UNIT - III

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LIMIT STATE OF COLLAPSE (SHEAR)

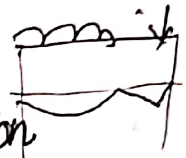
· Beams subjected to transverse loading experiene

① Bending Moment

② Shear Force

③ Deflection

when a beam is loaded with transverse loads, the BM varies from section to section.



Shearing stresses in the beam are caused by this variation of BM along the span. Due to variation of BM at two section distant 'dx' apart, there are unequal bending stresses at the same fibre. This inequality of bending stresses produces a tendency in each horizontal fibre to slide over adjacent horizontal fibre causing horizontal shear stress which is accompanied by complimentary shear stress in vertical direction.

BM and SF when act together causes resultant tensile force diagonally in the beam. This resultant force is known as Diagonal tension.

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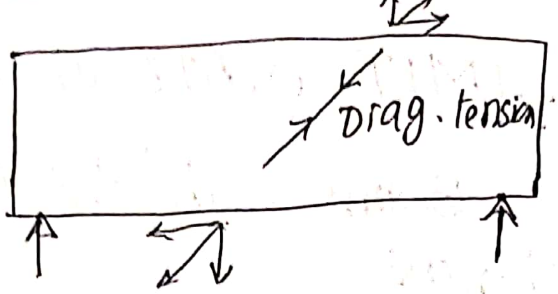
→ Steel bars are cranked at 45° to take up diagonal tension.



Design of Beams for Shear

SF ~~is~~ developed in the beam is taken up by

① Cross-section of beam i.e concrete.

② Bend up bars

③ Stirrups.

Steps for Design for SHEAR

Step 1 Given load acting on the beam 'w', span 'l'.

C.S b×d, Area of Steel Ast.

find S.F. $V_u = 1.5 \frac{wl}{2}$

Step 2: Calculate nominal Shear Stress $\tau_v = \frac{V_u}{bd}$

Step 3 Find shear taken up by Concrete τ_c from Table 19 IS 456 (P73).

τ_c depends on $\frac{A_{st}}{bd} \times 100$ and grade of concrete

SF taken up by concrete $V_{uc} = \tau_c bd$

$\Rightarrow$ Remaining SF $V_{urem} = (V_u - V_{ucont})$

Step 4 :- Crank (bend) suitable no: of bars and find shear taken up by bend up bars using

$V_{us} = 0.87 f_y \cdot A_{sv} \cdot \sin\alpha$ (IS 456 73)

Note:- Bent up bars should not take more than half the remaining S.F.

Step 5 Find the shear remaining, this will be taken up by stirrups.

Adopt 6mm or 8mm dia 2-legged stirrups and find the spacing of the stirrups.

Adopt 6mm or 8mm dia, 2legged stirrups and find the spacing of the stirrups using the eqn

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

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A SRB ~~measuring~~ of size 230mm X 400mm consists of 4-bars of 16mm ϕ at an eff. cover of 30mm. The beam is subjected to working load of 25kN/m on an Eff. span of 6m. Design the beam for shear cracking suitable number of bars. Use M_{20} conc. and Tor steel.

Sol **Design:** $b = 230mm$; $D = 400mm$; $d = (400-30)mm$

$$A_{st} = \frac{4 \times \pi \times 16^2}{4} = 804mm^2 \; ; \; W = 25kN/m.$$

→ Ult. S.F $= 1.5 \frac{wl}{2} = \frac{1.5 \times 25 \times 6}{2}$

$$\boxed{V_U = 112.5kN}$$

→ Shear Stress $\tau_{VU} = \frac{V_U}{b \cdot d} = \frac{112.5 \times 10^3}{230 \times 370}$

$$\Rightarrow \boxed{\tau_{VU} = 1.32N/mm^2}$$

→ Shear taken up by Concrete;

Refer table (19). Pg (73)

$$\frac{A_{st}}{b \cdot d} \times 100 = \frac{804}{230 \times 370} \times 100 = 0.94\%$$

Shear strength $\tau_c = 0.60N/mm^2$

SF taken by Concrete $V_{u\text{-}con} = \tau_v \times bd$

$= 0.6 \times 230 \times 370 \times 10^{-3}$

$= 51.06 kN.$

→ Remaining S.F $V_{u\text{-}rem} = 112.5 - 51.06$

$= 61.44 kN$

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→ Crank bars (Bend):

Crank 2 bars of 16mm at 45°.

Shear taken up by Bent up bars } $V_{us} = 0.87 f_y A_{sv} \sin 45$

$= 0.87 \times 415 \times 2 \times \frac{\pi}{4}(16)^2 \sin 45 \times 10^{-3}$

$\boxed{V_{us} = 102.63 kN}$

→ Bent up bars cannot take more than half the Remaining S.F.

∴ SF taken by bentup bars, $V_{us} = \frac{61.44}{2} = 30.72 kN$.

∴ → S.F left out for Stirrups, $V_{us}' = 61.44 - 30.72$

$= 30.72 kN.$

Design of Stirrups

Provide 6mm dia – 2legged mild steel Stirrups

Spacing of Stirrups from eqn } $V_{us}' = \frac{0.87 f_y A_{sv} d}{S_v}$.

where S_v = Spacing of stirrups

A_{sv} = Area of c.s of stirrup.

on substituting values

$\therefore 30.72 \times 10^3 = \frac{0.87 \times 415 \times 2\pi(16)^2/4}{S_v} \times 370$ (250)

$\Rightarrow \boxed{S_v = 243.52 \text{ mm c/c}}$.

As per IS 456 (47 & 48)

→ spacing of stirrups should not be greater than

i) $0.75d = 0.75 \times 370$
$= 277 \text{ m c/c}$

(ii) Not greater than 300 mm c/c.

→ Min. Shear Reinforcement shall be as per Eqn $\frac{A_{sv}}{b \cdot S_v} \geq \frac{0.4}{0.87 f_y}$ (pg 48 - IS 456).

$\frac{2\pi(6)^2/4}{230 \times S_v} = \frac{0.4}{0.87 \times 250} \Rightarrow S_v = 132 \text{ mm c/c}$

∴ provide the stirrups @ 130 mm c/c.

#2) An RCC beam measuring 230mm x 400mm consists of 2-16mmφ and 2-12mmφ on the tension face. The beam. The beam is provided with 6mmφ M.S stirrups at a spacing of 150mm c/c. and 2-bars of 12mmφ on compression face. Determine the shear capacity of the beam if 2-12mmφ are bend at 45°. use M_{20} concrete and Fe415 steel.

<u>sol</u> <u>step 1</u>

$b = 230mm$; $D = 400mm$, provide eff cover of 30mm

$\therefore d = 400 - 30 = 370mm$; $A_{st} = 2\left(\frac{\pi(16)^2}{4} + \frac{\pi(12)^2}{4}\right)$

$A_{st} = 628 mm^2$

Stirrups : 6mm φ MS - 2 legged

<u>Step 2</u> :

Shear capacity of Beam = Shear taken by concrete + Shear taken by bend bars + Shear taken by stirrups

→ Shear taken by concrete = $V_{us} = \tau_c \times b \times d$

$$\frac{A_{st}}{bd} \times 100 = \frac{628}{230 \times 470} \times 100 = 0.74\,\%$$

From table 19, $\tau_c = 0.56 N/mm^2$

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$\Rightarrow \therefore V_{us} = \dfrac{0.56 \times 230 \times 370}{10^3} = 47.65$ KN.
(Conc)

ii) → Shear taken by bend bars } · $V'_{us} = 0.87 f_y A_{sv} \sin\alpha$

$= 0.87 \times 415 \times \dfrac{2\pi(12)^2}{4} \sin 45 / 10^3$

$= 57.698 \simeq 57.7$ kN

iii) → Shear taken by Stirrups } → $V''_{us} = \dfrac{0.87 f_y A'_{sv} \cdot d}{s_v}$

$= \dfrac{0.87 \times 415 \times 2\frac{\pi}{4} \times 6^2 \times 370}{150 \times 10^3}$

$= 30.04$ kN.

$\therefore$ Shear Capacity of BEAM $= (47.66 + 57.7 + 30.04)$ kN

Min Shear reinfor.

$\dfrac{A_{sv}}{b \cdot S_v} \geq \dfrac{0.4}{f_y}$

$\Rightarrow S_v \leq \dfrac{2.5 A_{sv} \cdot f_y}{b}$

Shear resistance of Min reinforcement is found by subst.

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

$V_{usmin} = 0.87 \left(\dfrac{f_y A_{sv}}{S_v}\right) \cdot d$

$V_{usmin} = 0.87 \times 0.4 bd$
$\simeq 0.35 bd$.

#3 A RCC beam 250mm wide and 400mm Eff depth is subjected to ultimate design SF of 150kN at ~~as~~ the Critical section near support (i.e at a dist d' from support). The tensile reinforcement ~~reinforce-~~ ~~ment~~ at the section near support is 0.5 percent. Design shear stirrups near the support. Also design the ~~main~~ min shear reinforcement at mid span. Assume M20 concrete and M.S bars of Fe250 grade.

sol
Step 1
Given: b = 250mm, d = 400mm ; $\frac{A_{st}}{bd} = 0.5\% = 0.005$
$V_u = 150kN$

step 2 $\tau_{v} = \frac{V_u}{bd} = \frac{150\times10^3}{250\times400} = 1.5 N/mm^2$

from table 19; $\tau_c = 0.48 N/mm^2$ ((M20) + $\frac{A_{st}}{bd}\times100 = 0.5$)

Also from table .. $\tau_{cmax} = 2.8 N/mm^2$ (M_{20})

Thus $\tau_v < \tau_{cmax} > \tau_c$; Hence shear reinforcement is necessary.

step 3
→ $V_{uc} = \tau_c \cdot bd = 0.48\times250\times400\times10^{-3} = 48.0 kN$

→ Hence $V_{us} = V_u - V_{uc} = 150 - 48 = 102 kN$. (Bend)

→ Shear resistance of nominal stirrups (pg 47/48)
$V_{us}(min) = 0.4\, bd$
0.35

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$V_{us-min} = 0.4 \times 250 \times 400 = 40kN < V_{us}$ 51kN

Hence nominal stirrups are not sufficient at the section near supports.

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

using 2-legged stirrups of 10mmϕ bar

$$S_v = \frac{0.87 \times 250 \times \pi(10)^2/4}{102000} \times 400 = 134mm \ c/c$$

Max spacing $= 0.75d = 0.75(400)$ or $= 300mm$ } less

Hence provide 10mmϕ-2legged stirrups @ 130mm c/c at the section near the support.

At mid span, the spacing of min shear reinforcement for 10mmϕ-2legged stirrups is

$$S_v = \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 250 \times \pi\frac{(10)^2}{4}}{0.4 \times 250} = 341.6mm$$

However max spacing is limited to $0.75(400) = 300mm$ or 300mm whichever is less. Hence $S_v = 300mm$

$\therefore$ provide 10mmϕ-2legged ~~bars~~ stirrups @ 300mm c/c at mid span.

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LIMIT STATE OF COLLAPSE IN BOND

one of the most important assumption in the ~~beati~~ behaviour of Reinforced Concrete structure is that there is proper bond between concrete and reinforcing bars. When RC Element is loaded, the load is first borne by concrete and then by it's transferred to steel reinforcement. This transfer of force from concrete to steel can be effected only when there is no relative movement or slip or sliding between them.

"The force which prevents this slippage b/w the two constituent materials is known as bond". The intensity of this adhesive force is called bond stress. The bond stresses are the longitudinal shearing stresses acting on the surface between the steel and concrete along its length. Hence bond stress is the shear stress acting parallel to the reinforcing bar on the interface b/w the bar and the concrete.

Type of Bond

1) Flexural bond or local bond
2) Anchorage bond or development bond

Anchorage bond (τ_{bd}) is that which arise over the length of anchorage provided for a bar. Since bond stresses are developed over a specified length 'L_d', it is also known as development bond stress.

The required length necessary to develop full resisting force is called anchorage length in (L_d) case of axial tension and development length in case of flexural tension.

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

The design bond stress τ_{bd} is specified in the code IS 456-2000

Development Length of Bars (IS-456 2000)

the development length is defined as the length of the bar required on either side of the section under consideration to develop the required stress in steel at that section through bond. (L_d)

LIMIT STATE OF COLLAPSE IN TORSION

Beams which are curved in plan experience a torsional moment 'T'. As per IS 456-2000 Pg (75). Torsional moment is converted into Equivalent BM and Equivalent Shear and the beam is designed structurally for this moment & shear.

→ Longitudinal reinforcement shall be designed to resist an Equivalent BM, 'Me' given by

$M_{e1} = M_u + M_t$ where M_u = BM at the c-s due to applied load.

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

T_u = Torsional moment

D = overall depth of beam

b = width of beam.

→ Equivalent Shear 'Ve' shall be calculated from

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

where V_u = S.F due to applied load $= 1.5 \times wl/2$

Design a beam for equivalent shear and Equivalent BM for the following data:

$M_u = 70$ kN-m ; $V_u = 40$ kN

$T_u = 9$ kN-m ; use M_{20} & Fe 415 grades

provide 30mm Eff. cover.

sol
Design:

Equivalent BM, $M_{e1} = M_u + M_t$

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

$$= 9\left(\frac{1 + 450/230}{1.7}\right)$$

$$\boxed{M_t = 15.65 \text{ kN-m}}$$

$\therefore$ $M_{e1} = 70 + 15.65$

$$\boxed{M_{e1} = 85.65 \text{ kN-m}}$$

→ Moment Capacity of beam (M_{ulim}) = $2.76bd^2$
(M20 + Fe

$M_{ulim} = 2.76 \times 230 \times 420^2 \times 10^{-6}$

$$\boxed{M_{ulim} = 111.98 \text{ KN-m}}$$

$M_{e1} < M_{ulim}$

The beam is design as SRB.

Area of Steel Required:

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

$85.65 \times 10^6 = 0.87 \times 415 \times A_{st} (\cancel{4}20 - 0.42 \times 0.48 \times 420)$

$\Rightarrow$ $\boxed{A_{st} = 708 \text{ mm}^2}$

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Provide 16mmϕ bars.

No. of bars $= \frac{708}{\pi(16)^2/4} \simeq 4$ bars.

<u>Equivalent Shear:</u>

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

$= 40 + 1.6 \times 9/0.23$

$\boxed{V_e = 102.6\ kN}$

→ Shear Stress $\tau_{ve} = \frac{V_e}{bd} = \frac{102.6 \times 10^3}{230 \times 420} = 1.06\ N/mm^2$

→ Shear taken up by Concrete τ_c:

→ $\frac{A_{st}}{bd} \times 100 = \frac{\frac{4\pi(16)^2}{4}}{230 \times 420} \times 100 = 0.83\%$

for 0.83% A_{st} & M_{20} ⇒ $\tau_c = 0.58\ N/mm^2$

∴ Remaining Shear Stress $\tau_{v\,rem} = \tau_{ve} - \tau_c$

$= 1.06 - 0.58$

$= 0.48\ N/mm^2$

∴ Remaining S.F $= 0.48 \times 230 \times 420 \times 10^{-3} = \underline{46.368\ kN}$

→ <u>Bend up bars:</u>

Crank 2 bars of 16mmϕ @ $45°$

→ Shear taken by bent up bars } $V_{us} = 0.87 f_y A_{sv} \sin\alpha$

$= 0.87 \times 415 \times \frac{\pi \times 16^2}{4} \times 2 \times \sin 45$

$= 102.63\ kN$

Bent up bars shall not take more than half the remaining SF.

$\therefore$ Shear taken up by bent bars $\left.\right\}$ $V_{us} = \frac{46368}{2} = 23184\,N$.

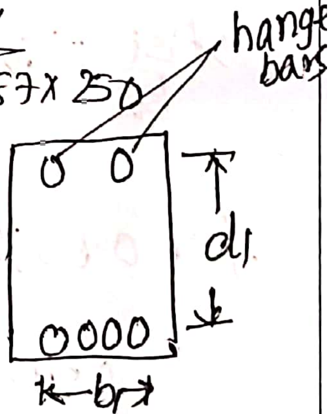
$\therefore$ Shear left out for stirrups $\left.\right\}$ $V_{us}' = 23184\,N$.

Provide 6mm ϕ MS stirrups

→ spacing may be calculated using the Eqn

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

$$2\frac{\pi(6)^2}{4} = \frac{9\times10^6 S_v}{170\times390\times0.87\times250} + \frac{40\times10^3 \times S_v}{2.5\times390\times0.87\times250}$$



$\Rightarrow S_v = 68\,mm\ c/c$

→ The transverse reinforcement A_{sv} shall not be less than $A_{sv} \geq \frac{(\tau_{ve} - \tau_c) b S_v}{0.87 f_y}$

$$2\frac{\pi(6)^2}{4} = \frac{(1.06-0.56)230 S_v}{0.87\times250} = 110\,mm\ c/c = S_v.$$

$\therefore$ Provide 6mm dia - 2legged Stirrups (MS) @ 68mm c/c

LIMIT STATE OF DEFLECTION AND CRACKING

→ **Limit state of deflection:**

Beams subjected to transverse loading experience deflection.

→ Total deflection in R.C.C members shall be taken as the sum of the

1. short term deflection
2. Long term deflection.
   - (a) due to shrinkage
   - (b) Due to Creep.

Short term deflection: may be calculated by usual methods i.e

(i) $\delta = \frac{Wl^3}{48EI}$ (S.S.B with pt. load)

(ii) $\delta = \frac{Wl^3}{3EI}$ (Cantilever with point load)

iii) $\delta = \frac{5wl^4}{384EI}$ (S.S.B. with udl)

where $E = 5000\sqrt{f_{ck}}$ – shortterm modulus of Elasticity of concrete. (IS456) (16)

$I = I_{effective}$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \cdot \frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$ (ref: Pg: 88 IS 456)

but $I_r \leq I_{eff} \leq I_{gr}$

where I_r = M.I of Cracked Section

$M_r = \frac{f_{cr} \cdot I_{gr}}{y_t}$

f_{cr} = Modulus of Rupture of Concrete.

I_{gr} = M.I of gross section about the Centroidal axis neglecting Reinforcement

y_t = Distance of Centroidal axis of gross C.S

M = Max. moment under service load

z = lever arm ; x = depth of N.A

d = Eff. depth ; b_w = breadth of web.

b = breadth of comp. face.

→ The final deflection due to all loads including effects of temperature, creep and shrinkage ≯ $\frac{span}{250}$ before construction of partition walls. and deflection ≯ $\frac{span}{350}$ after construction of partition walls

Control of Deflection:

Deflection in RCC members may be controlled by semi empirical methods based on calculations and tests on beams.

The permissible values for span/depth ratio depends on

(i) Span & support conditions.

ii) percentage of tension reinforcement

iii) percentage of comp "

iv) Type of beam

* For beams & slabs the vertical deflection limits may be assumed to be satisfied provided the following span/depth ratios are not exceeded.

(a)

Beam type	span/depth ratio
Cantilever	7
Simply supported	20
Continuous	26

(b) For spans above 10m values given in (a) may be multiplied by $\frac{10}{span}$ in mts. except for cantilever.

(c) Depending upon the area and type of tension steel the values of span/depth ratio be further modified as per fig (4) IS 456 (38)

(d) Depending upon the area of compression steel the values of span/depth ratio be further modified as per fig (5) IS-456 (38)

→ For beams & slabs it is assumed that vertical deflection limits are satisfied if

$$\frac{\text{Span}}{\text{Eff. depth}} \leq \alpha \beta \gamma \delta \lambda$$

α = basic values of $\frac{\text{span}}{\text{depth}}$ ratio = 7, 20, 26

$\beta = \frac{10}{\text{span}}$

γ = modification factor for tension reinforcement

δ = " " " " compn "

λ = factor for flanged beams which depends on $\frac{b_w}{b_f}$

A rectangular beam has a width of 250 mm and an Eff. depth of 700 mm. The area of tension steel provided is 3927 mm^2 and that of compression is 981 mm^2; Check for deflection requirements for the beam according to IS 456-2000, if it has Simply supported span of 12m. Use M20 & Fe415 grade

Sol $b = 250mm$; $d = 700mm$; $A_{st} = 3927 mm^2$

$A_{sc} = 981 mm^2$; $l = 12m$; given beam is S.S

→ For the beam to be safe in deflection

$\frac{span}{eff\ depth} \leq \alpha\beta\gamma\delta\lambda$ where α = Basic value of $\frac{span}{depth}$

$\boxed{\alpha = 20}$ (IS 456 pg 37)

$\boxed{\beta = \frac{10}{span} = \frac{10}{12} = 0.83}$

γ = modification factor for tension steel (pg 38 - IS456)

→ percentage of tension steel = $\frac{A_{st}}{bd} \times 100$

$= \frac{3927}{250 \times 700} \times 100 = 2.24\%$

from fig (4) $f_s = 0.58 f_y$

$= 0.58 \times 415 = 240 N/mm^2$

from fig(4) $\boxed{\gamma = 0.8}$

δ = modification factor for comp. steel.

$$\left.\begin{array}{l}\text{percentage of}\\ \text{comp. steel}\end{array}\right\} = \frac{A_{sc}}{bd} \times 100$$

$$= \frac{981}{250 \times 700} \times 100 = 0.56\%$$

from fig ⑤ $\boxed{\delta = 1.15}$

λ = Reduction factor for flanged beam.

$\boxed{\lambda = 1.0}$ since beam is rectangular.

Substituting $\alpha, \beta, \gamma, \delta, \lambda$ in standard eqn

$$\frac{\text{Span}}{\text{Eff depth}} = \frac{12 \times 10^3}{700} = 17.14 \leq \alpha\beta\gamma\delta\lambda$$

$$\leq 20 \times 0.83 \times 0.8 \times 1.0$$

$$\leq 13.28$$

$\therefore \frac{\text{Span}}{\text{depth}} \nless 13.28$

$\therefore$ The beam is not safe in deflection.

A rectangular SSB of span 4m has c.s 300x400mm. It is subjected to a moment of 75 kN-m and consists of 4-12ϕ bars on the tension face. Adopt M_{20} & Fe415 grades. Calculate the deflection for applied load (udl).

sol

step1: Given: $b = 300mm$; $D = 400mm$

let $d' = 40mm \Rightarrow d = 360mm$

$A_{st} = \frac{4\pi(12)^2}{4} = 452mm^2$; $M = 75kN\text{-}m$

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

$l = 4m$;

Deflection of a SSB with udl 'δ' $= \frac{5wl^4}{384EI}$

step2 Deflection of a S.S.B with udl

$\Rightarrow E = 5000\sqrt{f_{ck}} = 0.224 \times 10^5$ MPa

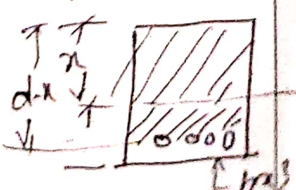
$\Rightarrow w = \frac{M \times 8}{l^2} = \frac{75 \times 10^6 \times 8}{4^2} = 37.5kN/m$

$\therefore I_{eff} = \frac{I_{cr}}{1.2 - \frac{M_{cr}}{M} \cdot \frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b_f}}$

$\boxed{I + Ad^2}$

$\rightarrow I_{cr}$ = M.I of cracked section about NA

$= \frac{bx^3}{3} + mA_{st}(d-x)^2$



x = depth of N.A. :

Taking moment of areas about N.A

$$b \cdot x \cdot \frac{x}{2} = m \cdot A_{st}\,(d - x)$$

$$300\,\frac{x^2}{2} = 13.33 \times 452\,(360 - x)$$

$$\boxed{x = 101.83\,\text{mm}}$$

→ Lever arm, $Z = d - x/3 = 360 - \frac{101.83}{3}$

$$\boxed{Z = 326\,\text{mm}}$$

$$\therefore I_{cr} = \frac{300 \times 101.83^3}{3} + 13.33 \times 452\,(360 - 101.83)^2$$

$$\boxed{I_{cr} = 5.07 \times 10^8\,\text{mm}^4}$$

$$\Rightarrow M_{cr} = \frac{f_{cr} \cdot I_{gross}}{y_t}$$

pg. 15

$$= \frac{0.7\sqrt{20}}{360/2} \times \frac{300 \times 360^3}{12}$$

$$\boxed{M_{cr} = 20.28\,\text{kN.m}}$$

$$\therefore I_{Eff} = \frac{5.07 \times 10^8}{1.2 - \frac{20.28}{75} \times \frac{326}{360}\left(1 - \frac{101.83}{360}\right) \times \frac{1}{1}}$$

$$\boxed{I_{Eff} = 4.95 \times 10^8\,\text{mm}^4}$$

$$\therefore \quad \delta = \frac{5}{384} \times \frac{37.5 \times 4000^4}{0.224 \times 10^5 \times 4.95 \times 10^8}$$

$$\boxed{\delta = 11.24mm}$$