- Characteristic Strength → below which atmost 5% of tests fail.

  f_k = Mean strength − 1.64 * Standard Deviation

- Characteristic Load

  f_{kl} = Mean Load + 1.64 * Standard Deviation

- Ductility
  ↳ Ability of Material to change shape (capacity to undergo large inelastic deformation) without fracture (w/o significant loss of strength or stiffness)
  ↳ Gauge Length = $5.65\sqrt{A_0}$
  ↳ % elongation = $\dfrac{(\text{Final elongated gauge length} - \text{Initial gauge length})}{\text{Initial gauge length}} \times 100$

-

Ajanta Page No. Date

Home assignment

(Q.1) Please classify steel sections according to their local buckling behaviour.

→ 1. Plastic cross-sections (Class 1)
↳ cross-sections which can develop plastic hinges can attain full plastic moment M_p, and can undergo large inelastic rotation required for failure of structure by formation of a plastic mechanism.
↳ allows re-distribution of moments
↳ only sections used in indeterminate frames forming plastic collapse mechanism.

2. Compact (class 2)
↳ develop plastic hinge and attain full plastic moment
↳ but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism before buckling. (not adequate ductility)
↳ stress-distributions → rectangular

3. Semi-compact (Class 3)
↳ extreme fiber in compression can reach yield stress (assuming an elastic distribution of stress), but cannot develop plastic moment of resistance due to local buckling.
↳ not capable of reaching a fully plastic stress distribution
↳ stress-distributions → triangular.

4. Slender (Class 4)
↳ elements buckle locally even before attainment of yield stress; at moment less than yield moment
↳ such sections used in cold-formed members

Ajanta Page No. Date

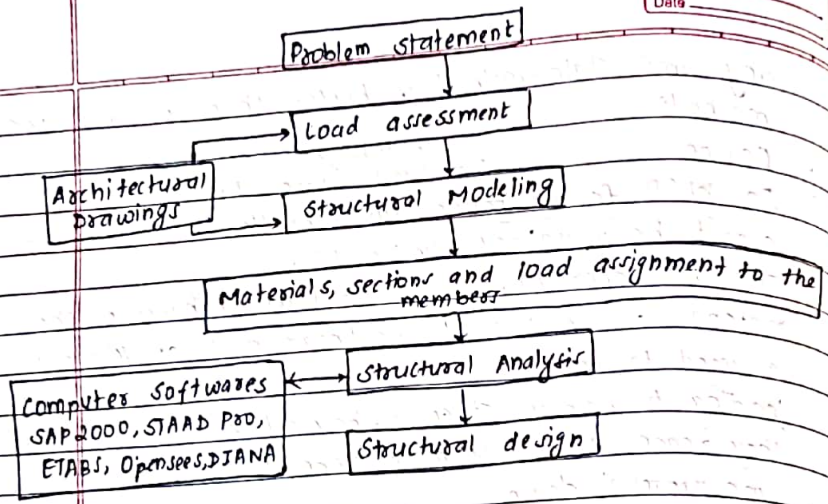
(Q.2) Why limit state method is better than working stress method?

~~WSD → Working stress Design.~~
LSD → limit state Design

→ Reasons:
(i) In WSD, failure mode (ductile/brittle) of structure cannot be visualized. unlike LSD
(ii) In WSD approach, the stress-strain behiour of the material is considered to be linear and structure is assumed to behave linearly in an elastic manner, leading to reserve strength. beyond elastic limit until they reach their ultimate strength and lead to uneconomical design.
(iii) LSD is recently developed approach based on probabilistic theory for designing steel structure while WSD is traditional approach based on elastic theory.
(iv) LSD leads to conservative design, than WSD
(v) LSD is suitable for all types of structure while WSD is suitable only for determinate structure.
(vi) LSD makes use of partial safety factors applied to both loads and strength, calibrated using reliability methods while WSD uses factor of safety based on engineering judgement.

(Q.3) Write design process and basis of design of steel structure.

→ A structure should satisfactorily support the anticipated loads for the intended time for which the structures are designed and serve the useful function. Steel structures should be designed to satisfy design requirements for stability, strength, serviceability, brittle facture, fire and durability



(Q.4) What is structural steel? Explain classification of structural steel sections.

→ Structural steel possesses attributes such as strength, ductility, stiffness, and toughness that are most desirable in modern steel constructions.

Bureau of Indian standards classifies structural steel as:

Grade Designation	UTS (MPa)	Yield strength (MPa)	Minimum % Elongation	Internal Bend Diameter Min (≤25)
Fe 410 (E250)	410	250	23	2t
E 275*	430	265	22	2t
E 300 (Fe 440)	440	290	22	2t
E 350 (Fe 490)	490	330	22	2t
E 410 (Fe 540)	540	390	20	2t
E 450 (Fe 570)	570	430	20	2.5t
E 550*	650	530	12	3t
E 600*	730	580	12	3.5t
E 650*	780	630	12	4t

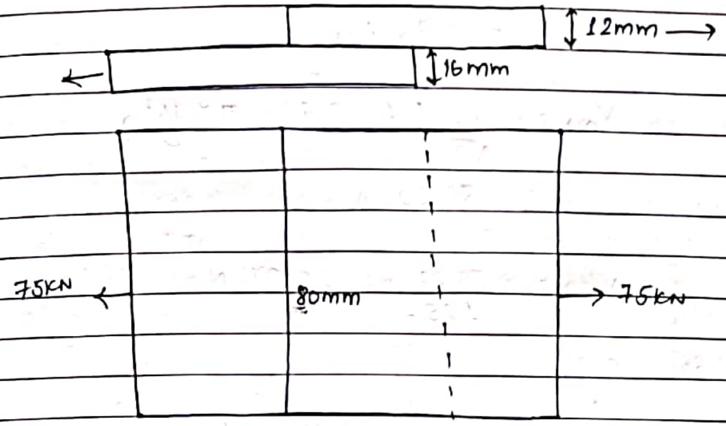
* → Grade designation not listed in IS:800-2007

t → thickness of test piece

Connection in steel structures

$A_{nb} = 0.78 A_{sb}$

Q(1) Design a lap joint b/w two plates to transmit a factored load of 75kN using Grade 4.6 M16 bolts and grade 410 steel plates



→ Solution,

For grade 4.6 bolts, $f_{ub} = 400\ N/mm^2$

For grade 410 plates: $f_u = 410\ N/mm^2$

(a) Check for shear (single shear)

No. of shear planes = 1

Nominal diameter of bolt = 16mm (d)

Hole diameter, $d_0 = 16 + 2 = 18mm$ (Table 19)

$\boxed{\text{Clause 10.3.3}}$

Design strength of bolt in shear,

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

working load →

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

Here,

$n_n = 1$ (shear plane is through threaded portion, Assume) ← (better than assuming shank region)
(safer side)

$n_s = 0$

$A_{nb} = 0.78\, A_{sb}$

$$V_{nsb} = \frac{400}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi \times 16^2}{4}\right]$$

$= 36.22$ kN

Table 5 (clause 5.4.1) → [$\gamma_{mb} = 1.25$]

$$V_{dsb} = \frac{36.22}{1.25} = 28.97 \text{ kN}$$

(b) check for bearing: [clause 10.3.4]

Design bearing strength of bolt,

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$V_{npb} = 2.5 k_b d t f_u \quad \text{(nominal value)}$$

f_{ub} → ultimate tensile strength of bolt

f_u → of plate

k_b (minimum of :) (clause 10.2.4.2)

(i) $\frac{e}{3d_0} = \frac{(1.5 * d_0)}{3 d_0} = 0.5$ ($p = 2.5d$ ⇒ cl. 10.2.2)

(ii) $\frac{p}{3d_0} - 0.25 = \frac{(2.5 * 16)}{3 * 18} - 0.25 = 0.49$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(iv) 1.0

Adopt, minimum of above

i.e. $k_b = 0.49$ ← minimum

$$V_{npb} = 2.5 \times 0.49 \times \underset{(d)}{16} \times \underset{(t)}{12} \times 410$$

$= 96.432$ kN

$$V_{dpb} = \frac{96.432}{1.25} = 77.1 \text{ kN}$$

Thus,

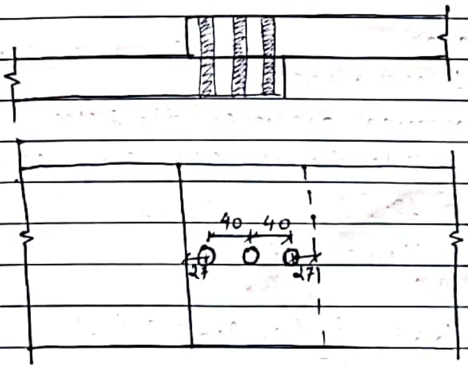
Bolt value (strength) $= 28.97$ kN ← Design strength

Required number of bolts $= \frac{\text{Factored load}}{\text{Bolt value}}$

$= \frac{75}{28.97}$

$= 2.5$

≈ 3



Load → (per unit width)

Let's provide 3-M16 bolts of grade 4.6
with pitch, $p = 2.5 * 16 = 40mm$ (Cl. 10.2.2)
end distance, $e = 1.5 * 18 = 27mm$
(Cl. 10.2.4.2)

Q.(2)
(a) Calculate the strength of a 20mm diameter bolt of grade 4.6 to join two main plates (Fe 415) 10mm thick with lap joint.

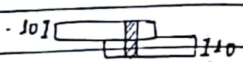
(b) Calculate the strength of a 20mm dia. bolt of grade 4.6 to join two main plate (Fe 415) 10mm thick with single cover butt joint with the 8mm thick cover plate (Fe415)

(c) Calculate the strength of a 20mm dia. bolt of grade 4.6 to join two main plates (Fe415) 10mm thick with double cover butt joint with the 6mm thick cover plate (Fe 415)

→ Solution,

(a) $f_u = 415 MPa$ (Fe415)
$f_{ub} = 400 MPa$ (for 4.6 grade bolt)

For shear:



1 shear plane
Nominal diameter $(d) = 20mm$
Hole diameter $(d_0) = 20+2 = 22mm$ [Table 19]

Clause 10.3.3:

$$\text{design } V_{dsb} = \frac{V_{nsb} \text{ (nominal)}}{\gamma_{mb}}$$

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$n_n = 1, n_s = 0$

$A_{nb} = 0.78 \times A_{sb}$

$$V_{nsb} = \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \pi \times \frac{20^2}{4} \right)$$

$= \cancel{245.044} = 56.590 kN$

From table 5, $\gamma_{mb} = 1.25$

Thus,

$$V_{dsb} = \frac{56.590}{1.25} = 45.272 kN$$

For bearing:

Clause 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} \text{ — (1.25) — table 5}$$

$V_{npb} = 2.5 k_b d t f_u$
$\cancel{= 2.5 \times}$

k_b — 10.2.4.2

(i) $\frac{e}{3d_0} = \frac{(1.5 d_0)}{3d_0} = 0.5$ — 10.2.2

(ii) $\frac{p}{3d_0} - 0.25 = \frac{2.5 \times 20}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{415} = 0.964$

(iv) 1.0

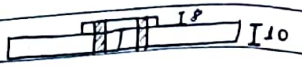
Adopting, $k_b = 0.5$
$d = 20mm, t = 10mm$

$$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 10 \times 415}{1.25}$$

$= 83 kN$

So,
strength of bolt = bolt value = 45.272 kN.

(b) 8mm thick cover plate, single cover butt joint



For shear

$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ (clause 10.3.3)

$= \frac{f_{ub}}{1.25} \cdot \frac{1}{1.25} \times \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$

[taking $n_n = 1$ & $n_s = 0$] → both in single shear

$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} (1 \times 0.78 \times \frac{\pi \times 20^2}{4})$

$= 45.272 kN$

For bearing: (clause 10.3.4)

$V_{dpb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{2.5 k_b d t f_u}{1.25}$ — Table 5

$k_b = 0.5$ (same as a)

$d = 20mm$

$d_0 = 22mm$

$t = 8mm$ (taking least of 10mm or 8mm) — As Bearing is governed by 8mm thick cover plate

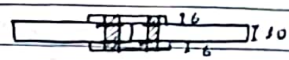
$f_u = 415 MPa$

$V_{dpb} = \frac{2.5 * 0.5 * 20 * 8 * 415}{1.25} = 66.4 kN$

So,
strength of bolt = bolt value = 45.27 kN.

(c) double cover butt joint with 6mm thick cover plate.

(double shear)



For shear :

$V_{dsb} = \frac{1}{\gamma_m} \times \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$ (Clause 10.3.3)

[$n_n = 2$ & $n_s = 0$] two shear planes on threaded region only.

$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} (2 \times 0.78 \times \frac{\pi \times 20^2}{4})$

$= 90.544 kN$

For bearing: (clause 10.3.4)

$V_{dpb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{1.25} \times 2.5 \times k_b \times d \times t \times f_u$ — minimum govern

$k_b = 0.5$ (same as a)

$d = 20mm$

~~t = 6 mm~~ : t = 10mm (Strength is governed by 10mm thick main plate) — sum of two cover plates thickness (12mm) > thickness of main plate (10mm)

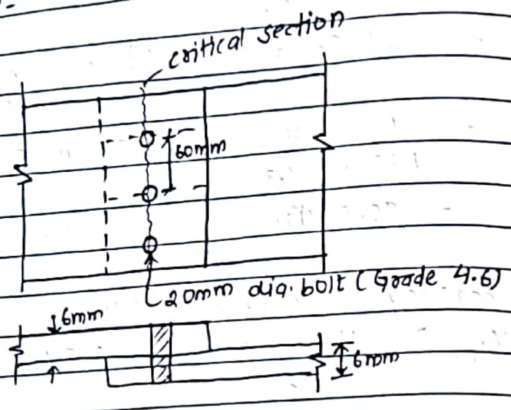
$V_{dpb} = \frac{2.5 * 0.5 * 20 * 10 * 45}{1.25}$ N

$= 83 kN$

Strength of bolt = bolt value = 83 kN

(Q.3) Calculate the efficiency of the joints connecting plates (Grade 410) of 6mm thick using a single bolted lap joint with a 20mm diameter bolts (4.6 grade) at 60mm pitch.

→ Solution:-



Bolt:

(i) Strength in single shear

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \quad \text{(Clause 10.3.3)}$$

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$f_u = 410\,MPa$
$f_{ub} = 400\,MPa$ ~~(for 4.6 grade)~~
Not from code.

$$= \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \pi \times \frac{20^2}{4} \right)$$

$$= 56.59\,KN$$

$$\therefore V_{dsb} = \frac{56.59}{1.25} = 45.272\,KN \text{ (for single bolt)}$$

(ii) Strength in bearing (Clause 10.3.4.)

Assume, $e = 1.5 d_0 = 33mm$ [Cl. 10.2.4.2]

$p = 2.5d = 2.5 \times 20$ [Cl. 10.2.2]

$= 50mm$

K_b →
- $\frac{e}{3d_0} = 0.5$
- $\frac{p}{3d_0} - 0.25 = 0.667$ (take 60)
- $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.96$
- 1

Adopt, 0.5.

6mm (main plate thickness)

$$V_{npb} = 2.5 k_b d t f_u$$

$$= 2.5 \times 0.5 \times 20 \times 6 \times 410$$

$$= 61.5\,KN$$

$$V_{dpb} = \frac{61.5}{1.25} = 49.2\,KN \text{ (for single bolt)}$$

Thus,

Strength of bolt = Bolt value = 45.272 KN (per pitch length)

Nob (overall width not given)

Plate: (Per pitch length मा कति वटा hole पर्छ)
Here 1 hole per pitch length.

Strength of net section for pitch length

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \quad (\text{Cl. } 6.3.1)$$

$$= \frac{0.9 \times [(60-22) \times 6] \times 410}{1.25}$$

$$= 67.3 \text{ KN}$$

$$A_n = \left[b - n d_h + \sum \frac{p_{si}^2}{4 g_i}\right] t$$

comes into play only for zig-zag bolts.

→ Strength of joint (per pitch length) is governed by 45.27KN (i.e. strength of bolt in single shear)

For efficiency compare with original strength of plate
↳ no holes

Original strength of plate

$$= \frac{0.9 \times f_u \times A_n}{\gamma_{m1}}$$

$$= \frac{0.9 \times 410 \times 60 \times 6}{1.25}$$

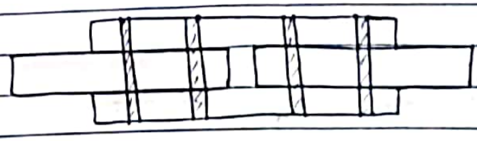
$$= \cancel{10.6} \; 106.3 \text{ kN}.$$

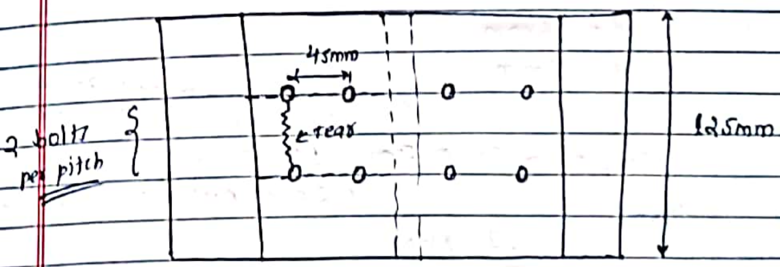
For joint efficiency ...
↳ Bearing → bolt/plate
'or' strength of joint → ↳ Tearing → plate

Thus,

$$\text{Joint efficiency} = \frac{45.27}{106.3} \times 100 = 42.6\%$$

(Q.4) Figure below shows a double-cover butt joint connecting two plates 8mm thick with cover plates 6mm thick using 16mm diameter bolts of 4.6 Grade. Calculate the efficiency of joint per pitch length. Assume Fe410 steel plates.





→ Solution:-

For shear: (Cl. 10.3.3.)
(double shear)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}} \times \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \left(2 \times 0.78 \times \frac{\pi \times 16^2}{4}\right)$$

$$= 57.95 \text{ KN}.$$

Page No.
Date

For bearing:- (cl. 10.3.4.)

$$V_{dpb} = \frac{V_{npb}}{\gamma_m} = \frac{2.5 k_b d t f_u}{\gamma_m}$$

10.2.4.2

$$k_b \quad \frac{e}{3d_0} = \frac{1.5 d_0}{3 d_0} = 0.5$$

$$45mm \leftarrow \frac{p}{3d_0} - 0.25 = \frac{\cancel{2.5 \times 16}\ 45}{3 \times 18} - 0.25 = 0.583$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

1.0

Taking, $k_b = \cancel{0.491}\ 0.5$

$d = 16mm$
$t = 8mm$

strength governed by main plate.

$$V_{dpb} = \frac{2.5 \times 0.5 \times 16 \times 8 \times 410}{1.25} \leftarrow \text{partial safety factor.}$$

design

$= 52.48\,kN$

Bolt value = strength of bolt = 52.48 kN (min^m of above 2)

- In one pitch, length there are two bolts.

(6.3.1) Net tensile strength of ~~bolt~~ plate per pitch length $(T_{dn}) = \frac{0.9 f_u A_n}{\gamma_{m_1}}$

Tear होने portion की Area

$A_n = (45 - 18) \times 8 = 216\,mm^2$

(minimum of 8mm or 2*6mm)

Page No.
Date

$$T_{dn} = \frac{0.9 \times 410 \times 216}{1.25} = 63.76\,kN.$$

Solid → strength of solid plate per pitch length

$$= \frac{0.9 \times 410 \times 45 \times 8}{1.25}$$

$= 106.27\,kN$

Efficiency of joint (per pitch length) $= \frac{63.7}{106.27} \times 100$

$= 59.9\%$

why 63.7?

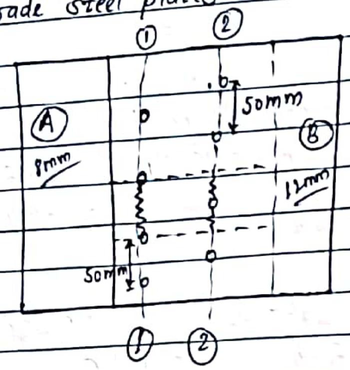
strength of joint is governed by (as 2 bolts per pitch) → 2 * Bolt value (2*52.48) → $T_{dn} = 63.7kN$ (Minimum of two) i.e., 63.7kN

Bolt value ↓ Always for single bolt

SURENDRA SHARMA

$d_0 = 16+2 = 18mm$ (Table 19)

(Q.5) Figure below shows a zigzag double-bolted lap joint connecting two plates 8mm thick using 16mm diameter bolts of 4.6 Grade. Calculate the efficiency of joint. Assume Fe410 grade steel plates.



← tear section

Ⓐ → ① critical
Ⓑ → ② critical.

[If A 't' = 8mm & B 't' = 12mm, A becomes critical]

→ Soln:-

For 4.6 Grade bolt

$f_{ub} = 400$ MPa

(Fe 410) $f_u = 410$ MPa.

Bolt:
(In single shear)

① Shear: $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ [Cl. 10.3.3]

$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi \times 16^2}{4}\right]$

$= 36.217$ kN.

$\Rightarrow \left\{ V_{dsb} = 28.97 \text{kN} \right.$

② Bearing: [Cl. 10.3.4]

$V_{dpb} = \frac{V_{npb}}{\gamma_m} = \frac{2.5 k_b d t f_u}{\gamma_m}$

[If A = 8mm & B = 12mm, 8mm becomes critical for bearing so take that]

K_b — 10.2.4.2

$\frac{e}{3d_0} = \frac{1.5 d_0}{3 d_0} = 0.5$

$\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.676$

$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

1.0

Adopt, $k_b = 1.0$

$V_{dpb} = \frac{2.5 * 0.5 * 16 * 8 * 410}{1.25}$

$= 52.48$ kN.

Thus, bolt value = 28.974 kN (~~per pitch length~~) of single bolt

(iii) Plate A (8mm thick)

For complete failure,

Line 2-2 : Tearing of plate + 1 bolt of 1-1

'or' Line 1-1 : Tearing of plate "

[Cl. 6.3.1]

Net strength of plate Ⓐ (1-1) $T_{dn} = 0.9 * A_n * \frac{f_u}{\gamma_m}$

$A_n = \left[b - n d_h + \sum_i \frac{p_{si}^2}{4g}\right] t$ — not considering this

Per pitch,

b = 50mm

b-

$A_n = [50 - 18 \times 1] \times 8 = 256 mm^2$

$T_{dn} = 0.9 \times \frac{256 \times 410}{1.25}$

$= 75.57 KN$ (per pitch length)

But, since per pitch length there are 2 bolts so need to compare with 2 * bolt value.

strength of 2 bolts $= 2 \times 28.974 = 57.948 KN$ (per pitch length)

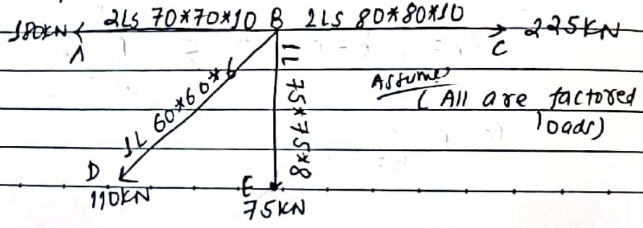
Thus,
strength of joint $= 57.94 KN$
(iii) (min = strength per pitch of plate)

(iv) For efficiency,
Strength of solid plate $= 0.9 \times 50 \times 8 \times \frac{410}{1.25}$

$= 118.08 KN$

Efficiency $= \frac{57.94}{118.08} \times 100 = 49.068\%$

(Ex. 7) Design the joint B of the truss as shown in figure below. The members are connected with 20 mm diameter bolts (Grade 4.6) to the gusset plate 12 mm thick.



Assume (All are factored loads)

2Ls → double angle section
1L → single angle section
[2 Ls]
[1 L]

→ Solution:-

Assume, Gusset plate strength Fe 410
so,
$f_u = 410$ ~~KN~~ MPa
$f_{ub} = 400$ MPa (4.6 Grade)

Assume
$p = 2.5d = 50mm$ (d = 20)
$e = 1.5 d_0 = 33 mm$ (d_0 = 22)
cl. 10.2.2
cl. 10.4.2.2

(1) Shear:
[Cl. 10.3.3]
For AB, & BC → double shear
For BD & BE → single shear

$\boxed{AB}$

$V_{dsb} = \frac{V_{nsb}}{\gamma_m} = \frac{1}{\gamma_m} \times \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s \cdot A_{ns})$

$[n_n = 2 \; \& \; n_s = 0]$

$V_{dsb} = \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \left(2 \times 0.78 \times \frac{\pi \times 20^2}{4} \right)$

$= 90.545 KN$

$\boxed{BC}$ $V_{dsb} = \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \times \left(2 \times 0.78 \times \frac{\pi \times 20^2}{4} \right)$

$= 90.545 KN$

[BD & BE] → single shear, $n_n = 1$ & $n_s = 0$

$V_{dsb} = \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi \times 20^2}{4}$

$= 45.272 KN.$

Page No.
Date

(ii) Bearing.
[Cl. 10.3.04]

$d = 20mm$
$d_0 = 22mm$ → Table 11

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 K_b d t f_u}{\gamma_{mb}}$$

K_b

$$\frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5$$

$$\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

1.0

Adopt, 0.5 → for all member.

[AB & BC] → $t = 12mm$ governs strength.
$(2 \times 10 > 12)$.

So,

$$V_{dpb} = \frac{2.5 * 0.5 * 20 * 10 * 410}{1.25} = 98.4 KN.$$

1.25 → tables

[BD] → $t = 6mm$ governs strength.

$$V_{dpb} = \frac{2.5 * 0.5 * 20 * 6 * 410}{1.25} = 49.2 KN.$$

[BE] → $t = 8mm$ governs strength.

$$V_{dpb} = \frac{2.5 * 0.5 * 20 * 8 * 410}{1.25} = 65.61 KN$$

IS 873:1994
(4th rev.)

Steel table
IS 808:1989
(APB tables)
where to keep bold

Bolt values
[AB] → ~~82KN~~ ~~98.4~~ 90.54KN
[BC] → ~~82KN~~ 90.54KN
[BD] → 45.272KN
[BE] → 45.272KN

Since factored load given.
Number of bolts: = Factored Load / Bolt values

$$[AB] \rightarrow \frac{180}{\cancel{82}\ 90.54} = 1.99 \approx 2$$

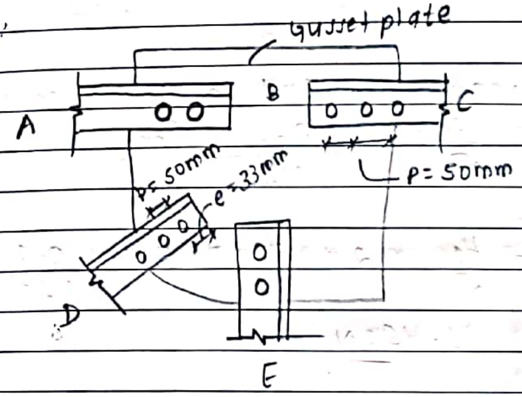
$$[BC] \rightarrow \frac{225}{\cancel{82}\ 90.54} = 2.48 \approx 3$$

$$[BD] \rightarrow \frac{110}{45.272} = 2.43 \approx 3$$

$$[BE] \rightarrow \frac{75}{45.272} = 1.66 \approx 2.$$

Total number of bolts = 10.

Figure:



(Cl. 10.3.3.3)

(packing plate needed)

[Q. 6] Design a double-cover butt joint to connect two plates 10mm and 18mm thick (Fe 410) using 4.6 Grade bolts of 20mm diameter. Use 8mm thick cover plates on both sides. The joint takes a factored design load of 750 kN.

Here,
Packing plates (8mm thick) is required.

[Cl. 10.3.3.3]
Strength of bolt will be reduced by a coefficient, $\beta_{pkg} = 1 - 0.0125 * t_{pkg}$

$= 0.9$

① Shear strength of bolt in double shear,

(Cl. 10.3.3) $V_{dsb} = \dfrac{f_{ub}(2 * A_{nb}) * \beta_{pkg}}{\sqrt{3}\,\gamma_{mb}}$

$= \dfrac{400}{\sqrt{3}} \times 2 * 0.78 * \pi * \dfrac{20^2}{4} * \dfrac{0.9}{1.25}$

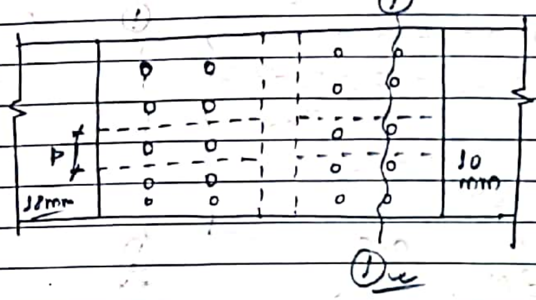
$= 81.49 kN$

Assuming the strength of bolt is governed by shear

Bolt value = 81.5 kN

No. of bolts required $= \dfrac{750}{81.5} = 9.2 \approx 10$ (in each plate)

(Arrange in two columns)
(10 on each side of joint)



Assuming two bolts per pitch length,
[Cl. 6.3.1]

(For efficient joint)

Net tensile strength of plate $= 0.9 \times (p-22) * 10 * 410 / 1.25$ — governed by 10mm thick plate.

$\dfrac{0.9 * (p-22) * 10 * 410}{1.25}$ (per pitch length) $= 2 *$ Bolt value.
(81.5 kN×1000)

$\Rightarrow p = \dfrac{2 * 81.5 * 1.25 * 1000}{0.9 * 10 * 410} + 22$

[$p = 77.217 mm$]

CamScanner

From clause, 10.2.2,
[$P_{min} = 2.5d = 2.5 \times 20 = 50mm$]

Let's adopt, pitch, $p = 80mm$.

Edge distance, $e = 1.5d_0 = 1.5 * 22 = 33mm$

for, $p = 80mm$, strength of plate will increase
thus, will be safe in tension.

Now,
check on Bearing:

$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$$

k_b

$$\frac{e}{3d_0} = 0.5$$

$$\frac{p}{3d_0} - 0.25 = \frac{80}{3*22} - 0.25 = 0.962$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

1.0

Adopt $k_b = 0.5$.
$d = 20mm$
$t = 10mm$
$f_u = 410KN$

Thus,

$$V_{dpb} = \frac{2.5 * 0.5 * 20 * 10 * 410}{1.25}$$

$= 82KN$

[$V_{dpb} = 82KN > 81.5KN$]

If $V_{dpb} \leq 82KN$
↓
Just change edge distance
make it higher
as k_b governed by e
[easier approach]

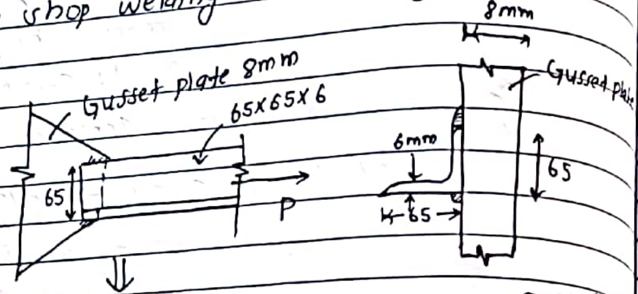
But if p governs → redesign

SURENDRA SHARMA

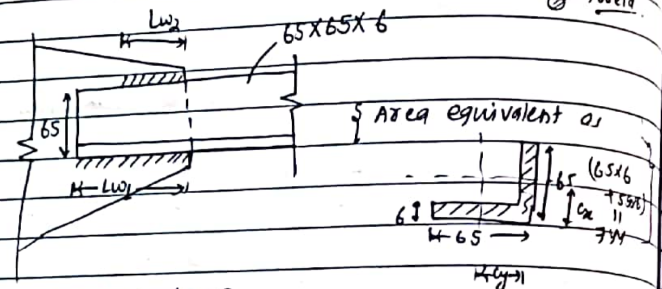
E250 ≡ Fe410

Problem 1: A tie member of a truss ~~section~~ consists of an angle section ISA 65x65 x 6. The angle section is welded to a gusset plate 8mm thick. Design a weld (fillet) to transmit a load equal to the full strength of the member. Assume shop welding and E250 grade steel.

→ Soln.



⇓



From IS 808: 1989,

$A = 7.44 \times 10^2 \ mm^2 = 7.44 cm^2$

Since, equal angle section,

$c_x = c_y = 1.81 cm = 18.1 mm$

For E250 (Fe 410) steel grade,

$f_u = 410 MPa$

$f_y = 250 MPa$

For shop welding, $\gamma_{mw} = 1.25$ (Table 5)

Partial safety factor for yielding,

$\gamma_m = 1.10$ (Table 5)

Full strength of member = (fully yield भएर plastic strain attain गर्ने बेलाको)

= Design strength of member governed by yielding of gross-section

$= \frac{A \times f_y}{1.1}$

$= \frac{744 \times 250}{1.1}$

$= 169.1 KN$

(यो strength angle-section को तल पट्टी र माथीपट्टी कति share गर्दछ? →) $c_x = 18.1 mm$ (nearer to lower plate)

Force resisted by the weld at lower side of angle (P_1),

$P_1 * 65 = 169.1 * (65 - 18.1)$ {Moment about upper part}

$\Rightarrow [P_1 = 121.9 KN]$

Force resisted by weld at upper side of angle (P_2),

$[P_2 = 169.1 - 121.9 = 47.2 KN]$

Size of weld: → (i) >3mm [Cl. 10.5.2.3]

→ (ii) $\leq 6 - 1.5 = 4.5 mm$ → square (1.5mm घटाउने)

→ (iii) $\frac{3}{4} * 6 = 4.5 mm$

Thus,

$s = 4.5 mm$,

Throat thickness, $t = k * s = 0.7 * 4.5$

$= 3.15 mm$

Page No.
Date

Strength of weld at lower side

$= (L_{w_1} * t) * \frac{f_u}{\sqrt{3}} * \frac{1}{\gamma_{mw}}$

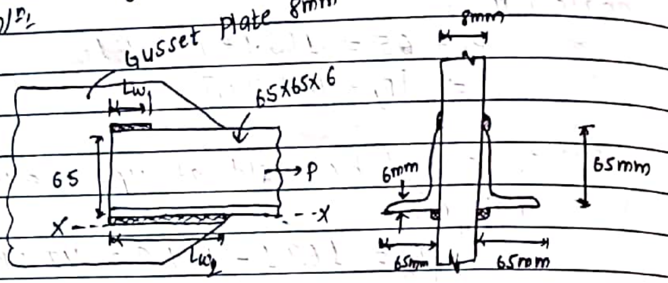
or, $121.9 \times 10^3 = L_{w_1} * 3.15 * \frac{410}{\sqrt{3}} * \frac{1}{1.25}$

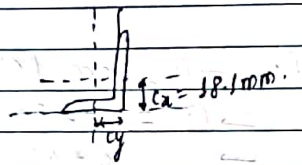
$\Rightarrow \boxed{L_{w_1} = 204.35 mm}$

↑ Length of weld.

(Q.2) A tie member of a truss consists of double angle section ISA 65*65*6. The angle sections are welded on opposite sides of a gusset plate 8mm thick. Design a weld (fillet) to transmit a factored tensile force of 200 kN. Assume shop welding (in workshop) and E250 grade steel.

→ Soln:





Page No.
Date

From IS 808:1989, (For 65x65x6 : equal angle section)

$A = 7.44 cm^2 = 744 mm^2$

$C_x = C_y = 1.81 cm = 18.1 mm$

For E250 grade steel,

$f_u = 410 MPa$

$f_y = 250 MPa$

For shop welding, $\gamma_{mw} = 1.25$ (Table 5)

Partial safety factor for yielding,

$\gamma_m = 1.10$ (Table 5)

Let, lengths of weld at top and bottom of the angle sections be l_{w_1} & l_{w_2}, respectively.

Total lengths of weld $= 2 \times (l_{w_1} + l_{w_2})$

(Table 21)

Minimum size of weld = 3mm (For 8mm thick plate) [Cl. 10.5.2.3]

Maximum size of weld = 6 − 1.5 = 4.5mm (on square edge)

$= \frac{3}{4} * 6 = 4.5 mm$ (on round toe)

Thus,

Adopting, S = 4.5mm (size of weld)

Effective throat thickness, $t_t = KS = 0.7 * 4.5 = 3.15 mm$

[K = 0.7 for 60°−90° angle b/w fusion face; here '90°']

10.5.3.1 (Table 22)

Design strength of weld, (10.5.7.1.1.)

$P_{dw} = l_w * t * \frac{f_u}{\sqrt{3} * \gamma_{mw}}$

Design strength of weld per mm length of weld,

$= 1 \times 3.15 * \frac{410}{\sqrt{3} * 1.25}$

$= 596.52 N/mm$

(Cl.10.5.1.2.) ~~(10.5.3.2)~~

Equating design strength of weld to factored tensile force,

$596.52 * 2(l_{w_1} + l_{w_2}) = \underbrace{200 \times 10^3}_{\text{(factored tensile load)}}$

$\Rightarrow [l_{w_1} + l_{w_2} = 167.639 mm]$

Taking moment about line X-X,

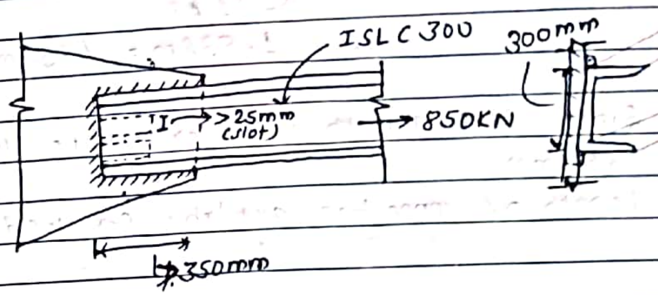
$596.52 * 2 l_{w_1} * 65 = 200 \times 10^3 \times 18.1$

$\Rightarrow [l_{w_1} = \cancel{70.02}\ 46.68 mm]$

$[l_{w_2} = \cancel{97.612}\ 120.960 mm]$

(Q3) An ISLC 300 member (Fe 410 grade) is to transfer factored tensile force of 850 KN. The channel section is to be welded at the site to a gusset plate 12mm thick. Design a fillet weld, if the overlap is limited to 350 mm.

→ Soln:-



(From table 4.1) ISLC300: $A_g = 42.2 cm^2 = 4220 mm^2$
(IS 808:1989) (Gross sectional area)

$t_f = 11.6 mm$ (thickness of flange)

$t_w = 6.7 mm$ (thickness of web)

Size of weld: → $> 5 mm$ (Table 21) (for t = 12 mm)
→ $< 6.7 - 1.5 = 5.2 mm$ (for square edge)
(10.5.8.1.)

Adopt, size of weld, $S = 5mm$

Throat thickness (t) = $0.7S$ {for 90° connection, $k = 0.7$ from table 22} (Cl. 10.5.3.2.)

$= 0.7 \times 5$

$= 3.5 mm.$

(Here adopting

Design strength of weld,

$$P_d = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} * (l_w * t)$$

$\gamma_{mw} \rightarrow 1.5$ (Table 5 for field fabrication)

[Cl. 10.5.7.1.1.]

$$850 = \frac{410}{\sqrt{3} \times 1.25} * l_w * 3.5$$

[l_w = 1.538m = 1538mm].

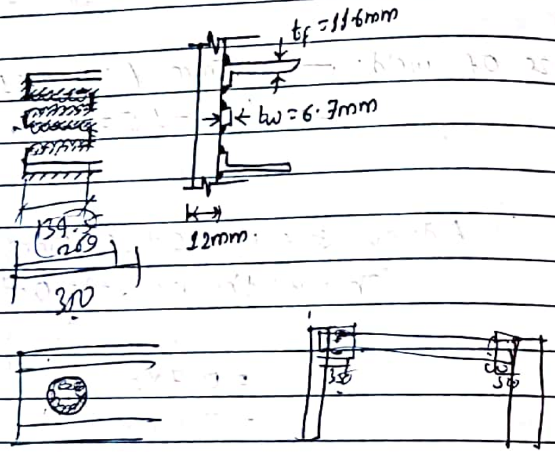
Since the length of overlap is limited to 350mm,

length of connection available for weld = 350×2 + 300

= 1000mm (< 1538mm)

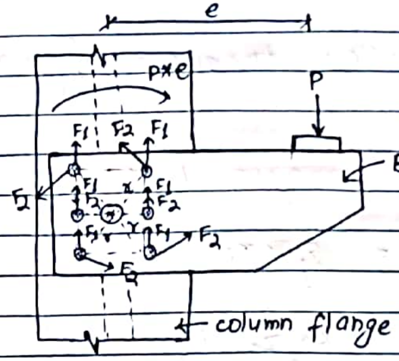
∴ Provide two slot welds of width 25mm (> 3×t)

and length = $\frac{(1538-1000)}{4}$ = 134.5mm

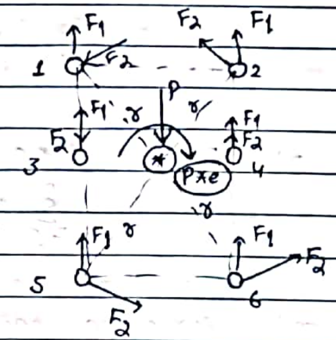


[Adopt l = 135mm]

Bracket connections



Enlarged



[$F_2 \rightarrow$ for balancing moment P×e
$F_1 \rightarrow$ for balancing/resisting load on bolts

Thus

Forces resisted by each bolts due to direct load,

$$F_1 = \frac{P}{n}$$ ← Number of bolts.

Force in each bolt due to torque,

[$F_2 \propto r_i \Rightarrow F_2 = K r_i$]   [$r_i$ → is distance of respective bolt from CG]

directly proportional

Torque about the center of rotation (c.g.) of bolt group,

$$= F_2 * r_i$$
$$= k r_i^2$$

Thus, Total resisting torque $= \sum_{i=1}^{n} k r_i^2$

Thus, Applied moment $= p * e$

$$p * e = \sum_{i=1}^{n} k r_i^2$$

or, $$p * e = k \sum_{i=1}^{n} r_i^2$$

$$= \frac{F_2}{r_i} \sum_{i=1}^{n} r_i^2$$ [As we are formulating for particular bolt]

$$\Rightarrow \boxed{F_{2_i} = \frac{p * e * r_i}{\sum_{i=1}^{n} r_i^2}}$$ i_{th} bolt

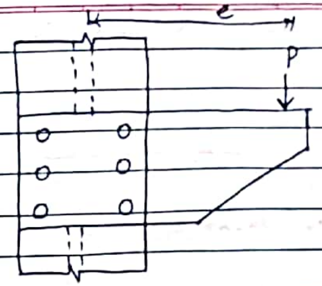
Resultant force in each bolt

$$\boxed{F_r = \sqrt{F_1^2 + F_2^2 + 2 F_1 \cdot F_2 \cdot \cos\theta}}$$

(depends on 'θ')

Just to find number of safe bolts (emperical formula)

(Not in code)



No. of bolts in each verticle line

$$\boxed{n = \sqrt{\frac{6M}{n' p V_{sd}}}}$$

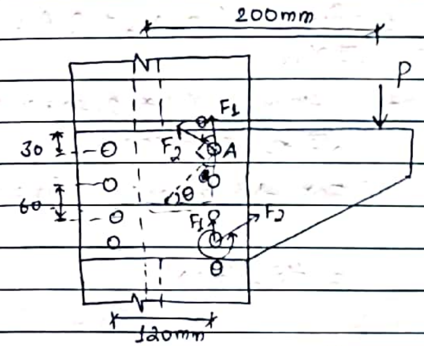
n' = no. of vertical lines

V_{sd} = design strength of bolt

p = pitch

M = Applied moment $= p * e$ (due to eccentric load)

(Q)



[E 250 grade steel]

Determine safe load for the given connection.
Use M20 bolt of grade 4.6
Take bracket plate thickness = 10mm
Thickness of flange of I section = 8 mm

(8) → distance ... C.G. F_R → b/w F_1 & F_2

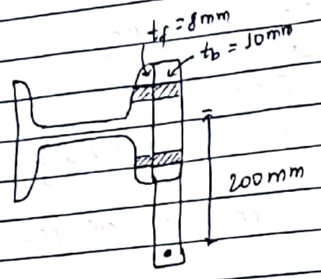
extreme stress → ~~bear~~ extreme bolts (at most ...)

→ least angle θ (b/w F_1 & F_2)

$$\left[F_R = \sqrt{F_1^2 + F_2^2 + 2 F_1 . F_2 \cos\theta} \right].$$

∴ single shear.

→ Plan



Strength of bolt in single shear, [Cl 10.3.3]

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \pi \times \frac{20^2}{4}\right)$$ ← table 5

[For 4.6 grade, $f_{ub} = 400MPa$]

$= 45.272kN$

Strength of bolt in bearing (at 8mm thick flange)

$$V_{dpb} = 2.5\, k_b\, d\, t \frac{f_u}{\gamma_{mb}}$$

(Cl. 10.3.4.)

$e = 30mm$, $p = 60mm$

$d = 20mm$, $d_0 = 22\ mm$ } [Table 19]

K_b

$$\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.455$$

$$\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

1

Adopting, ($K_b = 0.455$)

Strength of bolt in bearing

$$V_{dpb} = 2.5 \times 0.455 \times 20 \times 8 \times \frac{410}{1.25}$$

$= \cancel{59.04 kN}$ $59.696kN$

Thus, Strength of bolt = 45.27KN.

Again, let P be the factored load.

Direct force in bolt, $F_1 = \frac{P}{8}$ ← no. of bolts

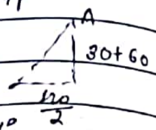
Force in bolt due to torque, (extreme bolt) $F_2 = \frac{P \times e \times r_n}{\Sigma r_i^2}$

$r_n = \sqrt{60^2 + 90^2}$
$= 108.16mm$

$\Sigma r_i^2 = 4 \times 108.16^2 + 4 \times (\sqrt{30^2 + 60^2})^2$
$= 64800\ mm^2$

F_2 (for extreme bolt A)

$= \frac{P * 200 * 108.16}{64800} = 0.334P$



$\theta = \tan^{-1}\left(\frac{90}{60}\right) = 56.31°$

NOW,

Resultant force on extreme bolt $F_R = \sqrt{F_1^2 + F_2^2 + 2F_1 \cdot F_2 \cdot \cos\theta}$

$= \sqrt{\left(\frac{P}{8}\right)^2 + (0.334P)^2 + 2 * \frac{P}{8} * 0.334P \times \cos(\ldots)}$

$= 0.416P$

But,

Resultant force, $F_x = 0.416P \le 45.27$

$\therefore \boxed{P = 108.6 KN}$

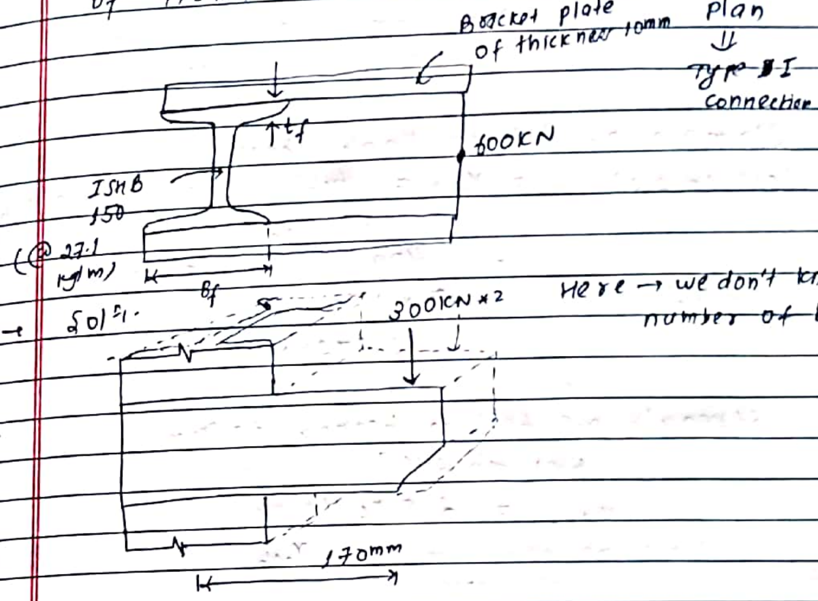
Thus,

Safe load, $P_s = \frac{P}{1.5} = 72.4 KN$

Partial safety factor for Load (IS 875) of ~~Table '4' for general loading~~

(Steel Fe 410
Bolt M20, Grade 4.6)

(8) Design a bolted bracket connection to transfer a factored load of 600KN with an eccentricity of 170 mm as shown in figure below.



Here → we don't know number of bolt

For ISHB 150, (Table 2.1)

$M = 27.1 \text{ kg/m}$ (Mass)

Sectional area, $a = 34.5 cm^2 = 3450 mm^2$

Flange thickness $= 9mm$ (t_f).

Flange width, $B_f = 150 mm$

Distance b/w two vertical lines of bolts,

$g = 90 mm$

(Design aid)

Diameter of bolt, $d = 20mm$

" " hole, $d_0 = 20+2 = 22mm$

(Table 19)

Pitch; $p_{min} = 2.5d = 50mm$ [Cl 10.2.2.]

edge distance, $e_{min} = 1.5 d_0 = 33mm$ (Cl. 10.2.4.2.)

Say,
Adopting, $p = 60mm$, $e = 35mm$ } (Setting more than minimum)

Strength of bolt in single shear,
[Cl. 10.3.3]
$$= V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} * (n_n * A_{nb} + n_s A_{sb})$$
$$= \frac{400}{\sqrt{3} * 1.25} * \left(1 \times 0.78 * \pi \times \frac{20^2}{4}\right)$$
$$= 45.272 kN.$$

Strength of bolt in bearing ($t = 9mm$)
[Cl. 10.3.4.]
$$V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$$

k_b

$$\frac{e}{3d_0} = \frac{35}{3 \times 22} = 0.53$$
$$\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$$
$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$
1

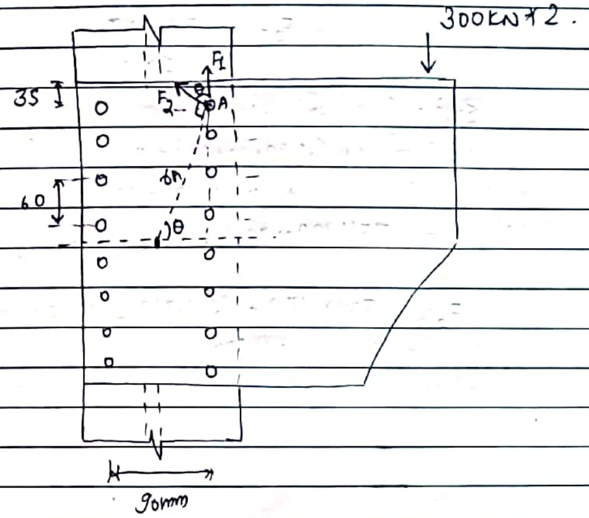
Adopt, $k_b = 0.53$

$$V_{dpb} = 2.5 * 0.53 * 20 * 9 * \frac{410}{1.25}$$
$$= 78.23 kN$$

(Strength of bolt = 45.272 kN)

Let us provide two verticle lines of bolts.
Number of bolts required in each vertical line,
$$n = \sqrt{\frac{6M}{n' p V_{sd}}}$$
$$= \sqrt{\frac{6 \times 300 \times 10^3 \times 170}{2 * 60 * 45.2 \times 10^3}}$$
$$= 7.5 \approx 8. \text{ (In each verticle lines)}$$

Final figure,



Next check F_R for extreme bolts

(F_R must be less than V_{sd})

i.e. F_R for extreme bolt < V_{sd} (45.2 kN)

[If not satisfy, increase number of bolts so that resultant force will be reduced].

? F_1 F_2 θ

F_R

Let P be the factored load.

Direct force on bolt, $F_1 = \frac{P}{16} = \frac{300}{16} = 18.75$ kN

Force on bolt due to torque, (extreme bolt)

$$F_2 = \frac{P \times e \times r_A}{\Sigma r_i^2}$$

$\Sigma r_i^2 = 4 \times \{(30+60+60+60)^2 + (45)^2\} + 4 \times \{(30+60+60)^2 + 45^2\} + 4 \times \{(30+60)^2 + 45^2\} + 4 \times \{30^2 + 45^2\}$

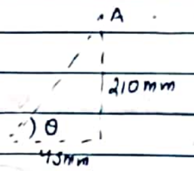
$= 334800 \text{ mm}^2$

$r_A = \sqrt{210^2 + 45^2} = 214.767$ mm

$$F_2 = \frac{300 \times 170 \times 214.767}{334800} = 32.715 \text{ kN.}$$

$\theta = \tan^{-1}\left(\frac{210}{45}\right)$

$= 77.90°$



$F_R = \sqrt{F_1^2 + F_2^2 + 2 \times F_1 \times F_2 \times \cos\theta}$

$= \sqrt{(18.75)^2 + (32.715)^2 + 2 \times 18.75 \times 32.715 \times \cos(77.90)}$

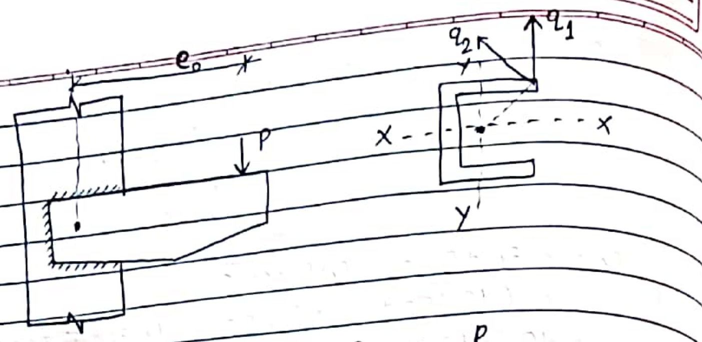
$= 40.976$ kN

Since,

$(F_R)_{\text{extreme bolt}}$ i.e. 40.976 kN < Design strength of bolt (V_{sd}) i.e. 45.2 kN

Okay.

SURENDRA SHARMA



Direct shear stress, $q_1 = \dfrac{P}{\text{Effective area of weld}}$

Torsion equation:

$$\frac{T}{I_p} = \frac{q_2}{r}$$

or, $q_2 = \dfrac{T*r}{I_p} = \dfrac{P*e_0*r}{I_p}$

Also,
Resultant stress in weld,

$$q_r = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$$

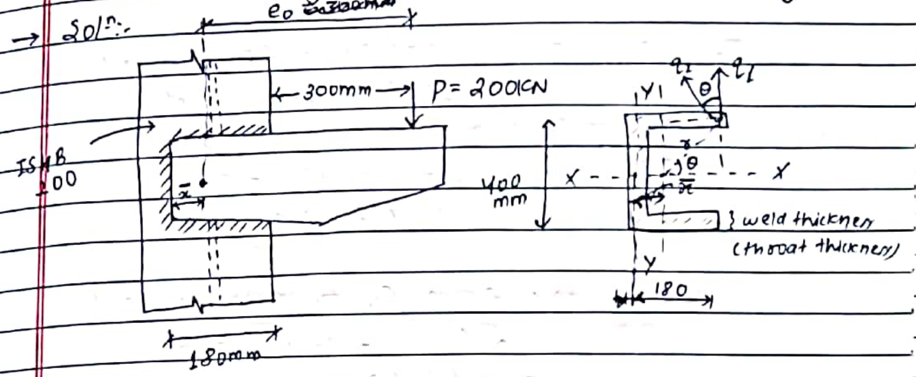
From [Cl. 10.5.7.1.1.]

$$q_r \le f_{wd}\left(= \frac{f_{wn}}{\gamma_{mw}}\right)$$

Polar moment of inertia $(I_p) = I_{x-x} + I_{y-y}$

(9) A 400mm ~~beam~~ deep bracket plate is welded to the flange of a column ISHB 200 as shown in figure below. It is subjected to an eccentric factored load of 200 kN at an eccentricity of 300mm from the face of column. Calculate the size of the weld required to support the load. Use steel of grade E250 and shop welding.

→ Soln:.



Let, $\bar{x}$ = distance of centroid of weld group from left edge of bracket plate.

$$\bar{x}*(400+2*180)*t_t = 2*180*\cancel{20}*t_t*90 + 400*t_t*0$$

$$\Rightarrow \boxed{\bar{x} = 42.63\,mm}.$$

$(t_t \to 0)$

$\bar{y} = 200mm$

Parallel Axis theorem: $I_{x-x} = \dfrac{t_t*400^3}{12} + \left(\dfrac{180*\cancel{t_t^3}}{12} + 180*t_t*200^2\right)*2$

$= 1973.3\times10^4\, t_t\ mm^4$

$$I_{yy} = \left[\frac{180^3 \times t_t}{12} + t_t \times 180 \times (190 - 42.63)^2 \right] \times 2$$

$$+ \cancel{\frac{t_t^3 \times 400}{12}}^{\to 0} + 400 \times t_t \times (42.63)^2]$$

$$= 250.67 \times 10^4 \text{ mm}^4$$

$$I_p = I_{xx} + I_{yy} = (1973.3 t_t + 250.67 t_t) \times 10^4$$
$$= 2223.97\, t_t \times 10^4 \text{ mm}^4$$

$$\text{Eccentricity, } e_0 = 300 + 180 - \overset{42.63}{\bar{x}}$$
$$= 437.37 \text{ mm}$$

r = Distance from C_g to extreme weld
$$= \sqrt{(180 - 42.63)^2 + 200^2}$$
$$= 242.63 \text{ mm}$$

$$\theta = \tan^{-1}\left(\frac{200}{180 - 42.63}\right) = 55.52^\circ$$

$$\cos\theta = \cos(55.52^\circ) = 0.566$$

Now,

$$q_1 = \frac{P}{\text{Effective area of weld}}$$

(Direct shear stress)

$$= \frac{200}{t_t \times 400 + 2 \times 180 \times t_t}$$

$$= \frac{5}{19 t_t} \text{ kN/mm}^2$$

$$= \frac{263.157}{t_t} \text{ N/mm}^2$$

$$q_2 = \frac{P \times e_0 \times r}{I_p} \quad \text{(Due to twisting moment)}$$

$$= \frac{200 \times 437.37 \times 242.63}{2223.97 \times 10^4\, t_t}$$

$$= \frac{0.954}{t_t} \text{ kN/mm}^2$$

$$= \frac{954}{t_t} \text{ N/mm}^2$$

Resultant stress,

$$q_r = \sqrt{q_1^2 + q_2^2 + 2 q_1 q_2 \cos\theta} \quad (\le f_{wd})$$

$$= \sqrt{\left(\frac{263.157}{t_t}\right)^2 + \left(\frac{954}{t_t}\right)^2 + 2 \times \frac{263.157}{t_t} \times \frac{954}{t_t} \times 0.566}$$

$$= \frac{1124.08}{t_t} \text{ N/mm}^2$$

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}} \quad [\text{Cl. } 10.5.7.1.1]$$

$$= \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Tables (for shop weld)

$$= 189.37 \text{ N/mm}^2$$

$$= \cancel{189.37 N}$$

Thus,

$$\boxed{t_t = 5.936 \text{ mm}}$$

throat thickness

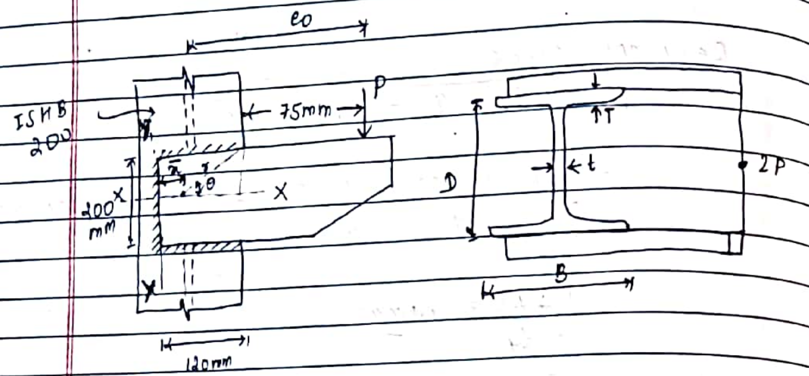
Size of weld, $s = \frac{t_t}{0.7} = 8.48$ mm

(Cl. 10.5.3.2)

Adopt size of weld = 9mm

Q A bracket plate of depth 200mm is used to transfer the reaction of beam to a column flange as shown in figure below.

Q A bracket plate is connected to the column flange of ISHB 200 by 6mm fillet weld. Compute the maximum surface load that can be placed over bracket plate at a distance of 75mm from flange of column.



→ Sol^n:-

From table 3.1 (IS-808-1989)

For ISHB 200,

Mass, M = 37.3 kg/m

Sectional area, a = 47.5 cm^2

D = 200 mm

B = 200 mm

t = 6.1 mm

T = 9.0 mm

Let, P be factored load on each bracket plate

(I) Size of weld, S = 6mm

(II) Effective throat thickness,

$t_e = 0.7 * 6 = 4.2mm$ (for 60-90°)

• $\bar{x}$ = distance of centroid of weld group from left edge of bracket plate.

$$\bar{x} = \frac{(120*4.2*2*60 + 200*4.2*0)}{2*120*4.2 + 200*4.2}$$

$= 32.73mm.$

• Polar moment of inertia, I_p

$$I_x = \frac{4.2*200^3}{12} + 2*\left[\frac{4.2^3*120}{12} + 120*4.2*100^2\right]$$

$= 1288.15 * 10^4 mm^4$

$$I_y = \frac{4.2^3*200}{12} + 2*\left[\frac{120^3*4.2}{12} + 120*4.2*(60-32.73)^2\right]$$

$\cancel{= 196.04*10^4 mm^4}$

$+ (4.2 * 200 * 32.73^2)$

$= 286.02 * 10^4 mm^4$

$I_p = I_x + I_y = 1574.17 \times 10^4 mm^4$

→ r = distance of extreme point of weld from CG of weld group.

$= \sqrt{(100)^2 + (120-32.73)^2}$

$= 132.72$ mm

Direct shear stress, $q_1 = \frac{P}{\text{effective area of weld}}$

$$= \frac{P}{2*120*4.2 + 4.2*200}$$

$= \frac{P}{1848}$ N/mm^2.

- Stress due to twisting moment,

$$q_2 = \frac{P*e_o*r}{I_p} = \frac{P*(120-32.73+75)*132.72}{1574.17*10^4}$$

$$= 0.0013681P \ N/mm^2.$$

- $$\cos\theta = \frac{120-32.73}{132.72} = 0.657$$

- Resulting shear stress,

$$q_R = \sqrt{q_1^2+q_2^2+2q_1q_2\cos\theta}$$

$$= \sqrt{\left(\frac{P}{1848}\right)^2+(0.0013681P)^2+2*\frac{P}{1848}*0.0013681P*0.657}$$

$$= 0.0017712P \ N/mm^2$$

- Since, q_R should be $\le \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

For shop weld, from Table 5, $\gamma_{mw} = 1.25$

For Fe410 Grade, $f_u = 410$ MPa.

Thus,

$$0.001712P \le \frac{410}{\sqrt{3}*1.25}$$

$$\Rightarrow [P \le 110.614 \text{ kN}]$$

- Service load, $= \frac{P}{\text{Load factor}} = \frac{110.614}{1.5} = 73.74$ kN.

Maximum load on bracket connection $= 2*73.74$

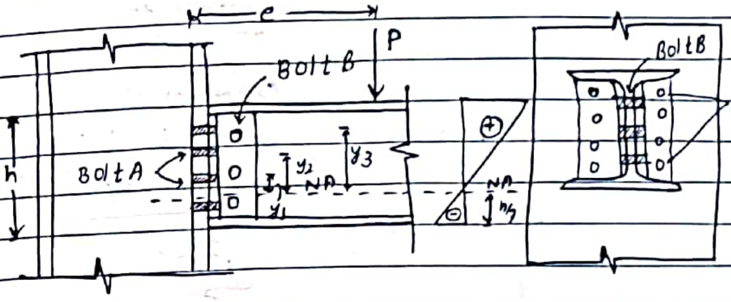
$= 147.48$ kN

Bolt A: out of plane connection (Type I)

Bolt B: In plane connection (Type II)

Bracket connection at right angle to the flange of column (Type-II)

[Load perpendicular to the plane of bolt]



Side view

Assumption :-

① Neutral Axis depth $= \frac{h}{7}$ (from the bottom of bracket)

NA भन्दा माथि → Bolts in tension (usually bolts designed in tension)

Below NA → Bolts in compression.

- Factored moment $= P*e$ (should be resisted by tension developed in bolts above the NA)

- T_1, T_2, T_3 = Tension in bolts at distance y_1, y_2 and y_3 from NA.

Thus,

$$2*(T_1*y_1 + T_2*y_2 + T_3*y_3) = P*e \quad \text{—(i)}$$

Also,

$$\frac{T_1}{y_1} = \frac{T_2}{y_2} = \frac{T_3}{y_3} = K \quad \text{—(ii)}$$

$(y_n) \rightarrow$ for individual bolt (n)

- Tension developed in bolt, $T_b = \dfrac{P \times e \times y_n}{2 \times \sum_{i=1}^{n} y_i^2}$

when bolts on both sides (symmetric)

{ else, for Asymmetric bolts:

$$T_b = \frac{P \times e \times y_n}{\sum_{i=1}^{n} y_i^2}$$

(n^{th} bolts)

(for all bolts) }

→ For above case ÷ by (I) $2 \times \sum_{i=1}^{n=3} y_i^2$

'or'

(II) $\sum_{i=1}^{n=6} y_i^2$

- Direct shear developed in each bolt

$$V_{sb} = \frac{P}{n}$$

- [Cl. 10.3.6.]

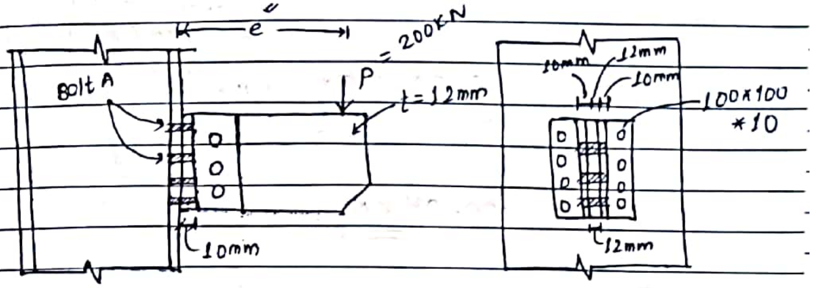
Bolt subjected to combined shear & Tension:-

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

{ $V_{sb} \rightarrow$ factored shear force acting on bolt
$V_{db} \rightarrow$ design shear capacity [Cl. 10.3.2.]
$T_b \rightarrow$ factored tensile force acting on the bolt
$T_{db} \rightarrow$ design tension capacity [Cl. 10.3.5.]

(Q) Design a bracket connection to carry a factored vertical load of 200 kN acting at an eccentricity of 300 mm from the ~~base~~ face of the column. Steel is of grade E250 and use M20 bolts of grade 4.6. A 12 mm thick bracket plate is connected to column flange through double angle section (2 ∠ 100*100*10mm).

→ Sol^n:-



Design of bolt group A (single shear)

(1) Bolt group A is subjected to single shear [Cl. 10.3.3]

$$V_{db} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} * (n_n A_{nb} + n_s A_{sb})$$

$$V_{db} = \frac{400}{\sqrt{3} * 1.25} * \left(0.78 \times \pi \times \frac{20^2}{4} * (1)\right)$$

$$= 45.27 \text{ kN.}$$

- Pitch, $p = 2.5d = 2.5 \times 20 = 50$ mm [Cl. 10.2.2.]

Edge distance, $e = 1.5 * d_0 = 1.5 \times 22 = 33$ mm [Cl. 10.2.4.2.]

In bearing,

$$V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$$ [Cl. 10.3.4]

SURENDRA SHARMA

[Adopt, p = 60mm & e = 40mm]

(K_b)

(i) $\frac{e}{3d_0} = \frac{40}{3\times 22} = 0.61$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.66$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410}$

(iv) 1

Adopt, 0.61

Adopt 10mm (minimum of thickness of angle section & flange thickness of column) → t

$V_{dpb} = \frac{2.5 * 0.61 * 20 * 10 * 410}{1.25}$ — [Table 5]

$= 100.04\ KN$

Thus,

Design bolt) V_{db} = Minimum of V_{dsb} & V_{dpb} = 45.27 KN.
↳ (Design shear capacity)

Next, [Cl. 10.3.5] $\left[T_{db} = \frac{T_{nb}}{\gamma_{mb}}\right]$

T_{db} = design tension capacity

nominal tensile capacity { $T_{nb} = 0.9 * f_{ub} * A_n < f_{yb}\, A_{sb} \left(\frac{\gamma_{mb}}{\gamma_{m0}}\right)$

[$A_n = \left(\frac{\pi \times d^2}{4}\right) * 0.78 = \frac{\pi \times 20^2}{4} \times 0.78 = 245.01$

[$0.9 * 400 * 245.01 = 88.203\,KN$]

$A_{sb} = \frac{\pi d^2}{4} = 314.16\ mm^2$

$\frac{f_{yb} * A_{sb} * \gamma_{mb}}{\gamma_{m0}} = 85.68\ KN$ (240, 314.16, 1.25, 1.1 from table 5)

Since,

$88.203 \nless 85.68$

Take, $[85.68\,KN] = T_{nb}$

Design Tension capacity,

$\left[T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{85.68}{1.25} = 68.5\,KN\right]$

⇒ (No. of bolts??)

Let's provide two vertical lines of bolt in each side of bracket,

No. of bolts in each vertical line, $n = \sqrt{\frac{6M}{pn'V_{sd}}}$
(gives tentative number)

$n = \sqrt{\frac{6 * 200 * 300}{60 * 2 * 45.27}}$ ← fig 2

$= 8.15$

$\simeq 9$

Total depth of bracket plate, $h = 8*p + 2*e$

$= 8*60 + 2*40$

$= 560\,mm$

N.A. from bottom of bracket plate $= h/7$

$= \frac{560}{7} = 80\,mm$

Direct shear in bolt, $V_{sb} = \frac{200*10^3}{9*2}$

$= 11.11\,KN.$

Tension developed in bolt (extreme), $T_b = \frac{P*e*y_n}{\sum_{i=1}^{16} y_i^2}$

(Below NA 2 bolts are in compression)

$= \frac{200*300*440}{2*(20^2+80^2+140^2+200^2+260^2+320^2+380^2+440^2)}$

$= 22.981\,KN$

Check (for combined shear & tension)

From Cl. 10.3.6.

$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 < 1.0$

$V_{sb} = 11.11\,KN$

$V_{db} = 45.27\,KN$

$T_b = 22.981\,KN$

$T_{db} = 68.5\,KN$

$\Rightarrow (0.173 \le 1.0)$

Okay

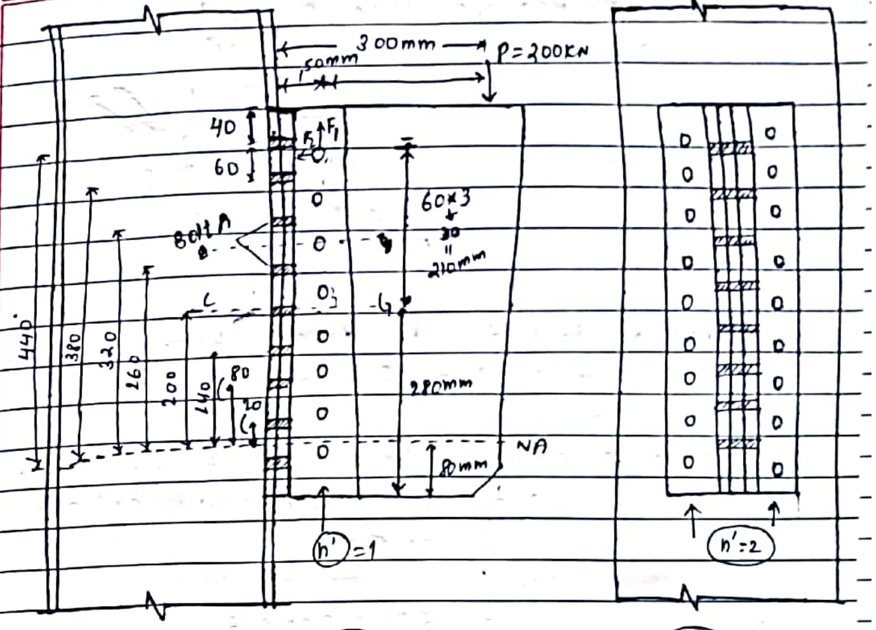


fig. (1) fig fig 2

Design of bolt group B

Bolts are subjected to double shear

→ shear, $V_{dsb} = \frac{2 * f_{ub}}{\sqrt{3}*\gamma_{mb}} * 1 * 245.01 = 90.5\,KN$

[Cl. 10.3.3.]

→ Bearing, $V_{dpb} = \frac{2.5*k_b*d*t*f_u}{\gamma_{mb}}$ [Cl. 10.3.4.]

Adopting same p & e as before $p = 60\,mm$ & $e = 40\,mm$

$k_b = 0.61$, $d = 20\,mm$ } $t = 12\,mm$ {$2*10 < 12$}, $f_u = 410\,KN$

$\Rightarrow [V_{dpb} = 120.048\,KN]$

Thus, → $[V_{db} = 90.5\,KN]$

ple → min^m → 60mm
40mm
refp →

Page No.
Date

- For one vertical line of bolt,

$$\text{No. of bolts}, n = \sqrt{\frac{6M}{p\,n'\,V_{sd}}}$$

$$= \sqrt{\frac{6 * 200 * 250}{60 * (1) * 90.5}}$$

fig (1)

$$= 7.43$$
$$\approx 8$$

- Force on bolt due to direct shear,

$$F_1 = \frac{200}{8} = 25\,kN.$$

- Force at extreme bolt due to twisting moment,

$$F_2 = \frac{P * e * r_n}{\Sigma r_i^2}$$ — CG to extreme

$$= \frac{200 * 250 * 210}{2 * (210^2 + 150^2 + 90^2 + 30^2)}$$

$$= 69.44\,kN.$$

- Resultant force on bolt,

$$F_R = \sqrt{F_1^2 + F_2^2}$$

$$= \sqrt{25^2 + 69.44^2}$$

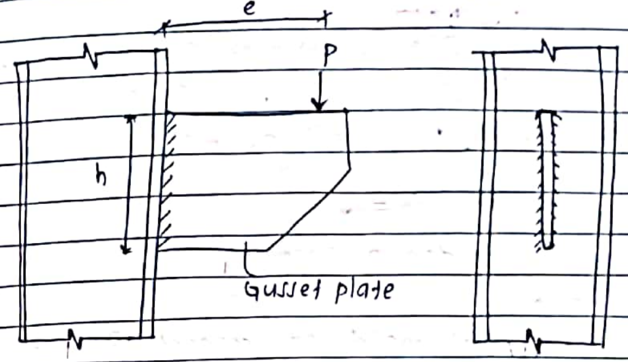
$$= 73.8\,kN \le \text{Bolt value}$$

90.5 kN

OK

Page No.
Date

Welded Bracket connection (Type II)

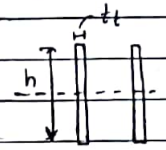


→ Size of weld, s
Throat thickness, t_t

(i) Direct shear stress, $q = \dfrac{P}{\text{Effective area of weld}} = \dfrac{P}{2 * h\,t_t}$

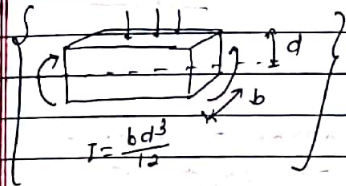
[Cl. 10.5.9.]

(ii) Bending stress, $f_a = \dfrac{M}{Z} = \dfrac{P * e}{\frac{t_t * h^2}{6} \times 2}$ दुहेरा (weld on both sides of gusset plate)



$$\left[f_a = \frac{3\,Pe}{t_t h^2} \right]$$

$Z \rightarrow$ section Modulus



$$I = \frac{bd^3}{12}$$

design stress

→ Resultant stress $f_e = \sqrt{f_a^2 + 3q^2} \le f_{wd} = \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}}$

[Cl. 10.5.10.1] emperical

Page No.
Date

- Design problem (weld depth, h = ?)

$$\left\{ h = \sqrt{\frac{3Pe}{t_t f_{wd}}} \right\} \rightarrow \left\{ \text{for first trial for depth of weld} \right\}$$

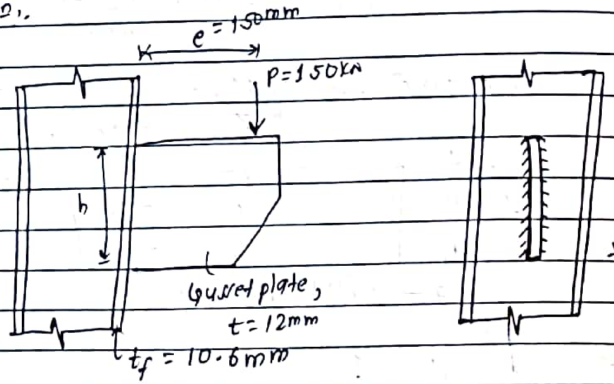
h = height or depth of weld

(ii) { $f_a \ngtr f_{wd}$ }

f_{wd} → design strength of weld.

(Q) Design a fillet weld to connect Gusset plate of 12mm thickness with the flange of column section with flange thickness 10.6mm. The load ~~ID~~. 150KN is applied at an eccentricity of 150mm perpendicular to the plane of weld. Take steel of grade E250 and assume shop fabrication.

→ Soln:



(I) Size of weld:

Minimum value for (thickness of thicker part = 12mm)

s = 5mm [Cl. 10.5.2.3]

Maximum value for s = 10.6 − 1.5 = 9.1mm

[Cl. 10.5.8.1]

Page No.
Date

Adopt size of weld, s = 8mm

- Throat thickness, $t_t = Ks = 0.7 * 8 = 5.6$ mm

[Cl. 10.5.3.2]

- Design strength of weld, $f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$ [Cl. 10.5.7.1.1]

$$= \frac{410}{\sqrt{3} * \underbrace{1.25}_{\text{Table 5 (for shop weld)}}}$$

$= 189.37$ N/mm²

Now,

For h,

$$h = \sqrt{\frac{3Pe}{\underbrace{t_t}_{\text{throat thickness}} f_{wd}}} = \sqrt{\frac{3 * 150 \times 1000 * 150}{5.6 * 189.37}}$$

$= 252.29$ mm.

(take 10-15% larger than h to be in safe side)

Trial 1: Adopt, depth of weld, h = 255mm

Now,

$$q = \frac{P}{2 * h * t_t} = \frac{150 \times 1000}{2 * 255 * 5.6} = 52.52 \text{ N/mm}^2$$

$$f_a = \frac{3Pe}{t_t (h)^2} = \frac{3 * 150 \times 1000 * 150}{5.6 * 255^2}$$

$= 185.37$ N/mm²

Thus,

Equivalent stress: $\left\{ f_e = \sqrt{f_a^2 + 3q^2} = \sqrt{(185.37)^2 + 3 * (52.52)^2} \right.$

$= 206.49$ N/mm²

$> (f_{wd} = 189.37$ N/mm²)

Not safe

Page No.
Date

Trial 2: Adopt, $h = 285mm$

$$q = \frac{P}{2 \times h \times t_t} = \frac{150 \times 1000}{2 \times 285 \times 5.6} = 46.99 N/mm^2$$

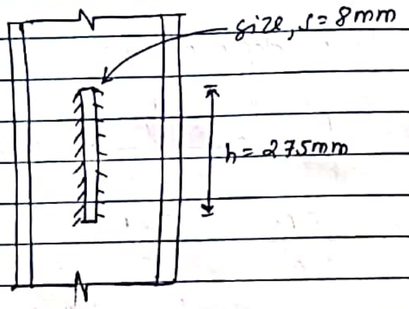
$$f_a = \frac{3Pe}{t_t h^2} = \frac{3 \times 150 \times 1000 \times 150}{5.6 \times 285^2} = 148.40 N/mm^2.$$

Thus,

$$f_e = \sqrt{f_a^2 + 3q^2} = \sqrt{(148.40)^2 + 3 \times 46.99^2}$$
$$= 169.25 N/mm^2$$
$$[\le f_{wd} = 189.37 N/mm^2]$$

Thus,
Design is safe.

Thus,
size of weld, $s = 8mm$
depth of weld, $h = 285mm$
throat thickness, $t_t = 5.6mm$.



Page No.
Date

Seat angle connections:-

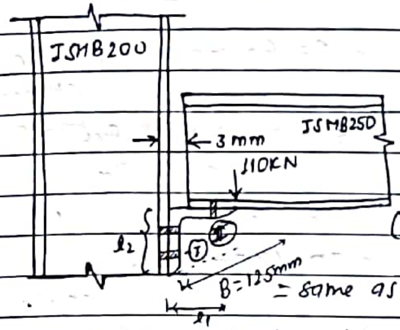
→ may be unstiffened or stiffened
(w)
study

→ Beam & column has some clearance (3 - 10mm)
i.e. It is not flushed

(Q) Design a seat connection for a factored beam end reaction of 110KN. Beam section ISMB 250 is connected to a flange of column section ISHB 200 using bolted connection. Use steel of grade E250 and bolts of grade 4.6

→ Sol^n:-

(II) → no need to design
ISMB 250 rests on seat angle
& its strength determined by (I) bolt.



(usually l_2 · double of l_1 is taken)

= same as flange width of ISMB250

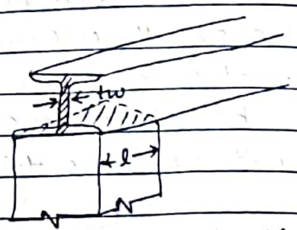
For ISMB 250 : (T) $t_f = 12.5mm$
(B) $b_f = 125mm$ ← width of flange
(t) $t_w = 6.9mm$
$r_1 = 13mm$

For ISHB 200 : $t_f = 9mm$ ← thickness of flange
$b_f = 200mm$ ← width of flange
$t_w = 6.1mm$ ← thickness of web.

Members → yield
Bolt → ultimate

Length of seat angle, $B = 125mm$

(1) Bearing area for seat angle = length of bearing * Thickness of web (ISMB 250)

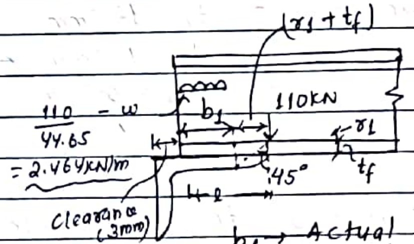


(tw ले मात्र load transfer गरीरा हुन्छ as flange माथि र मुनीको portion मा मात्र छ). member भरि

End reaction load, P = Bearing area * Design yield stress.

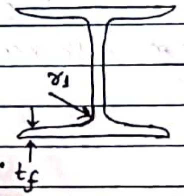
or, $110 \times 10^3 = (l \times t_w) \times \frac{f_{yb}}{\gamma_{mo}}$ (t_w = 6.9, f_{yb} = 250, γ_{mo} = 1.1)

[$l = 70.145mm$] → not a actual length.



$\frac{110}{44.65} = w$
$= 2.464 kN/m$

b_1 → Actual bearing length



r_1 → flange भन्दा अगाडि curve i.e. r को portion बाट load disperse हुदै जान्छ

b_1 → portion मा 110KN load udl to form मा uniformly load transfer हुन्छ

point load → disperse to udl on seat angle

Actual length of bearing on seat angle,

$b_1 = l - (t_f + r_1) = 70.14 - (12.5 + 13)$
$= 44.64mm$

But, ultimately we need size of seat angle.

Provide a clearance of 3mm between beam & column.

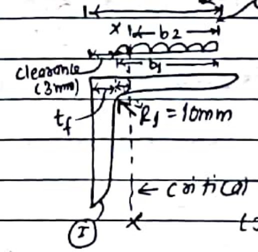
Length of seat angle resting beam $= 70.14 + 3 = 73.14mm$

Thus,
select seat angle section of ISA 150*75*12mm (150)

• Critical section on seat angle:

$(44.64 + 3 - (12 + 10)) = 25.64mm$

thickness of angle leg; R1

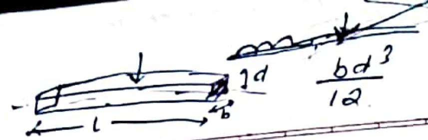


b_2 = Distance of critical section x-x for seat angle leg.

$= 44.64 + 3 - (12 + 10)$

$= 25.64mm$

(as हामीले critical moment निकाल्न हो so, we computed b_2)



Page No.
Date

- Moment at critical section x-x,

$$M = w * b_2 * \frac{b_2}{2}$$

$$= \left(\frac{110}{44.64}\right) * 25.64 * \frac{25.64}{2}$$

$$= 809.98 \text{ KNmm}$$

But,

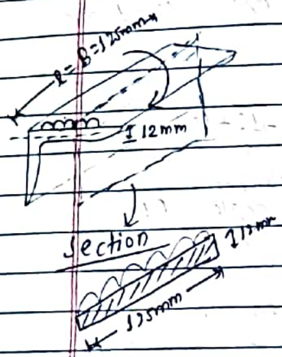
- Moment capacity of seat angle leg connected to beam

$$= \frac{1.2 Z_e f_y}{\gamma_{m_0}}$$

[Cl. 9.2.2.]

$Z_e \to$ Section modulus.

$f_y \to$ yield stress)



$$= 1.2 * \frac{125 * 12^2}{6} * \frac{250}{1.1}$$

$$= 818.1 \text{ KN-mm} > 809.98 \text{ KN-mm}$$

(Thus, okay)

Check for shear:

- Shear capacity of seat angle leg

$$= (125 * 12 *) * \frac{f_y}{\sqrt{3}\gamma_{m_0}}$$

$$= 125 * 12 * \frac{250}{\sqrt{3} * 1.1}$$

$$= \underset{\text{shear capacity}}{196.82 \text{ KN}} > \underset{\text{applied shear}}{110 \text{ KN}}$$

Okay

Page No.
Date

Bolt

Let's use 20mm dia M20 grade 4.6 bolt,

Strength of bolt in shear, $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} (n_n A_{nb})$

[Cl. 10.3.3]

$$= \frac{400}{\sqrt{3} \times 1.25} * 0.78 * \pi * \frac{20^2}{4}$$

$$= 45.27 \text{ KN}.$$

Strength in bearing, $V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

[Cl. 10.3.4.]

Adopt, $p = 60mm$, $e = 40mm$

$$k_b \Rightarrow \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.61$$

$$\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

$$1$$

Adopt, $k_b = 0.61$

(of ISHB 200)

$$V_{dpb} = \frac{2.5 * 0.61 * 20 * 9 * 410}{1.25}$$

$$= 90.036 \text{ KN}.$$

Thus, [Bolt value = 45.27KN]

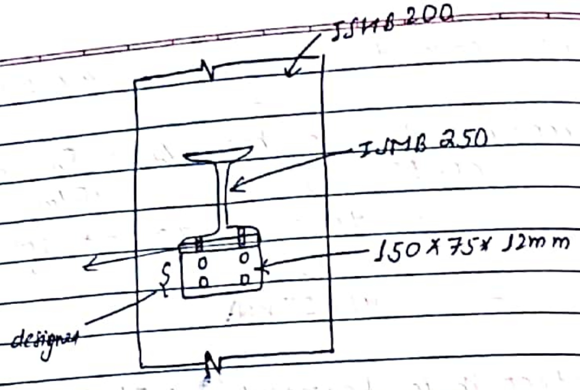
$$\text{Number of bolts} = \frac{110}{45.27} = 2.43 \approx 4$$

$\Rightarrow$ Provide 20mm dia. bolts of 4 nos in two vertical lines.

$\Rightarrow$ Provide cleat angle of nominal size 150 × 75 × 12mm

SURENDRA SHARMA

Page No.
Date



Tension Members

(Section 6) + (clause of connections)

- suspenders. (tension-only members)
- Tie members (works in tension only)
- stay cables in cable-stayed bridge

→ Member sections may be single or built-up

Design strength → 6.2, 6.3, 6.4

6.4.1 (Block shear failure)

(I) → Tension fracture and shear yield

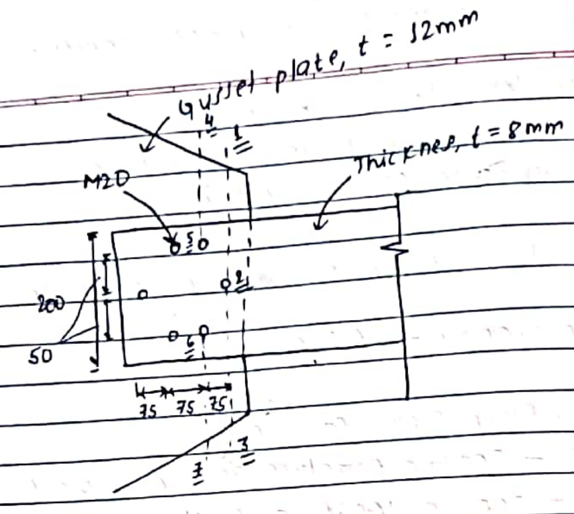
(II) → Tension yield and shear fracture

जो minimum take it for design.

ultimate → net area

Yield → Gross area.

(Q)



→ Hole diameter, $d_0 = 20+2 = 22$mm

Now,

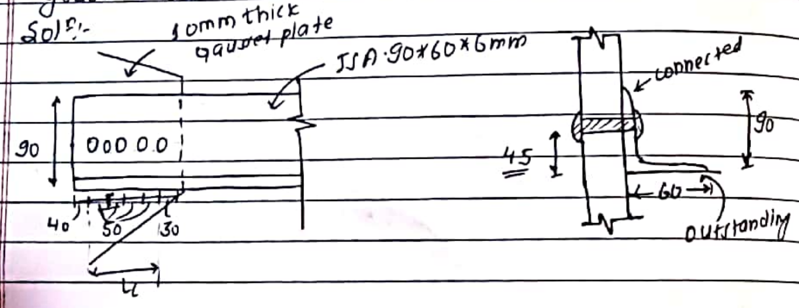
$A_n(1-2-3) = (200 - 1*22)*8 =$

$A_n(4-5-2-3) = \left(200 - 2*22 + \frac{75^2}{4*50}\right)*8$

(Q) An unequal angle section $90*60*6$ mm is connected to a 10mm gusset plate with 16mm dia bolt as shown in figure below to transfer tension. Determine the design tensile strength of the angle using steel of grade E250.

→ Soln:-

No. of bolts → given



(I) Design strength due to yielding of gross section [Cl. 6.2.]

$T_{dg} = A_g f_y / \gamma_{m_0}$

$A_g = 8.65\,cm^2$ (steel table)

A { 'OR' Ref. $A_g = (90 - 6/2 + 60 - 6/2)*6 = 864\,mm^2$.

$T_{dg} = \frac{865*250}{\underbrace{1.1}_{\text{Table 5}}} = 196.59$ kN

(II) Design strength due to rupture of critical section. [Cl. 6.3.3.]

$T_{dn} = 0.9 A_{nc} \frac{f_u}{\gamma_{m_1}} + \beta A_{go} \frac{f_y}{\gamma_{m_0}}$

A_{nc} = Net area of connected leg (with reference to A).

$= (90 - \frac{6}{2} - \underbrace{18}_{\text{for net area}})*6$

$= 414\,mm^2$

(connected & outstanding leg दोनों की thickness share $t/2$ & $t/2$ आयेगी)

A_{go} = Gross area of outstanding leg

$= (60 - 6/2)*6$

$= 342\,mm^2$

$\beta = 1.4 - 0.076(w/t)(f_y/f_u)(b_s/L_c)$ $\left\{ \begin{array}{l} \le 0.9\left(\frac{f_u \gamma_{m_0}}{f_y \gamma_{m_1}}\right) \\ \ge 0.7 \end{array} \right.$

w = outstanding leg width = 60mm

b_s = shear lag width = $w + w_1 - t$ = 60+45-6 = 99mm

L_c = length of end connection (distance b/w extreme bolts in direction of tension) = 50*4 = 200mm

t = 6mm

$$\beta = 1.4 - 0.076\left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\left(\frac{99}{200}\right)$$

$$= 1.17 \begin{cases} \le 0.9 \times \dfrac{410 \times 1.1}{250 \times 1.25} = 1.29 \\ \ge 0.7 \end{cases}$$

Okay

Thus, adopt

$[\beta = 1.17]$

So,

$$T_{dn} = 0.9 \times 414 \times \frac{410}{1.25} + 1.17 \times 342 \times \frac{250}{1.1}$$

$$= 213.153\,kN$$

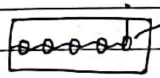
(III) Design strength Due to Block Shear [Cl-6.4.1]

$$T_{db_1} = \frac{A_{vg}\, f_y}{\sqrt{3}\,\gamma_{m_0}} + \frac{0.9 A_{tn}\, f_u}{\gamma_{m_1}}$$

$$T_{db_2} = \frac{0.9 A_{vn}\, f_u}{\sqrt{3}\,\gamma_{m_1}} + \frac{A_{tg}\, f_y}{\gamma_{m_0}}$$

A_{vg} = Min^m gross area in shear along bolt line ∥ to external force

$= (40+5\times4)\times6 = 1440\,mm^2$



$[d_0 = 16+2 = 18\,mm]$

A_{vn} = Net area

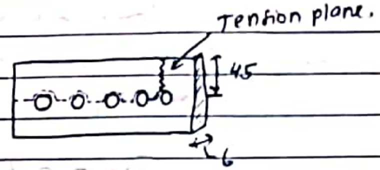
$= (40+50\times4 - 4.5\times18)\times6$

$= 954\,mm^2$

{ extreme hole को आधा part का fall होता so, 0.5 }

A_{tg} = Minimum gross area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force.

$= 45\times6 = 270\,mm^2$



$A_{tn} = (45 - 0.5\times18)\times6 = 216\,mm^2$

Thus,

$$T_{db_1} = \frac{1440\times250}{\sqrt{3}\times1.1} + \frac{0.9\times216\times410}{1.25} = 252.714\,kN$$

$$T_{db_2} = \frac{0.9\times954\times410}{\sqrt{3}\times1.25} + \frac{270\times250}{1.1} = 223.957\,kN.$$

Thus,

$[T_{db} = 223.957\,kN]$ Taking minimum

⇒ Design strength of tension member (T_d):

= Minimum of 3 cases

= 196.59 kN

Design section and connection for tension member

Q. Design a bridge-truss diagonal (tension member) subjected to factored tensile load of 275 kN. The length of the diagonal member is 3m. The member is connected to a gusset plate 16mm thick with single line of 20mm diameter sub-bolts of grade 4.6. Assume E250 grade steel.

→ Soln:-

Trial 1 (Trial section)

[Cl. 6.3.3.] Required net area of member based on rupture strength,

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$$

$$\Rightarrow A_n = \frac{T_{dn} \times \gamma_{m1}}{\alpha f_u}$$

{Let's assume more than 4 bolts are required, $\alpha = 0.8$}.

steel table → gross area

$$A_n = \frac{275 \times 10^3 \times 1.25}{0.8 \times 410}$$ [for E250]

net-area

[A_n = 1048.02 mm²]

net-area of member section

[Normally, Gross area $A_{g1} = 1.15 \times A_n$]

1.15 to 1.20 (Increasing net area by 15% or 20%.)

Thus,

Gross area (A_{g1}) = $1.15 \times A_n$ = 1205.22 mm²

But if ... → can take I-sectional way

truss member → generally of (6 section) → moreover area is less

(Also, Gross-area can be assumed on the basis of yielding)

Required gross area of member based on gross-section yielding, [Cl. 6.2.]

$$A_{g2} = \frac{T_{dg} \times \gamma_{m0}}{f_y} = \frac{275 \times 10^3 \times 1.1}{250}$$ ← Table 5

$= 1210\ mm^2$

{Gross-area taken greater of A_{g1} & A_{g2} → safety.}

[From IS808: 1989] (Steel table)

Taking sectional area a bit greater than 12.1 cm² i.e. 13 cm²

i.e. equal angle section of 70×70×10

Try angle section: 70×70×10 mm

A_g = 13 cm² = 1300 mm².

• Design of connection: (Bolt design)

Check: 6.2, 6.3, For 6.3 No of bolts required in prior

[Cl 10.3.3] $V_{dsb} = \frac{f_{ub} \times A_{nb}}{\sqrt{3} \times (1.25)}$ ← γ_{mb} (single shear)

$= \frac{400 \times 245}{\sqrt{3} \times 1.25}$

= 45.26 kN

[Cl 10.3.4] $V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

10.2.4.2

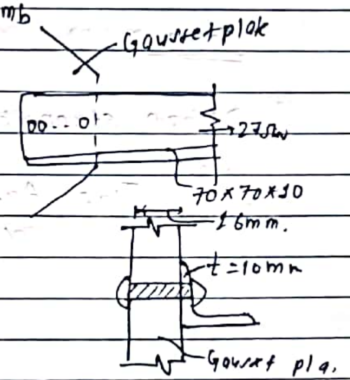
k_b → $\frac{1.5 d_0}{3 d_0} = 0.5$,

$\frac{2.5d}{3d_0} - 0.25 = 0.51$

$\frac{f_u}{f_{ub}} = 0.975$

d_0 = 22 mm

Table 19



Adopt k_b = 0.5

$V_{dpb} = 2.5 * 0.5 * 20 * 10 * \frac{410}{1.25}$

$= 82\text{ kN}$.

Thus, Bolt value = 45.27 kN.

{Tension capacity of bolt → not used [$T_{anb} = 0.9 A_{nb} f_u$]}

No. of bolts required $= \frac{275}{45.27} = 6.07 \approx 7$

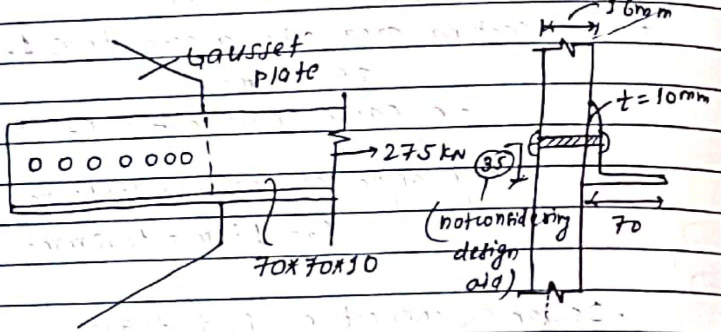


Fig:

Check:

(I) Design strength due to yielding of gross section

[Cl. 6.2] $T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1300 * 250}{1.1} = \cancel{484.59\text{ kN}}$ 295.45 kN

[> 275 kN]

safe.

(II) Design strength due to rupture of critical section -

[Cl. 6.3.3] $T_{dn} = \cancel{0.9 A_n f_u / \gamma_{m1}}\ \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta A_{go} \frac{f_y}{\gamma_{m0}}$

$\ne 0.9$

$A_{nc} = (70 - \frac{10}{2} - 22) * 10$

$= 430\text{ mm}^2$

$A_{go} = (70 - \frac{10}{2}) * 10 = 650\text{ mm}^2$

$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$

≥ 0.7

$w = 70\text{mm}$

$t = 10\text{mm}$

$L_c = (2.5 * 20) * 6 = 300\text{mm}$

$b_s = w + w_1 - t = 70 + 35 - 10 = 95\text{mm}$

$\beta = 1.4 - 0.076 * \left(\frac{70}{10}\right)\left(\frac{250}{410}\right)\left(\frac{95}{300}\right) = 1.297$

$\le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 * 1.1}{250 * 1.25} = 1.44$

≥ 0.7

okay

$\beta = 1.297$

$T_{dn} = 0.9 * 430 * \frac{410}{1.25} + 1.297 * 650 * \frac{250}{1.1}$

$= 318.54\text{ kN}$ (> 275 kN)

okay

Page No.
Date

(III) Design strength due to block shear [Cl. 6.4].

$$T_{db_1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m_0}}\right] + \left[\frac{0.9 A_{tn} \times f_u}{\gamma_{m_1}}\right]$$

$$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m_1}}\right] + \left[\frac{A_{tg} f_y}{\gamma_{m_0}}\right]$$

$A_{vn} = (33 + 50 \times 6 - 6.5 \times 22) \times 10$
$= 1900\ mm^2$

33, 50, 6.5, 1, 3

$A_{vg} = (33 + 50 \times 6) \times 10$
$= 3330\ mm^2$

$A_{tn} = (35 - 0.5 \times 22) \times 10 = 240\ mm^2$
$A_{tg} = (35) \times 10 = 350\ mm^2$

$$T_{db_1} = \frac{3330 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times \cancel{350}\ 240 \times 410}{1.25} = \cancel{540.27kN}\ 507.8kN$$

$$T_{db_2} = \frac{0.9 \times 1900 \times 410}{\sqrt{3} \times 1.25} + \frac{350 \times 250}{1.1} = \cancel{378.37kN}\ 403.37kN$$

$[T_{db_1} > T_{db_2} > 275kN]$

okay

#
Minor

Page No.
Date

[For tension-only member → slenderness ratio is also to be checked]
(If length given) → Table 3
↓
length of diagonal member given

(IV) Check for slenderness

$L = 3m$

radius of gyration: $r = r_x = r_y = 21\ mm$ (steel table for $70 \times 70 \times 10$)

effective length.

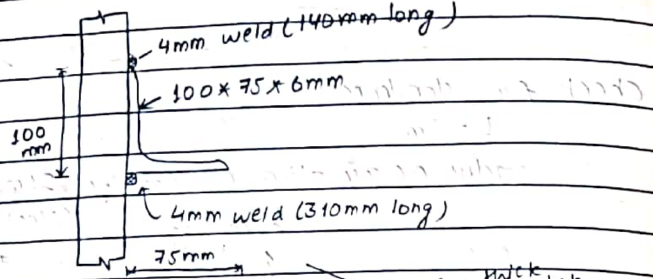
Table 3 $\quad \frac{KL}{r} < 400$

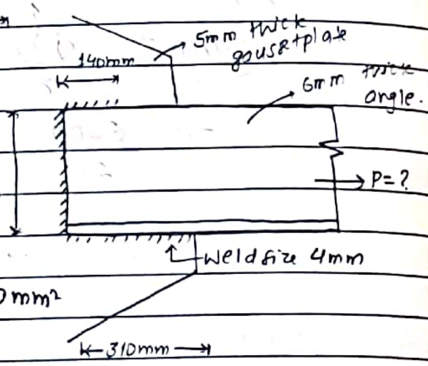
$$\Rightarrow \frac{3000}{21} = 142.86 < 400$$

okay

Page No.
Date

(9) Design the tensile strength of roof truss diagonal 100×75×6mm of E250 grade steel connected to the gusset plate 5mm thick by 4mm weld as shown in figure below.





(IS 808:1989)
(steel table)
$A_g = 10.1\,cm^2 = 1010\,mm^2$

Soln.

(I) Design strength due to yielding of gross-section. [Cl. 6.2.]

$$T_{dg} = \frac{A_g f_y}{\gamma_{m_0}} = \frac{1010 * 250}{1.1}$$

Table5

$$= 229.54\,kN$$

(II) Design strength due to Rupture of critical section (shear lag effect for angle section) ⇒ 6.3.3

Cl. 6.3.3

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m_1}} + \frac{\beta A_{go} f_y}{\gamma_{m_0}}$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u \gamma_{m_0}}{f_y \gamma_{m_1}}\right)$$

$$\ge 0.7$$

where,

$w = 75mm$ (outstanding leg)
$t = 6mm$
$f_y = 250\,MPa$
$f_u = 410\,MPa$ } for E250

$b_s = w + w_1 - t$
$= w = 75mm$

L_c = weld to weld distance. (along load direction)
$= \frac{140+310}{2}$
$= 225\,mm$

(if both way 310mm $L_c = 310mm$, but not in these case, 100mm weld ⇒ not in load direction.

$A_{nc} = \left(100 - \frac{6}{2}\right) \times 6 = 582\,mm^2$ (no bolt)

$A_{go} = \left(75 - \frac{6}{2}\right) \times 6 = 432\,mm^2$

$\gamma_{m_1} = 1.25$ (Table 5)

$\gamma_{m_0} = 1.10$ (Table 5)

$$\beta = 1.4 - 0.076 * \left(\frac{75}{6}\right)\left(\frac{250}{410}\right)\left(\frac{75}{225}\right)$$

$$= 1.2087 \qquad \left(\le \frac{410 * 1.1}{250 * 1.25} = 1.443\right)$$

$$(\ge 0.7)$$

Adopt, $\beta = 1.209$

Thus, $T_{dn} = \frac{0.9 * 582 * 410}{1.25} + \frac{1.209 * 432 * 250}{1.1}$

$= 290.51\,kN$.

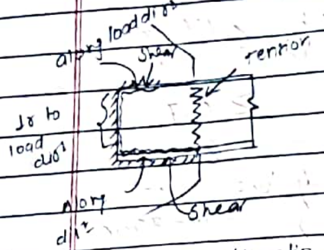
(III) Strength in block shear (Cl.6.4.2)

ℓ पूरै angle section फुत्केर/ चुँडिएर जाने)

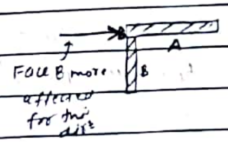
ℓ can be angle चुँडिने, gausset वा फुत्केर or weld फुत्केर

ℓ Min. weld ⇒ 4mm ← size

ℓ Throat thickness (t_t) = 0.7*4 mm



eg: In earthquake inplane wall is more ~~affected~~ resist so, while designing primary direction मा load participate गर्ने wall i.e. out plane more affected



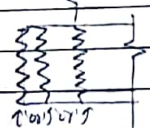
ℓ inplane / load-direction मा shear off हुने weld मात्र design गरिन्छ

ℓ out-plane ⇒ not designed
↳ its strength not considered (i.e. it is not activated)

⇒ Tension को लागि Gausset plate र यसैले बिग्रिन्छ c/s. compromised हुन्छ thus 5mm governs block shear in tension.

ℓ Gausset plate को separate भएर जान्छ

Tension
↳ gausset plate / Angle section connected part ले govern गर्छ



Shear ⇒ out-of-plane weld लाई discard गरिन्छ to be in safer part (out-of plane weld को मतलब भएन)

$A_{vn} = A_{vg} = (310+140)*4*0.7 = 1260\ mm^2$

$A_{tg} = A_{tn} = 100*5 = 500\ mm^2$

$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m_0}} + \frac{0.9 \times f_u \times A_{tn}}{\gamma_{m_1}}$$

$$= \frac{1260 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times \cancel{250}^{410} \times 500}{1.25}$$

$= \cancel{255.33kN}$ 312.93 kN

$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m_1}} + \frac{A_{tg} f_y}{\gamma_{m_0}}$$

$$= \frac{0.9 \times 1260 \times 410}{\sqrt{3} \times 1.25} + \frac{500 \times 250}{1.1}$$

= 328.38 kN

Thus, Design ~~strength~~ load = 229.5 kN

Safe load = $\frac{229.5}{1.5}$

= 153 kN

[Though stated maximum, unless the load must be factored.
300 kN → safe load tension member

[Thus, factored load = 1.5 × 300 = 450 kN]
Generally angle section

(Q) A diagonal member of a roof truss carries a maximum pull of 300 kN. Design the section and its connection with a 16mm thick gusset plate. {The length of the connection is limited to 340mm.} Design the lug angles if required. Use steel of grade E250 and 20mm dia bolts of grade 4.6.

add^n

→ Solution:-

• ~~Designing~~ Defining size:-

Factored load = 1.5 × 300 = 450 kN

From the strength (i) [Cl. 6.3.1]

Required net area of member based on ruptured strength

$T_{dn} = \frac{0.8 A_n f_u}{\gamma_{m_1}}$ (For more than 4 bolts) $\alpha = 0.8$

or, $450 \times 1000 = \frac{0.8 \times A_n \times 410}{1.25}$

$\Rightarrow A_n = \cancel{1524.39\ mm^2}\ 1714.94\ mm^2$

$A_g = 1.15 \times \cancel{1524.39}\ 1714.94 = \cancel{1753.05}\ 1972.18\ mm^2$.

(ii) Design strength due to yielding of gross section:-
[Cl. 6.2.]

$T_{dg} = \frac{A_g f_y}{\gamma_{m_0}}$

Or, $450 \times 1000 = \frac{A_g \times 250}{1.1}$

$\Rightarrow [A_{g_2} = 1980\ mm^2]$

{Normally gusset plate taken thicker}

{Normally thicker angle section is designed}

2

Thus,
From (IS 808: 1989)
Adopt, For ∠ 90 × 90 × 12 mm,
$A_g = 20.2\ cm^2 = 2020\ mm^2$

• Design of connection (Bolt design)
(i) Shear capacity
[Cl. 10.3.3]

$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{m_1}}(A_{nb} n_n + n_s A_{sb})$

Single shear:

$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi \times 20^2}{4}\right)$

$= 45.27\ kN$

(ii) Bearing capacity:-
[Cl. 10.3.4.]

$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{m_1}}$

k_b Adopt {
$e = 1.5 d_0 = 1.5 \times 22 = 33\ mm$ (10.2.4.2.)
$p = 2.5 \times d = 2.5 \times 20 = 50\ mm$ (10.2.2.)

$\frac{e}{3d_0} = 0.5$

$\frac{p}{3d_0} - 0.25 = 0.51$

$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

1

Adopt, $k_b = 0.5$

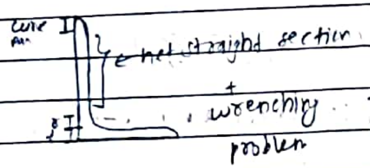
weld ⇒ length limited ⇒ slot
Bolt ⇒ length limited ⇒ lug angle
in Gusset length

Thus $V_{dpb} = \frac{2.5 * 0.5 * 20 * 12 * 410}{1.25} = 98.4\,kN$
(12mm governs)

Bolt value = 45.27kN

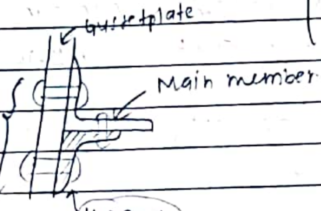
No. of bolts = $\frac{450}{45.27} = 9.94 \approx 10$

~~Bolt~~
But length of connection limited to 340mm.
So, 10 bolts cannot adjust in that length



Thus, for angle section generally single row of bolts is provided
⇓
Solution, lug angle

Additional angle member provided to outstanding leg.
(As far as possible plate symmetrically dispose करें S.t. load को C.G. offset न करें)

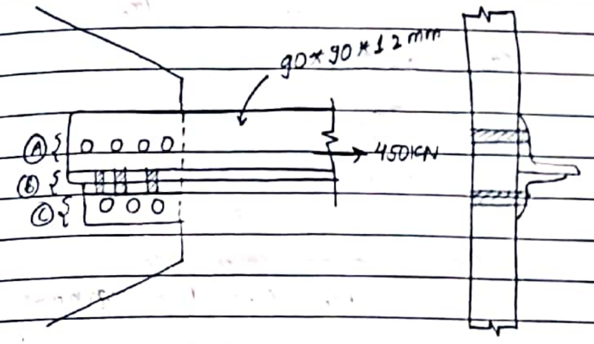


All three bolts must be designed
↓
Magnified design

(450 को certain portion of load share करेंगे)

(10.12) → lug angles.
10.2.2
outstanding leg at share को जितने load होगा 20% extra load को लगाकर lug angle is designed i.e. for 1.2 * load on outstanding leg को लगाके designing

For single line of bolt, length of connection required = 2*33 + 9*50 = 516 mm > 340mm.
Design of lug angle is required. [Cl. 10.12]



(For proportion of load in connected & outstanding load)

Main angle

Of Main Angle
Area of connected leg, $A_c = (90 - \frac{12}{2}) * 12 = 1008\,mm^2$ (Gross)
Area of outstanding leg, $A_o = (90 - \frac{12}{2}) * 12 = 1008\,mm^2$ (Gross)

[Load shared to connected & outstanding in proportion of area]
↳ For equal angle, half load shared by each angle.

Load shared by connected leg of main angle, $F_c = \frac{450 * 1008}{(1008 + 1008)} = 225\,kN$

Similarly,
Load shared by outstanding leg, F_o = 225kN
But from [clause 10.12]
Design of lug angle for $1.2 * F_o = 1.2 * 225$
$= 270\,kN$

[Cl. 6.2]

Approximate area required for lug angle,

(Gross) $A_g = \frac{270\times10^3}{250} \times 1.1 = 1188 mm^2$

(250 = f_y, 1.1 = γ_{m_0})

$\left[T_{dg} = \frac{A_g f_y}{\gamma_{m_0}}\right]$

[Use thickness < 12mm
& angle length < 90mm]

But, to be on safer side
take A_g around 1200 mm².

IS 808: 1989. Try, Angle section of 80×80×8mm
with $A_g = 1220 mm^2$

Thus,

(I) Design of bolt connecting main angle with Gusset plate: (A)

$F_c = 225 kN$

No. of bolts required = $\frac{225}{45.27} = 4.95 \approx \underline{5}$

Length of connection,

$2\times33 + 4\times50 = 266mm$ (< 340mm)
okay

[If length comes > 340mm,
take angle s.t. $A_o > A_c$]

(II) Design of connection between lug angle and Gusset plate: (C)

∴ No. of bolts required (N_l) = ?

Thus,

For lug angles, Bolts to be designed for $1.2F_o$ i.e. 270kN.

(Bolt value may depend on thickness of thinner plate, but since here B.V governed by shear we need not check for Bearing)

Bolts value governed by shear, $V_{dsb} = 45.27 kN$
(as previous)

Bolts value governed by bearing, $V_{dpb} = \frac{2.5\times0.5\times20\times8\times410}{1.25}$

$= 65.6 kN$

$BV = 45.27 kN$

Thus,

N_l = No. of bolts required = $\frac{270}{45.27} = 5.8 \approx 6$

Length of connection = $33\times2 + 5\times50 = 316mm < 340mm$
OK

[Cl. 6.2.]

Approximate area required for lug angle.
(Gross)
$$A_g = \frac{270\times10^3}{250} \times 1.1 = 1188\,mm^2$$
f_y γ_{m_0}

$$\left[T_{dg} = \frac{A_g f_y}{\gamma_{m_0}}\right]$$

[use thickness < 12mm
& angle length < 90mm]

But to be on safer side
take A_g around 1200 mm^2.

IS 808: 1989.
{Try, Angle section of 80*80*8mm
with $A_g = 1220\,mm^2$

Thus,

(I) Design of bolt connecting main angle with Gusset plate: (A)

$F_c = 225\,kN$

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$2*33 + 4*50 = 266mm$ (< 340mm)
okay

[If length comes > 340mm,
take angle s.t. $A_0 > A_c$]

(II) Design of connection between lug angle and Gusset plate: (C)

∴ NO. of bolts required $(N_L) = ?$

Thus,

For lug angles, Bolts to be designed for $1.2F_0$ i.e. 270kN.

(Bolt value may depend on thickness of thinner plate, but since here B.V governed by shear we need not check for Bearing)

Bolts value governed by shear, $V_{dsb} = 45.27\,kN$
(as previous

Bolts value governed by bearing, $V_{dpb} = \frac{2.5*0.5*20*8*410}{1.25}$

$= 65.6\,kN$

$BV = 45.27\,kN$

Thus,

N_0 = of bolts required $= \frac{270}{45.27} = 5.8 \approx 6$

length of connection $= 33*2 + 5*50 = 316mm < 340mm$

OK

Gusset मा connect हुने bolt ≤ 340mm must
But Main र lug लाई No problem

(III) Design of connection b/w lug angle & main angle (B)

From clause 10.12.2.

To be designed for $1.4F_0 = 1.4 * 225 = 315 kN$

For Bolt value (Taking 20mm bolt as before)

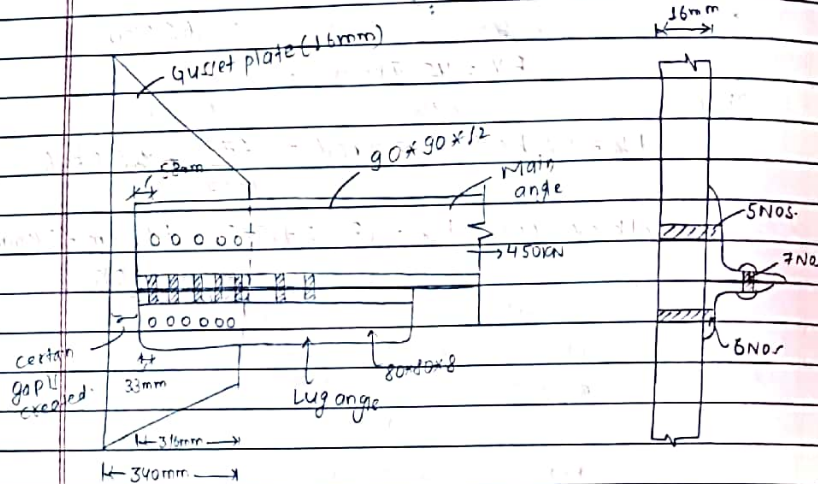
$V_{dsb} = 45.27 kN$

$V_{dpb} = 65.6 kN$

No. of bolts required, $= \frac{315}{45.27} = 6.9 \approx 7$.

(lug angle र main angle connect गर्दा length को limitation छैन)

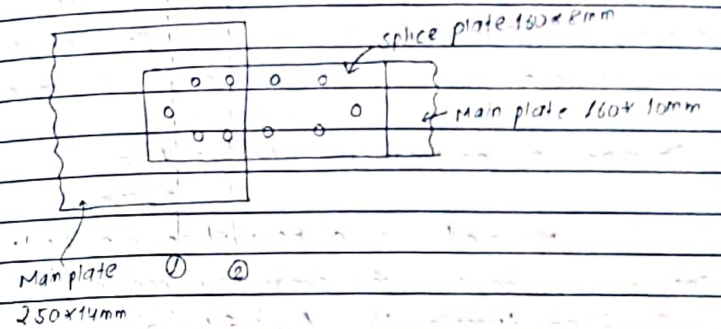
↳ Length limitation → only for angle connecting Gusset plate.



General (mostly) ↳ Gen. connected plate of greater length is taken i.e. unequal angle is taken ⇒ In that case more bolts will be seen in Main angle and less bolts on lug angle

Tension Splices (Similar as defining double cover butt joint (Packing plate) ...)

(Q) Design a splice for joining tension member sections 160*10mm and 250*14mm. The member is subjected to a factored tensile load of 300kN. Assume Fe 410 grade of steel. Provide 20mm diameter bolts of grade 4.6 for making the connections.



→ Soln.

For E250 grade of steel: $f_u = 410 MPa$, $f_y = 250 MPa$

For bolts of grade 4.6, $f_{ub} = 400 MPa$

Diameter of bolthole; $d_h = 22mm$ (Table 19)

Splice plate will be provided one on each side of the tension member section. The width of splice plate will be equal to width of the main plate & then thickness can be calculated.

• Design tensile strength of the member

Due to gross section yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = (160*10) * \frac{250}{1.1} * 10^{-3} = 363.63 kN$$

Due to net section rupture

$$T_{dn} = 0.9 * A_n \frac{f_u}{\gamma_{m1}}$$

Assuming the section to be weakened by one bolt hole (since the number and arrangement of bolts is not still known)

$A_n = (b - d_h)t = (180 - 22) * 10 = 1380\ mm^2$

$T_{dn} = 0.9 * 1380 * \frac{410}{1.25} = 407.37 KN$

Hence, design tensile strength of member is 363.63 KN.

The splice connection for tension members are designed for 0.3 * 363.63 = 109 KN or the factored tensile load (design action) of 300 KN whichever is ~~less~~ more

Thickness of packing, $t_{pkg} = 14 - 10 = 4mm$ $(< 6mm)$

No, reduction in shear strength of bolts required

The bolts will be in double shear and bearing,

For 20mm diameter bolt: $A_{nb} = 245 mm^2$.

Strength of bolt in double shear,

$V_{dsb} = 2 * A_{nb} * \frac{f_{ub}}{\sqrt{3}\, \gamma_{mb}}$

(10.3.3.)

$= 2 * 245 * \frac{400}{\sqrt{3} * 1.25} = 90.52 KN.$

Strength of bolt in bearing

(10.3.4.) $V_{dpb} = 2.5\, K_b\, d\, t\, \frac{f_u}{\gamma_{mb}}$

$= 2.5 * 1 * 20 * 10 * \frac{410}{1.25}$

$= 164 KN$

(Assuming $K_b = 1$ and aggregate thickness of splice plates to be more than the minimum thickness of sections (10mm and 14mm to be spliced))

Hence,

strength of bolt = 90.5 KN.

Number of bolts required, $n = \frac{\text{design force}}{\text{strength of bolt}} = \frac{300}{90.5}$

$= 3.31$

≈ 5

Provide 5, 20mm φ bolts and arrange them as shown in figure

Size of splice plate (Sp)

Width of Sp = Width of main plate = 160mm

Critical section for main plate will be 1-1 and for splice plate will be 2-2.

For thickness of splice plate, equate design action at section 2-2 to be design tensile strength.

$300 = 0.9 A_n \frac{f_u}{\gamma_{m1}} = 0.9 * (160 - 2 * 22) * 2 t_{sp} * \frac{410}{1.25} * 10^{-3}$

$t_{sp} = 4.38mm \simeq 6mm$

The aggregate thickness of splice plates should not be less than max^m thickness of main connected plate. Hence, provide 8mm thick splice plates.

Let us provide bolts at a pitch of 2.5 * 20 = 50mm

Edge distance = 33mm ≈ 35mm

Length of splice plate = 4 * (50 + 35) = 340mm

Hence, provide two splice plates 340 * 160 * 8mm, one on each side of main plate as shown

SURENDRA SHARMA

(Q) Calculate the design compressive load for ISHB 300 @ 58.8 kg/m, ~~Calculate the design~~ 5m height. The column is fixed in position but not in direction at both ends. Use steel of grade E250.

→ Solution:-

From IS808:1989,

For ISHB 300 @ 58.8 kg/m

$a = 74.8 cm^2$

$D = 300 mm$

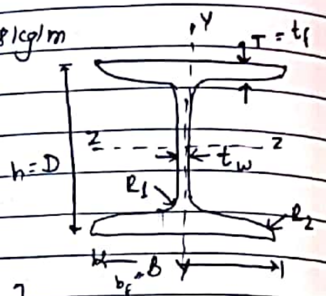
$B = 250 mm$

$t = 7.6 mm$

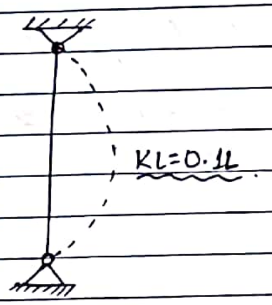
$T = 10.6 mm$

$r_{zz} = 13 cm$

$r_{yy} = 5.41 cm$] $r_{min} = 5.41 cm$



For column fixed in position but not in direction at both ends,



For Buckling class Identification, Table 10 [Cl. 7.1.2.2.]

$$\frac{h}{b_f} = \frac{D}{B} = \frac{300}{250} = 1.2 \quad (\leq 1.2)$$

$t_f = T = 10.6 mm$ ($\leq 100 mm$)

Thus, buckling about axis : z-z → Buckling class → b

" " " : y-y → " " → c

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$T = 10.6 mm$

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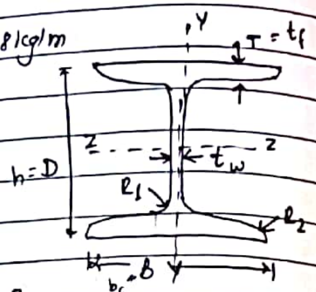
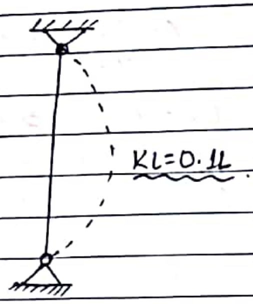


For column fixed in position but not in direction at both ends,



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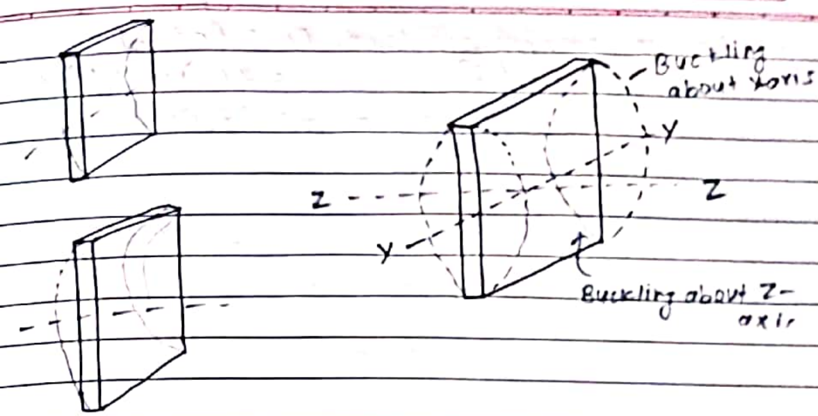


As, class c is lower class than class b.

∴ critical class is class 'c'

To get design compressive load, select class ©

Table 9©.

→ Slenderness ratio $= \frac{KL}{r_{min}} = \frac{5000 \times 1.0}{54.1}$

$= 92.42 < 180$ (Table 3)

okay.

→ $f_{cd} = 117.61 N/mm^2$

Design compressive stress, Table 9©

KL/r	fy 250
90 →	121
100 →	107

) 92.42 →

||

~~119.308~~

117.612 N/mm²

$f_{cd} = 117.61 N/mm^2 \leq \frac{f_y}{\gamma_{mo}}$

||

$\frac{250}{1.1} = 227.27 N/mm^2$

okay

class a → b → c → d
superior ⟶ inferior.

Gross area

* Design compressive load, $P_d = A_e \times f_{cd}$
[Cl. 7.1.2] $= 74.8 \times 10^2 \times 117.61$
$= 879 kN$

(Q) A single Angle discontinuous member 130 × 130 × 10 with simple bolted connection is 3m long, and the ends are hinged. calculate design compressive strength of the section using steel of grade E250.

[Cl. 7.5.1] → Single Angle strut
But except for single angle strut use table 9.(a), 9.(b), 9.(c), 9.(d)

* Cl. 7.5.1.2 → amendment

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

E → Elastic modulus.

* If not mentioned, assume hinge connection (for K_1, K_2, K_3)

→ Solution:-
Single angle strut with single bolted connection (hinged)
From table 12
constants: $K_1 = 1.25$, $K_2 = 0.50$, $K_3 = 60$

From Cl. 7.5.1.2

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

For steel, $E = 2 \times 10^5 N/mm^2$ (Remember)

$$\varepsilon = \left(\frac{250}{f_y}\right)^{0.5} \quad [\text{For E250, } f_y = 250 MPa]$$

so, $\varepsilon = 1$

From IS808: 1989

$a = 25.1\,cm^2$

$r_{vv} = r_v(min) = 2.57\,cm. = 25.7\,mm$

Cl. 7.5.1.2

$$\lambda_{vv} = \frac{\left(\frac{3000}{25.7}\right)}{1 * \sqrt{\frac{\pi^2 * 2*10^5}{250}}} = 1.31.$$

$$\lambda_{\phi} = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

$$= \frac{\cancel{130}\ (130+130)/2*10}{1*\sqrt{\frac{\pi^2*2*10^5}{250}}}$$

$= 0.146$

Thus,

Equivalent slenderness ratio, λ_e

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_{\phi}^2}$$

$$= \sqrt{1.25 + 0.50 * 1.31^2 + 60 \times 0.146^2}$$

$= 1.84.$

f_{cd} Cl. 7.1.2.1.

Design compressive stress,

$$f_{cd} = \frac{f_y/\gamma_{m_0}}{\phi + [\phi^2 - \lambda_e^2]^{0.5}}$$

where,

$$\phi = 0.5[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2]$$

equivalent

α = Imperfection factor (Table 7)

For angle section → class 'c' = (Table 10)

$[\alpha = 0.49]$

$$\phi = 0.5[1 + 0.49(1.84 - 0.2) + 1.84^2]$$

$[\phi = 2.59]$

$$f_{cd} = \frac{250/1.1}{2.59 + [2.59^2 - 1.84^2]^{0.5}}$$

$= 51.50\,N/mm^2$

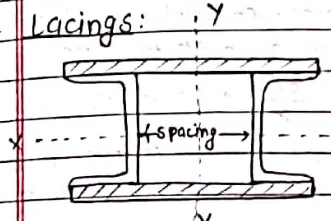
From Cl. 7.1.2

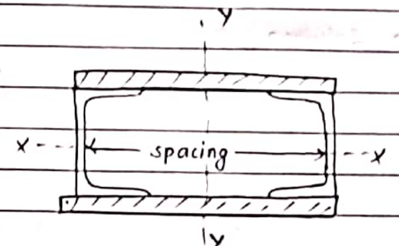
Design compression strength, $P_d = f_{cd} * A_e$

$= 51.50 * 2510$

$= 129.27\,kN$

Lacings:





Fig①

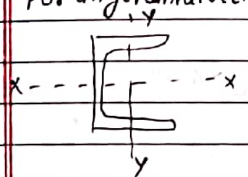
Two channel sections back to back

Two channel sections toe to toe

→ ① distance b/w channel section/spacing

Cl. 7.6.

For any channel section, $I_{xx} >> I_{yy}$ } MOI



Lacing टाकले plane → XX

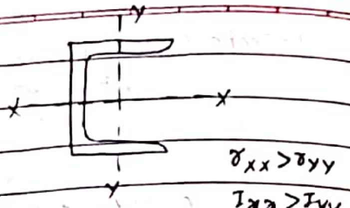
Lacing टाकले ⊥r plane → YY ([illegible] I_{xx} [illegible] [illegible])

7.6.1.1 → Relating rad. of gyration with MOI:

MOI Axis ⊥r to plane of lacing ≥ MOI about Axis ∥ to plane to lacing

Page No.
Date

[code]



$r_{xx} > r_{yy}$

$I_{xx} > I_{yy}$

For fig ①

Maintain spacing s.t.

$[r_{yy} \geq r_{xx}]$

⇓ means

$[I_{yy} \geq I_{xx}]$

(Q.) Design a laced column with two channels back to back of length 10 m → carrying an axial factor load of 1450 kN. The column is restrained in position but not in direction at both the ends.

→ Solution:

Generally for built-up section
Assume, $f_{cd} \approx 110$ to 150 MPa
'or' $\frac{KL}{r}$ → around 80

~~Normally, assume 130 MPa (or lower) (but not lower value)~~

① Assume $f_{cd} = 130\,N/mm^2$

Approximate area of section,

$$A_{approx} = \frac{1450 \times 10^3}{130}$$

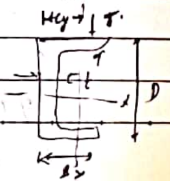
$$= 111.53\,cm^2.$$

[choose single channel section of > 60 cm²]

② From IS 808:1989

For ISMC 400, $a = 63.8\,cm^2$.

D = 400 mm	r_x = 15.4 cm
B = 100 mm	r_y = 2.82 cm
t = 8.6 mm	c_x = 200 mm
T = 15.3 mm	c_y = 2.42 cm



Page No.
Date

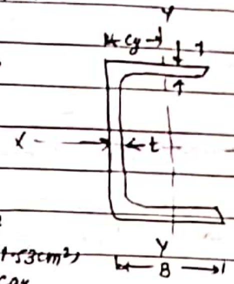
Let's adopt ISMC 400 connected back to back,

$A = 63.8\,cm^2 = 6380\,mm^2$

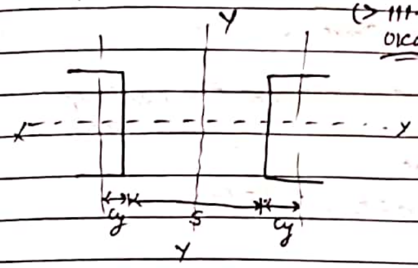
$I_{x-x} = 15200\,cm^4$

$I_{y-y} = 508\,cm^4$

$c_y = 2.42\,cm = 24.2\,mm$



$A_{provided} = 2 \times 63.8 = 127.6\,cm^2$ (> 111.53 cm²) Okay



③ Spacing b/w channel section

Cl. 7.6.1.1

$r_{yy} \geq r_{xx}$

$I_{yy} \geq I_{xx}$

or

$$\left[I_{yy} + A\left(\frac{s}{2} + c_y\right)^2\right] \times 2 \geq 2 \times I_{x-x}$$

i.e.

$$\left[508 \times 10^4 + 6380\left(\frac{s}{2} + 24.2\right)^2\right] \times 2 \geq 2 \times 15200 \times 10^4$$

$\rightarrow [s \geq 255.1\,mm]$

(Adopting this spacing ⇒ $I_{yy} \geq I_{xx}$ & $r_{yy} \geq r_{xx}$)

Adopt spacing between channels,
$s = 260mm$

check λ for whole section

(IV) Check for slenderness ratio (λ) & compressive strength (P)
Cl-7-6-1-5

$\Rightarrow \lambda_{eff} = \left(\frac{KL}{r_{min}}\right)_0 * 1.05$

(As, r_{min} is always r_{xx}) ⇒ For built-up section (so designed)

Thus,
$r_{min} = r_{x-x} = 154mm$

Restraining position but not direction ⇒ hinged (both)

Thus,
$(KL = 1.0L) \rightarrow$ Table 11

Thus,
$$\lambda_{eff} = \left(\frac{1.0 * 10000}{154}\right) * 1.05$$
$= 68.18$

Design compressive stress,
$\Rightarrow f_{cd} = 154.91$ MPa

KL/r (λ)	f_{cd}
60	168
70	152

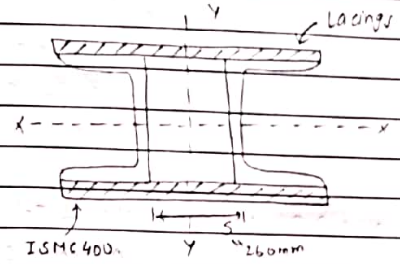
68.18

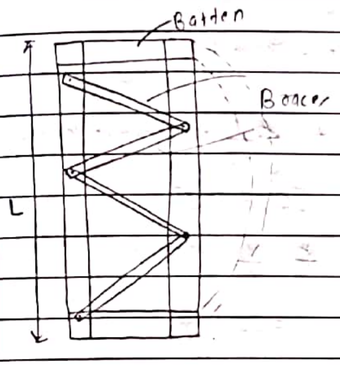
For E250
Table 9 (c)

As, Channel → class 'c' (Buckling class).

Design compressive strength, $P_d = f_{cd} * A_e$ (Ae → Approximated)
$= 154.91 * 127.6 * 10^2$
$= 1976.65$ kN
$(> 1450$ kN$)$
Okay
[Overall section is safe]

[Design of channel section] → Done ✓
[Now, Design of lacing ⇒].
7.6.2 onward,

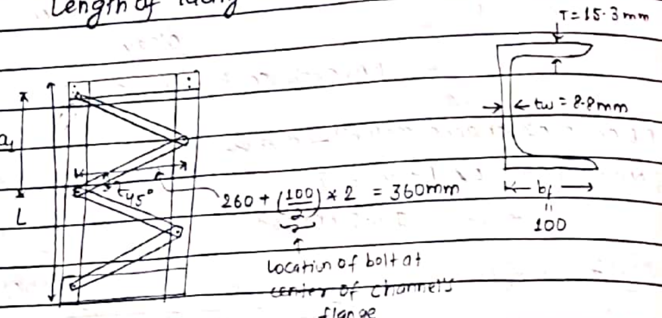




⑤ Lacings

Let's keep lacings at 45° inclination [Cl. 7.6.4 ⇒ 40° to 70°]

Length of lacing = $360\sqrt{2} = 509.12mm$ (45° inclination)



$260 + \left(\frac{100}{2}\right) \times 2 = 360mm$

location of bolt at ~~center of channel's~~ flange

[usually, taken as gauge length ⇒ but not mentioned in IS 800:2007]

Width of lacing flats ≮ 3d [Cl. 7.6.2.]

(Assume 20mm dia bolts)

Thus,

width of lacing flat = 60mm

Thickness of lacing flats, $t \nless \frac{1}{40} \times$ effective length

[Cl. 7.6.3]

i.e.

$t \nless \frac{1}{40} \times 509.12$

$= 12.7mm$

Adopt lacing of size 60mm × 13mm

Next, check for slenderness ratio (buckling tendency) in component level [local buckling & inclined lacing bar buckling]

⑥ Check for slenderness of component of main member.

a_1 = Unsupported length between lacing points

$= 360 \times 2$

$= 720mm$

$\frac{a_1}{r_{min}}$

$r_1 = r_{min}$ → seen for individual component (not as a whole)

i.e. For ISMC 400,

$r_{y-y} = 28.2mm$] → r_{min}

$r_{xx} = 154mm$

Thus,

[considering unit value of K]

$\frac{a_1}{r_{min}} = \frac{720}{28.2} = 25.53$

As per Cl. 7.6.5.1,

Max. slenderness ratio of components of main member $\left(\frac{a_1}{r_1}\right)$

≯ 50

or

0.7 × most unfavourable slenderness ratio of member as a whole

i.e. $0.7 \times 68.18 = 47.7$

Taking lesser value 47.7.

Thus,

$\frac{a_1}{r_1} = 25.53$ (< 47.7)

okay.

⑦ Check for slenderness ratio of lacing flat

Length of lacing = 509.12mm

Radius of gyration, $r_{lacing} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{\frac{bt^3}{12}}{b*t}}$

$= \frac{t}{\sqrt{12}}$

$= \frac{13}{\sqrt{12}}$

$= 3.75$ mm

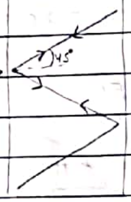
Bolted ↓ both pinned ↓ (k=1)

$\frac{KL}{r} = \frac{l_{lacing}}{r_{lacing}} = \frac{509.12}{3.75} = 135.76\ (<145)$ [K = 1.0]

[Cl. 7.6.6.3.]

* In case of comes unsafe increase thickness of lacing flats (t) → not used in calculation before.

⑧ Check for strength of lacing member



Transverse shear V_t = 2.5% of axial force on member [Cl. 7.6.6.1] → divided to 2 lacing flats

Transverse shear, V_t = 2.5% of 1450

= 36.25 kN

Compressive force in lacing = F_{lacing} (see component of V_t)

$= \frac{V_t}{2*\sin 45°}$

= 25.63 kN

Now,

For $\lambda_{lacing} = 135.76$, $f_{cd} = ?$

For buckling class c?

↳ plates / Angles ⇒ Buckling class 'c'

Table 9(c)

130 → 74.3

140 → 66.2

135.76 → 69.63

$74.3 - \frac{74.3 - 66.2}{140 - 130} \times 5.76$

Thus,

From table 9(c)

$f_{cd} = 69.63$ N/mm²

Now,

Design compressive strength of lacing flat,

$F_{d\,lacing} = f_{cd} * (60*13)$

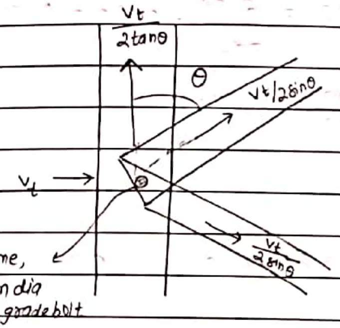
$= 69.63 * 60 * 13$

$= 54.31$ kN ($> F_{lacing}$)

Safe ok

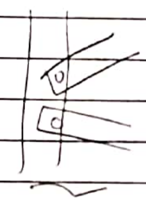
→ Next, is design of connections.

⑨ Design of connection:-



θ angle → member और lacing के बीच का angle

Assume, 20mm dia 4.6 grade bolt



The bolt will be in double shear.

(two lacings connected together)

Strength of bolt in double shear
[Cl. 10.3.3]

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

$$= \frac{f_{ub}}{\gamma_{mb}\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3} * 1.25} * 2 * 0.78 * \frac{\pi * 20^2}{4}$$

$$= 90.54 \text{ KN}$$

Strength of bolt in bearing
[Cl. 10.3.4]

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$= \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$$

$p = 2.5d$
$e = 1.5d_0$

$k_b \rightarrow \frac{e}{3d_0} = \frac{1.5d_0}{3d_0} = 0.5$

$\rightarrow \frac{p}{3d_0} - 0.25 = \frac{2.5d}{3d_0} - 0.25 = 0.508$ (with $d = 20$, $d_0 = 22$)

$\rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

$\rightarrow 1.0$

$\therefore k_b = 0.5$

t = minimum of thickness of lace plate & t_{flange}
= 13mm

$$V_{dpb} = \frac{2.5 * 0.5 * 20 * 13 * 410}{1.25}$$

$V_{dpb} = 106.60 \text{ kN}$

Thus,

Bolt value = 90.54 KN

Force on bolt from lacing $= \frac{V_t}{2\tan\theta} * 2 = 36.25$ KN [< 90.54 KN]

[Most critical condition मा $\frac{V_t}{2\tan\theta}$

यो component of ~~each lacing~~ एउटै dir मा जानेको i.e. both upward]

No. of bolts required $= \frac{36.25}{90.54} \approx 1$ bolts.

SURENDRA SHARMA

Ajanta
Page No.
Date

Ajanta
Page No.
Date

Ajanta

Page No.

Date

Ajanta

Page No.

Date

Page No.
Date

Q. Design a column of effective length 6m subjected to a factored axial load of 1200kN. Design the system with battens and bolted connections. Use two channel sections toe-to-toe and steel of grade E250.

→ Solutions:

Step I:

Assume $f_{cd} = 130 N/mm^2$

Approximate area of sections $= \frac{1200 \times 10^3}{130}$

$= 9230.77 mm^2$

$A_{approx. required} = 92.308 cm^2$

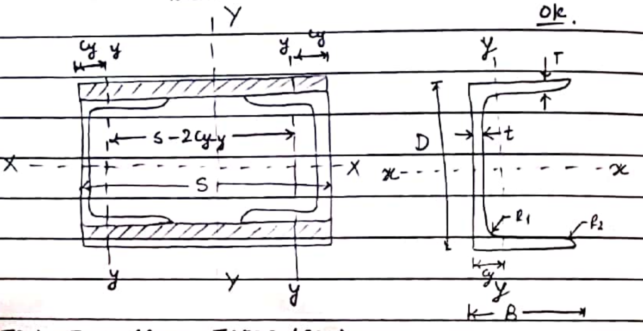
Choose single channel section of ~~size~~ area greater than $46.15 cm^2$.

Choose ISMC 350 @ 42.5kg/m placed toe to toe.

$A_{gross} = 54.40 cm^2$

Thus,

$A_{taken} = 54.40 \times 2 = 108.80 cm^2 > 92.308 cm^2$ ok.



For ISMC 350 (from IS808:1989)

$a = 54.4 cm^2$	$t = 8.3 mm$	$I_{x-x} = 10000 cm^4$
$D = 350 mm$	$T = 13.5 mm$	$I_{y-y} = 434 cm^4$
$B = 100 mm$	$c_y = 24.4 mm$	$r_{x-x} = 13.6 cm$
		$r_{y-y} = 2.82 cm$

Page No.
Date

(I) Spacing between channel sections

[Cl. 7.7.1.1] $r_{y-y} \geq r_{x-x}$

$I_{y-y} \geq I_{x-x}$

or,

$[I_{yy} + A(\frac{s}{2} \mp c_y)^2] \times 2 \geq 2 \times I_{x-x}$

or, $[434 + 54.4 \times (\frac{s}{2} - 2.44)^2] \times 2 \geq 2 \times 10000$

solving, $s = 31.401 cm$ {adopt s greater than this}

i.e. [$s = 314.01 mm$]

Let's adopt spacing between channels, $s = 320 mm$.

(II) Slenderness ratio of whole section:-

$\left(\frac{KL}{r_{min}}\right)_0 = \frac{6000}{\underbrace{r_{x-x}}_{r_{x-x} = 136 mm}} = 44.11$ ← effective length.

As per clause. 7.7.1.4.

$\left(\frac{KL}{r_{min}}\right)_{eff} = 1.1 \times \left(\frac{KL}{r_{min}}\right)_0 = 1.1 \times 44.11 = 48.53$

From table 9(c), [for buckling class c]

$f_{cd} = 185.205 N/mm^2$ $\left\{\begin{array}{l} 40 \rightarrow 198 \\ 50 \rightarrow 183 \end{array}\right] 48.53 \rightarrow 185.205 N/mm^2$ (f_{cd})

Design compressive strength, $F_d = f_{cd} * 2 * 5440$

$= 185.203 * 2 * 5440$

$= 2015.03\,kN > 1200\,kN$ ($F_{applied}$)

OK

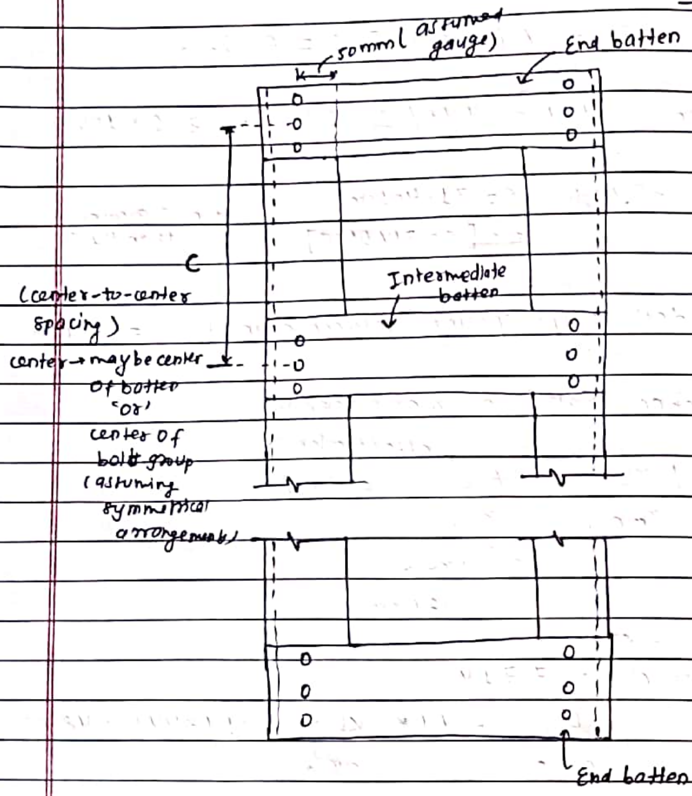


Fig: Batten with assumed number of bolts.

(IV) Spacing of battens (C)

$\frac{C}{r_{min}} < 50$ 'or' $\frac{C}{r_{min}} < 0.7 * \lambda_{whole\,section}$ (taking lesser one for safety)

[This slenderness ratio is checked for spacing b/w battens i.e. to observe component level buckling thus component level λ].

Thus,

$r_{min} = r_{y-y} = 28.2\,mm$ } Minimum r for member level buckling is r_{y-y}.

$\Rightarrow \frac{C}{28.2} < 50$ 'or' $\frac{C}{28.2} < 0.7 * 44.1$

$\Rightarrow C < 1410\,mm$ 'or' $C < 870.5\,mm$

Thus, C should be $< 870.5\,mm$

[lesser the spacing b/w battens ⇒ more safe].

Provide battens at 850mm spacing.

[Member divided into more than 3 bays Cl. 7.7.1.3] ok

⇒ {As, minimum 3 bays of battens i.e. 4 battens are to be provided check for it}.

[Cl. 7.7.1.3]

{If < 3 bays for given length, decrease C value s.t. to incorporate 3 bays}

{At least 3 bays → to use the code (thus take care of it)}.

(v) Size of battens: [Use 20mm dia bolts]

End battens:

[Cl. 7.7.2.3] → effective depth, longitudinally, not less than perpendicular distance b/w the centroids of main member.

i.e.

Effective depth, $d_{eff} \geq s - 2 * C_{y-y}$

(top bolt को center दूरी अर्को end bolt सम्मको distance)

$d_{end} \geq 320 - 2 * 24.4$

≥ 271.2 mm.

But also,

$\geq 2 *$ width of one member in plane of battens

i.e.

$\geq 2 * b_f$

$\geq 2 * 100 = 200$ mm

[$d_{end} \geq 271.2$ mm]

Governed by 271.2mm

Intermediate battens:

[Cl 7.7.2.3]

Effective depth, $d_{eff} \geq \frac{3}{4} (s - 2 * C_{yy})$

$d_{intermediate} \geq \frac{3}{4} (320 - 2 * 24.4) = 203.4$ mm

But also,

[$d_{intermediate} \geq 203.4$ mm]

$\geq 2 * b_f = 2 * 100 = 200$ mm

Governed by 203.4mm.

Overall depths.

For end battens,

$D_{end} = 271.2 + 2 * \underbrace{\text{edge distance}}_{1.5 * d_0} = 271.2 + 2 * 33$

$= 271.2$ mm

For intermediate battens,

$D_{intermediate} = 203.4 + 2 * 33 = 269.4$ mm.

Thus,

Adopt overall depths,

$D_{end} = 340$ mm

$D_{intermediate} = 270$ mm

Now,

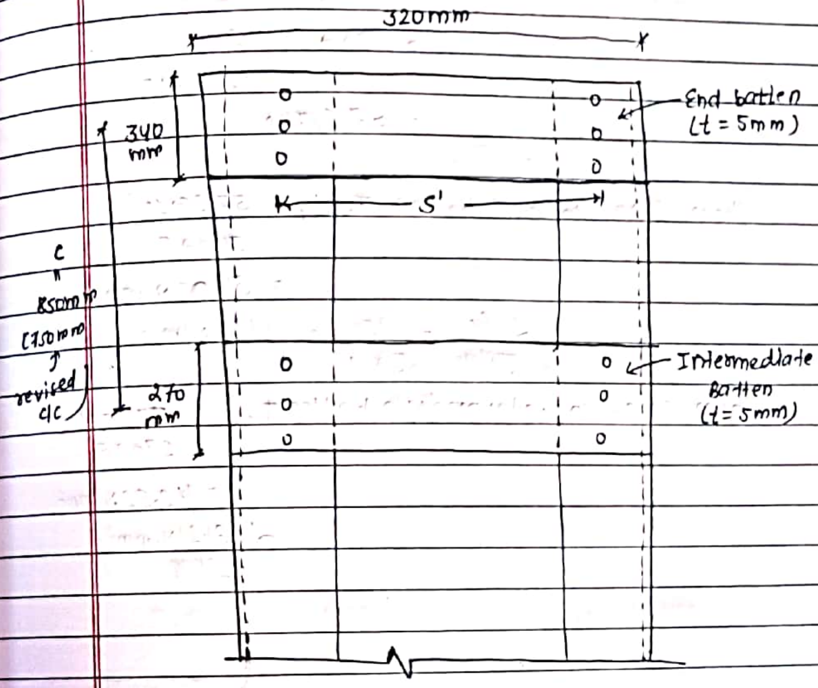
Thickness of batten, $t \geq \frac{1}{50} (s - 2 * g)$

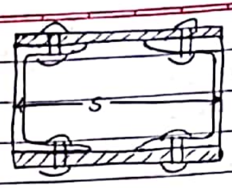
[Cl 7.7.2.3]

$\geq \frac{1}{50} (320 - 2 * 50)$

≥ 4.4 mm

Thus, Adopt thickness of 5mm.





(VI) Check for Shear & Moment:
(Cl. 7.7.2.1.)

Transverse shear,

$V_t = 2.5\%$ of 1200

$= 30\,kN$

Longitudinal shear, $V_b = \frac{V_t C}{N S'}$
[Cl. 7.7.2.1]

$= \frac{(30 \times 10^3) \times (850)}{(2) \times (320 - 2 \times 50)}$

$= 57.95\,kN$

Shear stress in end batten, $\tau_{vend} = \frac{57.95 \times 10^3}{340 \times 5}$

$= 34.09\,N/mm^2$

$\left(< \frac{f_y}{\sqrt{3}\,\gamma_{m0}} = \frac{250}{\sqrt{3} \times 1.1} = 131.21\,N/mm^2 \right)$ Okay

Shear stress in intermediate batten, $\tau_{vint} = \frac{57.95 \times 10^3}{270 \times 5}$

$= 42.92\,N/mm^2$

$(< 131.21\,N/mm^2)$ okay.

Moment, $M = \frac{V_t C}{2N}$
[Cl. 7.7.2.1.]

$= \frac{30 \times 850 \times 10^3}{2 \times 2}$

$= 6375\,kNmm.$

Bending stress in end batten, $\sigma_{bend} = \frac{M}{Z}$

$= \frac{6.375 \times 10^3}{\frac{1}{6} \times 5 \times 340^2}$

$= 0.066\,kN/mm^2 = 66.18\,N/mm^2$ $(< 227.27\,N/mm^2)$ okay

Bending stress in intermediate batten, $\sigma_{bint} = \frac{M}{Z}$

$= \frac{6.375 \times 10^3}{\frac{1}{6} \times t \times 270^2}$

$= \frac{6.375 \times 10^3}{\frac{1}{6} \times 5 \times 270^2}$

$= 0.10494\,kN/mm^2$

$= 104.94\,N/mm^2$ $\left(< \frac{f_y}{\gamma_{m0}} = \frac{250}{1.1} = 227.27\,N/mm^2 \right)$

Okay

(VII) Design of connection:-

Let us provide 20mm dia bolts of grade 4.6

Strength in single shear $= \frac{f_{ub}}{\gamma_{mb}\sqrt{3}}(A_{nb} \times n_n + n_s A_{sb})$

[Cl. 10.3.3]

$= \frac{400}{1.25\sqrt{3}} \times 1 \times 0.75 \times \frac{20^2}{4} \times \pi$

$= 45.27$ ~~kN/mm²~~ kN

Strength in bearing $= \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

$k_b \rightarrow \frac{e}{3d_0} = \frac{\cancel{1.5d_0}\ 50}{3d_0} = 0.75$

$\rightarrow \frac{p}{3d_0} - 0.25 = \frac{2.5d}{3d_0} - 0.25 = 0.51$ ($d = 20$, $d_0 = 22$)

$\rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

$\rightarrow 1$

Adopt $k_b = 0.51$

$V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 5 \times 410}{1.25}$ (5: governed by batten thickness)

$= 41.82$ kN

Bolt value $= 41.82$ kN

No. of bolts required $= \frac{57.95}{41.82} = 1.39 \approx 3$

(keeping it 3 s.t. the central bolt will be in c/L)

Let's provide 3 bolts in single vertical row. (Edge distance = 50mm, pitch = $\frac{340-50\times 2}{2}$ = 120mm)

Direct shear in bolts, $F_1 = \frac{57.95}{3} = 19.32$ kN (3 = no of bolts)

: Actual provided ↓ ⇒ k_b ↑ ⇒ BV ↑ ⇒ less bolt ⇒ Thus 3 bolts will keep it safe in any case

Force due to moment, $F_2 = \frac{(P \times e) \times r}{\sum r^2}$ (P × e = M (VI))

$= \frac{6375 \times 120}{2 \times 120^2}$

$= 26.55$ kN

(50mm, 120mm, 120mm, 50mm)

Resultant Force in extreme bolt,

$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$ ($\theta = 90°$)

$= 32.83$ kN $\leq$ (BV = 41.82 kN)

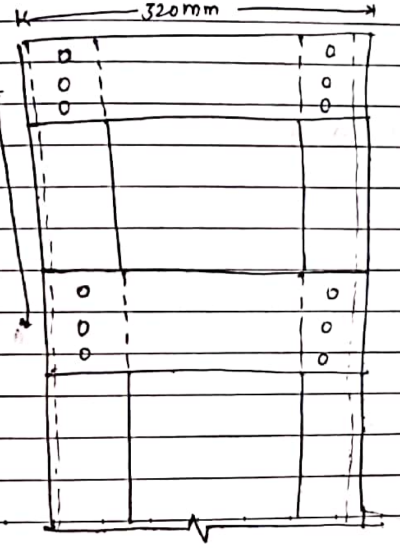
OK

(ht. same no. bolt at end)

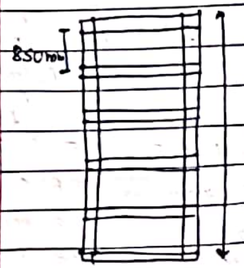
Fig

320mm

e = 750mm (revised)



Number of battens =



$$\text{No. of battens} = \frac{6000}{850} + 1$$

$$= 7.05 + 1$$

$$= 8.05$$

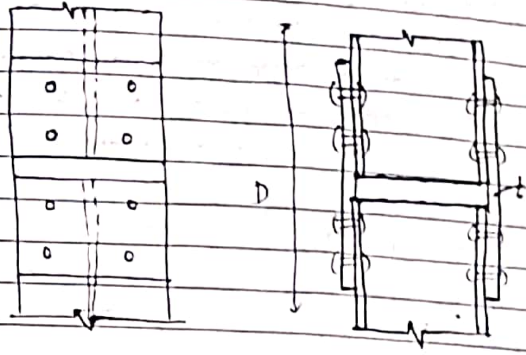
$$\approx 9$$

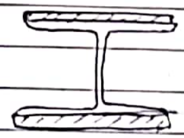
Revised c/c spacing between the battens

{1 c/c spacing = 2 battens}

$$= \frac{6000}{8} = 750\,mm$$

Splicing

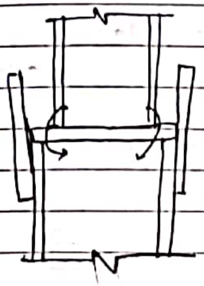




Load taken by splice plate depends whether it is well machined or not

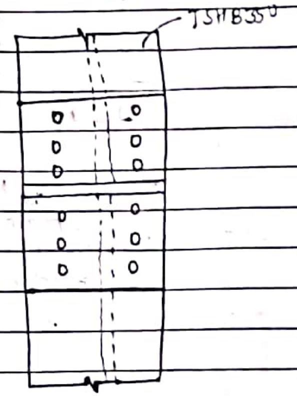
Column Splicing

Two Different sizes column



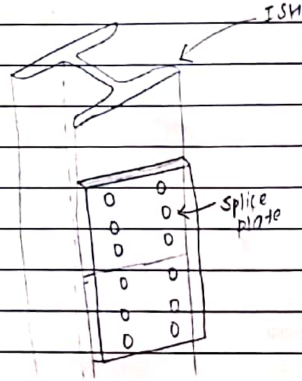
Q. A column section ISHB 350 is required to support a factored axial load of 1200kN. The section is to be spliced at a height of 2.5m. Design the splice plate & the connection using M16 bolts of grade 4.6. Assume the column ends are machined.

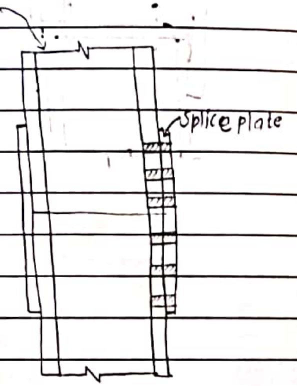
→



Machined → Force to be carried by splice plate = 50% of factored axial load

Non machined → Force to be carried by splice plate = 100% of factored axial load





Load in each splice plate, $P_s = \frac{50\% \text{ of } 1200}{2} = 300\text{kN}$

Cross-section required for each splice plate $= \frac{P_s}{(f_y/\gamma_{m_0})}$

$= \frac{300 \times 10^3}{(250/1.1)}$

$= 1320\text{mm}^2$

Width of splice plate (b_s) = Flange width of ISHB350
$= 250\text{mm}$
(IS808 : 1989)

Thickness of splice plate (t_s) $= \frac{1320}{250} = 5.28\text{mm} \nless 6\text{mm}$

Adopt, $t_s = 6\text{mm}$

* Connection design:

- Strength of bolt in shear $= \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(A_{nb}n_n + n_s A_{sb})$
[Cl. 10.3.3]
(Single shear)
$= \frac{400}{\sqrt{3} \times 1.25} \times 1 \times 0.78 \times \frac{\pi \times 16^2}{4}$
$= 28.97\text{kN}$

- For bearing,
Adopt, pitch, $p = 50\text{mm}$
end-distance, $e = 30\text{mm}$

Strength of bolt in bearing $= 2.5 k_b d t \frac{f_{ub}}{\gamma_{mb}}$
[Cl. 10.3.4]

k_b → $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.56$

→ $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.67$

→ $\frac{f_{ub}}{\ } = \frac{400}{\ } = 0.97\ldots$

Adopt $k_b = 0.56$

$t \rightarrow$ smaller of splice thickness and flange thickness

Min (6mm, 11.6mm)

~~Thus~~

$t = 6mm$ governs the bearing

$$V_{dpb} = \frac{2.5 \times 0.556 \times 16 \times 6 \times 410}{1.25}$$

$$= 43.77 kN$$

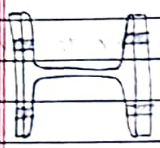
Thus,

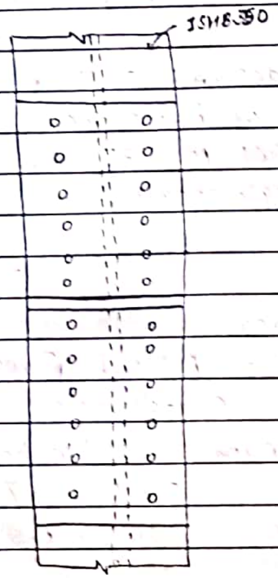
Bolt value = 28.97kN

$$\text{No. of bolts required} = \frac{300}{28.97} = 10.35 \approx 12$$

Let's provide 12 no. of bolts.

Plan





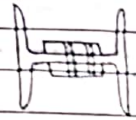
length of splice plate $= l_s = 4e + 10 \times p$

$= 4 \times 30 + 10 \times 50$

$= 620mm$

Provide two splice plates of size 620mm × 250mm × 6mm

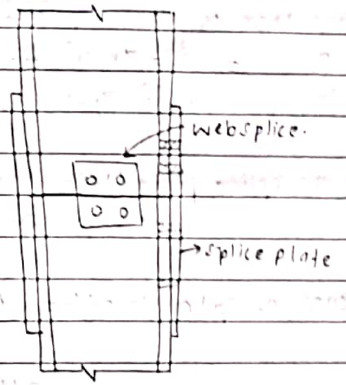
(8) Web splice:-



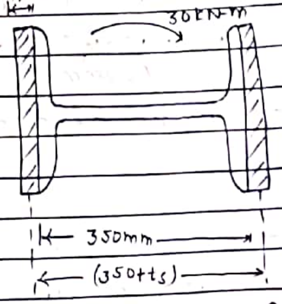
Here, bolts will be in double shear

web & splice needed

A column section ISHB350 is required to support a factored axial load of 800kN, bending moment of 30kNm and shear force of 100kN. The section is to be spliced at the height of 2.5m. Design the splice plates and the connection using M16 bolt of grade 4.6. Assume the column ends are machined.



ts (assumed first & BM is found and checked if it is within limit)



$P_s(\text{moment}) = \dfrac{M}{(350+t_s)}$

Load in each splice plate (P_s) = P_s(axial load) + P_s (moment)

Let's assume thickness of splice plate, $t_s = 8mm$ (>6mm)

$$P_s(\text{moment}) = \frac{30}{\frac{(350+8)}{10^3}} = 83.7kN$$

$$P_s = \frac{800*50\%}{2} + 83.7 = 283.7kN$$

Cross-section area of each splice plate,

$$= \frac{P_s}{(f_y/\gamma_{m_0})} = \frac{283.7*10^3}{(250/1.1)} = \cancel{1031.64mm^2}\ 1248.28mm^2$$

Width of splice-plate (b_s) = Flange width of ISHB 350
= 250mm

Thickness of splice plate (t_s) = $\dfrac{1248.28}{250}$
= 4.99mm $\underbrace{(< t_{s\,adopted} = 8mm)}$

└ यदि less t adopted किये redesign नहीं करेंगे
But if greater need to redesign

Connection design:-

Strength of bolt in shear (V_{dsb}) $= \dfrac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$
[Cl. 10.3.3]

(single shear) $= \dfrac{400}{\sqrt{3}*1.25}\left(1*0.78*\dfrac{\pi*16^2}{4}\right)$

= 28.97kN

Strength of bolt in bearing (V_{dpb}) = $\dfrac{2.5 k_b d t f_u}{\gamma_{mb}}$
[Cl. 10.3.4]

k_b Adopt, p = 50mm & e = 30mm

→ $\dfrac{e}{3d_0} = \dfrac{30}{3*18} = 0.556$

→ $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3*18} - 0.25 = 0.67$

→ $\dfrac{f_u}{f_{ub}} = \dfrac{410}{400} = 0.976$

→ 1.0

Adopt, $k_b = 0.556$

t → governed by t = 8mm (min of t_s & t_f)

$$V_{dpb} = \frac{2.5*0.556*16*8*410}{1.25}$$
= 58.36kN

Bolt value = 28.97kN

No. of bolts = $\dfrac{283.7}{28.97} = 9.79 \simeq 10$

Length of flange splice plate, l_s = 4*30+8*50
= 520mm

Use 2 splice plates (flange) of size 520*250*8mm

* Web splice-plate

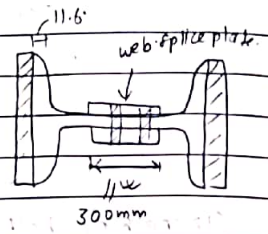
Design of web splice plate:

Here,

$D - 2t_f$

$= 350 - 2*11.6$

$= 326.8mm$



Let's adopt width of splice plate (web), $b_w = 300mm$

($D - t_f$ से 30-40 mm कम होना चाहिए)

Shear force, $V_u = 100kN = \frac{f_y}{\sqrt{3}\gamma_{m0}} *$ Area of X-section of web-splice.

Area of web-splice $= \frac{100*10^3 * \sqrt{3} * 1.1}{250}$

$= 762.10 mm^2$

Thickness of web-splice (t_w) $= \frac{762.10}{300} = 2.54mm \not< 6mm$

[Adopt, t_w = ~~2.54m~~ 6mm.]

Connection design: (web-splice):

Strength of bolt in double shear (V_{dsb}) $= \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$

(Cl. 10.3.3) $= \frac{400}{\sqrt{3}*1.25} * 2*0.78*\frac{\pi*16^2}{4}$

$= 57.95kN.$

Strength of bolt in bearing (V_{dpb}) $= \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

(Cl. 10.3.4)

Adopt, $p = 50mm$ & $e = 30mm$

Similar as before,

Adopt, $k_b = 0.556$.

t = Minimum of $(2*t_s, t_w)$

$=$ Min. $(2*6, 8.3)$

$= 8.3mm$

$V_{dpb} = \frac{2.5*0.556*16*8.3*410}{1.25}$

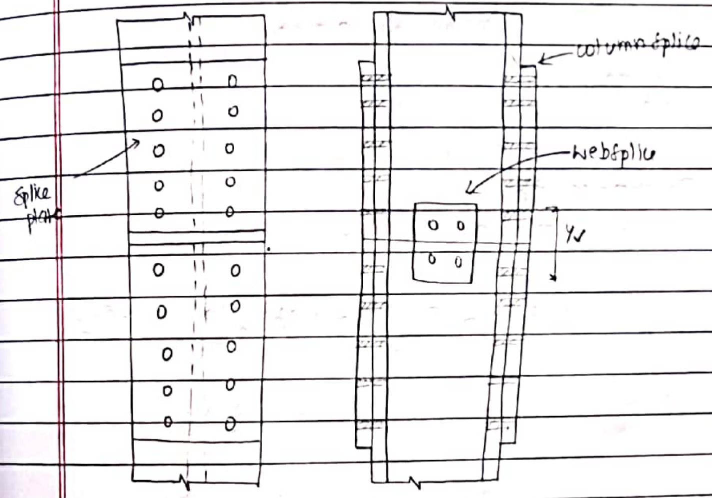
$= 60.55kN$

Bolt value $= 57.95kN$

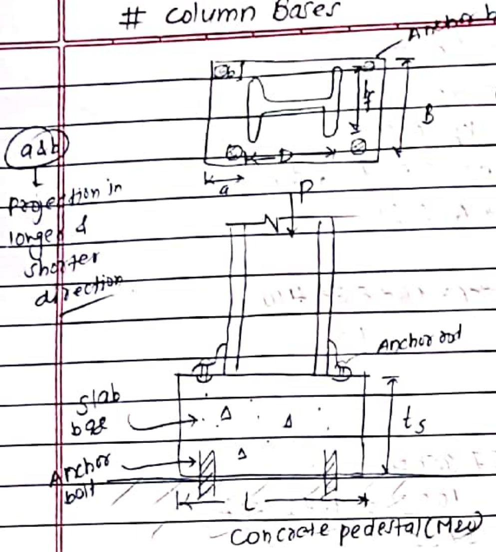
No. of bolts $= \frac{100}{57.95} = 1.72 \approx 2$

Length of web splice, $l_w = 4*30 = 120mm$

Use 2 web-splice plates in either side of web of size $120*300*6mm$.



Column Bases



Cl. 7.4.1

Max^m Bearing capacity = $0.6 * f_{ck}$

Characteristic strength of concrete cube

Approximate area of column base = $\frac{\text{Factored Load}}{\text{Max}^m \text{ Bearing capacity}}$

$L * B$

Gen. $\left\{ \frac{L}{B} = \frac{D}{b_f} \right\}$ — D: depth of I-section, b_f: width of flange; L, B: column base.

Thickness, t_s

(Cl. 7.4.3.1)

$$t_s = \sqrt{2.5 w (a^2 - 0.3 b^2) \frac{\gamma_{mo}}{f_y}} > t_f$$

(Q) A column ISHB350 carries an axial compressive factored load of 1000 kN. Design a suitable slab base & the base rests on M20 Grade concrete pedestal & use steel of grade 4.6.

→ Soln:-

For ISHB350, [Steel code]

$D = 350\,mm$

$b_f = 250\,mm$

$t_f = 11.6\,mm$

For M20 grade concrete, $f_{ck} = 20\,MPa$

Max^m bearing capacity (Cl. 7.4.1.) $= 0.6 * f_{ck} = 0.6 * 20 = 12\,MPa = 12\,N/mm^2$

(0.45 to 0.6)

Approximate area of base slab $= \frac{\text{Factored load}}{\text{Max}^m \text{ Bearing Capacity}}$

$= \frac{1000 * 10^3}{12}$

$= 83333.33\,mm^2$

$\left[\frac{L}{B} = \frac{350}{250} = 1.4 \right]$ → Ratio proportion to $\frac{D}{b_f}$

Thus, $1.4\,B^2 = 83333.33$

$\Rightarrow B = 243.97\,mm < 250\,mm$

$L = 1.4 * 243.97 = 341.57\,mm < 350\,mm$

(Adopt L & B such that to keep I-section & anchor plate)

Adopt, $L = 350 + 50 + 50 = 450\,mm$

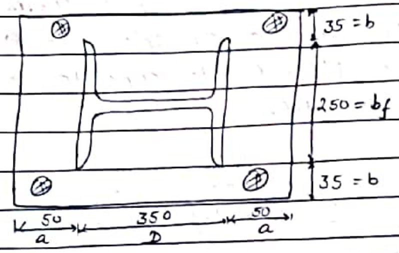
$B = 250 + 35 + 35 = 320\,mm$

(Longer projection → a → 40-50 mm
Shorter " → b → 30-35 mm)

Max^m bearing stress = $\frac{(0.45 \text{ to } 0.6) f_{ck}}{1}$

0.45 लिएर जहाँ L र B greater

But can do 0.6 लिएर then later increase L & B as per requirement to keep it in safer side

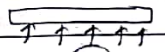


Now

Thickness of base slab

$$t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\,\gamma_{m0}/f_y} > t_f$$

[Cl. 7.4.3.1.]



$$w = \frac{1000 \times 10^3}{450 \times 320}$$

area of Base slab adopted

$$w = \frac{1000 \times 10^3}{450 \times 320} = 6.94 \text{ N/mm}^2$$

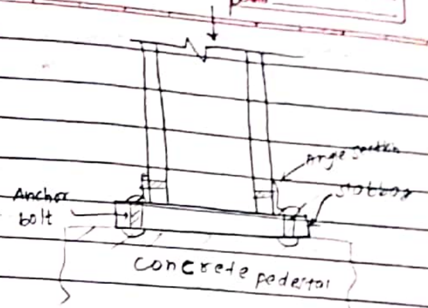
$$t_s = \sqrt{2.5 \times 6.94 \times (50^2 - 0.3 \times 35^2) \times \frac{1.1}{250}}$$

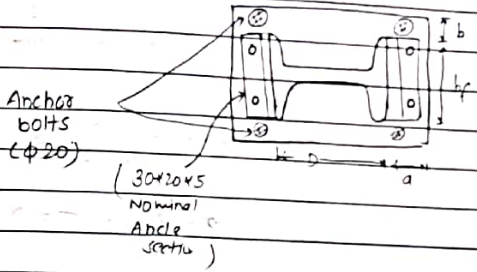
$$= 12.76 \text{mm} > (t_f = 11.6 \text{mm})$$

OK.

Let's a', Adopt $t_s = 13$mm

→ [Let's provide slab base of size 450 × 320 × 13 mm]

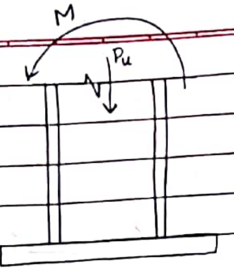


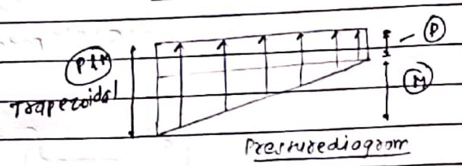


→ Let's provide 4 nos. of 20mm diameter anchor bolts to connect slab base with concrete pedestal.

→ To connect slab base of ISHB 350, nominal angle sections of size 30 × 20 × 5 are used.

* When moment also acts along with P.





Pu → gives uniform stress

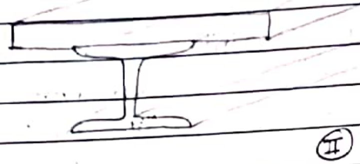
M → Triangular pressure dis (Tension - compression)

$$\text{Total stress} = \frac{P}{L * B} \pm \frac{6M}{B L^2}$$ (P*e)

$e = \frac{L}{6}$, $\sigma = 0$

$e < L/6$, total stress = compressive

Design of Flexure Members



'or'

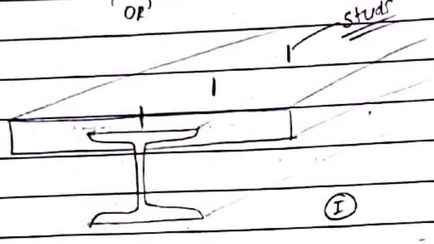


Fig: Methods of casting slab to the beam

Two cases

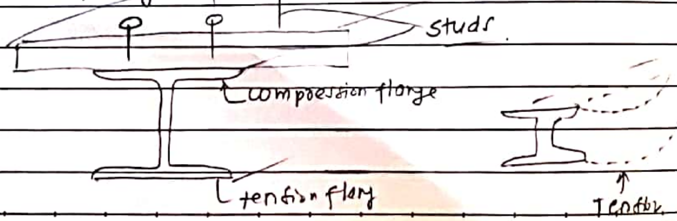
(I) Laterally restrained beams

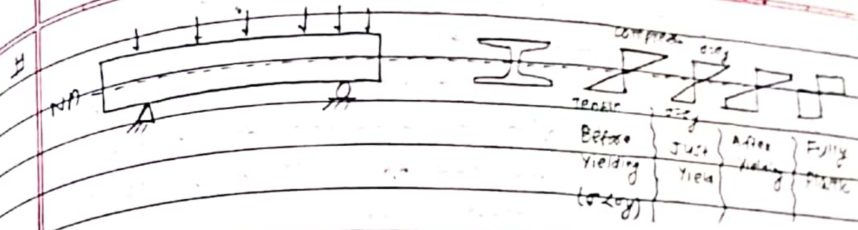
↳ slabs integrally cast into compression flange of beam

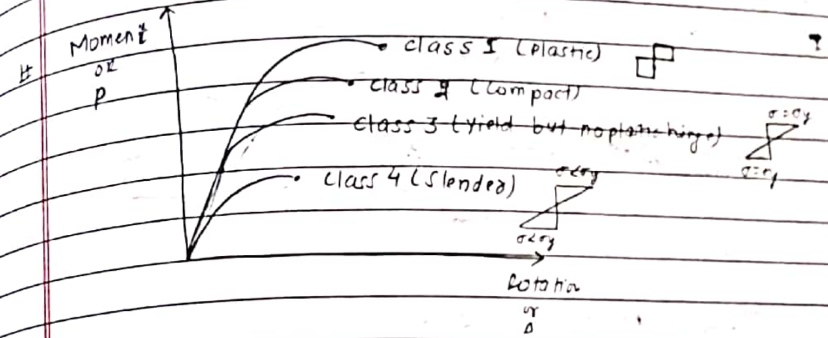
(II) Laterally non-restrained beams:

↳ Slabs resting on beams

But, if studs are kept in fig (II) & welded to compression flange then the case becomes completely restrained case







* In class 4 case, the web might undergo local buckling or cripple failure modes

Section 8

Shear force → High shear → $> 0.6 V_d$ (V_d = design shear strength)
Shear force → Low shear → $\leq 0.6 V_d$

(diff. design procedures.)

8-4 — High shear & low shear

colum ⇓ CSA

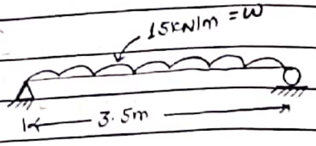
[Section Modulus (in Beam)] → $\frac{Moment}{\sigma}$

(Q) Design an appropriate joist section simply supported with 3.5m eff. span and laterally supported throughout. The member carries a uniformly distributed load of 10kN/m including its self weight. The beam is supported on walls of width 100mm thick. Use E250 Grade steel.

(low shear condition)

→ Soln:-

(I) Factored load $= 1.5 \times 10$ (laterally restrained)
(UDL) $= 15 kN/m$



$(BM)_{max} = \frac{wl^2}{8} = \frac{15 \times 3.5^2}{8}$
$= 22.97 kNm$

$(SF)_{max} = \frac{15 \times 3.5}{2} = 26.25 kN$

Plastic Section modulus required $(Z_{reqd.}) = \frac{M}{(\sigma_y/\gamma_{m_0})}$
$= \frac{22.97 \times 10^6}{(250/1.1)}$
$= 101068 \ mm^3$
$= 101.068 \times 10^3 mm^3$

[ANNEX H → plastic properties of beam]

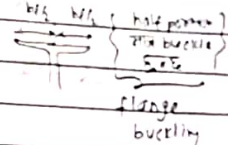
For ISWB150 section modulus both plastic & elastic exceeds $101.068 \ cm^3$

Flex → plast

Let's adopt ISWB 150@17.0kg/m (Annex H, IS 800:2007)

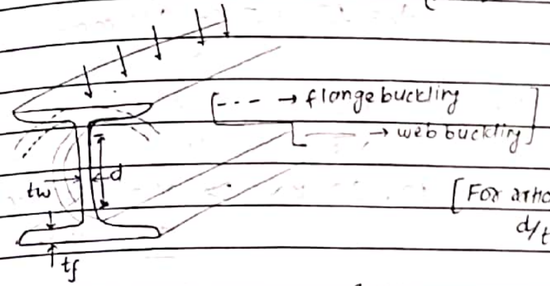
Plastic section modulus, $Z_p = 126.86 cm^3$
Elastic section modulus, $Z_e = 111.90 cm^3$
$b_f = 100mm$, $t_f = 7mm$, $t_w = 5.4mm$, $D = 150mm$
steel table → [$r_1 = 8mm$, $I_{xx} = 839.1 cm^4$] ← (IS 808: 1989)

(II) Section classification:-



Flange outstand, $b = \frac{b_f}{2} = 50mm$
$\frac{b}{t_f} = \frac{50}{7} = 7.14 < 9.4\varepsilon = 9.4$ OK (Table 2) — flange

Outstand width = $\frac{b_f}{2}$ (flange # center to end of flange)



[d → excluding curved portion]

Fig:

[For ratio limit of b/t_f & d/t_w see Table 2] → limiting width to Thickness ratio

(Here, see 9.4ε & 84ε)
plastic (Flange) — web (neutral axis at mid depth)

Web: Depth of web, $d = D - 2t_f - 2r_1 = 150 - 2 \times 7 - 2 \times 8$
[$d = 120mm$]
$\frac{d}{t_w} = \frac{120}{5.4} = 22.22 < 84\varepsilon = 84$ OK. (Table 2)

→ $\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

→ $9.4\varepsilon = 9.4$

→ $84\varepsilon = 84$

⇒ Section classified as plastic
(class I, plastic) (Table 2)

(III) Check for design shear

→ Factored shear force = 26.25 KN (V_u)

⇒ [Clause 8.4]

→ Design shear strength, $V_d = \frac{A_w f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{m_0}}$
[Cl. 8.4]

[8.4.1] ⟶ A_v = shear area

For hot rolled: $A_v = h \times t_w$ (8.4.1)

$$A_v = 150 \times 5.4 = 810 mm^2$$

$$V_d = \frac{810 \times 250}{\sqrt{3}} \times \frac{1}{1.1}$$

$$= 106.284 KN > (V = 26.25 KN)$$

OK (safe)

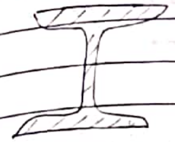
Now, shear classification → low-shear / → high-shear [8.2.1.2]

→ High/Low shear condition [Cl. 8.2.1.2]

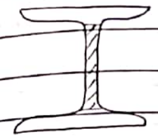
$0.6 V_d = 0.6 \times 106.28 = 63.76 KN > 26.25 KN$

The section is in low shear condition

low shear → whole section can be used → to resist shear
high shear → only web portion in use



$V_u < 0.6 V_d$ (low shear)
↑
Full section modulus in use



$V_u > 0.6 V_d$ (high shear) ⇒ [Cl. 9.2]
↑
~~Section modulus of~~
web portion only in use
(Z_w)

(IV) Check for design moment
[Cl. 8.2.1.2]

Design moment, $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m_0}} \leq \frac{1.2 Z_e f_y}{\gamma_{m_0}}$

$\beta_b = 1.0$ for plastic section

$M_d = \frac{1.0 \times 126.86 \times 10^3 \times 250}{1.1}$ (Design Moment)

$= 28.83$ KNm $>$ 22.97 KNm

OK.

$\frac{1.2 Z_e f_y}{\gamma_{m_0}} = \frac{1.2 \times 111.9 \times 10^3 \times 250}{1.1}$

$= 30.5 KNm > M_d$

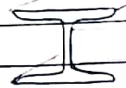
* [Check for buckling and crippling 'OR' check for lateral stability]

(v) Check for web buckling and web crippling:-
[Cl. 8.2.1.1.] ← web buckling.

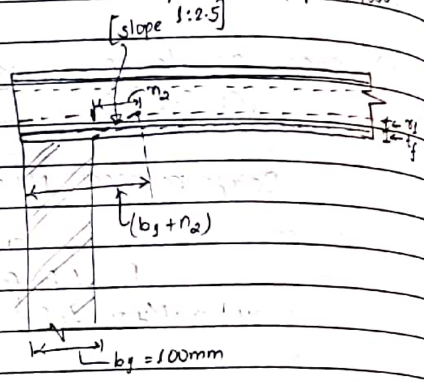
- $\frac{d}{t_w} = \frac{120}{5.4} = 22.22 < 67\varepsilon$ (No check required for web buckling)
[Cl. 8.2.1.1.]

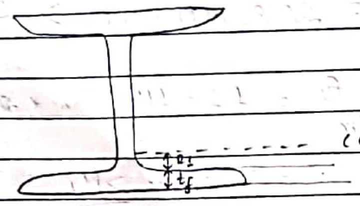
- Possible section for web buckling & web crippling ⇒ support
- If point load applied → then at the position of point load

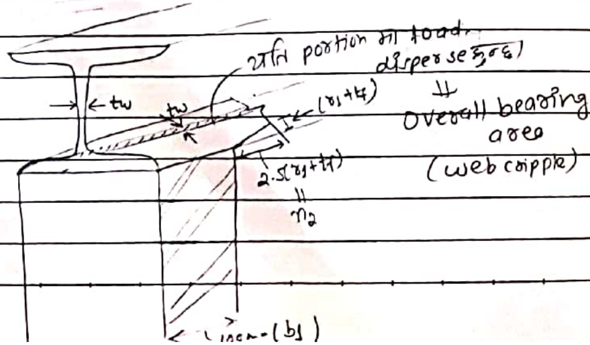
- Web crippling



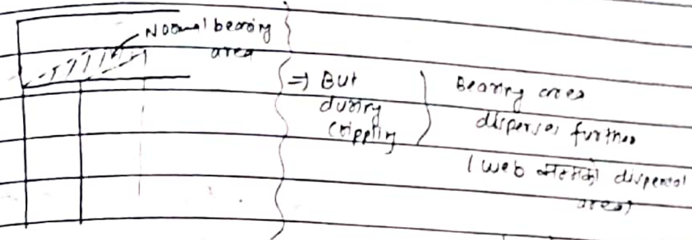
$[n_2 = 2.5(t_f + r_1)]$







$n_2 = 2.5(t_f + r_1) = 2.5(7+8) = 7.5mm$



$F_{web\,crippling} = (b_1 + n_2) * (t_w) \frac{f_y}{\gamma_{m_0}}$ ← web portion ले load लाई

$= \frac{(100 + 7.5) * 5.4 * 250}{1.1}$

$= 168.7 kN > V_u = 26.25 kN$

(OK)

(vi) Check for deflection:- ⇒ [deflection check done for serviceable load] [Applied load → 10kN]
(timber code)

Maximum deflection, $\Delta_{max} = \frac{5}{384} * \frac{w_s l^4}{EI}$ (safe load)

$= \frac{5}{384} * \frac{(15/1.5) * (3500)^4}{\left(\frac{2*10^5}{1000} kN/mm^2\right) * 839*10^4}$ (kN, mm, mm⁴)

[$E_{steel} = 2*10^5 N/mm^2$]

$= \frac{5}{384} * \frac{(15/1.5) * 10^3 * (3.5)^4}{(2.5*10^5*10^6) * (839*10^{-8})}$ (N/m^2, m^4)

$= 11.64mm$

$\Delta_{permissible}$ ~~(for elastic cladding)~~ $= \frac{span}{240}$

[Table 6] $= \frac{3500}{240}$

(Live load ⇒ simple span ⇓ ~~Elastic cladding~~) $= 14.5mm > \Delta_{max}$

(OK)

⇒ [If $\Delta_{max} < \Delta_{permissible}$, (I) बढाउन पर्छ।

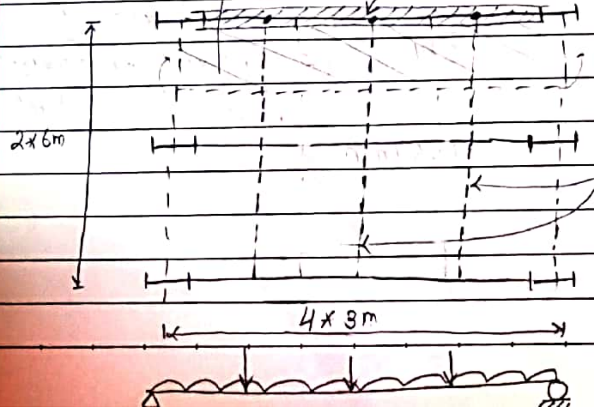
only I can be changed here.

(Q) A floor 12m × 12m in plan & as shown in figure below is to be covered with a floor made up of secondary and main beams. The secondary beams of 5kN/m are spaced at 3m intervals. It ~~so~~ supports reinforced concrete slab 125mm thick and floor finish 0.5kN/m². If the live load on the floor is 3kN/m². Design main beam located at end of the floor for bending. The compression flange of main beam is laterally restrained.

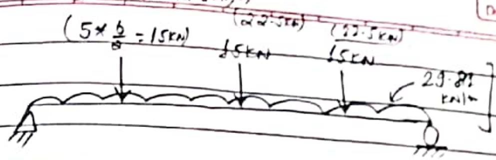
(RC slab (125mm) built integrally)

{~~Main beam~~ + column लाई जोड्छ}

{Main beam लाई + secondary beam}



For middle main beam → both side को load को half हुन्छ
[i.e. $5 \times 2 \times \frac{6}{2} = 30kN$]



→ Secondary beam (Load) $= 5 \times \frac{6}{2} = 15kN$

Slab load, $w_{slab} = (25 \times 0.125 \times 3) = 9.375 kN/m$

γ_{rcc} (25kN/m³); thickness of slab; $(\frac{6}{2} m)$

(Only half of slab gives load to end beam)

$w_{floor finish} = 0.5 \times 3 = 1.5 kN/m$ $\left(\frac{6}{2}\right)$

$w_{live load} = 3 \times 3 = 9kN/m$ $\left(\frac{kN}{m^2}\right)$ $\left(\frac{6}{2}\right)$

Factored UDL $= (9.375 + 1.5 + 9) \times 1.5 = 29.81 kN/m$

→ If intermediate beam दिएको छ भने माथिको सबै values ×2 गर्नुपर्यो।

(Δ को exact value आउँदैन वाको check गर्न पर्छ)

(Q) Design a simply supported beam of 6m span carrying a reinforced concrete floor capable of providing lateral restrain to the top compression flange. The beam carries maximum factored moment of 150 kNm & maximum factored shear load of 210 kN near supports.

→ Solution:-

(1) For E250 grade of steel:

$f_y = 250\,MPa$

(web) $f_{yw} = 250\,MPa$

partial safety factor, $\gamma_{m_0} = 1.1$

Factored Bending moment = 150 kNm = $(BM)_{max}$

Factored Shear load = 210 kN = $(SF)_{max}$

plastic section modulus required ($Z_{reqd.}$) $= \dfrac{M}{(f_y/\gamma_{m_0})}$

$= \dfrac{150 \times 10^6\ Nmm}{(250\,N/mm^2)/1.1}$

$= 660000\ mm^3$

$= 660 \times 10^3\ mm^3$

$= 660\ cm^3$

Let's adopt ISLC 350 @ 38.8 kg/m (Annex H, IS800:2007)

Plastic section modulus (Z_p) = $622.95\,cm^3$

Elastic section modulus (Z_e) = $532.1\,cm^3$

$b_f = 100mm$, $t_f = 12.5mm$, $t_w = 7.4mm$, $D = 350mm$

$[r_1 = 13mm$, $I_{xx} = 9330\,cm^4]$ ← Steel table (IS808:1989)

[If adopted ISMB 250 @ 51 kg/m] ⇒ $Z_p = 678.73\,cm^3$, $Z_e = 618.9\,cm^3$, $b_f = 250mm$, $t_f = 9.7mm$, $t_w = 6.9mm$, $D = 250mm$

$[r_1 = 10\,mm$, $I_{xx} = 7740\,cm^4]$

let's adopt ISLB 350 @ 49.5 kg/m (Annex H: IS:800:2007)

Relevent properties of the sections are:-

$D = 350mm$, $b_f = 165mm$, $t_f = 11.4mm$, $t_w = 7.4mm$

[$r_1 = 16mm$, $I_{xx} = 13200\ cm^4$] ← [IS 808: 1989].

Plastic section modulus (Z_p) = $851.11\ cm^3$

Elastic section modulus (Z_e) = $751.9\ cm^3$

(I) Section classification.

Flange outstand, $b = \frac{b_f}{2} = \frac{165}{2} = 82.5mm$

$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.24 < 9.4\varepsilon = 9.4$ (OK) } flange

$\varepsilon = \sqrt{\frac{250}{f_y}} = 1$ → (Table 2)

$f_y = 250$

Depth of web, $d = D - 2t_f - 2r_1 = 350 - 2*11.4 - 2*16$

$= 295.2mm$

$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.95 < 84\varepsilon = 84$ (OK) } web

(Table 2)

Section classified as class I, plastic.

(II) Check for design shear

→ Factored shear force (V_u) = 210 kN

→ Design shear strength, $V_d = \frac{A_w f_{yw}}{\sqrt{3}} * \frac{1}{\gamma_{m_0}}$

[Cl 8.4]

[8.4.1] → A_w = shear area

For hot rolled, $A_w = h * t_w$ (8.4.1)

$= 350 * 7.4$

$= 2590\ mm^2$

$V_d = \frac{2590 * 250}{\sqrt{3} * 1.1} = 339.849\ kN$

($> V_u = 210kN$)

OK

→ High/Low shear condition [Cl. 8.2.1.2]

$0.6 V_d = 0.6 * 339.849 = 203.91\ kN$

($< 210kN$)

The section is in high shear condition.

High shear ⟹ (Cl. 9.2.2. → combined shear & Moment)

High shear → check for moment

(High shear → plastic shear of web portion निकालेंगे उसे घटा ⇓ Additional than low shear condition)

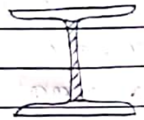
(IV) Design moment (Moment capacity)

[Cl. 9.2.2]

∵ Plastic or compact section

$$M_{dv} = M_d - \beta (M_d - M_{fd}) \le \frac{1.2 Z_e f_y}{\gamma_{m_0}}$$

flange की part को घटाके (after high shear web portion को निकाल जाने)



$$\beta = \left(\frac{2V_u}{V_d} - 1\right)^2$$

$$= \left(\frac{2 \times 210}{339.85} - 1\right)^2$$

$$= 0.0556$$

[Cl. 8.2.1.2] $M_d = \beta_b Z_p \frac{f_y}{\gamma_{m_0}} \le \frac{1.2 Z_e f_y}{\gamma_{m_0}}$ (Z_p = 851.11 cm³, f_y = 250, γ_{m_0} = 1.1)

for plastic ($\beta_b = 1$)

$$\frac{1 \times 851.11 \times 10^3 \times 250}{1.1} \le \frac{1.2 \times 751.9 \times 10^3 \times 250}{1.1}$$

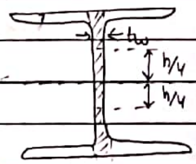
$$193.43 \text{ kN-m} \le 205.06 \text{ kNm}$$

(Okay)

M_{fd} = plastic design strength as governed by web yielding or web buckling.

$$M_{fd} = Z_{fd} \times \frac{f_y}{\gamma_{m_0}}$$

only flange [$Z_{fd} = Z_p - Z_{pw}$

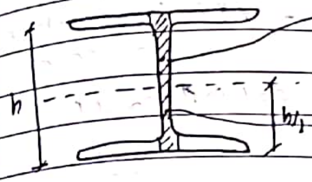


Plastic section modulus → $[Z_p = A_c \bar{y}_c + A_T \bar{y}_T]$

(Shear सिर्फ only web portion)

Similarly, $Z_{pweb} = A_c \times y_c + A_T \times y_T$

$$= \left(t_w \times \frac{h}{2}\right) \times \left(\frac{h}{4}\right) + (t_w \times h/2) \times (h/4)$$



$Z_{pweb} = (t_w \times h/2) \times h/4 + (t_w \times h/2) \times h/4$

$$\left[Z_{pweb} = t_w \times \frac{h^2}{4}\right]$$

$$Z_{fd} = 851.11 \times 10^3 \text{ mm}^3 - (7.4) \times \frac{350^2}{4} \text{ mm}^3$$

$$= 851.11 \times 10^3 - 226.625 \times 10^3$$

$$= 624.48 \times 10^3 \text{ mm}^3$$

$$M_{fd} = Z_{fd} \times \frac{f_y}{\gamma_{m_0}}$$

$$= 624.48 \times 10^3 \text{ mm}^3 \times \frac{250 \text{ N/mm}^2}{1.1}$$

$$= 141.93 \text{ kNm}$$

Design moment,

$$M_{dv} = 193.43 - 0.056(193.43 - 141.93)$$

$$= 190.546 \text{ kN-m} > 150 \text{ kNm (applied moment)}$$

$$\le 205.06 \text{ kN-m}$$

$(1.2\, Z_e f_y/\gamma_{m_0})$

OK.

Ajanta Page No. Date

(V) Check for web buckling & web crippling (lateral stability)

web buckling:-

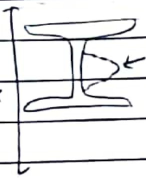
$d =$ ht. of web $= 350 - 2*11.4 - 2*16$ (D, t_f, r_1)

$= 295.2$ mm

[Cl. 8.2.1.1]

$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 67\varepsilon$ (No need to check for web buckling)

($\varepsilon = 1$ for $f_y = 250$ MPa)



← web buckling

(can also check for detailed web buckling)

(done as in laterally unsupported beam)

Web crippling:

(Even if support width not given assume & check for crippling)

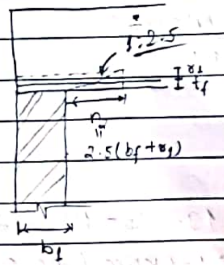
considering load dispersion

{Assume bearing depth (support width) $= b_1 = 100$mm} If not given

$n_2 = 2.5(t_f + r_1)$

$= 2.5(11.4 + 16)$

$= 68.5$mm



← [Cl. 8.7.4]. 8.7.3

$F_w = (b_1 + n_2) * t_w * \frac{f_y}{\gamma_{m_0}}$

$= \underbrace{(100 + 68.5) * 7.4}_{\text{Bearing area}} * \frac{250}{1.1}$

$= 283.39$kN > 210kN (applied)

OK

Ajanta Page No. Date

[If this is unsafe, assume b_1 high value]

[If asked for check for deflection, Find w from moment & check for deflection]

[Cl. 8.2.2.]

(Q) Design a laterally unsupported beam of 4m span carrying a reinforced concrete floor. The beam carries maximum factored moment of 550 kNm & Maximum factored shear force of 200kN near supports. Check for its lateral stability and deflection.

→ Sol^n :-

(I) Assume, E250 grade steel

$f_y = 250 N/mm^2$

$f_{yw} = 250 N/mm^2$

$\gamma_{m_0} = 1.1$

$M_u = (BM)_{max} = 550 kNm$

$V_u = (SF)_{max} = 200 kN$

Plastic section modulus required

$$(Z_{reqd.}) = \frac{M_u}{(\sigma_y/\gamma_{m0})}$$

$$= \frac{550 * 10^3 * 10^3 \ Nmm}{(250 N/mm^2 / 1.1)}$$

$$= 2.42 * 10^6 \ mm^3$$

$$= 2420 * 10^3 \ mm^3$$

$$= 2420 \ cm^3$$

For laterally unsupported Zp → 30-40% increased value लेकर adopt करना from Annex H table

[40% increased value]

Let's adopt $Z_{preq} = 1.4 \times 2420 = 3388 \ cm^3$

Adopt, ISMB 600, $Z_p = 3510.63 \ cm^3$

(Annex H IS 800:2007) $Z_e = 3060.4 \ cm^3$

↳ [laterally unsupported ⇒ governed by Moment
↓
(high/low shear) → not mentioned]

$t_w = 12 mm$, $t_f = 20.8 mm$, $b_f = 210 mm$, $D = 350 mm$

$[r_1 = 20 mm, \ I_{xx} = 91800 \ cm^4]$ ← Steel table (IS SP: 1989)

(II) Section classification

Flange

$$\frac{b}{t_f} = \frac{210/2}{20.8} = 5.05 < 9.4\varepsilon = 9.4 \ [\text{Plastic}]$$

(Table 2)

b = Flange outstand $= \frac{b_f}{2} = \frac{210}{2} = 105 mm$

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

Web

$$\frac{d}{t_w} = \frac{(600 - 2*20.8 - 2*20)}{12} = 43.2 < 84\varepsilon = 84$$

(Plastic) (Table 2)

The adopted section is 'Plastic'

(III) Check for design moment : [Cl. 8.2.2.]

$$M_d = \beta_b * Z_p * f_{bd}$$

laterally unsupported beam.

[f_{bd} (Cl. 8.2.2.) → $f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}$] → difficult to find (lots of parameters)

Better

8.2.2.1 ⇒ $f_{cr,b}$ & or

~~approx calculate from~~ Table 14

[Cl. 8.2.2.1]

$f_{cr,b}$ of non-slender rolled steel sections

$$f_{cr,b} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left\{\frac{L_{LT}/r_y}{h_f/t_f}\right\}^2\right]^{0.5}$$

L_{LT} ⟹ table 15 (2.3)

acc. to condition of restraint

(here, laterally unsupported)

⇓

(i) to (iv) ⇓ laterally restrained

(v) warping not restrained in both flanges

(if not mentioned in question)

Adopt,

effective length for lateral torsional buckling [L_{LT} = 1.0L (for restrained in torsion and not restrained for warping)

= 1.0 * 4000

= 4000mm.

h_f = c/c distance between flanges

$= D - \frac{t_f}{2} \times 2 = 600 - 20.8$

$h_f = 579.2$mm

α_{LT} = 0.21 for rolled steel section (imperfection factor)

→ [8.2]

(table 13(a), (for $\alpha_{LT} = 0.21$)) → $f_{cr,b}$ → for f_{bd}

$$f_{cr,b} = \frac{1.1\pi^2 \times 2.0\times10^{11}}{(4000/41.2)^2}\left[1+\frac{1}{20}\left\{\frac{4000/41.2}{579.2/20.8}\right\}^2\right]^{0.5}$$

r_y → Min. r04 (Annex H)

$= \cancel{306.69 \times 10^6\ N/m^2}$ = 292.08×10^6 N/m²

$= \cancel{306.69\ N/mm^2}$ = 292.08 N/mm²

From table 13(a), for $\alpha_{LT} = 0.21$

$f_{cr,b}$ = ~~306.69 N/mm²~~ 292.08 N/mm²

f_y = 250 MPa

$f_{cr,b}$	
~~350~~ →	
300 →	163.6
250 →	152.3

] ~~292.08~~ ⇒ 161.81 N/mm²

$\left[163.6 - \frac{(163.6-152.3)}{(300-250)} \times \frac{7.92}{\cancel{292.08}}\right]$

300−292.08

$[f_{bd} = 161.81 N/mm^2]$

Design bending compressive stress, f_{bd} = 161.81 N/mm²

$M_d = \beta_b Z_p f_{bd}$ (β_b = 1 for plastic)

$= 1 \times 3510.63 \times 10^3 \times 161.81$

= 568 kN-m > 550 kN-m (OK)

(benefit of taking 40% excess)

IV) Check for shear:-

Design shear,

$$V_d = (t_w * h) \Big/ \left(\frac{f_y}{\sqrt{3}\,\gamma_{m_0}}\right)$$

A_w

[Cl. 8.4]

[Cl. 8.4.1] ⇒ $A_w = h * t_w$

$$V_d = (12 * \cancel{350}\ 600) * \left(\frac{250}{\sqrt{3} * 1.1}\right)$$

$= 944.75\text{KN}$ ~~551.107KN~~ $> (V_u = 200\text{KN})$

V) Check for lateral stability

web crippling :-

Bearing area

$$F_w = (b_1 + n_2) * t_w * \frac{f_y}{\gamma m_0}$$

$n_2 = 2.5(t_f + r_1) = 2.5(20.8 + 20) = 102\text{mm}$

$$F_w = \cancel{102*}\ (100 + 102) * 12 * \frac{250}{1.1}$$

[Assume support width, $b_1 = 100\text{mm}$]

$= 550.91\ \text{KN} > 200\text{KN}$ (OK)

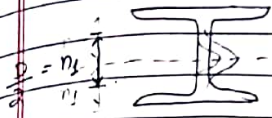
crippling → web को मात्र पड्किन्छ (flange को nature सकिएको)

buckling → Laterally.

web buckling (detailed not as previous check of 67ε)

check गर्नेर हेर्नकोलागि { $\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 67\varepsilon = 67$] ← ओके हुन पर्दछ

[Cl. 8.2.1.1.] $\left(\varepsilon = \sqrt{\frac{250}{f_y}} = 1\right)$



(buckling हुने tendency ⇒ max at middle of web)

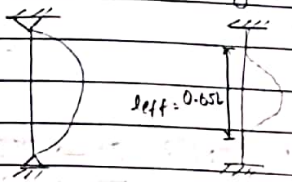
$$F_w = (b_1 + n_1) * t_w * f_{cd}$$

← compression

(यो specifically निकाल्न पर्छ for buckling)

$n_1 = \frac{600}{2} = 300\text{mm}$

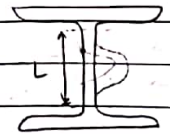
(web को buckling both end fixed जस्तो हुन्छ)



So, Use $l_{effective}$ for both end fixed.

[Table 11 for both end fixed]

For web, $l_{eff} = 0.65(600 - 2t_f - 2r_1)$



$= 0.65(600 - 2*20.8 - 2*20)$

$= 336.96\text{mm}$

[r_{web} ⇒ not directly I_{web}/A_{web}]

b_1 t_w

Page No.
Date

$r = \sqrt{\frac{I}{A}}$

$I = \frac{b_1 * t_w^3}{12} = \frac{100 * 12^3}{12} = 14400\ mm^4$

$A = b_1 * t_w = 100*12 = 1200 mm^2$

$r = \sqrt{\frac{14400}{1200}} = 3.464 mm$

$\lambda = \frac{l_{eff}}{r} = \frac{336.96}{3.464} = 97.27$ $\left(\frac{KL}{r}\right)$

Table 9.ⓐ f_{cd}

(Buckling class → Just web को मात्र मात्र)
→ web → as a plate element considered जाते → class 'c'
→ But overall section हेर्दा its → class 'a'
⇓
class I, plastic section)

Table. 9.ⓐ, $f_{cd} = 136.64 N/mm^2$

90 →	149	97.27 →
100 →	132	136.64

(can adopt any (more safer ⇓ इत्ते लिन्ने) 110.822)

If considered, ⓒ

9.ⓒ, $f_{cd} = 110.822\ N/mm^2$

90 →	121	97.27
100 →	107	↳ 110.822

Page No.
Date

If adopting ⓒ,

$F_w = (100+300) * 12 * 110.65 = 531.5 kN > 200 kN$ (OK)

If adopted ⓐ,

$F_w = (100+300) * 12 * 136.45 = 654.9 kN > 200 kN$ (OK)

Ⓥ Check for deflection:- (deflection check → governing moment here)

$M_u = 550 kN$

$\left(M_{max} = \frac{wl^2}{8}\right)$

*Assuming UDL, $w_u = \frac{550 * 8}{4^2} = 275\ kN/m$

Safe, $w_s = \frac{w_u}{1.5} = 183.33\ kN/m$
↑ factor

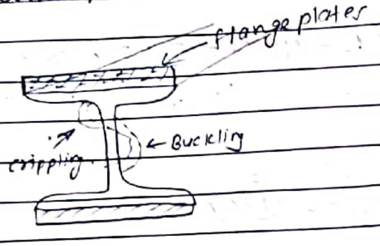
$\Delta max = \frac{5}{384} \frac{w_s L^4}{EI}$

$\Delta permissible = \frac{L}{240}$

$\Delta max = \frac{5}{384} * \frac{183.33 * (4000)^4}{2*10^5 * (91800*10^4)} = 3.328 mm$
↑ (I_{xx})

$\Delta permissible = \frac{4000}{240} = 16.67 mm > \Delta max$ (OK)
↑
(Table 6)
(Deflection limits).

Built-up section



* Providing flangeplates & web stiffners (vertical & Hz.)

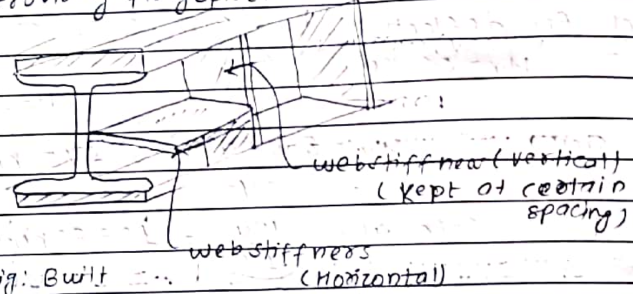


Fig: Built

* Plate girders

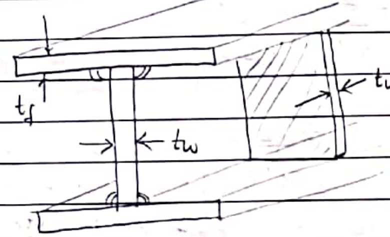


Plate girder.

* For built-up section:-

$$\left[\underbrace{Z_p \text{ required}}_{\frac{M_u}{f_y/\gamma_{m_0}}} = Z_{Ip} + Z_{\text{flange plate}}\right]$$

(Q) Design a steel beam section for supporting roof of a big hall for a data given below:-

Clear span = 7m

End-bearing = 150mm (Support width)
↳ for buckling, crippling checks

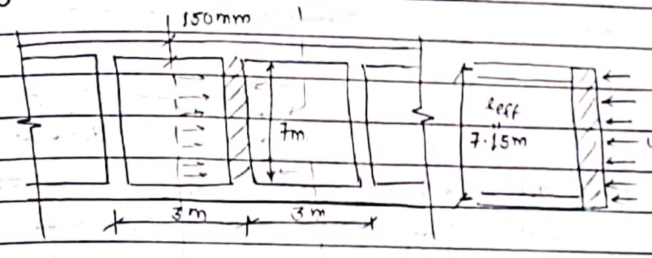
c/c spacing of beams = 3m

Imposed load on beam = 10 kN/m²

Dead load including self weight = 4 kN/m²

The depth of the beam is restricted to 375mm. The compression flange of the beam is laterally supported throughout.

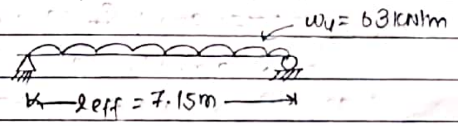
→ Soln:-



Factored imposed load = 1.5 × 10 × 3 = 45 kN/m

Factored dead load = 1.5 × 4 × 3 = 18 kN/m
↳ ($\frac{3}{2}$ m span load on each side of beam)

∴ Total factored load, w_u = 63 kN/m



(I) $M_u = (BM)_{max} = \frac{wl^2}{8} = \frac{63 * (7.15)^2}{8} = 402.59 kNm$

$V_u = (SF)_{max} = \frac{wl}{2} = \frac{63 \times 7.15}{2} = 225.22 kN$

Required plastic section modulus,

$Z_{preqd.} = \frac{M_u}{(f_y/\gamma_{m0})} = \frac{402.59 kNm}{(250 N/mm^2)/1.1}$

$= 1771396 \ mm^3$

(beam depth restricted to 375mm but for this size $Z_{preqd.}$ as above cannot be provided. Additionally, we need to provide flange plate as well)

So, choose beam of depth < 375mm say 350mm s.t. remaining space provided for flange plates)

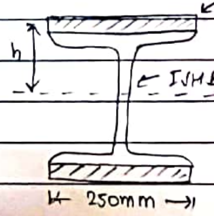
Since the depth of beam is limited to 375mm
Let's adopt ISHB 350 @ $\underbrace{72.4 kg/m}_{\text{higher unit wt. of available}}$

For ISHB 350, @ 72.4 kg/m (From Annex H: IS 800:2007)

$(Z_p)_I = 1268.69 \times 10^3 mm^3$

Required plastic section modulus for plate

$(Z_p)_{plate} = 1771.35 * 10^3 - 1268.69 * 10^3$
$= 502.71 * 10^3 mm^3$



ISHB 350 @ 72.4 kg/m
$\rightarrow D = 350 \ mm$
$b_f = 250 \ mm$
$t_f = 11.6 \ mm$
$t_w = 10.1 \ mm$
Annex H.

$\left[\begin{matrix} r_1 = 12 \ mm \\ I_{xx} = 19800 cm^4 \end{matrix} \right]$ steel plate.

Now,

Area of flange plate $= \frac{(Z_p)_{plate}}{D}$

(Neglecting thickness of flange plate, tp)

or, $250 * t_p = \frac{502.7 * 10^3}{350} \Rightarrow \boxed{t_p = 5.74 mm} < 6mm$

$\Rightarrow (I_p)_{plate} = \left(\frac{b_f * t_p^3}{12} + A_{plate} * h^2 \right) * 2$ [tp → plate thickness] (negligible (assume))

$\rightarrow$ MOI of plate about centroidal axis (- - - -)

$= A_{plate} * h^2 * 2$

$= A_{plate} * \left[\frac{D}{2} + \frac{t_p}{2} \right]^2 * 2$ (negligible (assume)) [D → depth of I-section]

$= A_{plate} * \frac{D^2}{2}$

Thus, $(Z_p)_{plate} = \frac{(I_p)_{plate}}{y} = \frac{A_{plate} * D^2/2}{D/2}$

$\boxed{(Z_p)_{plate} = A_{plate} * D}$

But, $\boxed{t_p \geq 6mm}$ { tp such that the given depth 375mm doesn't exceed. Here, can provide tp upto 12.5mm }

~~Adopt, thickness~~

Let's provide 8mm thick flange plate on both sides

$(Z_p)_{provided} = (Z_p)_I + (Z_p)_{plate}$ (parallel axis theorem)

(centroidal axis)

$= 1268.69 \times 10^3 + \left[\frac{250 * 8^3}{6} + (250 * 8) * \left(\frac{350}{2} + \frac{8}{2} \right) \right] * 2$

$= 1990.0 * 10^3 mm^3$

Total depth of beam $= 350 + 2 * 8 = 366 mm < 375mm$
OK

(Laterally supported → check for shear first)

(II) Section classification:-

ISHB 350 @ 72.4 kg/m

$b_f = 250mm$, $t_f = 11.6mm$, $t_w = 10.1mm$, $D = 350mm$

$r_1 = 12\,mm$, $I_{xx} = 19800\,cm^4$

Flange: $\frac{b}{t_f} = \frac{250/2}{11.6} = 10.77 < 15.7\varepsilon = 15.7$ ← flange outstand $\left(\frac{b_f}{2}\right)$ (Table 2) → semi-compact

$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

Web: $\frac{d}{t_w} = \frac{(350 - 2*11.6 - 2*12)}{10.1} = 29.9 < 84\varepsilon$ (Table 2) → plastic

[Of web & flange classify the section as lower class of two here it would be semi-compact]

Section is semi-contact

(III) Check for design shear:-

Factored shear force, $V_u = 225.22\,kN$

Design shear strength, $V_d = \frac{A_v f_{yw}}{\sqrt{3}} * \frac{1}{\gamma_{m_0}}$ [Cl. 8.4.1]

[8.4.1.1] → $A_v = h * t_w = 350 * 10.1$

$= 3535\,mm^2$

$V_d = \frac{(3535) * 250}{\sqrt{3} * 1.1}$

$= 463.85 > (225.22 = V_d)$

ok

High/low shear condition

[8.2.1.2.]

$0.6V_d = 278.3\,kN > 225\,kN$

[Low-shear condition]

(IV) Check for bending strength.

[Cl. 8.2.1.2]

$M_d = \beta_b Z_p f_y / \gamma_{m_0} < \frac{1.2 Z_e f_y}{\gamma_{m_0}}$

Elastic section modulus, $Z_e = \frac{I_{xx}}{y}$

overall built-up section $\left[I_{xx} = 19800 * 10^4 mm^4 + 2 * \left\{ \frac{250 * 8^3}{12} + (250*8) * \left(\frac{350}{2} + \frac{8}{2}\right)^2 \right\} \right]$

$= 19800 * 10^4 + \cancel{50128.53 * 10^4}\ 12818.53 * 10^4$

$= 32618.53 * 10^4 mm^4$

$Z_e = \frac{32618.53 * 10^4}{\underbrace{\left(\frac{350}{2} + 8\right)}_{y}} = 1782.43 * 10^3\,mm^3$

$\left[\beta_b = \frac{Z_e}{Z_p}\right]$ → For semi-compact section (8.2.1.2)

$M_d = \frac{Z_e}{Z_p} * Z_p * \frac{\cancel{250}\ f_y}{\cancel{1.1}\ \gamma_{m_0}}$

$= 1782.43 * 10^3 * \frac{250}{1.1}$

$= 405.10\,kNm > 402.5\,kNm$ (ok)

benefit of taking thicker flange plate (it would be better if adopted $t_p = 10mm$)

$\frac{1.2 Z_e f_y}{\gamma_{m_0}} = 486.12 \text{ kNm}$ $(\geq M_d)$ OK

(like 20% increment to Md for semi-compact section)

$\left(M_d \leq \frac{1.2 Z_e f_y}{\gamma_{m_0}}\right)$ ok

(V) check for web-buckling & web-crippling (lateral stability)

web buckling

d = ht. of web = 350 − 2×11.6 − 2×12 = 302.8 mm

[Cl. 8.2.1.1]

$\frac{d}{t_w} = \frac{302.8}{10.1} = 29.98 \leq (67\varepsilon = 67)$

[No need to check for web buckling]

leave space for detailed web buckling check if not mentioned in question no need to do but write if ok

web crippling

[Cl. 8.7.4]

$F_w = \frac{(b_1 + n_2)\, t_w\, f_y}{\gamma_{m_0}}$

$b_1 = 150$ mm

$n_2 = 2.5(t_f + r_1) = 2.5(11.6 + 12) = 59$ mm

$F_w = (150 + 59) \times 10.1 \times \frac{250}{1.1}$

$= 479.75 \text{ kN} > (225 \text{ kN} = V_u)$

Ok

(VI) Check for deflection

must use safe load for deflection check

$\Delta_{max} = \frac{5}{384} \frac{w_s l^4}{EI}$ $\left[w_s = \frac{w_u}{1.5}\right]$

$= \frac{5}{384} \frac{(63/1.5) \times 7.15^4}{2 \times 10^8 \times \frac{3.2618.53 \times 10^4}{(1 \times 10^3)^4}}$ (kN/m, m)

$= 21.90$ mm

$\Delta_{permissible} = \frac{l}{240}$ (for elastic loading) Table 6

$= \frac{7.15}{240}$

$= 29.8$ mm

$\Delta_{max} < \Delta_{permissible}$

ok

(Q) Find a design wind pressure on a sloping roof of span 4m and pitch 1:4. The heights of the eaves is 6m above the ground and the building is situated in Madras with low permeability.

→ (k_1, k_2, k_3 → not given so assume suitable condition)

Soln:- Use IS875: Part3 (1987)

→ ① . Let's assume building design life as 50 years, then risk coefficient, $k_1 = 1.0$

k_1, k_2, k_3

[Clause 5.3.1]
[Table 1]

. Let's take terrain category 2 & class A for 10m high, $k_2 = 1.0$
[Table 2]
[Cl. 5.3.2.2.]

. Let's assume upwind slope, $\theta < 3°$, topography factor, $k_3 = 1.0$
[Cl. 5.3.3.1.]

② Basic wind speed, $V_b = 50 m/s$ [Appendix A for Madras]

③ Design wind speed, $V_z = k_1 \cdot k_2 \cdot k_3 \cdot V_b$
(Cl. 5.3.)

$= 1.0 * 1.0 * 1.0 * 50$

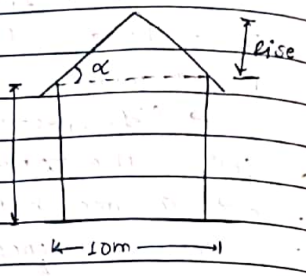
$= 50 m/s$

④ Design wind pressure, $P_z = 0.6 V_z^2 = 0.6 * 50^2$
[Cl. 5.4.]
$= 1500 N/m^2$

② External and Internal pressure coefficients

• pitch = $1/4$ = $\frac{\text{Rise}}{\text{span (whole span)}}$

=> Rise = $\frac{10}{4}$ = 2.5m



• Sloping roof angle,

$\alpha = \tan^{-1}\left(\frac{2.5}{5}\right)$

$= 26.56°$

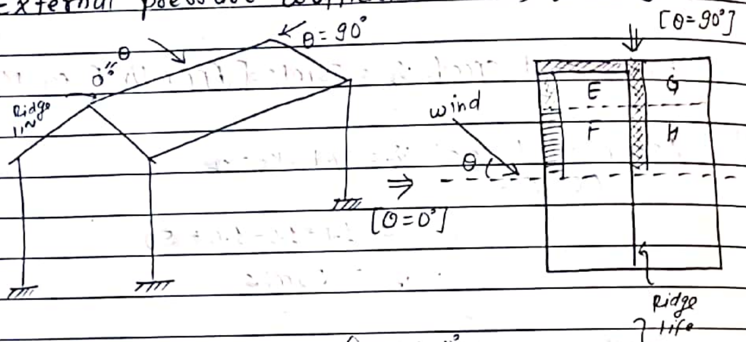
normal permeability — less opening (gen. < 5%)

• For low permeability, internal pressure coefficient,

$C_{p_i} = \pm 0.2$

[Cl. 6.2.3.1]

• External pressure coefficient : (C_{pe}) [Table 5]



Building Height ratio:

$\frac{h}{w} = \frac{6}{10} = 0.6$

$\frac{1}{2} < \frac{h}{w} = 0.6 \leq \frac{3}{2}$

{ For θ = 0°,
EF → windward slope
GH → leeward slope }

{ For θ = 90°
EG = windward
FH = leeward }

ⓐ Wind normal to ridge (wind angle, θ = 0°)

$\frac{h}{w} = \frac{6}{10} = 0.6 \quad \left\{ \frac{1}{2} < \frac{h}{w} \leq \frac{3}{2} \right\}$

Roof angle, α = 26.56°

α	EF (windward)	GH (leeward)
20°	−0.7	−0.5
30°	−0.2	−0.5
[26.56°	−0.372	−0.5]

ⓑ Wind ~~nor~~ parallel to ridge (wind angle, θ = 90°)

α	EG (windward)	FH (leeward)
20°	−0.8	−0.6
30°	−0.8	−0.8
[26.56°	−0.8	−0.731]

Pressure

* wind normal to ridge

⇒ Wind ward slope,

Wind pressure, $P_1 = (C_{pe} - C_{pi}) * P_z$ $\{ \text{i.e. } P = \frac{F}{A} \}$

[Cl. 6.2.1] $= [-0.372 \pm 0.2) * 1500$

$= -858 \text{ N/m}^2$ {Taking −ve]

{for safe design to take max.}

Leeward slope, $P_2 = (-0.5 \pm 0.2) * 1500$

$= -1050 \text{ N/m}^2$ [Taking −ve]

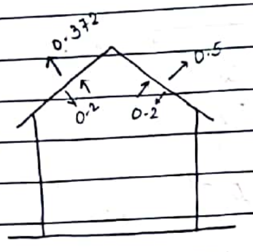
{ All $C_{pe} > C_{pi}$ → uplift pr...
If $C_{pe} < C_{pi}$ → then downshift pressure)
↳ by taking +ve C_{pi} find downshift press)

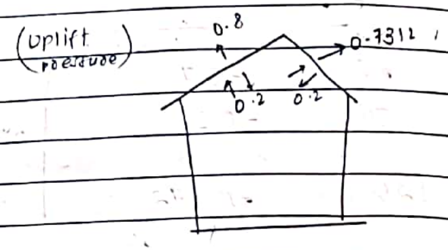
Pressure

* wind parallel to ridge

windward slope: $P_3 = (-0.8 \pm 0.2) * 1500$
$= -1500\ N/m^2$

leeward slope: $P_4 = (-0.731 \pm 0.2) * 1500$
$= -1396.5\ N/m^2$





Design wind pressure

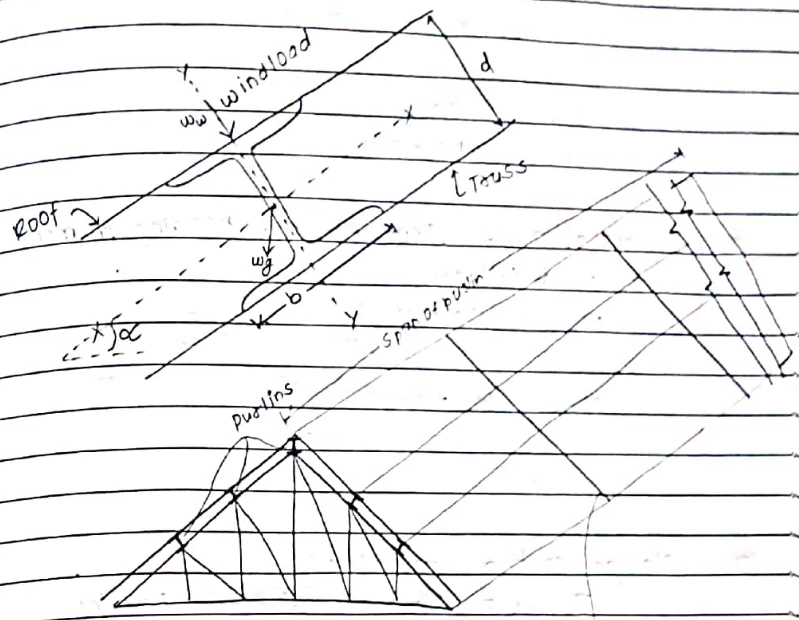
Uplift pressure = Maximum value of P for $\theta = 0°$ & $90°$
$= -1500\ N/m^2$

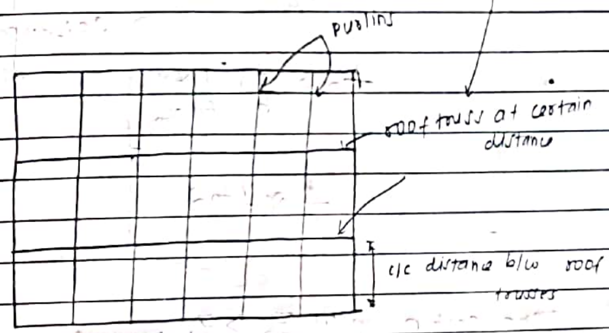
Downshift pressure = All values of P obtained are negative
(when $C_{pe} < C_{pi}$)

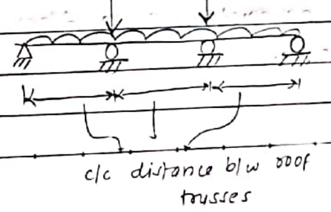
{ If medium or high permeability then downshift pressure may exist}

[Design force = Design wind Pressure * Cross-sectional area of member].
← [Cl. 6.2.1].

Design of Purlin (using design force)







Page No.
Date

- Design steps:

(I) ⇒ W_g = Dead load on purlins
W_w = wind load
α = sloping roof's angle

$W_x = W_g \sin\alpha$
$W_y = W_w + W_g \cos\alpha$

(II) Determination of shear force & bending moment about x-x & y-y axis.

$$M_{x-x} = (W_g \sin\alpha)\frac{l^2}{10}$$

$$M_{y-y} = (W_w + W_g \cos\alpha)\frac{l^2}{10}$$

For continuous support

(III) Plastic section modulus:

$$Z_{p\,reqd.} = \frac{M_{x-x}}{(f_y/\gamma_{m_0})} \quad \{\text{Biaxial Bending}\}$$

$$+ 2.5\left(\frac{d}{b}\right)\frac{M_{y-y}}{f_y/\gamma_{m_0}}$$

{Bi-directional Moment}

$\frac{d}{b} \approx 1$ to 2

(IV) Take section with $Z_p > Z_{p\text{-required}}$

(V) (a) Check for all parameters as in flexural member or beam design.

Page No
Date

(V) Md (Design moment check)
- Laterally supported
- Laterally unsupported

(VI) Shear-design check

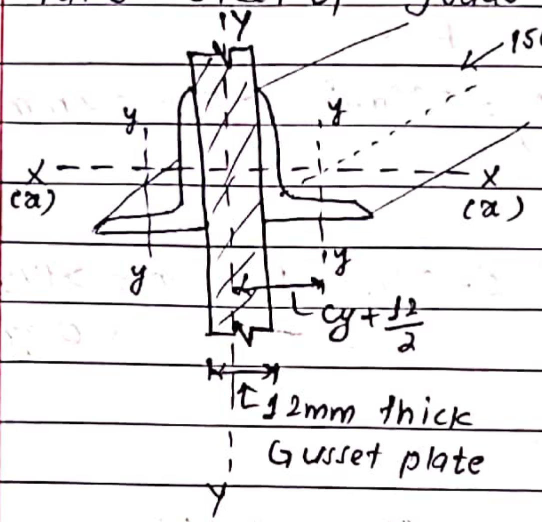
(VII) Deflection check (limit)

(VIII) Buckling & crippling (web)

Cl. 7.5.2

(Q) Design a double angle discontinuous strut to carry a factored load of 150kN. The length of strut is 3m between the intersection. The two angles are placed on back to back on 12mm thick Gusset plate. Take steel of grade E250.

opposite ~~on two sides of gusset plate~~



Y-Y; X-X → composite section

y-y; x-x → single member

* CG of double angle section shifts :
  ↳ so modification for radius of Gyration required (centroidal axis changes)
  (For Rad. of Gyration ∥ axis theorem may be used)
  X-X → same as for single angle section

(I) Assume slenderness-ratio = 100 < 180 (Table 3)

(Assume SR)

$f_y = 250 N/mm^2$.

For angle-section member ⇒ Buckling class 'c' (Table 10)

From table 9(c),

Design compressive stress, $f_{cd}^{*} = 107 N/mm^2$. (* first trial)

Approximate area (for first trial *) $= A_e = \frac{P_d}{f_{cd}}$

$= \frac{150 \times 1000}{107}$

$= 1401.87 mm^2$.

For 1-single angle section

$$A_e = \frac{1401.8}{2} = 700.54\ mm^2.$$

From IS 808: 1989,

Assume angle section of $60 \times 60 \times 8$ mm

which has,

$$A_g = 896 mm^2. \quad (> 700.54\ mm^2)$$

for no complication for $r_x = r_y$ → equal ang.

Thus,

Provided area, $A = 2 \times 896 = 1792\ mm^2 > 1401.8\ mm^2$ (okay)

Now, (components)

Radius of Gyration

$r_{x-x} = 18 mm$ (From table)

(r_{X-X})

$I_{y-y} = 29 \times 10^4\ mm^4$ $\{c_x = c_y = 17.7 mm\}$

$$I_{y-y} = \left\{29 \times 10^4 + 896 \times \overset{12/2 + 17.7}{\left(\cancel{6} - \cancel{17.7}\right)}^2\right\} \times 2$$

i.e. $\left\{I_{y-y} + A_s \times \left(C_y + \frac{12}{2}\right)^2\right\} \times 2$

gauset plate thickness

individual member

$$= 79.33 \times 10^4\ mm^4 \times 2$$

$$= 158.65 \times 10^4\ mm^4$$

Now,

$$r_{y-y} = \sqrt{\frac{I_{y-y}}{A}} = \sqrt{\frac{158.65 \times 10^4}{1792}} = 29.76 mm$$

(for composite)

comparing r_{x-x} & r_{y-y}

Minimum, $r = \cancel{29.76 mm}\ 18 mm$

slenderness ratio
(actual)

(II) From Cl. 7.5.2.1.

For effective length KL,

$KL = (0.7 - 0.85) * \text{distance between intersection}$

(No support condition given so to be on safer side assume, $K = 0.85$)

∴ Thus,

$\boxed{KL = 0.85L}$

Now,

$$\frac{KL}{r_{min}} = \frac{0.85 \times 3000}{18} = 141.67 < 180 \text{ (OK)}$$

which is okay as per Table 3.

Again,

Table 9(c), buckling class ©, f_{cd}

f_y (250

Thus,

By interpolation,

$f_{cd} = 65.03 \text{ N/mm}^2$.

$$\left.\begin{array}{l} 140 \rightarrow 66.2 \\ 150 \rightarrow 59.2 \end{array}\right\}$$

$\uparrow \frac{KL}{r}$ $\quad \uparrow f_{cd}$

(III) Design compressive strength, $P_d = f_{cd} * A_e = 65.03 * 1792$

$= 116.5 \text{ kN}$

$(< 150 \text{ kN})$

unsafe.

(?) r_{min} बढाउन पर्छ so that SR ↓ & f_{cd} ↑.

Date
Page

- Revise section using 70*70*8 mm,

$A_g = 1060\ mm^2$

Provided area, $A = 2 * 1060 = 2120\ mm^2$

$r_{x-x} = r_{xx} = \cancel{21.4}\ 21.2\ mm$

$r_{yy} = \cancel{20.2}\ 21.2\ mm$ $C_x = 20.2\ mm$.

$I_{yy} = 47.4 \times 10^4\ mm^4$

IS 808: 1989

$I_{y-y} = [47.4 \times 10^4 + 1060 * (20.2 + \frac{12}{2})^2] \times 2$

1 ∡

$= 2403252.8\ mm^4$

$= 240.33 \times 10^4\ mm^4$ ← (for 2 ∡)

Thus,

$r_{y-y} = \sqrt{\frac{I_{y-y}}{A}} = \sqrt{\frac{240.33 \times 10^4}{2120}} = 33.67\ mm$

(2 ∡)

Adopting minimum,

$r_{min} = 21.2\ mm.$

From Cl. 7.5.2.1.

Effective length, $KL = 0.85 \times L$
$= 0.85 \times 3000$
$= 2550\ mm$

Thus,

$SR = \frac{KL}{r_{min}} = \frac{2550}{21.2} = 120.28 < 180$ (OK)

From table 9(c)

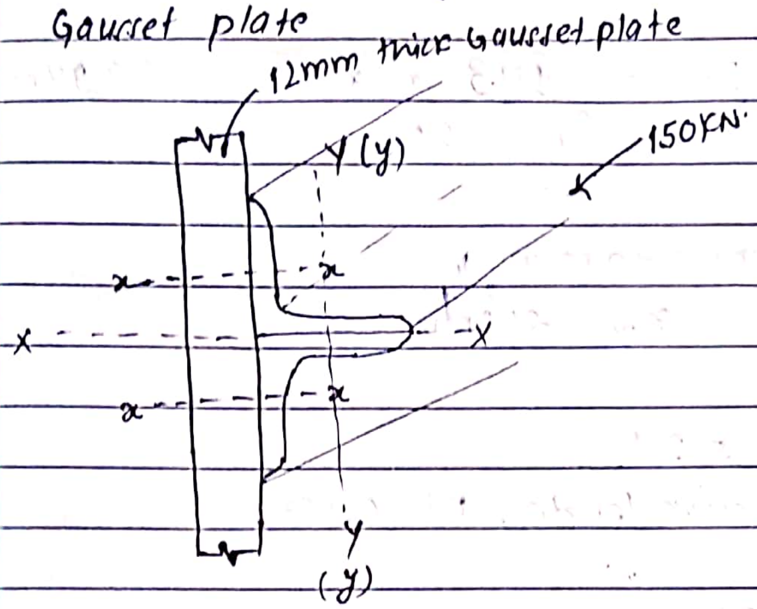
$f_{cd} = \cancel{72.78\ N/mm^2}$
$83.45\ N/mm^2$

120 → 83.7
130 → $\cancel{74}$ 74.3 } 120.28

Design compressive strength, $P_d = f_{cd} \times A$
$= 83.45 * 2120$
$= 176.91\ kN > 150\ kN$ (safe).

Thus, adobt 70 × 70 × 8 angle section on both sides of Gusset plate.

(9) Prev. qsn. but Angle sections on same side of Gausset plate



Assume slenderness ration, $\lambda = 100 < 180$ (table 3)

From table 9(c)

$$f_{cd}{}^{*} = 107 \text{ N/mm}^2.$$

Approximate area, $A_e = \dfrac{P_d}{f_{cd}{}^{*}} = \dfrac{150 \times 1000}{107}$

$$= 1401.87 \text{mm}^2$$

For single angle section,

$$A_e = \frac{1401.87}{2} = 700.54 \text{ mm}^2$$

From IS 808 : 1989

Assume angle section of 70 × 70 × 8 mm.
(safer one as per previous)

$$A_g = 1060 \text{mm}^2$$

Provided area ; $A = 2 \times 1060 = 2120 \text{ mm}^2$

$r_{yy} = r_{xx} = 21.2 \text{mm}$

$I_{xx} = I_{yy} = 47.4 \times 10^4 \text{ mm}^4$

$I_{xx} = 2 * [I_{x-x} + A h^2]$, $h = c_x$

Date ______
Page ______

$r_{yy} = 21.2mm$ $c_x = 20.2mm$

$I_{xx} = [47.4 * 10^4 + 1060 * (20.2 - 0)^2] * 2$
$= \cancel{190.08 * 10^4 mm^4}$ $181.30 * 10^4 mm^4$

$r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{181.30 * 10^4}{2120}} = 29.24mm$

Adopting minimum,
$r_{yy} = 21.2mm$

From Cl. 7.5.2.1.
Effective length, $KL = 0.85 * L$

$SR = \frac{KL}{r_{min}} = \frac{0.85 * 3000}{21.2} = 120.28 < 180$
(Table 3)
ok

From table 9(c)
$f_{cd} = 83.45 N/mm^2$
$120 \rightarrow 83.7$
$130 \rightarrow 74.3$

Design compressive strength, $P_d = f_{cd} * A$
$= 83.45 * 2120$
$= 176.91kN > 150kN$
(safe)

SURENDRA SHARMA