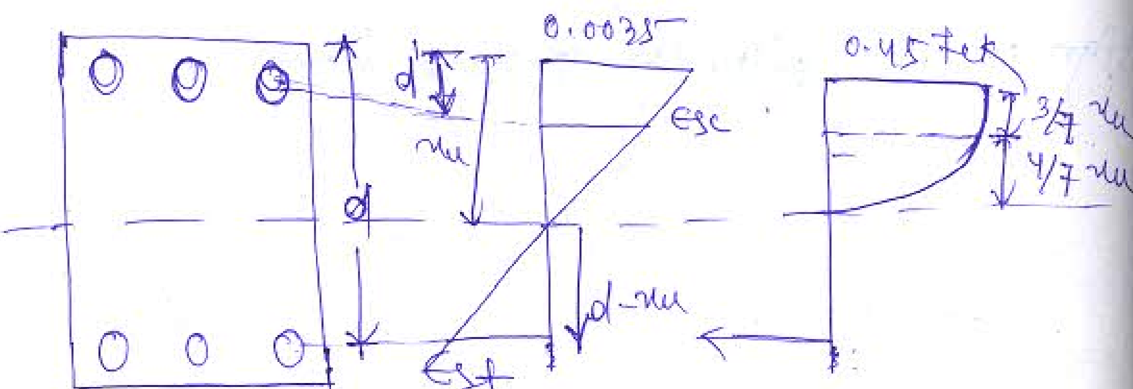
Date - 05/02/2019

Design of Doubly reinforced beam :-

When the size of the beam is restricted & the beam has to ~~resist~~ restrict ~~more~~ moment more than the moment of resistance of the balanced section then we need a doubly reinforced beam.



$$\frac{\epsilon_{st}}{0.0035} = \frac{d - x_u}{x_u}$$

$$\Rightarrow \epsilon_{st} = 0.0035\left(\frac{d}{x_u} - 1\right)$$

$$\frac{\epsilon_{sc}}{0.0035} = \frac{x_u - d'}{x_u}$$

$$\Rightarrow \epsilon_{sc} = 0.0035\left(1 - \frac{d'}{x_u}\right)$$

$$C = T$$

$$0.36 f_{ck} x_u b - 0.45 f_{ck} A_{sc} + A_{sc} f_{sc} = f_{st} A_{st}$$

$$f_{sc} = f_{st} = 0.87 f_y .$$

$0.36 f_{ck} x_{ub} + A_{sc}(f_{sc} - 0.45 f_{ck}) = f_{st} A_{st}$.

M.O.R for compression :-

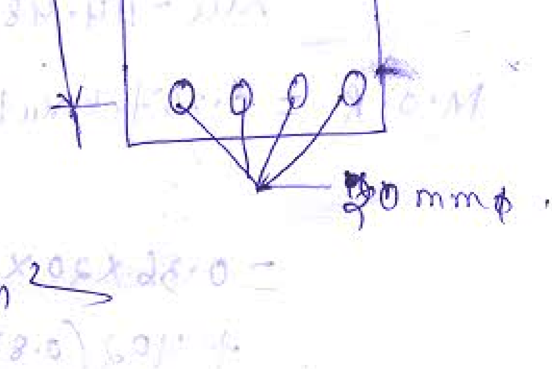
M.O.R $= 0.36 f_{ck} x_{ub}(d - 0.42 x_u) + A_{sc}(f_{sc} - 0.45 f_{ck})(d - d')$

HW
Page - 6.5, 6, 7, 8, 9

Date - 07/02/2019

Q:- Find the factored moment of resistance of a beam section 230mm wide & 460mm effective depth with two 16mm dia bar as compression reinforcement ~~& the~~ at an effective cover of 40mm & four 20mm diameter bar as tension reinforcement. Use M20 grade concrete & mild steel reinforcement.

Soln



$b = 230\,mm$

$d = 460\,mm$

$d' = 40\,mm$

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

$= 402.12\,mm^2 \simeq 402\,mm^2$

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

$\simeq 1256.63\,mm^2$

$\simeq 1257\,mm^2$

$f_{ck} = 20\,MPa$

$f_y = 250\,MPa$

$C = T$

$0.36 f_{ck} x_u b + A_{sc}(f_{sc} - 0.45 f_{ck}) = 0.87$

$\Rightarrow x_u = \dfrac{0.87 f_y A_{st} - A_{sc}(f_{sc} - 0.45 f_{ck})}{0.36 f_{ck} b}$

$= \dfrac{0.87 \times 250 \times 1257 - 402(0.87 \times 250 - 0.45 \times 20)}{0.36 \times 20 \times 230}$

$= 114.48$ mm.

$\dfrac{x_{umax}}{d} = 0.53$ (Page - 70)

$\Rightarrow x_{umax} = 0.53 \times d$

$= 0.53 \times 460$

$= 243.8$ mm.

$x_u < x_{umax}$

So, the section is under reinforced.

$x_u = 114.48$ mm.

$M.O.R = 0.36 f_{ck} x_u b (d - 0.42 x_u) + A_{sc}(f_{sc} - 0.45 \ldots)(d - \ldots)$

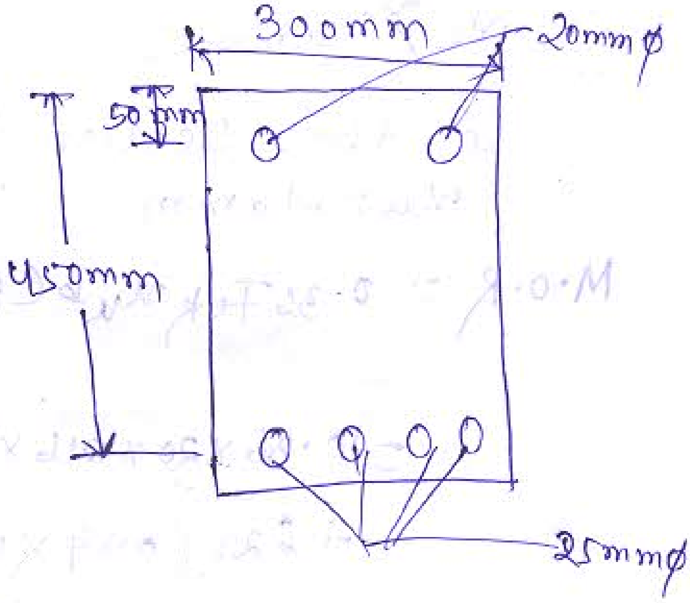
$= 0.36 \times 20 \times 114.48 \times 230 (460 - 0.42 \times 114.48)$
$+ 402(0.87 \times 250 - 0.45 \times 20)(460 - 40)$

$= 113294168.9$ N mm

$= 113.29$ kNm.

Q:- Find out the factored moment of resistance of a beam having 300mm wide & 450mm effective depth reinforced with two 20mm diameter bar as a ~~tension~~ compression reinforcement ~~the materials are~~ at an effective cover of 50mm & four 25mm dia bar as tension reinforcement, the material are M_{20} & HYSD bar.

Sol^n)



$b = 300mm$

$d = 450mm$

$d' = 50mm$

$f_{ck} = 20 MPa$

$f_y = 415 MPa$

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

$= 628.31 mm^2$

$\simeq 628 mm^2$

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

$= 1963.49 mm^2$

$= 1964 mm^2$

$C = T$

$0.36 f_{ck} x_u b + A_{sc}(f_{sc} - 0.45 f_{ck}) = 0.87 f_y A_{st}$

$\Rightarrow x_u = \dfrac{0.87 f_y A_{st} - A_{sc}(f_{sc} - 0.45 f_{ck})}{0.36 f_{ck} b}$

$= \dfrac{0.87 \times 415 \times 1964 - 628(0.87 \times 415 - 0.45 \times 20)}{0.36 \times 20 \times 300}$

$= 225.93$

$\simeq 226 mm$

$$\frac{x_{umax}}{d} = 0.48$$

$$\Rightarrow x_{umax} = 0.48 \times d$$

$$= 0.48 \times 450$$

$$= 216\,mm$$

$$x_u > x_{umax}$$

So, the section is over reinforced.

$$x_u = 216\,mm$$

$$M.O.R = 0.36 f_{ck} x_u b (d - 0.42 x_u) + A_{sc}(f_{sc} - 0.45 \ldots)(d - d')$$

$$= 0.36 \times 20 \times 216 \times 300 (450 - 0.42 \times 216) + 628 (0.87 \times 415 - 0.45 \times 20)(450 - 50)$$

$$= 256060636.8\,Nmm$$

$$= 256.06\,kNm$$

Date - 08/02/2019

$$\cancel{(M_u)_{lim}}$$

$$M_u = (M_u)_{lim} + M_{u2}$$ (Page-96)

$$\Rightarrow M_{u2} = M_u - (M_u)_{lim}$$

$$\boxed{M_{u2} = f_{sc} A_{sc} (d - d')}$$

Q:- Design the doubly reinforced rectangular beam of size 230mm wide 500mm effective depth is subjected to a factored moment 200 KNm. Find the reinforcement for flexure. The materials are M_{20} grade concrete & HYSD bar of Fe415.

Sol^n

Given data,

$b = 230mm$

$d = 500mm$

$M_u = 200\ KNm$

$f_{ck} = 20 MPa$

$f_y = 415 MPa$

Let us assume $d' = 50mm$

$(M_u)_{lim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 230 \times 500^2$

$= 158.7\ KNm$

$M_{u2} = M_u - (M_u)_{lim}$

$= 200 - 158.7$

$= 41.3\ KNm = f_{sc} A_{sc} (d - d')$

$41.3 = 0.87 f_y A_{sc} (d - d')$

$\Rightarrow 41.3 = 0.87 \times 415 \times A_{sc} (500 - 50)$

$\Rightarrow A_{sc} = \dfrac{41.3 \times 10^6}{0.87 \times 415 (500 - 50)}$

$= 254.196\ mm^2$

$\simeq 260\ mm^2$

Let us provide 16 mm dia

$n \times \frac{\pi}{4} \times 16^2 = 260$

$\Rightarrow n = 260 \times \frac{4}{\pi} \times \frac{1}{16^2}$

$= 1.29$

$\simeq 2 \text{ nos}$.

Let us provide 2 - 16 mm dia bar.

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

$= 402.12 \text{ mm}^2$

$\simeq 402 \text{ mm}^2$

$(A_{st})_2 = \frac{f_{sc} A_{sc}}{0.87 f_y}$

$= \frac{0.87 f_y A_{sc}}{0.87 f_y}$

$= A_{sc} = 260 \text{ mm}^2$.

$(M_u)_{lim} = 0.87 f_y (A_{st})_1 (d - 0.42 (x_u)_{lim})$

$\frac{(x_u)_{lim}}{d} = 0.48$

$\Rightarrow (x_u)_{lim} = 0.48 \times 500$

$= 240 \text{ mm}$.

$\Rightarrow 158.7 \times 10^6 = 0.87 \times 415 (A_{st})_1 (500 - 0.42 \times$

$\Rightarrow (A_{st})_1 = \frac{158.7 \times 10^6}{0.87 \times 415 \times (500 - 0.42 \times 240)}$

$= 1101.08$

$\simeq 1110 \text{ mm}^2$

$A_{st} = (A_{st})_1 + (A_{st})_2$

$= 1110 + \cancel{260}\ 402$

$= \cancel{1370}\ mm^2$.
1512

Let us use 20 mm dia bar,

$n \times \frac{\pi}{4} \times 20^2 = \cancel{1370}\ 1512$

$\Rightarrow n = \cancel{1370}\ 1512 \times \frac{4}{\pi} \times \frac{1}{20^2}$

$= 4.81$

$\simeq 5$ no.

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

$= 1570.79\ mm^2$

$\simeq 1570\ mm^2$.

$C = T$

$0.36 f_{ck} x_u b + A_{sc}(f_{sc} - 0.45 f_{ck}) = 0.87 f_y A_{st}$

$\Rightarrow x_u = \frac{0.87 f_y A_{st} - A_{sc}(f_{sc} - 0.45 f_{ck})}{0.36 \times f_{ck} b}$

$= \frac{0.87 \times 415 \times 1570 - 402(0.87 \times 415 - 0.45 \times 20)}{0.36 \times 20 \times 230}$

$= 256.84$

$x_u > x_{u\,lim}$

The section is over reinforced, Let us assume 4 nos. of 16 mm ϕ bar at compression side.

$$A_{sc} = 4 \times \frac{\pi}{4} \times 16^2$$
$$= 804.24 \text{ mm}^2$$
$$\simeq 804 \text{ mm}^2$$

$$0.36 f_{ck} x_u b + A_{sc}(f_{sc} - 0.45 f_{ck}) = 0.87 f_y A_{st}$$

$$\Rightarrow x_u = \frac{0.87 f_y A_{st} - A_{sc}(f_{sc} - 0.45 f_{ck})}{0.36 \times f_{ck} \times b}$$

$$= \frac{0.87 \times 415 \times 1570 - 804(0.87 \times 415 - 0.45 \times 20)}{0.36 \times 20 \times 230}$$

$$= 171.37 \text{ mm}$$

$x < (x_u)_{lim}$,

So, the section is under reinforced

Flanged beam:—

(page – 37 ~~3~~ 23.1.2)

(a) For T-beams, $b_f = \frac{l_o}{6} + b_w + 6D_f$

(b) For L-beams, $b_f = \frac{l_o}{12} + b_w + 3D_f$

— For monolythically casted beam.

For isolated beams,

T-beam, $b_f = \dfrac{l_o}{\left(\frac{l_o}{b}\right)+4} + b_w$

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

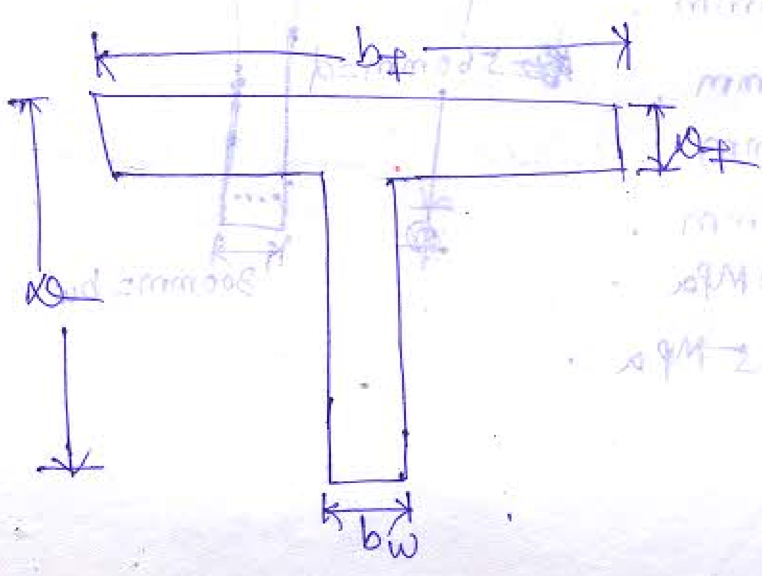
Where, b_f = effective width of flange.

• l_o = Distance between points of zero moments.

b_w = breadth of the web.

D_f = thickness of the flange.

b = Actual width of the flange.



$C = 0.36 f_{ck} \, b_f \, D_f$.

$T = 0.87 f_y A_{st}$.

When $C < T$, the nutreal axis is ~~above~~ in the web.

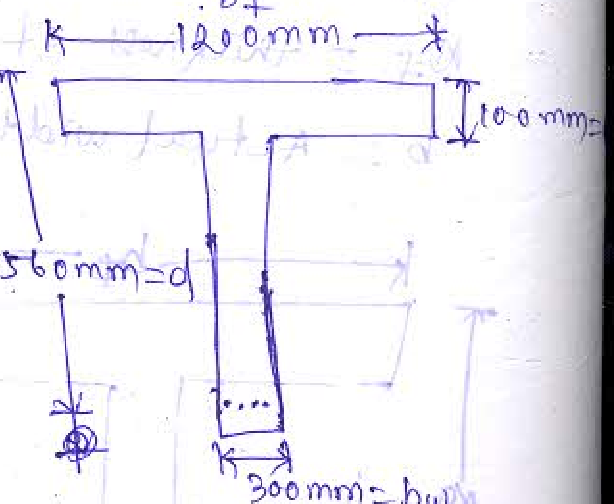
When $C = T$, the nutreal axis will lie at the bottom of the flange.

When $C > T$, the nutreal axis lie in the flange section.

Date - 12/02/2019

Q:- A T-beam of effective flange width of 1200mm, thickness of the slab 100mm, width of the rib 300 mm effective depth of 560 mm is reinforced with 4 no. of 25 mm dia HYSD Fe415 bar. calculate the factored moment of resistance. the grade of concrete is M20.

Sol:



$b_f = 1200mm$.

$D_f = 100mm$

$d = 560mm$

$b_w = 300mm$.

$f_{ck} = 20 MPa$.

$f_y = 415 MPa$.

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

$= 1963.49$

$\simeq 1964 \, mm^2$.

$F_{tc} = 0.36 f_{ck} \, b_f D_f$

$= 0.36 \times 20 \times 1200 \times 100$

$= 864000 \, N$

$= 864 \, KN$.

$F_{ts} = 0.87 f_y A_{st}$

$= 0.87 \times 415 \times 1964$

$= 709102.2 N$

$= 709 KN$

$F_{tc} > F_{ts}$ (N.A lies in the flange).

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

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

$= \frac{0.87 \times 415 \times 1964}{0.36 \times 20 \times 1200}$

$= 82.07$

$\simeq 82 \, mm$.

$\left(\frac{x_u}{d}\right) lim = 0.48$

$\Rightarrow x_{ulim} = 0.48 \times d$

$= 0.48 \times 560$

$= 268.8$

$\simeq 269 \, mm$.

$x_u < (x_u)_{lim}$

So, the beam is under reinforced.

$$\frac{D_f}{d} = \frac{100}{560}$$

$= 0.17 < 0.2$

$$M_u = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42 \frac{x_{umax}}{d}\right] f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

$$= 0.36 \times \frac{269}{560}\left[1 - 0.42 \times \frac{269}{560}\right] \times 20 \times 300 \times 560^2 + 0.45 \times 20 \times (1200 - 300) \times 100 \left(560 - \frac{100}{2}\right)$$

$= 672836500.8$ N mm

$= 672.83$ KNm.

→ Instead of 560mm to take 450 mm.

$A_{st} = 1964 \text{ mm}^2$
$d = 450$ mm.

$x_u = 82.07$.

$\frac{(x_u)_{lim}}{d} = 0.48$

$\Rightarrow (x_u)_{lim} = 0.48 \times 450$

$= 216$

$$\frac{D_f}{d} = \frac{100}{450}$$

$$= 0.22 > 0.2$$

~~$x_{u} = 0.36$~~

$$y_f = 0.15 x_u + 0.65 D_f$$

$$= 0.15 \times 82.07 + 0.65 \times 100$$

$$= 77.31 \text{ mm}$$

$$M_u = 0.36 \times \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

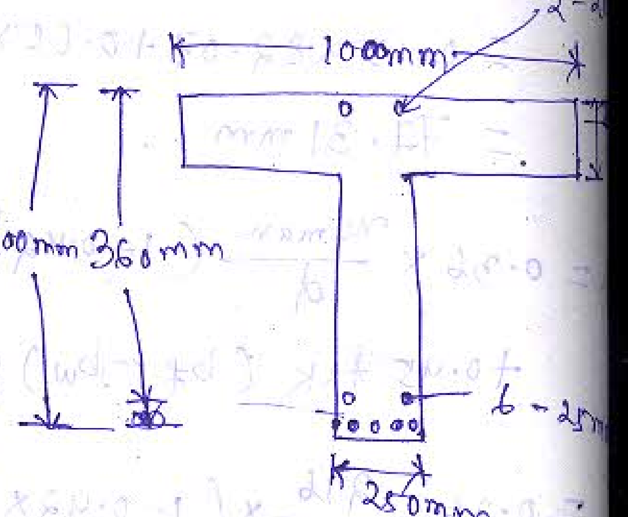
$$= 0.36 \times \frac{216}{450} \times \left(1 - 0.42 \times \frac{216}{450}\right) \times 20 \times 300 \times 450^2 + 0.45 \times 20 \times (1200 - 300) \times 77.31 \times \left(450 - \frac{77.31}{2}\right)$$

$$= 167625676.8 + 257588763.8$$

$$= 425214440.6 \text{ Nmm}$$

$$= 425.21 \text{ kNm}$$

Q:- Determine the moment of resistance of the section as shown in figure. The material are M20 grade concrete & HYSD bare of Fe415.



Sol:

$f_{ck} = 20 MPa$
$f_y = 415 MPa$
$b_f = 1000 mm$
$D_f = 100 mm$
$d' = 40 mm$
$d = 360 mm$
$D = 400 mm$
$b_w = 250 mm$

$A_{sc} = 2 \times \frac{\pi}{4} \times 20^2$
$= 628.32 \ mm^2$
$\simeq 628 \ mm^2$

$A_{st} = 6 \times \frac{\pi}{4} \times 25^2$
$= 2945.24 \ mm^2$
$\simeq 2945 \ mm^2$

$C = T$

$0.36 f_{ck} x_u b_f + A_{sc}(f_{sc} - 0.45 f_{ck}) = f_{st} A_{st}$

$\Rightarrow 0.36 \times 20 \times x_u \times 1000 + 628(0.87 \times 415 - 0.45 \times 2$
$= 0.87 \times 415 \times 2945$

$\Rightarrow x_u = \frac{0.87 \times 415 \times 2945 - 628(0.87 \times 415 - }{\cancel{0.36 \times 20 \times 1000} \ 0.36 \times 20 \times 1000}$

$\Rightarrow x_u = 116.97$
$\simeq 117 \text{ mm}$.

$\left(\frac{x_u}{d}\right)_{lim} = 0.48$

$\Rightarrow (x_u)_{lim} = 0.48 \times d$
$= 0.48 \times 360$
$= 172.8 \text{ mm}$.
$\simeq 173 \text{ mm}$.

$x_u < (x_u)_{lim}$ (under reinforced)

$\frac{D_f}{d} = \frac{100}{360} = 0.28 > 0.2$.

$y_f = (0.15 x_u + 0.65 D_f)$

$= 0.15 \times 117 + 0.65 \times 100$

$= 82.55 \text{ mm}$.

$$M_u = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{(x_u)_{max}}{d}\right) f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right) + A_{sc} f_{sc} (d - d')$$

$$= 0.36 \times \frac{173}{360}\left(1 - 0.42 \times \frac{173}{360}\right) \times 20 \times 250 \times 360^2 + 0.45 \times 20 (1000 - 250) \times 82.55 \left(360 - \frac{82.55}{2}\right) + 628 \times 0.87 \times 415 (360 - 40)$$

$= 89477676 + 177597554.1 + 72556608$

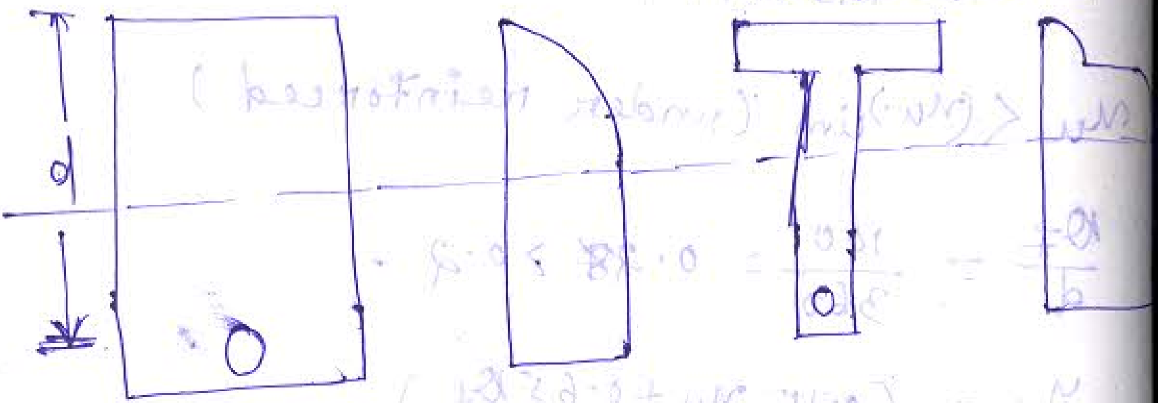
$\simeq 339631838.1 \text{ Nmm}$

$= 339.63 \text{ kNm}$.

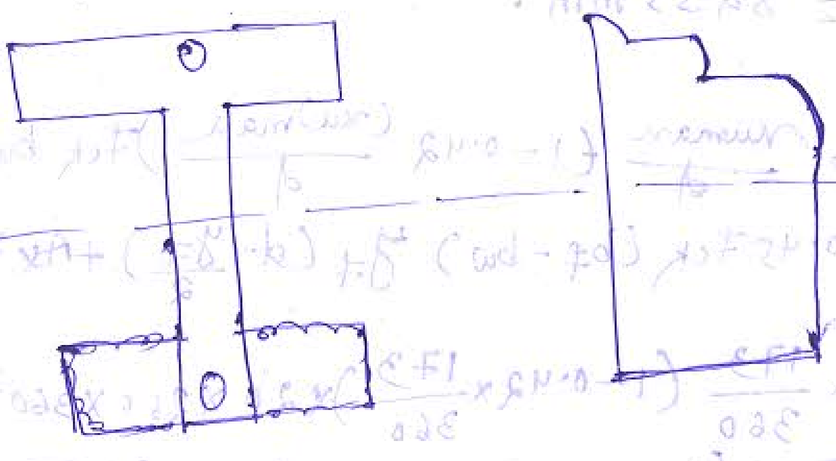
Design for shear

Shear stress distribution for a singly reinforced rectangular beam varies parabolically upto the nutral axis & then after remains constant upto the effective depth d.

For a singly reinforced flanged section



For a ~~singly~~ doubly reinforced flanged section



At the bottom fibre of any beam throughout the length shear stress is zero & bending stress is maximum. The nature of bending stress is tensile due to which ~~1st~~ crack is formed at the bottom which is having an angle of 90° with the beam axis.

Design steps for Shear :-

Step-1

Nominal shear stress = $\tau_v = \frac{V_u}{bd}$

For flange section,

$\tau_v = \frac{V_u}{b_w d}$.

Step-2

Check $\tau_v < \tau_{cmax}$ (Page-73, Table-20)

If $\tau_v > \tau_{cmax}$ redesign the beam or improve the grade of concrete.

Shear strength of concrete depends on the following factors:

1. Uncracked concrete in compression zone.
2. Aggregate interlocking.
3. Shear acting along the longitudinal bar.
4. Shear force across the steel bar.
5. Shear stirrups.

Step-3

Find out percentage of steel.

$$P_t = \frac{A_{st}}{bd} \times 100 \quad \text{(Page-73)}$$

From % of Steel, calculate the value of τ_c from table-19.

Step-4

→ Check if $\tau_v < \frac{\tau_c}{2}$, no shear reinforcement is required.

→ If $\tau_v > \frac{\tau_c}{2}$ & $\tau_v < \tau_c$, the shear reinforcement is provided as per ~~clause~~ clause - 26.5.1.6.

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

~~Asv = total cross-se No. of lag into $\frac{\pi}{4} \times \phi^2$~~

$$A_{sv} = \text{No. of leg} \times \frac{\pi}{4} \times \phi^2$$

Higher grade of steel Fe500 is restricted as a shear reinforcement because the area of steel required for higher grade steel is less due to which the ductility reduces.

Q: Why minimum shear reinforcement is required?

Ans:-

1. To resist any crack development due to creep & shrinkage.
2. To improve the ductility of the beam.
3. To improve dowel action of main reinforcement.
4. To resist diagonal tension.

Step-5

If $\tau_v > \tau_c$, then

$$\boxed{\tau_v = \frac{V_u}{bd}}$$

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

$$= \tau_v bd - \tau_c bd$$

$$= bd(\tau_v - \tau_c)$$

Q7. Inclined stirrups are more effective in resisting shear as it is provided at an angle 90° to the propagation of crack.

Step-6

(Page-47 26.5.1.5)

S_v =

0.75d - vertical

d - inclined

300mm which ever is the less for spacing.

Date - 16/02/2019

Q1- A T-beam section having 230 mm width of the web & 460mm effective depth is reinforced with 5 no. of 16 mm dia bar as tension reinforcement. The section is subjected to a factored shear force 52.5 KN. Check the shear stress & design the shear reinforcement, the materials are M_{20} & HYSD bar of Fe_{415} for stirrup use mildsteel.

→ What will be changed in shear reinforcement if the factored shear is increased to 90 KN & 6mm dia stirrups are used.

Soln

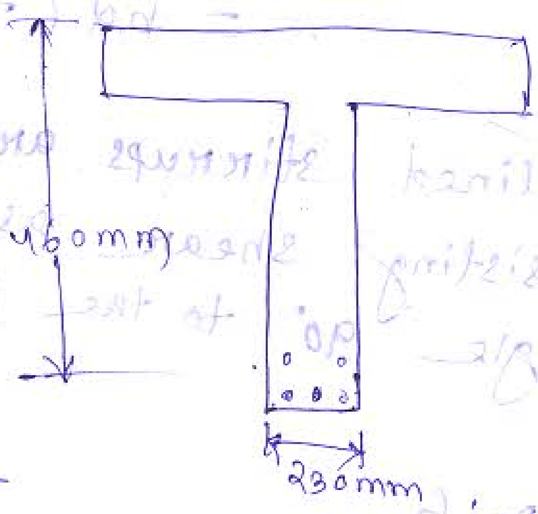
Given data

$b_w = 230$ mm

$d = 460$mm

$f_{ck} = 20$ MPa

$f_y = 415$ MPa



$A_{st} = 5 \times \frac{\pi}{4} \times 16^2$

$= 1005.31 \text{ mm}^2$

$\simeq 1006 \text{ mm}^2$

$V_u = 52.5 \text{ KN} = 52.5 \times 10^3 \text{ N}$

$$\tau_v = \frac{V_u}{bd} = \frac{52.5 \times 10^3}{230 \times 460} = 0.49 \text{ N/mm}^2$$

$\tau_v < (\tau_c)_{max}$ (safe).

$$P_t = \frac{100 A_{st}}{bd}$$

$$= \frac{100 \times 1006}{230 \times 460}$$

$$= 0.95$$

To calculate τ_c,

$$\tau_c = \frac{x - x_1}{x_2 - x_1} \times y_2 + \frac{x - x_2}{x_1 - x_2} \times y_1$$

x_1 0.75	—	0.56 y_1
x 0.95	—	? y
x_2 1.0	—	0.62 y_2

$$= \frac{0.95 - 0.75}{1 - 0.75} \times 0.62 + \frac{0.95 - 1}{0.75 - 1} \times 0.56$$

$$= 0.608$$

$$\frac{\tau_c}{2} = \frac{0.608}{2} = 0.304$$

$\tau_v > \frac{\tau_c}{2}$, $\tau_v < \tau_c$.

$$\frac{\tau_c}{2} < \tau_v < \tau_c$$

$$0.304 < 0.49 < 0.608$$

Minimum shear reinforcement will be provided,

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

For stirrups, $f_y = 250\ N/mm^2$.

$$A_{sv} = \text{No. of leg} \times \frac{\pi}{4} \times \phi^2$$

Let us provide 2 leg, 6mm dia stirrup.

$$A_{sv} = 2 \times \frac{\pi}{4} \times 6^2$$

$$= 56.54\ mm^2$$

$$\approx 57\ mm^2$$

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

$$\Rightarrow \frac{57}{230 \times S_v} \geq \frac{0.4}{0.87 \times 250}.$$

$$\Rightarrow S_v \leq \frac{0.87 \times 250 \times 57}{0.4 \times 230}$$

$$\Rightarrow S_v = 134.75 \text{ mm}$$

$\simeq 130$ mm.

(a) $S_v = 130$ mm.

(b) $0.75d = 0.75 \times 460 = 345$ mm

(c) 300 mm

Let us provide 2 leg 6 mm dia stirrups with 130 mm c/c spacing.

<u>case-2</u>

$V_u = 90 \text{ kN} = 90 \times 10^3 \text{ N}$.

$$\tau_v = \frac{V_u}{bd} = \frac{90 \times 10^3}{230 \times 460}$$

$$= 0.85 \text{ N/mm}^2$$

$\tau_c = 2.8 \text{ N/mm}^2$

$\tau_v < (\tau_c)_{max}$ (Safe)

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1006}{230 \times 460} = 0.95$$

$$\tau_c = \frac{0.95 - 0.75}{1 - 0.75} \times 0.62 + \frac{0.95 - 1}{0.75 - 1} \times 0.56$$

$$= 0.608 \text{ N/mm}^2$$

Now, we find

$\tau_v > \tau_c$.

Shear reinforced will be provided,

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

Here $f_y = 250$

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

$$= 90 \times 10^3 - 0.608 \times 230 \times 460$$

$$= 25673.6 \text{ N}.$$

$$= 25.6 \text{ KN}$$

Let us provide 2leg 6mm dia stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 6^2$$

$$= 56.55 \text{ mm}^2.$$

For vertical stirrups,

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

$$\Rightarrow 25673.6 = \frac{0.87 \times 250 \times 56.55 \times 460}{S_v}$$

$$\Rightarrow S_v = \frac{0.87 \times 250 \times 56.55 \times 460}{25673.6}$$

$$= 220.18 \text{ mm}.$$

$$\simeq 220 \text{ mm}$$

The spacing shall not exceeds

(a) $0.75 \times d = 0.75 \times 460 = 345$ mm.

(b) 300 mm.

(c) 220 mm.

Let us provide 2leg-6mm dia stirrups with

220 mm

Date - 26/02/2019

Design for Bond :-

When reinforcing bar is embeded in concrete the concrete adheres to its surface & resist any force that tries to cause slippage of bar related to its surrounding concrete. This phenomenon is called Bond.

Factors affecting development of bond stress

1. Pure adhesion.
2. Friction resistance.
3. Mechanical resistance.

Bond stress in plain bar is due to pure adhesion & frictional resistance while in deformed bar bond stress is due to pure adhesion, frictional resistance & mechanical resistance. This is why bond stress of deformed bar is more in compare to plain bar.

(Page - 42, 26.2

The development bond length, Page-

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

Where, ϕ = nominal diameter of the bar

σ_s = stress in bar at the section considered design load

$= 0.87 f_y$

τ_{bd} = design bond stress.

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

~~HYS 60.~~

(Page – 43, 26.2.1.1)

Q:– Findout the permissible stress of HYSD bar in tension & compression for M_{20} grade concrete.

Soln

For M_{20} grade concrete, $\tau_{bd} = 1.2\ N/mm^2$.

In tension, $\tau_{bd} = 1.2 \times 1.6$
$= 1.92\ N/mm^2$.

In compression, $\tau_{bd} = 1.2 \times 1.6 \times 1.25$
$= 2.4\ N/mm^2$.

Q:– Findout the development length required for Fe_{415} & M_{25} grade concrete.

(i) In tension
(ii) In compression.
(iii) Also find out L_d in above case if mild steel is used.

Soln

For M_{25} grade concrete,
$\tau_{bd} = 1.4\ N/mm^2$.

~~(i) in compression~~.

(i) in tension $\tau_{bd} = 1.4 \times 1.6 = 2.24\ N/mm^2$.

$\sigma_s = 0.87 f_y$
$= 0.87 \times 415$

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

$= \frac{\phi \times 361.05}{4 \times 2.24}$

$\Rightarrow L_d = 40.29 \phi$.

(ii) In compression, $\tau_{bd} = 1.4 \times 1.6 \times 1.25$

$= 2.8 \ N/mm^2$.

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

$= \frac{\phi \times 361.05}{4 \times 2.8}$

$\Rightarrow L_d = 32.23 \phi$.

(iii) For mild steel, $f_y = 250 \ N/mm^2$.

$\tau_{bd} = 1.4$.

<u>In tension</u>

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

$\Rightarrow L_d = \frac{\phi \, 0.87 f_y}{4 \times 1.4}$

$= \frac{\phi \, 0.87 \times 250}{4 \times 1.4}$

$\Rightarrow L_d = 38.84 \phi$.

In compression, $\tau_{bd} = 1.4 \times 1.25$
$= 1.75\ N/mm^2$

$$L_d = \frac{\phi\, 0.87 f_y}{4 \tau_{bd}}$$
$$= \frac{\phi \times 0.87 \times 250}{4 \times 1.75}$$
$$\Rightarrow L_d = 31.07\,\phi$$

IS code Provision for bond :-

(Page-44, 26.2.3)

curtailment of tension reinforcement

Page - 26.2.3.3

Positive moment reinforcement,

$$L_d \le \frac{M_1}{V} + L_o$$

For the reinforced confined by a compressive react, 'or'/ In hook case, $L_d \le 1.3 \frac{M_1}{V} + L_o$

Date - 07/03/2010

Q:- A S/S beam 25 cm x 50 cm has two bar of dia 20mm, if Shear force at centre of support is 110 kN & it is working load. Determine anchorage length. Use M20 Fe415, LSM. The effective cover is 35mm.

Sol

Given data,

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

$= 628.31 \ mm^2$

$\simeq 630 \ mm^2$.

$b = 25 \ cm = 250 \ mm$

$d = 50 \ cm = 500 \ mm$,

$d' = 35 \ mm$.

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

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

$V = 1.5 \times 110 = 165 \ kN$.

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

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

$= \frac{0.87 \times 415 \times 630}{0.36 \times 20 \times 250}$

$\Rightarrow x_u = 126 \ mm$.

$(x_u)_{lim} = 0.48 d$

$= 0.48 \times 465$

$= 223.2 \ mm$.

$(x_u) < (x_u)_{lim}$ (under Reinforced)

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

$= 0.87 \times 415 \times 630 (465 - 0.42 \times 126)$

$= 93.73 \, kNm$.

$L_d = \dfrac{\phi \sigma_s}{4 \tau_{bd}}$

$= \dfrac{\phi \times 0.87 \times 415}{4 \times 1.92 (1.6 \times 1.2)}$

$= 47\phi = 47 \times 20$

$= 940 \, mm$

Development length $= 940 \, mm$

— o —

Torsion :-

There are two types of torsion :-

(i) Primary torsion

(ii) Secondary torsion.

Primary torsion :-

Primary & equilibrium torsion are induced by an eccentric loading with respect to shear centre & equilibrium condition indetermining the twisting moment.

Secondary or compatability torsion :-

Torsion is induced by need for member undergoes angle of twist to maintain deformation compactibility & resulting twisting moment depends on torsional stiffness on the member.

Design step for torsion :-

Step-1

calculate Vequivalent (page-75-41.3.1

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

Step-2

calculate longitudinal reinforcement (page-75-

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

Where,

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

Step-3

calculate transverse reinforcement

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

$$A_{sv} < \frac{(\tau_{ve} - \tau_c) b S_v}{0.87 f_y},$$

Step-4

If τ_{ve} is less than τ_c minimum shear reinforcement will be provided $\frac{A_{sv}}{b S_v}$

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

Step-5

Maximum spacing equal to,

(i) x_1

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

(iii) 300

which ever is the less.

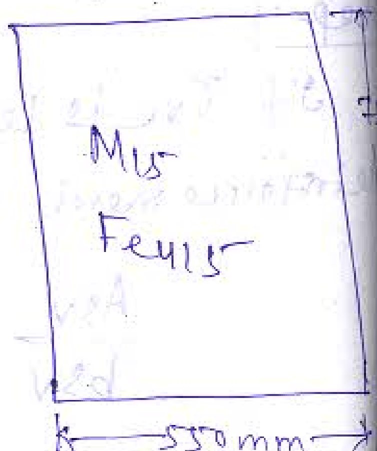
Step-6

Calculate the side reinforcement (page-47)

Date - 08/03/...

Q:- A RCC beam of 550 mm x 750 mm overall depth is subjected to ultimate shear force of 130 KN, ultimate bending moment 150 KNm & ultimate twisting moment of 50 KNm. Assume M15 & Fe415 steel. Determine the longitudinal & transverse reinforcement.

Sol^n



Given data,

$b = 550 mm$

$D = 750 mm$

$f_{ck} = 15 N/mm^2$

$f_y = 415 N/mm^2$

$V_u = 130 KN$

$M_u = 150 KNm$ (Page-75)

$T_u = 50 KNm$

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

$\Rightarrow V_e = 130 + 1.6 \times \frac{50}{550 \times 10^{-3}}$

$\Rightarrow V_e = 275.45 KN$

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

$= 50 \left(\frac{1 + 0.750/0.550}{1.7} \right)$

$= 69.51 KN\text{-}m$

$M_u = 150\ kN\text{-}m > M_t$

Longitudinal reinforcement,

$M_{e\cancel{1}} = M_u + M_t$

$= 150 + 69.51$

$= 219.51\ kNm$.

As $M_u > M_t$, so $M_{e2} \doteq 0$.

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

$\Rightarrow 219.51 = 0.138 \times 15 \times 550 \times d^2$

$\Rightarrow d = \sqrt{\dfrac{219.51 \times 10^6}{0.138 \times 15 \times 550}}$

$\Rightarrow d = 439.09\ mm$
$\simeq 450\ mm$.

The effective depth is less than overall depth, so ~~our~~ the design is safe.

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] bd.$$

$$= \frac{0.5 \times 15}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 219.51 \times 10^6}{15 \times 550 \times (450)^2}}\right] \times 550 \times 450$$

$= 1659.63\ mm^2$.

Let us provide 20mm dia bar

$n \times \frac{\pi}{4} \times 20^2 = 1659.63$

$\Rightarrow n = 1659.63 \times \frac{4}{\pi} \times \frac{1}{20^2}$

$= 5.28$

$\simeq 6$.

Let us Provide 6no. of 20 mm dia bar

Now to calculate equivalent shear Stress.

(Page-74)

$\tau_v = \frac{V_u}{bd}$

$= \frac{275.45 \times 10^3}{550 \times 450}$

$= 1.11 \; N/mm^2 < 2.5$

$\tau_v < \tau_{cmax}$ (Design is safe)

$\frac{100 A_{st}}{bd}$

$= \frac{100 \times 6 \times \frac{\pi}{4} \times 20^2}{550 \times 450}$

$= 0.76$

x_1 0.75 $\quad$ 0.54 y_1

x 0.76 $\longrightarrow$ ~~y~~

x_2 1.00 $\quad$ 0.60 y_2

$$\tau_c = \frac{0.76-0.75}{1.00-0.75} \times 0.60 + \frac{0.76-1.00}{0.75-1.00} \times 0.54$$

$= 0.54\,N/mm^2$.

$\tau_v > \tau_c$.

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

$= 275.45 - 0.54 \times 550 \times 450$

$= 141800\,N$

$= 141.8\,kN$

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

Letus Provide two leg 8mm dia stirrups

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

$= 100.53\,mm^2$.

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

$$= \frac{0.87 \times 250 \times 100.53 \times 450}{141800}$$

$= 69.38$

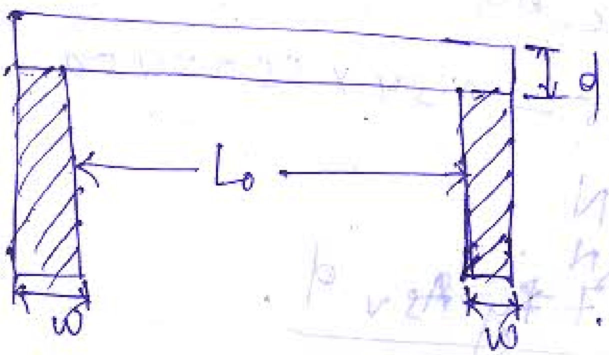
$\simeq 70\,mm$

Design of slab :-

Effective span :-

(Page - 34 - 22.2)

Simply supported beam :-



Effective span = clear span + width of the column.

$L_{eff} = L_0 + w$
'or'
$L_0 + d$.

Continuous beam :

case - 1

If width of support is (less than) $< \frac{\text{clearspan}}{12}$,

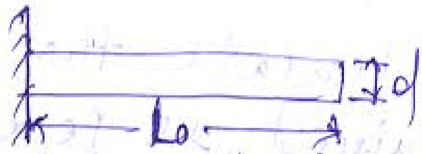
$L_{eff} = L_0 + w$ 'or' $L_0 + d$.

case - 2

If width of support $> \frac{\text{clearspan}}{12}$, ↑$L_{0,1}$ ↑$L_{0,2}$ ↑$L_{0,3}$

$L_{eff} = L_{0_3} + \frac{w}{2}$.

cantilever ~~Fixed~~ beam :-



case-1)

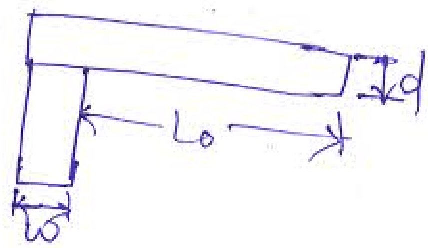
If one end fixed other end free,

$$L_{eff} = L_0 + \frac{d}{2}.$$

case-2

one end continuous, other end free,

$$L_{eff} = L_0 + \frac{w}{2}.$$



control of deflection :-

(~~Page-37~~ 23.2)

Q:- A span of s/s beam is 18 m. The minimum depth required as per deflection criteria is ?

Ans:-

For simply supported, $\frac{L}{d} \leq 20 \times \frac{10}{L}$.

$$\Rightarrow \frac{18}{d} \leq 20 \times \frac{10}{18}$$

$$\Rightarrow d \geq \frac{18 \times 18}{20 \times 10}$$

$$\Rightarrow d \geq 1.62 \text{ m}.$$

Q:- Calculate the minimum depth required as per deflection criteria for a cantilever span of 7m.

Sol^n

For cantilever beam, $\frac{L}{d} \leq 7m$.

$\Rightarrow \frac{7}{d} \leq 7$

$\Rightarrow d \geq \frac{7}{7}$

$\Rightarrow d \geq 1m$.

Q:- A cantilever beam of span 5m with dimension 250mm × 400mm, check the beam for deflection.

Sol^n $L = 5m = 5000mm$.

For cantilever beam,

$\frac{L}{d} \leq 7$

$\Rightarrow \frac{5000}{d} \leq 7$

$\Rightarrow \frac{5000}{7} \leq d$

$\Rightarrow d = 714.28mm$.

$d > 400mm$

So the beam is unsafe / Failure

Date - 12/03/2019

IS code Provision :-

(i) Minimum reinforcement :- (Page-47)

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

Minimum reinforcement is provided to resist possible load effect & to control cracking in concrete due to shrinkage & temperature variation.

(ii) Maximum tension Reinforcement

$= 4\%$ of bD

$= 0.04 \times bD$.

(iii) Maximum compression Reinforcement

$= 0.04 \times bD$.

Q:- Why maximum compression Reinforcement is used?

Ans:- To avoid congestion & for proper placement & compaction.

Side face Reinforcement :-

→ Side face reinforcement is provided to improve resistance under lateral buckling.

→ cracking can occur on large unreinforced face of concrete on acount of shrinkage & temperature.

Page-48 -26.5.2.1

Minimum reinforcement of slab
is 0.12% of bD (Fe415 & Fe500)
0.15% of bD (Fe250)

Maximum diameter (Page-48-26.5.2.2)
Maximum diameter of bar should not be greater than (≯) thickness of the bar / 8

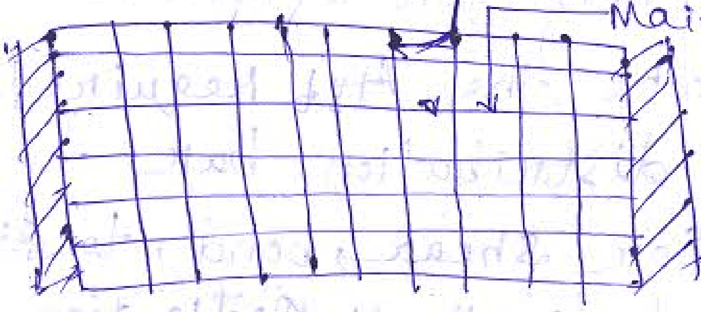
Maximum spacing :- (26.5.
Maximum spacing should not be greater than equal to (≯) 3d or 300mm } For main
& 5d or 450mm → Distr bar

Function of transverse Reinforcement or Distribution bar :-
1. It distributes the effect of point load on the slab more evenly & uniformely.
2. It distributes the shrinkage & temperature crack more uniformely
3. It keeps the main bar in position

Design of oneway slab :-

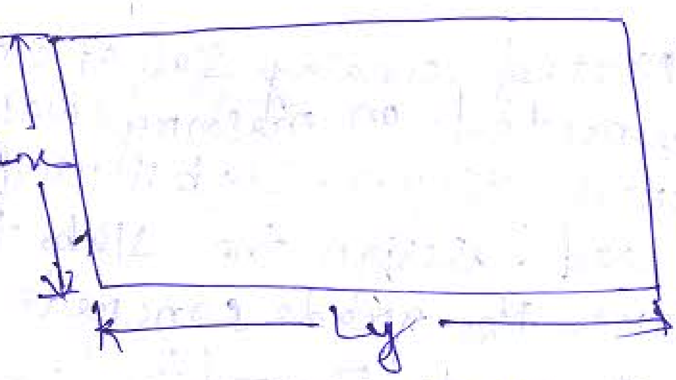
Slab is one way:

1. 97 supported on opposite side (either supported on shorter edge or longer edge)



(oneway slab).

Page - 90 ~~...~~



$\frac{Ly}{Lx} < 2 \rightarrow$ TWO ~~one~~way slab

$\frac{Ly}{Lx} > 2 \rightarrow$ one ~~two~~ way slab.

Design steps :-

1. Calculate the effective depth from $\frac{L}{d}$ ratio.

2. Calculation of ~~dead load~~ DL, L

3. ~~calcul~~ check for effective depth

4. Calculate the Ast required.

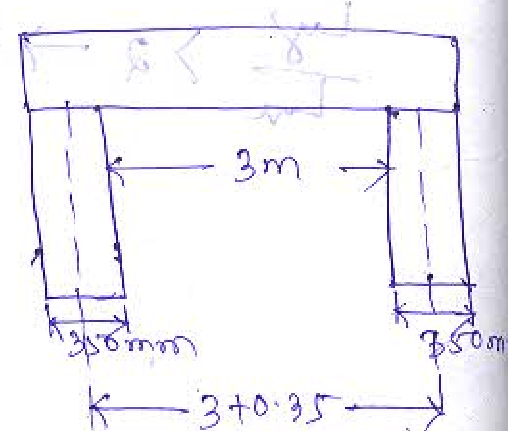
5. Design distribution bar.

6. Check for shear, bond, torsion, development length & Deflection.

Date - 14/03/2019

Q:- A simply supported oneway slab of clear span 3m is supported on masonry wall having thickness 350mm. Slab is used for residential load. Design the slab, the materials are M_{20} grade concrete & HYSD bar of grade Fe_{415}, live load is $2 kN/m^2$ & floor finish is $1 kN/m^2$.

Soln



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

Page-37

$\frac{Span}{d} = 20$.

$\frac{3000}{d} = 20$

$\Rightarrow d = \frac{3000}{20} = 150 mm$.

$L_{eff} = 3000 + 150 = 3150$ mm.

$L_{eff} = 3000 + 350 = 3350$ mm.

Now, effective span $= 3150$ mm.

Calculation of shear & moment :—

(Unit weight of concrete $= 25\ kN/m^3$)

Floor finish $= 1\ kN/m^2$

LL of floor $= 2\ kN/m^2$

~~DL of floor~~ $\cancel{= 0.15 \times 25}$
$\cancel{= 3.75\ kN/m^2}$.

overall depth $= 150 + 15 + \frac{12}{2}$
$= 171$ mm $\simeq 180$ mm.

(Let us provide 12mm dia bar with nominal cover 15 mm)

overall depth $= 180$ mm.

DL of floor $= 0.180 \times 25$
$= 4.5\ kN/m^2$.

Total load $= 1 + 2 + 4.5$
$= 7.5\ kN/m^2$

Factored load $= 7.5 \times 1.5$
$= 11.25\ kN/m^2$.

For 1m span the factored load is
$= 11.25 \times 1 = 11.25\ kN/m$.

$\frac{w}{2} + w \times l$

$S.F. = \frac{wl}{2} = \frac{11.25 \times 3.15}{2}$

$= 17.7\ldots$

$$B.M. = \frac{wl^2}{8}$$

$$= \frac{11.25 \times (3.15)^2}{8}$$

$$= 13.95\ KNm.$$

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

$$\Rightarrow 13.95 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$\Rightarrow d = \sqrt{\frac{13.95 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$\Rightarrow d = 71.09\ mm.$$

$(d)_{required} < (d)_{provided}$ (Safe)

$$Ast = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 Mu}{f_{ck} bd^2}}\right] bd$$

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 13.95 \times 10^6}{20 \times 1000 \times (150)^2}}\right] \times 1000 \times 150$$

$$\Rightarrow Ast = 267.618\ mm^2$$

Let us provide 12 mm bar

$$Ast = No.\ of\ bar \times \frac{\pi}{4} \times 12^2$$

$$\Rightarrow No.\ of\ bar = 267.618 \times \frac{4}{\pi} \times \frac{1}{12^2}$$

$$\Rightarrow n = 2.36$$

Let us provide 4no. of 12mm bar.

$$A_{st} = 4 \times \frac{\pi}{4} \times 12^2$$
$$= 452.38 \, mm^2$$

Check for shear :-

$D = 180 \, mm$

$d = 180 - 15 - 6$

$= 159 \, mm$.

$$\tau_v = \frac{V_u}{bd} = \frac{17.72 \times 10^3}{1000 \times 150}$$

$= 0.118$

$(\tau_c)_{max} = 2.8 \, N/mm^2$.

$\tau_v < (\tau_c)_{max}$.

$0.118 < 2.8$

$$P_t = \frac{A_{st}}{bd} \times 100$$
$$= \frac{452.38}{1000 \times 150} \times 100$$
$$= 0.301.$$

0.25 — 0.36

0.301 — ?

0.50 — 0.48

$$\tau_c = \frac{0.301 - 0.25}{0.50 - 0.25} \times 0.48 + \frac{0.301 - 0.50}{0.25 - 0.50} \times 0.36$$

$= 0.384.$

Date - 15/03

$$\frac{A_{sv}}{bS_v} \geq \frac{0.4}{0.87 f_y}$$ Page - 48

Let us provide 2 leg 6 mm dia stirrups

$$\frac{2 \times \frac{\pi}{4} \times 6^2}{1000 \times S_v} \geq \frac{0.4}{0.87 \times 250}$$

$$\Rightarrow S_v \cancel{\leq} \frac{2 \times \frac{\pi}{4} \times 6^2 \times 0.87 \times 250}{0.4 \times 1000}$$

$$\Rightarrow S_v \cancel{\leq} 30.74\ mm^2$$

~~$\Rightarrow S_v = 25$~~

Let us provide two leg 6 mm dia stirrups with 25 mm c/c spacing.

Check for deflection :-

Page - 37 - 23.2.1

$$\frac{Span}{d} = \frac{3000}{150}$$

$$= 20\ mm$$

'or'

$$\frac{3000}{159} = 18.86\ mm.$$

As the deflection value is within the limit so the slab is safe in deflec

Design of two way slab :-

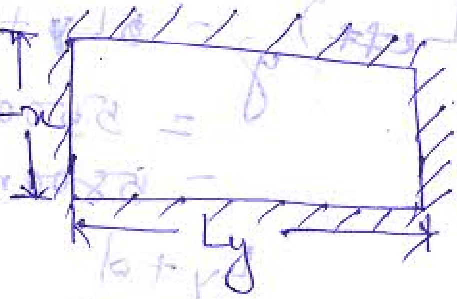
Q:- A drawing room of a residential building measures 4.3m × 6.55m. It is supported on 350mm thick wall on all 4 sides. The slab is simply supported at edges with no provision to resist torsion at corners. Design the slab using M_{20} grade concrete & HYSD reinforcement of grade Fe_{415}

Q:- Design a slab of size 4.6m & 5.5m. The slab is continuous over 2 adjacent edges & other two edges are discontinuous. The slab is subjected to live load of 8 KN/m² & Floor finish thickness is 100mm is required for water proofing of slab. Design the slab using M_{25} grade concrete & Fe_{415}. The slab is supported of 300mm wide support.

Date - 16/03/2019

2. Sol

Given data,



$L_x = 4.6\,m$

$L_y = 5.5\,m$

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

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

Page - 39

$$\frac{L}{D} = 40 \times \frac{3.5}{4.6}$$

$$= 30.43 \simeq 32.$$

$$\frac{span}{D} = 32$$

$$\Rightarrow \frac{4600}{D} = 32$$

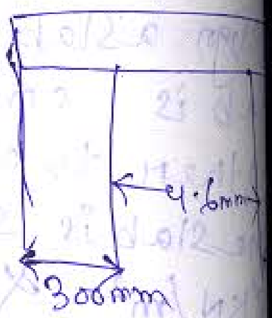
$$\Rightarrow D = \frac{4600}{32}$$

$$\Rightarrow D = 143.75\ mm.$$

$$\simeq 150\ mm.$$

~~Let us~~

$$d = 150 - 30 = 120\ mm.$$



$$L_{eff} = L_0 + w$$
$$= 4600 + 300$$
$$= 4900\ mm$$
$$= 4.9\ m.$$

or

$$L_{eff} = L_0 + d$$
$$= 4600 + 120$$
$$= 4720\ mm$$
$$= 4.720\ m$$

$$(L_{eff})_x = 4.72\ m.$$

$$(L_{eff})_y = L_y + w$$
$$= 5500 + 300$$
$$= 5800\ mm = 5.8\ m$$

$$L_y + d$$
$$= 5500 + 120$$
$$= 5620\ mm.$$
$$= 5.62\ m$$

$$(L_{eff})_y = 5.62\ m.$$

check

$$\frac{(L_{eff})_y}{(L_{eff})_x} = \frac{5.62}{4.72} = 1.19 \simeq 1.2 < 2.$$

(Two way slab).

$W_{DL} = 25 \times 0.15 \times 1 = 3.75$ kN/m.

$W_{LL} = 8 \times 1 = 8$ kN/m.

Unit weight of floor finish = 24 kN/m³

$W_{FL} = 24 \times 0.1 \times 1 = 2.4$ kN/m

Total Load = 14.15 kN/m.

Factored Load = 14.15 × 1.5
= 21.225 kN/m.

(Page - 91 - Table - 26)

As per table no. 26, $\alpha_x(-) = 0.060$.

$\alpha_x(+) = 0.045$

$\alpha_y(-) = 0.047$

$\alpha_y(+) = 0.035$

$M_x(-) = 0.060 \times 21.225 \times (4.72)^2$
$= 28.37$ kNm.

$M_x(+) = 0.045 \times 21.225 \times (4.72)^2$

$= 21.27$ kNm.

$M_y(-) = 0.047 \times 21.225 \times (4.72)^2$
$= \cancel{31.54}$ 22.22 kNm.

$My(+) = 0.035 \times 21.225 \times (4.72)^2$

$= \cancel{23.46\ KNm}\ 16.55\ KNm$

$Mu = 0.138\, fck\, bd^2$

$\Rightarrow 28.37 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$

$\Rightarrow d = \sqrt{\dfrac{28.37 \times 10^6}{0.138 \times 25 \times 1000}}$

$\Rightarrow d = \cancel{95.56}\ 90.68\ mm < 120\ mm$ ~~(Safe)~~

The slab is under reinforced & it safe.

Date - 18/03/

We know,

$Ast = \dfrac{0.5\, fck}{fy}\left[1 - \sqrt{1 - \dfrac{4.6 Mu}{fck\, bd^2}}\right] bd$

	Ast	Spacing Required
$Mu(+) = 21.28$	530.31	$\cancel{\dfrac{1000}{\left(\dfrac{530.31}{\frac{\pi}{4} \times 10^2}\right)}}$
$Mu(-) = 28.37$	728.56	
$My(+) = 16.55$	404.85	
$My(-) = 22.22$	555.85	

$Spacing = \dfrac{1000}{No.\ of\ bar}$

$= \dfrac{1000}{\left(\dfrac{Ast}{\frac{\pi}{4} \times \phi^2}\right)}$

Let us Provide 10mm dia bar

	Ast	Spacing required	Spacing Provided
$M_x(+) = 21.28$	530.31	$\frac{1000}{\left(\frac{530.31}{\frac{\pi}{4}\times10^2}\right)} = 148.1$ mm	140mm
$M_x(-) = 28.37$	728.56	$1000/\left(\frac{728.56}{\frac{\pi}{4}\times10^2}\right) = 107.81$ mm	100mm
$M_y(+) = 16.55$	404.85	$1000/\left(\frac{404.85}{\frac{\pi}{4}\times10^2}\right) = 193.99$ mm	190mm
$M_y(-) = 22.22$	555.85	$1000/\left(\frac{555.85}{\frac{\pi}{4}\times10^2}\right) = 141.25$ mm	140mm

Area of distribution bar is (Pag-48 26.5.2.1)

$$\frac{0.12}{100} \times 1000 \times 150 = 180\ mm^2$$

Let us provide 8mm bar as distribution steel.

$$\text{Spacing} = \frac{1000}{\frac{Ast}{\frac{\pi}{4}}}$$

~~between the distribution~~ bar.

Spacing between the distribution bar are $\frac{1000}{\left(\frac{180}{\frac{\pi}{4}\times8^2}\right)} = 279.25$ mm.

$\simeq 270$ mm.

Check for Shear :-

$V_x = \frac{wL_x}{3}$ $V_y = \frac{wL_xr}{2+r}$.

$= \frac{21.225\times4.72}{3}$ $V_y = \frac{21.225\times4.72\times1.2}{2+1.2}$.

$= 33.394$ kN. $= 37.57$ kN.

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

$$= \frac{37.57 \times 10^3}{1000 \times 120} = 0.313 \text{ MPa}$$

$(\tau_c)_{max} = 3.1 \text{ N/mm}^2$

$\tau_v < (\tau_c)_{max}$ (safe)

Check for bond :—

$$\tau_{bd,y} = \frac{V_x}{\Sigma o j d \times \text{no. of bar}}$$

ΣO = circumference $= 2\pi r / \pi d$

d = Effective depth

j = Modification factor = 0.8

V_x = shear force

$$\tau_{bd,y} = \frac{33.39 \times 10^3}{\pi \times 10 \times 0.8 \times 120 \times \left(\frac{1000}{190}\right)}$$

$= 2.1$ MPa

$$\tau_{bd,x} = \frac{V_y}{\Sigma o j d \times \text{no. of bar}}$$

$$= \frac{37.57 \times 10^6}{\pi \times 10 \times 0.8 \times 120 \times \left(\frac{1000}{140}\right)}$$

$= 1.74$ MPa

$\tau_{bd} = 1.4 \times 1.6 = 2.24$ MPa.

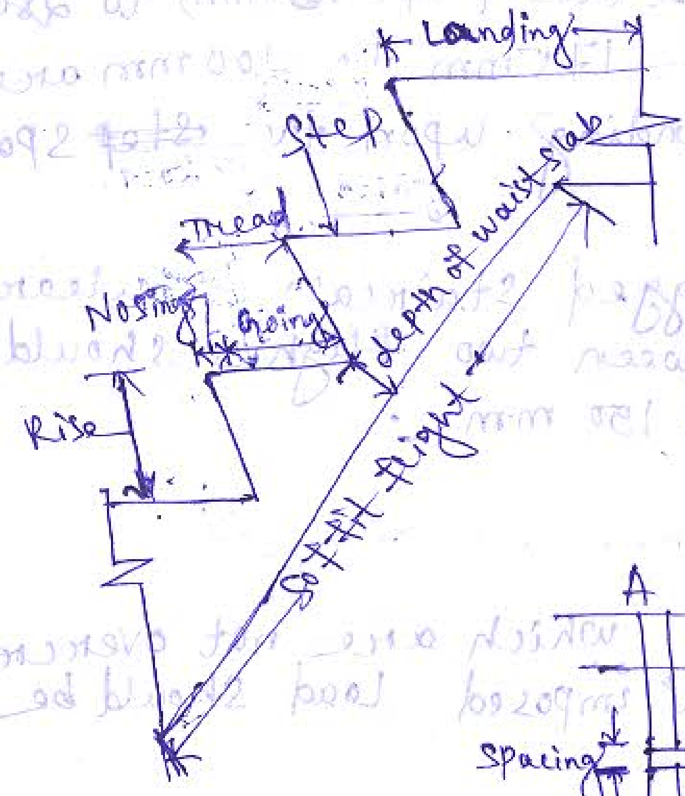
τ_{bd} provide < τ_{bd} permissible

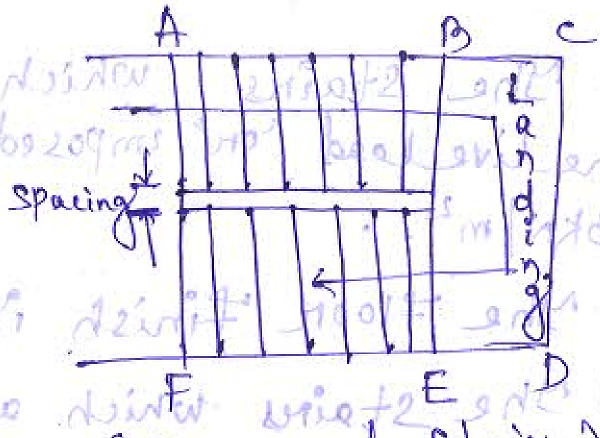
So, it is safe.

Hence all the reinforcement ~~at~~ two support has been considered for bond check. So no cuttlement of bar is required.

Date - 19/03/2019

Design of staircase :-





(Dogleged Stair)

The stairs are grouped into the following categories as per their use:

1. Private stairs.
2. common stairs.

1. Private stairs :—

→ For private stair the rise should not be more than 200mm & tread is not less than 230mm.

→ These are minimum requirements & usually a tread of 250mm to 280mm & a rise of 175mm to 200mm are provided depending upon the ~~step~~ space available.

→ For dog-legged staircase the clear distance between two flights should be between 0 to 150 mm.

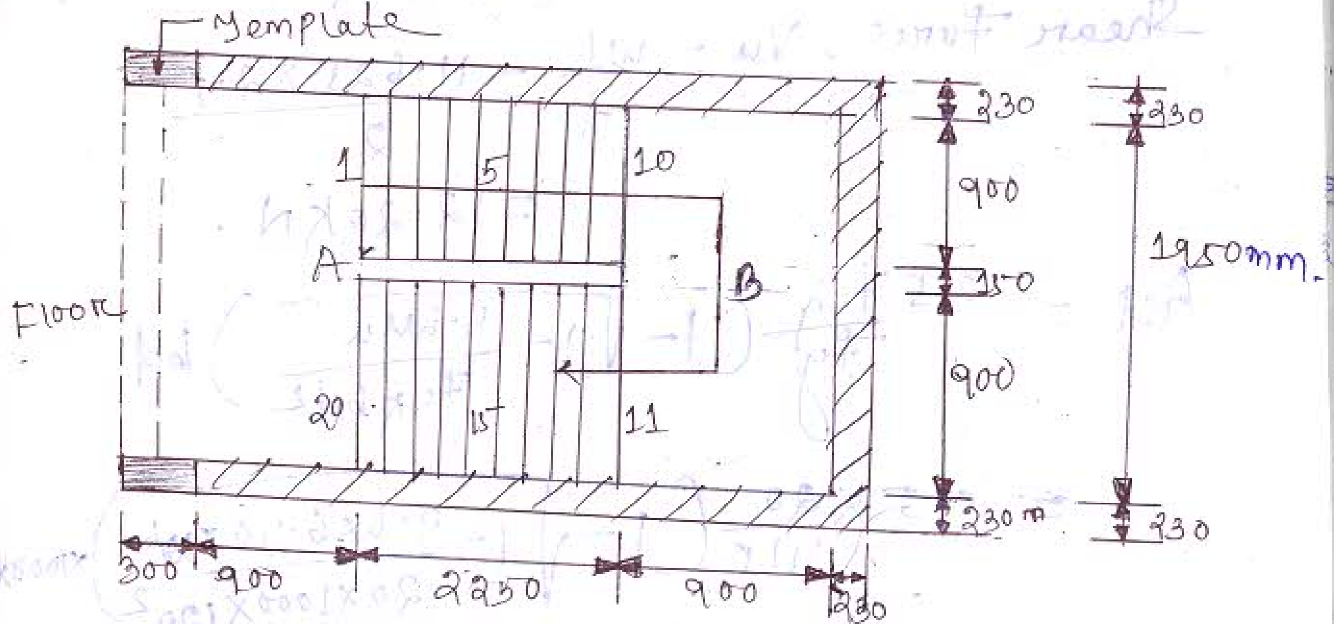
Design requirements for stairs :—

→ The stairs which are not overcrowded the live load 'or' imposed load should be 3kN/m².

→ The floor finish is 1kN/m².

→ The stairs which are overcrowded the live load should be 5kN/m².

Q: The arrangement of a dog-legged stair case in a residential building is shown in the figure. Rise of step is 160mm & tread is 250mm, nosing is not provided. The materials are M_{20} grade concrete & HYSD bar of Fe415. Design the stair case.



Date - 28/03/2019

Soln

Let us assume the thickness of waist slab is 150mm.

$DL = 0.15 \times 25 = 3.75 \; KN/m^2$

$LL = 3 \; KN/m^2$

$FF = 1 \; KN/m^2$

Total load $= 7.75 \; KN/m^2$.

Factored load $= 1.5 \times 7.75 = 11.625 \; KN/m^2$

The span length $= 1950 + 150 = 2100$ mm
$= 2.1$ m.

For 1m span the total load is $11.625 \times 1m$
$= 11.625$ kN/m.

Moment, $M_u = \frac{wl^2}{8} = \frac{11.625 \times (2.1)^2}{8}$
$= 6.40$ kN-m.

Shear force, $V_u = \frac{wl}{2} = \frac{11.625 \times 2.1}{2}$
$= 12.20$ kN.

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right) bd$$

$$= 0.5 \times \frac{20}{415}\left(1 - \sqrt{1 - \frac{4.6 \times 6.40 \times 10^6}{20 \times 1000 \times 120^2}}\right) \times 1000 \times 1$$

$= 151.77$ mm^2
$\simeq 152$ mm^2.

$D = 150$ mm.

Let us assume,
clear cover $= 25$mm & Provide 10mm dia bar
$\phi = 10$mm.

So, $d = 150 - 25 - \frac{10}{2}$
$= 120$mm.

$\cancel{Now} \; n \times \frac{\pi}{4} \times 10^2 = 152$

$\Rightarrow n = 152 \times \frac{4}{\pi} \times \frac{1}{10^2}$

$\Rightarrow n = 1.93$

$\simeq 2$ no.

Let us provide 2nos. of 10mm dia bar.

Check for Shear

$\tau_v = \frac{V_u}{bd} = \frac{12.20 \times 10^3}{1000 \times 120}$

$= 0.10 \; N/mm^2$.

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

$\tau_v << \tau_{c\,max}$ (safe)

Check for deflection :-

$\frac{span}{d} = 20$

$\frac{2100}{120} = 17.5 < 20$ (safe)

Check for development length

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$1.3 \times \frac{M_1}{V} + L_0$

L_0 = effective depth(d) or 12ϕ.

$1.3 \times \frac{6.40 \times 10^6}{12.20 \times 10^3} + 120$

$= 801.96$ mm