$$P_{safe} = \sigma_{ac} \times A_g$$

$$= \sigma_{ac} \times (\alpha \times t_w) \qquad \alpha = \frac{d}{2} + \frac{d}{2} + \frac{d}{2}$$

$$= f \times 1.5 d \times t \qquad = 1.5 d.$$

$$= 1.5\, f d \cdot t_f$$

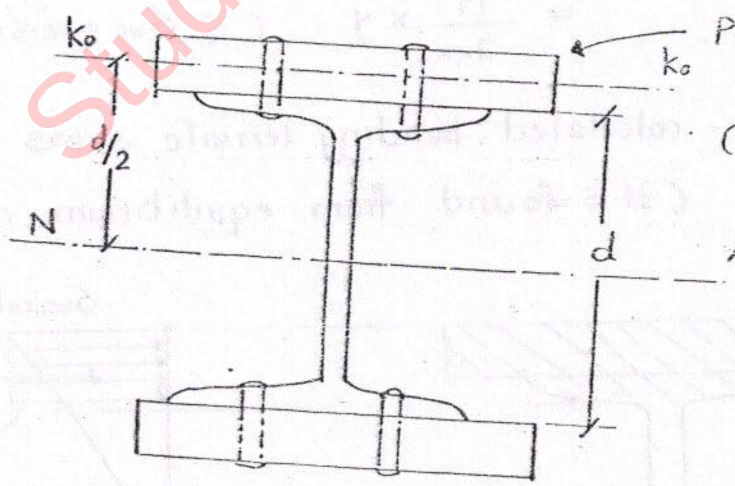
Design of built up beams
(Plated beams)

(i) If a single beam section is not capable of withstanding applied loads, then built up beams are used. (If built-up beams cannot withstand applied loads then plate girders are used.) If plate girders cannot withstand applied load then truss girders are used.

In a truss girders B.M. is taken by top and bottom members shear force is taken by vertical and diagonal members.

(ii)
$$Z_{reqd} = \frac{M}{\sigma_{bc}}$$

(iii) If cover plates are used in built up beams.



$$I_{k_o k_o} \simeq 0$$

For plates,

$$I_{xx} = I_{k_o k_o} + A \cdot h^2$$

$$= \left[0 + A_p \left(\frac{d}{2}\right)^2 \right] \times 2 \quad \text{2 plates, (at top \& bottom)}$$

$$I_{xx} = A_p \cdot \frac{d^2}{2}$$

$$Z_{xx} = \frac{I_{xx}}{y}$$

$$= \frac{A_p \cdot d^2/2}{d/2}$$

$$Z_{plates} = A_p \cdot d$$

$$Z_{reqd} = Z_{beam} + Z_{plates} \quad (Z_{plates} \to A_p \cdot d)$$

A_p - area of plates required on each side

$$A_p = \frac{Z_{reqd} - Z_{beam}}{d}$$

(iv) If riveting is done to take care of area of rivet holes in the tension zone, increase calculated A_p by 40-50%. (because rivets are useless in carrying tension in the plates). If welding is done to connect the plates, calculated A_p need not be increased.

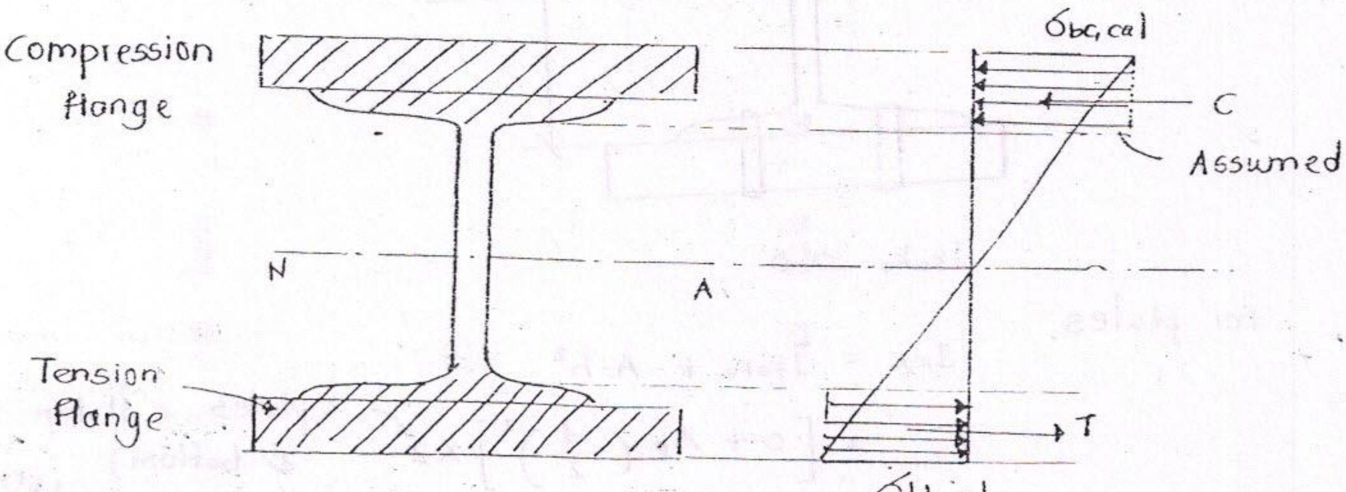
(v)

ⓐ $\sigma_{bc,cal}$ - calculated bending compressive stress in beam

$$= \frac{M}{I_{xx}} \times y \quad (\leq \sigma_{bc} = 0.66 f_y)$$

ⓑ $\sigma_{bt,cal}$ - calculated bending tensile stress in beam.

(It is found from equilibrium consideration)



$$-C+T=0$$

$$C=T$$

$$\sigma_{bc,cal} \times A_g = \sigma_{bc,cal} \times A_{nel}$$

$$\sigma_{bt,cal} = \sigma_{bc,cal} \times \frac{A_g}{A_{nel}} \quad (\leq \sigma_{bt} = 0.66 f_y)$$

where,

A_g - gross area of compression flange alone.

A_t - net area of tension flange alone.

Note:

(i) It is assumed that bending stress distribution over the depth of flange is uniform.

(ii) If welding is done, $A_g = A_{net}$

$\therefore$ $\sigma_{bt,cal} = \sigma_{bc,cal}$.

(iii) In a built up beam. gross area of compression flange is $300\ cm^2$ and net area of tension flange is $270\ cm^2$. If calculated bending compressive stress is 150 MPa then bending tensile stress is.

$$\sigma_{bt,cal} = \sigma_{bc,cal} \times \frac{A_g}{A_{nel}}$$

$$= 150 \times \frac{300}{270}$$

$$= 166.7\ MPa$$

(vi) Check for shear:

$\tau_{va,cal}$ - calculated avg. shear stress in beam

$$= \frac{V}{D \times t_w} \leq (\tau_{va} = 0.4 f_y) \text{ in WSM}$$

$$\tau_{va} = \frac{f_y}{\sqrt{3} \times 1.1} \text{ in LSM.}$$

(vii) Check for deflection:

$(y_{max})_{cal}$ - calculated max. deflection.

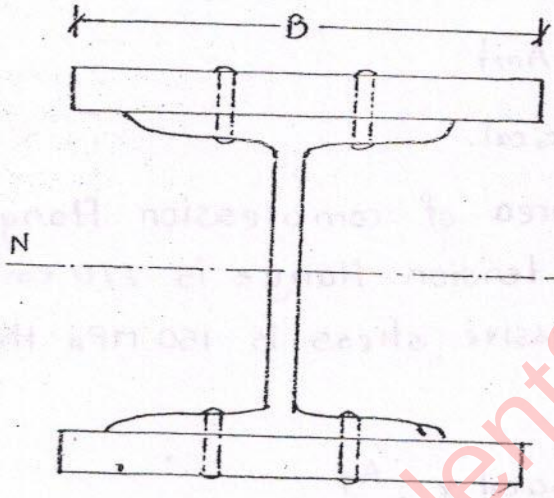
$$\leq y_{max} = \frac{span}{325} \quad \text{(in WSM)}$$

$$= \frac{span}{300} \quad \text{(in LSM) - (don't crack)}$$

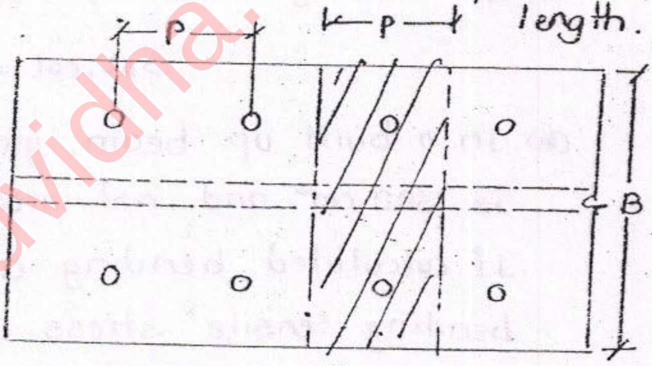
$$= \frac{span}{360} \quad \text{(in LSM)}$$

(if supported elements crack)

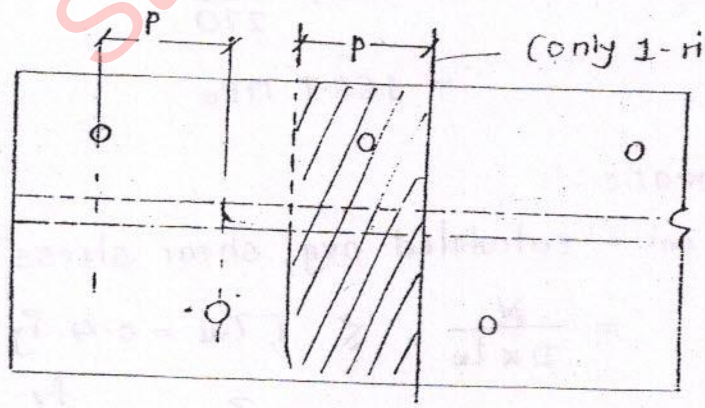
5 Marks (viii) Design of connection
(finding pitch of rivets)



c/s view



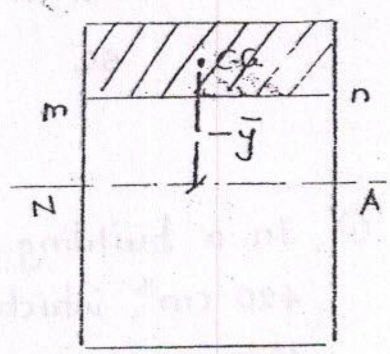
Top view
(When chain riveting is provided)



Top view
(When staggered riveting is provided)

ⓐ From strength of materials :

vertical shear stress at any level mn is given by



$$q = \frac{F(A\bar{y})}{I \cdot b}$$

where

F - S.F. acting at the c/s

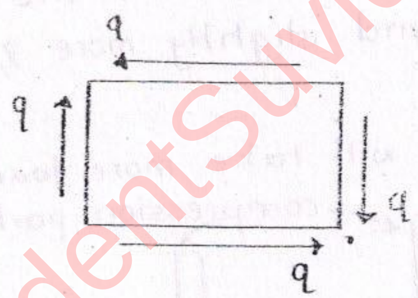
A - area of c/s above level 'mn' only.

$\bar{y}$ - distance between C.G. of shaded area and N.A.

I - M.I. of c/s area about N.A.

b - width of c/s at level 'mn' only.

ⓑ From principle of complementory shear stresses: i.e. the vertical shear stress is always associated with same amoun of horizontal shear stress at any level. to keep the element in equilibrium.



So horizontal shear stress at the junction of flange plate and flange of I-section is

$$q = \frac{F \cdot (A\bar{y})}{I \cdot B}$$

Horizontal shear force per pitch length $= q \times (p \times B)$

$$= \frac{FA\bar{y}}{I \times B} \times (p \times B)$$

$$= \frac{F \cdot A\bar{y}}{I} \times p$$

ⓒ If chain riveting is done

$$\frac{F \cdot A\bar{y}}{I} \times p \leq 2 R_r$$ (2 - because there are 2 rivets in pitch length)

(d) If staggered riveting is provided, there is only one rivet in a pitch length.

so, $\frac{F \cdot A \cdot \bar{y}}{I} \times p \leq R_v$, find 'p'

Q. In a building, a beam requires a section modulus of 420 cm^3, which of the following c/s is most suitable for this purpose...

(A) ISLB 200 @ 0.41 kN/m. $Z_{xx} = 500\ cm^3$

✓(B) ISLB 250 @ 0.37 kN/m $Z_{xx} = 550\ cm^3$ (less weight)

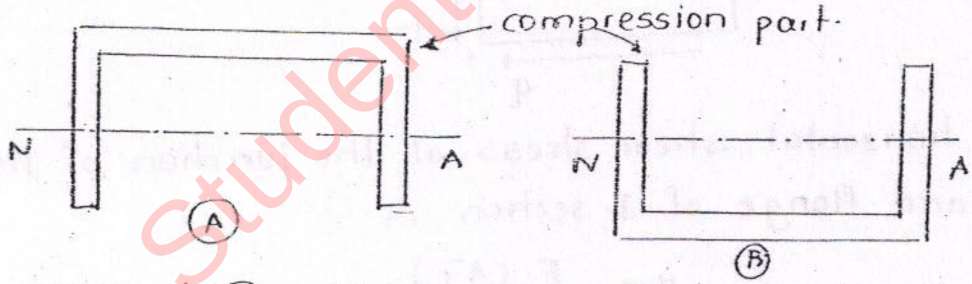
(c) ISWB 600 @ 1.41 kN/m $Z_{xx} = 6060\ cm^3$

(d) ISHB 600 @ 1.37 kN/m $Z_{xx} = 3060\ cm^3$

Note:

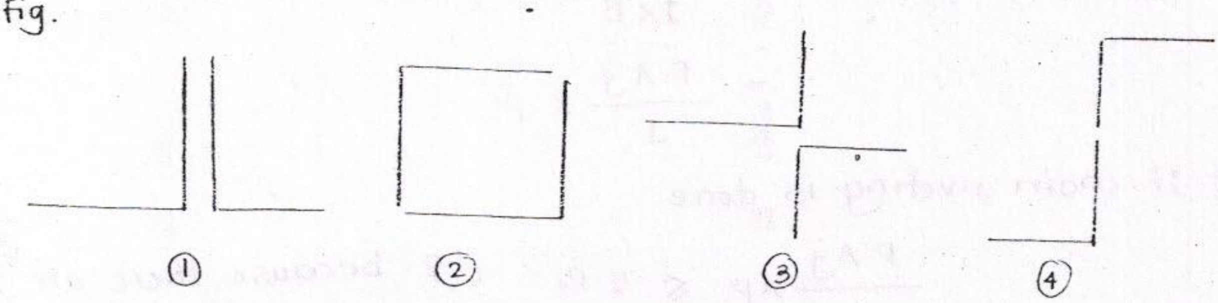
The most suitable section is the one which has a minimum weight and slightly more Z than required.

Q. Which of the following will take more load?



Arrangement (A) will take more load than (B) because the buckling possibility is less in (A), so it can take more load

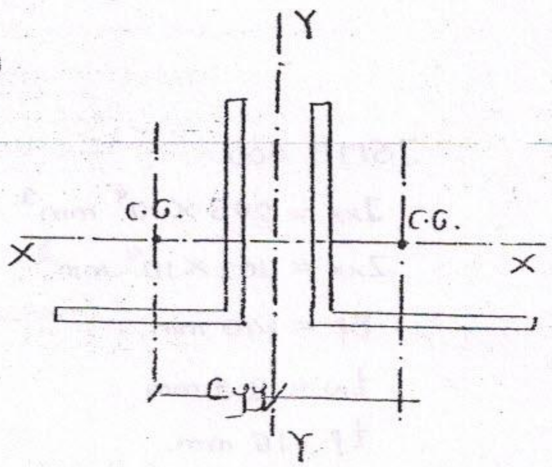
Q Two equal angles form compound column c.s. as shown in fig.



Amongst these, those which have some axial compression load carrying capacity include.

(A) 1 and 2　　(B) 1 and 3

(C) 2 and 3　　✓(D) 3 and 4.

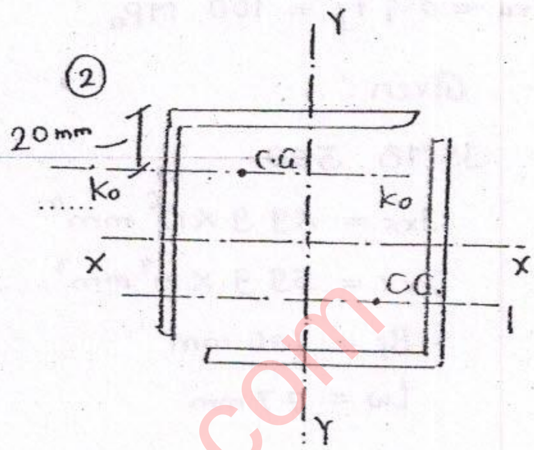
① 

$C_{yy} = 20$ mm (say)

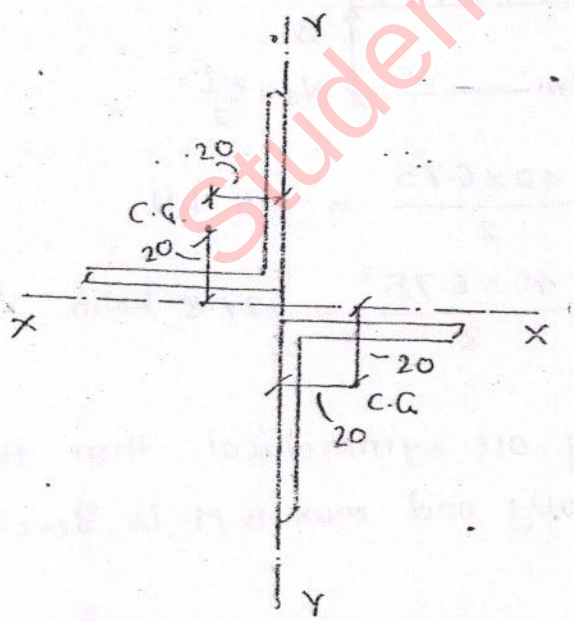
I_{xx} is minimum

(no tranfer formula)

$$P_{cr} = \frac{\pi^2 E I_{max}}{L^2}$$

② 

$$P_{cr} = \frac{\pi^2 E I_{xx}}{L^2}$$

③ 

$I_{YY} = I_{XX} = [I_{CG} + 20^2 \cdot A] \times 2$

Q. A simply supported beam of span 6.75 m to support U.D.L. of 40 kN/m, inclusive of its self weight. Design the beam by making use of ISMB 300 or ISMB 400 which are available. If required flange plates may be used. The beam is assumed to be laterally supported. Take $\sigma_{bc} = \sigma_{bt} = 165\ MPa$. $\tau_{va} = 0.4 f_y = 100\ MPa$.

Given:

ISMB 300

$I_{xx} = 89.9 \times 10^6\ mm^4$

$Z_{xx} = 59.9 \times 10^4\ mm^3$

$B_f = 140\ mm$

$t_w = 7.7\ mm$

ISMB 400

$I_{xx} = 205 \times 10^6\ mm^4$

$Z_{xx} = 102 \times 10^4\ mm^3$

$B_f = 140\ mm$

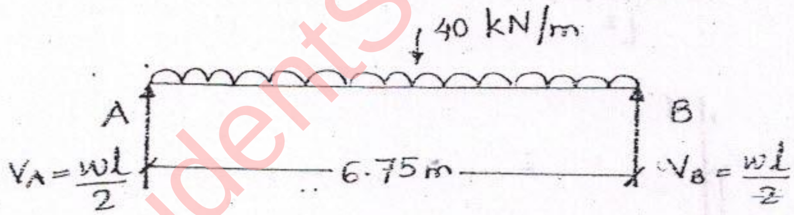
$t_w = 8.9\ mm$

$t_f = 16\ mm$.

(If t_f is not given assume it t_f is always greater than t_w)

Analysis:

(Finding max. S.F., max. B.M., in the beam)



$$\text{Max. S.F.} = \frac{wl}{2} = \frac{40 \times 6.75}{2} = 135\ kN$$

$$\text{Max. B.M.} = \frac{wl^2}{8} = \frac{40 \times 6.75^2}{8} = 227.8\ kNm.$$

Note:

If structure and loading are symmetrical, then the supports share load equally and max. B.M. is developed at centre.

Design :

(It means fixing size of beam, checking for primary criteria - bending. shearing. deflection)

(i) Size of beam :

$$Z_{reqd} = \frac{M}{\sigma_{bc}}$$

$$= \frac{227.8 \times 10^6 \text{ Nmm}}{165 \text{ N/mm}^2}$$

(Since beam is laterally supported $\sigma_{bc} = 0.66 f_y = 165$ MPa)

$$= 138.06 \times 10^4 \text{ mm}^3$$

Select ISMB 400 ($Z = 102 \times 10^4$ mm^3) and provide cover plates

Area of cover plates on each side

$$A_p = \frac{Z_{reqd} - Z_{beam}}{d}$$

$$= \frac{(138.06 - 102) \times 10^4}{400}$$

$$= 901.5 \text{ mm}^2$$

d - depth of beam (d = 400 mm)

Increase A_p by 50% to take care of rivets holes in tensile zone.

∴ A_p required on each side $= 1.5 \times 901.5$

$= 1352.5$ mm^2

∴ Provide each cover plate of size 200 × 8 mm

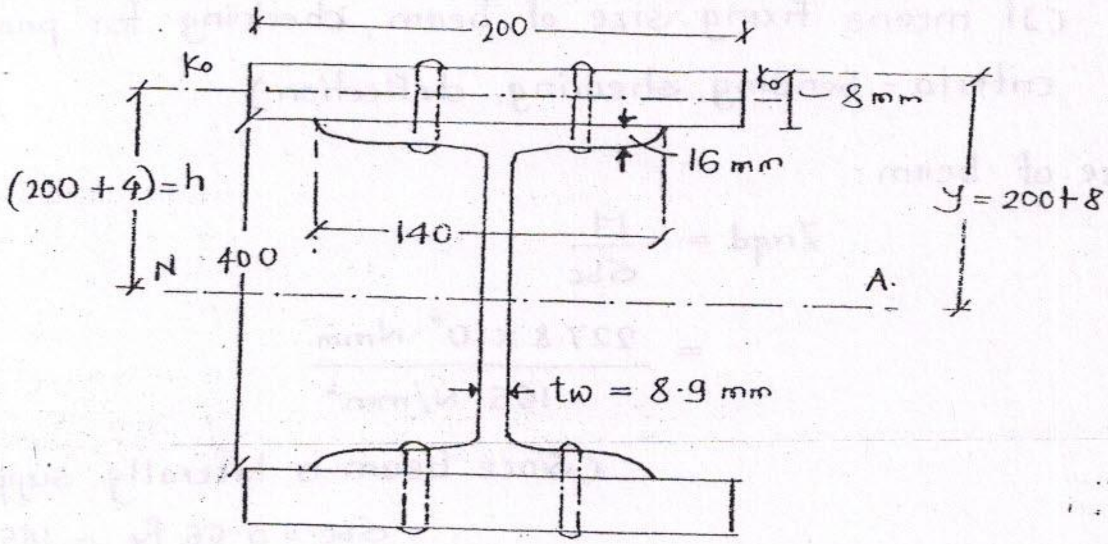
(or 160 × 10 mm or 200 × 10 mm)

Note:

If welding is done, then A_p is not required to be increased by 50%.

So provide A_p = 140 × 8 mm = 1120 mm^2

(or 100 × 10 mm)



c/s of beam

(ii) Check for primary criteria:

ⓐ $\sigma_{bc,cal}$ – calculated bending compressive stress

$$= \frac{M}{I_{xx}} \times y \leq \sigma_{bc} = 165\ MPa$$

$$M = 227.8 \times 10^6\ Nmm$$

$$y = 200 + 8 = 208\ mm$$

$$I_{xx} = 205 \times 10^6 + \left[\frac{200 \times 8^3}{12} + (200 \times 8) \times (200+4)^2\right] \times 2$$

I_{xx} for beam $\quad I_{koko} + (A \times h^2) \quad$ two plates

$$I_{xx} = 338.17 \times 10^6\ mm^4$$

$$\therefore \sigma_{bc,cal} = \frac{227.8 \times 10^6}{338.17 \times 10^6} \times 208$$

$$= 140.1\ N/mm^2 < \sigma_{bc} = 165\ N/mm^2$$

Hence safe.

ⓑ $\sigma_{bt,cal}$ – calculated bending tensile stress

• (it is found from equilibrium condition)

$$C = T$$

$$\sigma_{ct,cal} \times A_g = \sigma_{bt,cal} \times A_{net}$$

$$\sigma_{bt,cal} = \sigma_{bc,cal} \times \frac{A_g}{A_{net}}$$

A_g - gross area of compression flange alone

$= (200 \times 8) + (140 \times 16)$

↑ Flange plate ↑ Flange of I-section

$= 3840\ mm^2$

A_{net} - net area of tension flange alone.

$= A_g$ - area of rivet holes.

Note:

To get more A_{net}, always assume staggered riveting so that only 1 rivet hole is deducted from A_g.

Assum $\phi = 20\ mm$. ∴ $d = 21.5\ mm$

$A_{net} = 3840 - 21.5(8+16)$

↑ thickness of plate ↑ t_f - of I-section

∴ $A_{net} = 3324\ mm^2$

$$\sigma_{bt,cal} = 140.11 \times \frac{3840}{3324}$$

$= 161.85\ N/mm^2 < \sigma_{bt} = 165\ N/mm^2$.

Hence, safe.

Note:

If at all, chain riveting is provided

$A_{net} = A_g$ - area of rivet holes

$= 3840 - 2 \times [21.5 \times (8 + 16)]$

$= 2808\ mm^2$

$$\sigma_{bt,cal} = 140.11 \times \frac{3840}{2808}$$

$= 191.6\ N/mm^2 > \sigma_{bt} = 165\ N/mm^2$

Unsafe.

Always provide staggered riveting.

ⓒ Check for shear :

$\tau_{va,cal}$ - calculated shear stress in web

$$= \frac{V}{D \times t_w} \leq (\tau_{va} = 0.4 f_y)$$

$$= \frac{135 \times 10^3}{(400+8+8) \times 8.9}$$

D- overall depth

$$= 36.4 \ N/mm^2 < (\tau_{va} = 100 \ MPa)$$

Hence, safe.

ⓓ Check for deflection:

(stiffness criteria)

$(y_{max})_{cal}$ - calculated max deflection.

$$= \frac{5 w L^4}{384 EI}$$

$$(y_{max})_{cal} = \frac{5 \times 40 \ N/mm \times (6750)^4}{384 \times 2 \times 10^5 \times 338.17 \times 10^6 \ mm^4}$$

$$= 15.98 \ mm < \left(y_{max} = \frac{span}{325}\right)$$

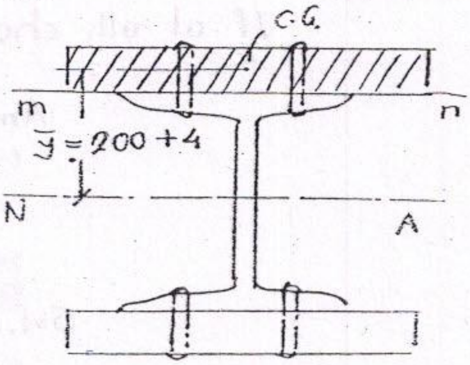
$$< \frac{6750}{325} = 20.76 \ mm$$

(iii) Design of connection :

(finding pitch 'p' of rivets)

Since, staggered riveting is provided.

$$\frac{F \cdot (A \bar{y})}{I} \times p \leq R_v$$



F - S.F in c/s = 135×10^3 N

A - area of c/s above level mn

$= 200 \times 8 = 1600 \ mm^2$

$\bar{y} = 200 + 4 = 204 \ mm$

Rivets are in single shear,

$$P_s = \frac{\pi}{4} \times d^2 \times f_s$$

$$= \frac{\pi}{4} \times (21.5)^2 \times 100$$

$$= 36.3 \text{ kN}.$$

$$P_b = d \times t \times f_b$$

$$= (21.5) \times 8 \times 300$$ t - lesser of 8 mm and 16 mm.

$$= 51.6 \text{ kN}$$

$\therefore$ $R_v = P_s = 36.3$ kN

$$\therefore \frac{135 \times 10^3 \times (200 \times 8) \times (204)}{338.17 \times 10^6} \times p \leq (36.3 \times 10^3)$$

$$p = 278 \text{ mm}$$

But. max. pitch of rivets in compression zone

$= 12t$ } whichever less
$= 200$ mm

$\therefore$ $12t = 12 \times 8 = 96$ mm

Provide pitch of rivets (p) = 96 mm in compression zone.

max. pitch of rivets in tension zone

$= 16t = 16 \times 8 = 128$ mm } whichever is less
$= 200$ mm

Provide, pitch of rivets (p) = 128 mm in tension zone

Note:

If distance between rivet lines is (g) $\not>$ 75 mm, above values can be increased by 50%