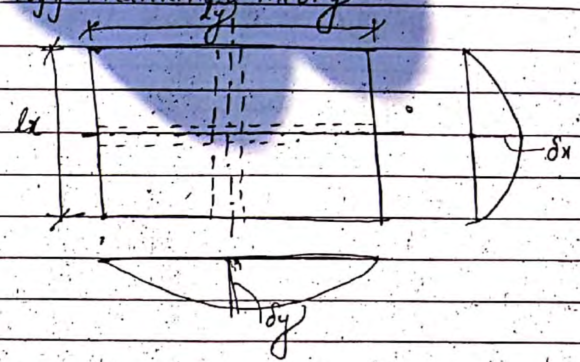
c) As per load distribution
i) One-way slab $\left(\frac{l_y}{l_x} > 2\right)$

ii) Two-way slab $\left(\frac{l_y}{l_x} \leq 2\right)$

where,
l_y = longer span
l_x = Shorter span



Grasshoff Ranking's Theory



Here,

$$\delta_x = \frac{5}{384}\frac{w_x l_x^4}{EI}$$

$$\delta_y = \frac{5}{384}\frac{w_y l_y^4}{EI}$$

Since,

$$\delta_x = \delta_y$$

$$\frac{5}{384}\frac{w_x l_x^4}{EI} = \frac{5}{384}\frac{w_y l_y^4}{EI}$$

$$w_x = w_y\left(\frac{l_y}{l_x}\right)^4 \quad \text{—— (i)}$$

Since, $w = w_x + w_y$ —— (ii)

Using eq^n (i) on eq^n (ii)

$$w = w_y\left(\frac{l_y}{l_x}\right)^4 + w_y$$

If $r = \frac{l_y}{l_x}$

$$\boxed{w_y = w\left(\frac{1}{1+r^4}\right)} \quad \text{—— (iii)}$$

Also, using eqn (iii) in eqn (ii)
we get,

$$w = w_x + w\left(\frac{1}{1+r^4}\right)$$

$$w\left(\frac{r^4}{1+r^4}\right) = w_x$$

$$\boxed{w_x = w\left(\frac{r^4}{1+r^4}\right)} \quad \text{(iv)}$$

Case-I $l_y = l_x$, $r = 1$

$w_x = 0.5w$

$w_y = 0.5w$

Case II If $l_y = 1.5 l_x$

$w_x = 0.835\ w$

$w_y = 0.165\ w$

Case III If $l_y = 2l_x$

$w_x = 0.941w$

$w_y = 0.059w$

Case IV: If $l_y = 3l_x$

$w_x = 0.988w$

$w_y = 0.012w$

For $l_y < 2l_x$, load carried by both axis is comparable. Hence, it is designed as shell section and is termed Two-way slab.

For $l_y > 2l_x$, load carried by longer axis is almost negligible. Hence, such slab are designed as ~~one~~ beam and are termed as one way slabs.

Design of one-way slab

Design steps

1. Calculate effective depth

$$\frac{l}{d} = 25 \quad \text{(simply supported)}$$

$$= 30 \quad \text{(continous slab)}$$

Overall depth (D) $= d + \phi/2$ + clear cover

2. Calculate effective length   Cl. 22.2 pg-34

3. Calculate total load

4. Calculate maximum bending moment and shear force.

5. Check for depth against bending

6. Calculate area of steel and appropriate spacing
Annex-G, 1-1 (b) pg-96

1. Check for shear force : (Cl-40 (40.2) Table-19)

2. Check for deflection. (Cl- ~~22.3~~ 23.2.1, pg-37),

9. Check for development length (cl 26.2.1, pg-42
cl 26.2.3.3 ©, pg-44)

0. Provide distribution bar and reinforcement detailing?

. Design a RC slab over a room 2.5m * 5.4m to carry imposed load of 5kN/m² and floor finish of 1kN/m². Use M20 concrete and torsteel.

l = (always shorter span)
(critical)

Step 1:

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

$$\text{or, } d = \frac{l}{25}$$

$$= \frac{2.5}{25}$$

$$= 0.1\,m$$

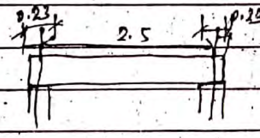
$d \approx 110\,mm$

Adopt $\phi = 10\,mm$

clear cover = 15mm (From Table-16)

$$\text{Overall depth } (D) = 110 + \frac{\phi}{2} + 15$$

$$= 130\,mm$$



Step 2: Along x-axis
(let support width of 230 mm)

i) $c/c = 2.5 + \frac{0.23}{2} + \frac{0.23}{2} = 2.73\,m$

ii) clear span + eff. depth $= 2.5 + 0.11$
$= 2.61\,m$

$l_{ex} = 2.61\,m$

Along Y-axis

i) $c/c = 5.4 + \frac{0.23}{2} + \frac{0.23}{2} = 5.63\,m$

ii) Clear span + eff. depth $= 5.4 + 0.1$
$= 5.5\,m$

$l_{ey} = 5.5\,m$

$\frac{l_{ey}}{l_{ex}} = \frac{5.5}{2.61} = 2.11 \geq 2$, one-way slab

Step-3: load calculation

i) Imposed load = $5 * 1 = 5 KN/m$

ii) Floor finish = $1 * 1 = 1 KN/m$

iii) Self-wt = $25 * 1 * 0.13 = 3.25 KN/m$

Total load = $9.25 KN/m$

Factored load = $1.5 * 9.25$
$= 13.875 KN/m$

Step-4:

$$M_{max} = \frac{wl^2}{8} = \frac{13.875 * (2.61)^2}{8} = 11.815 KN\text{-}m$$

$$V_{max} = \frac{wl}{2} = \frac{13875 * 2.61}{2} = 18.106 KN$$

Step-5:-

$M_{max} = \cancel{11.8}\ 0.138\ f_{ck}\ bd^2$

$11.815 * 10^6 = 0.138 * 20 * 1000 * d_{req}^2$ (for 1m, b = 1m)

$d_{req} = 65.43 mm \leq 110 mm$, OK

Step-6: Annex-G, 1.1(b) pg-96

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

or, $11.82 * 10^6 = 0.87 * 415 * (A_{st}) * 110 * \left(1 - \frac{415 * A_{st}}{20 * 1000 * 110}\right)$

$\therefore A_{st} = 316.5 mm^2$

$(A_{st})_{min} = 0.12\%$ of bD (Cl. 26.5.2.1, pg-48)
$= \frac{0.12}{100} * 1000 * 130$
$= \cancel{13.2}\ 156 mm^2$

Spacing $= \frac{1000}{\left(\frac{316.5}{78.53}\right)} = 248.1 mm$ (Area of 10mm bar = 78.
$\leq 300 mm$
$\leq 3 * d = 3 * 110 = 330 mm$ } 26.6.3

Provide 10mm dia bar @ 150 mm c/c.

(A_{st}) provided $= \frac{1000}{150} * 78.53 = 523.53 mm^2$

Step-7:

$V_{max} = \frac{wl}{2} = \frac{13.875 * 2.61}{2} = 18.11 KN$

always use max

$$\tau = \frac{V}{bd} = \frac{18.11 * 10^3}{1000 * 110} = 0.164 N/mm^2$$

Design shear capacity $= K\tau_c$ (40.2.1)

$= 1.3 * 0.28$

$= 0.36\ N/mm^2 > 0.164\ N/mm^2$ (ok)

Step 8:

$\frac{l}{d} = \frac{2.61}{0.11} = 23.73$

from cl 23.2.1 pg-37

$\alpha = 20$

$\beta = 1$

$\lambda = 1$

$\delta = 1$

For γ:

$\frac{A_{st}}{bD}\ \% = \frac{523.53}{1000 * 130} * 100\ \%$

$= 0.403\ \%$

$f_s = 0.58 * f_y * \frac{(A_{st})\ required}{(A_{st})\ provided} = 0.58 * 415 * \frac{316.5}{523.53}$

$= 145.52\ N/mm^2$

From fig. 4

$\gamma = 2$

$\alpha\beta\gamma\lambda\delta = 20 * 1 * 2 * 1 * 1 = 40$

$\therefore \frac{l}{d} \le \alpha\beta\gamma\lambda\delta$. ok

Step 9: Cl 26.2.1 pg-42

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

$= \frac{10 * (0.87 * 415)}{4 * (1.2 * 1.6)}$

$= 470.11\ mm$

Cl 26.2.3.3 (c)

$M_1 = 0.87 f_y A_{st} d * \left(1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right)$

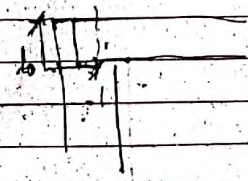
$= 0.87 * 415 * \frac{523.53}{2} * 110 * \left(1 - \frac{415 * \frac{523.53}{2}}{20 * 1000 * 110}\right)$

$= 9.88\ kN\text{-}m$

$\frac{1.3\,M_1}{V} = \frac{1.3 * 9.88 * 10^6}{18.11 * 10^3} = 709.22\ mm$

let $l_0 = 0$ (assume)

$L_d \le \frac{1.3 M_1}{V} + l_0$ ok

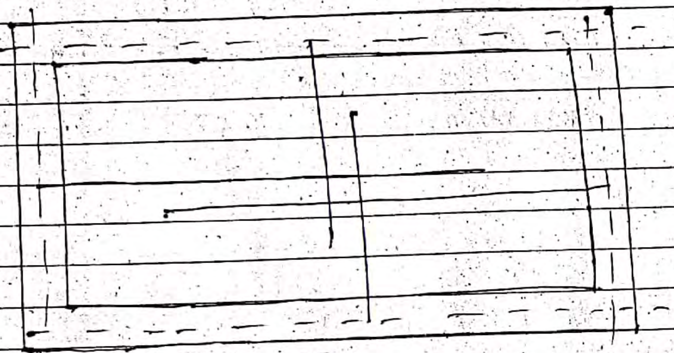


Step 10: Provide minimum (A_{st}) for distribution bar.

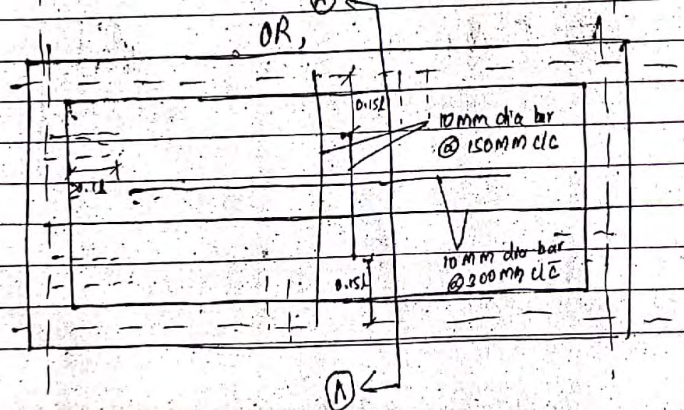
$$\text{Spacing} = \frac{1000}{\left(\frac{156}{78.53}\right)} = 503.4\,mm$$ shall not be more than 450mm or 5 times d

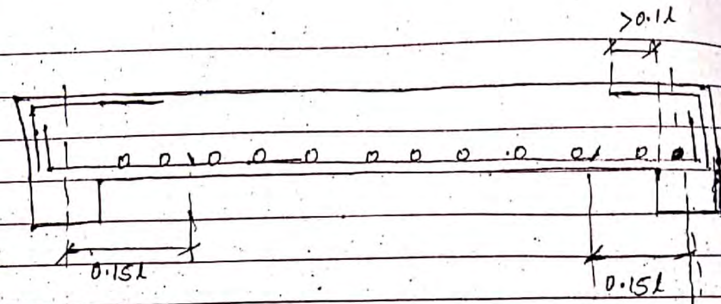
(Cl. 26.3.3 b-2)

Provide 10 mm dia bar @ 300mm c/c along y-axis.

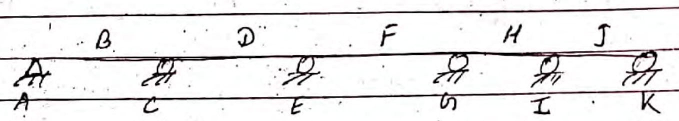


OR,



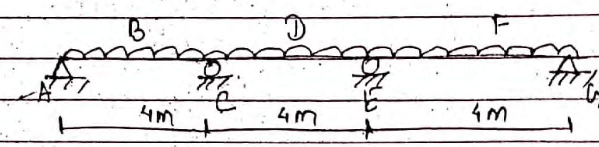


Continous slab



A.K. Jain

Q. Design an RC slab for a continous span of 4m as shown in fig below. It carries UDL (imposed load) of 3 KN/m² and floor finish of 1KN/m². Use M20 concrete and Torsteel.



Soln,

Step 1: $\frac{l}{d} = 25$

$d = \frac{4}{25} = 0.16\,m$

Adopt $\phi = 10\,mm$

Clear cover $= 15\,mm$

$D = 160 + \frac{10}{2} + 15 = 180\,mm$

Step 2:

effective length $(l_{eff}) = 4m$ (Use 22.2 (b))

Step 3:

load calculation

i) Imposed load $= 3 * 1\,KN/m$

ii) Floor finish $= 1 * 1 = 1\,KN/m$

iii) Self-wt $= 25 * 0.18 * 1 = 4.5\,KN$

Total load $= 8.5\,KN/m$

Factored load $= 1.5 * 8.5$

$= 12.75\,KN/m$

Step 4: $M_B = M_F = \frac{1}{12} * (w * l) * l$ (Table-12)

$= \frac{1}{12} * (12.75 * 4) * 4$

$= 17\,KN\text{-}m$

$M_D = \frac{1}{16} * (wl) * l$

$= \frac{1}{16} * (12.75 * 4) * 4 = 12.75\,KN\,m$

$M_C = M_E = -\frac{1}{10} * (w * l) * l$

$= -\frac{1}{10} * (12.75 * 4) * 4$

$= -20.4\,KN\text{-}m$

Shear force

$V_{max} = 0.6 * (w * l)$

$= 0.6 * (12.75 * 4)$

$= 30.6\,KN$

Step 5: ~~(for -ve rebar)~~

$M_{max} = 0.138\, f_{ck}\, bd^2$

$20.4 * 10^6 = 0.138 * 20 * 1000 * d_{req}^2$

$d_{req} = 85.97\,mm \leq 160\,mm$ OK

Step 6: for +ve rebar (tension bar)

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

$17 * 10^6 = 0.87 * 415 * A_{st} * 160 \left(1 - \frac{415 * A_{st}}{20 * 1000 * 160}\right)$

$A_{st} = 306.46\,mm^2$

For -ve bar

$20.4 \times 10^6 = 0.87 \times 415 \times A_{st} \times 160 \times \left(1 - \frac{415 \times (A_{st})}{20 \times 1000 \times 160}\right)$

$(A_{st})^- = 370.99 \ mm^2$

$(A_{st})_{min} = 0.12\%$ of bD

$= \frac{0.12}{100} \times 1000 \times 180$

$= 216 \ mm^2$

$(Spacing)^+ = \frac{1000}{\left(\frac{306.46}{78.53}\right)} = 256.25 \ mm$

$(Spacing)^- = \frac{1000}{\left(\frac{370.99}{78.53}\right)} = 211.68 \ mm$

Provide 10 mm dia bar @ 125 mm c/c at support

Provide " " @ 150 " " mid span

$(A_{st})^+_{provided} = \frac{1000}{150} \times 78.53$

$= 523.53 \ mm^2$

Step 7:

$V_{max} = 30.6 \ kN$

$\tau_{max} = \frac{V_{max}}{bd}$

$= \frac{30.6 \times 10^3}{1000 \times 160}$

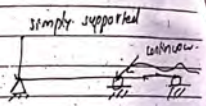
$= 0.19 \ N/mm^2 \leq 0.28 \ N/mm^2$ (OK)

Step 8:

$\frac{l}{d} = \frac{4000}{160} = 25$

Step 9:

$\alpha = \frac{20+26}{2}$



(Pg-37)

$\beta = 1$

$\lambda = 1$

$\delta = 1$

for γ

$\frac{A_{st}}{bD} \% = \frac{523.53}{1000 \times 180} \times 100 = 0.291 \%$

$f_s = 0.58 \times f_y \frac{(A_{st})_{req}}{(A_{st})_{provided}} = 0.58 \times 415 \times \frac{306.46}{523.53}$

$= 140.87 \ N/mm^2$

From fig:- 4
$\gamma = 2$

$\alpha\beta\gamma\lambda\delta = 23 * 1 * 2 * 1 * 1$
$= 46$

$\frac{l}{d} \leq \alpha\beta\gamma\lambda\delta$ (ok)

step 9:

$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} = \frac{10 * (0.87 * 415)}{4 * (1.2 * 1.6)}$
$= 470.11 mm$

$M_1 = 0.87 f_y A_{st} * d * \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$

$= 0.87 * 415 * \left(\frac{523.53}{2}\right) * 160 * \left(1 - \frac{415 * \frac{523.53}{2}}{20 * 1000 * 160}\right)$

$= \cancel{10.04}$ 14.61 KN-m

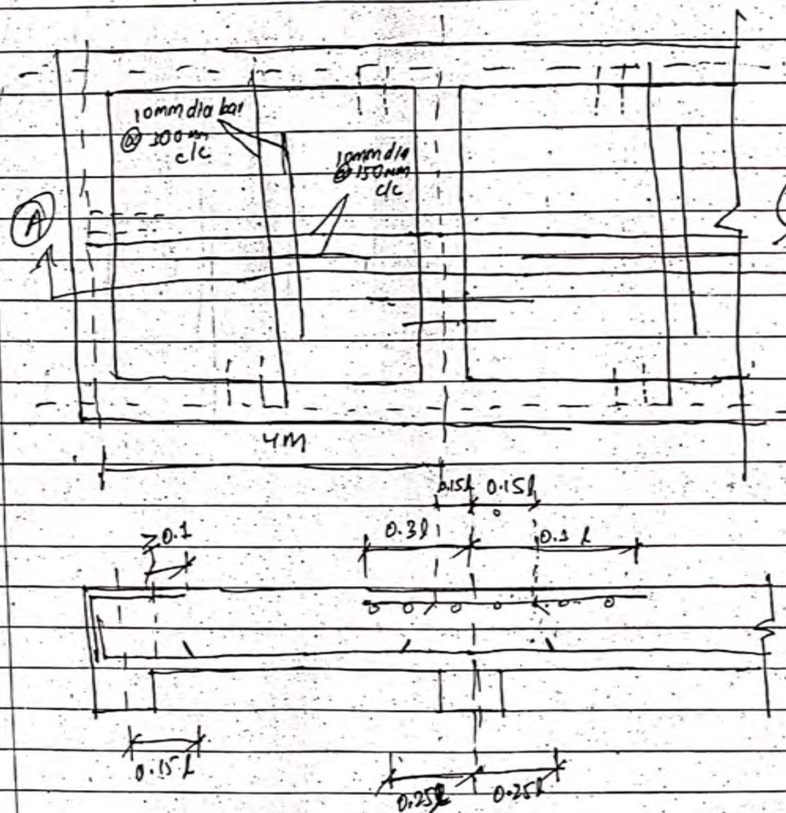
$1.3 \frac{M_1}{V} = \frac{1.3 * 14.61 * 10^6}{30.6 * 10^3}$
$= 620.7 MM$

Assume, $l_0 = 0$
$L_d \leq 1.3 \frac{M_1}{V} + l_0$ (ok)

Step 10 for distribution bar

Spacing $= \frac{1000}{\frac{216}{78.52}} = 363.56 mm$

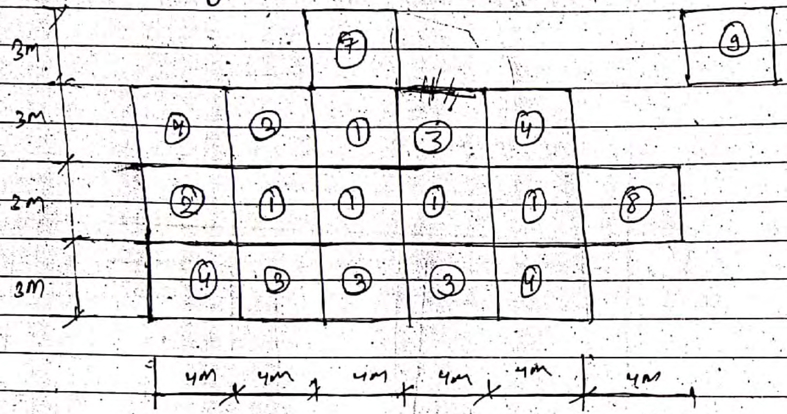
Provide 10mm dia bar @ 300mm as distribution bar.



Two-way slab

If $\frac{l_y}{l_x} \leq 2$, it is known as two-way slabs.

Types of two-way slab



$M_x = \alpha_x w l_x^2$

$M_y = \alpha_y w l_x^2$

Design steps

1. Calculate eff depth

$\frac{L}{d} = 28$ (simply supported)

$\phantom{\frac{L}{d}} = 32$ (continous)

2. Calculate eff length (Cl.22.2, pg.-34)

3. Calculate total load.

4. Calculate Mmax,

$M_x = \alpha_x w l_x^2$

$M_y = \alpha_y w l_x^2$

5. Check for depth against bending

6. Calculate area of steel

7. Check for shear force

8. Check for deflection

9. Check for development length

10. Provide torsion bar at discontinous corner

11. Reinforcement detailing

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