LECTURE NOTES

ON

DESIGN AND DRAWING OF STEEL STRUCTURES
ACADEMIC YEAR 2023-24

III B.Tech –II SEMESTER (R20)

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DESIGN AND DRAWING OF STEEL STRUCTURES

UNIT – I

Connections: Riveted connections – definition, rivet strength and capacity, Welded connections: Introduction, Advantages and disadvantages of welding-Strength of welds-Butt and fillet welds: Permissible stresses – IS Code requirements. Design of fillet weld subjected to moment acting in the plane and at right angles to the plane of the joints.

UNIT – II

Beams: Allowable stresses, design requirements as per IS Code-Design of simple and compound beams-Curtailment of flange plates, Beam to beam connection, check for deflection, shear, buckling, check for bearing, laterally unsupported beams.

UNIT –III

Tension Members and compression members: General Design of members subjected to direct tension and bending –effective length of columns. Slenderness ratio – permissible stresses. Design of compression members, struts etc.**Roof Trusses:** Different types of trusses – Design loads – Load combinations as per IS Code recommendations, structural details –Design of simple roof trusses involving the design of purlins, members and joints – tubular trusses.

UNIT – IV

Design of Columns: Built up compression members – Design of lacings and battens. Design Principles of Eccentrically loaded columns, Splicing of columns.

UNIT – V

Design of Column Foundations: Design of slab base and gusseted base. Column bases subjected moment.

UNIT – VI

Design of Plate Girder: Design consideration – I S Code recommendations Design of plate girder-Welded – Curtailment of flange plates, stiffeners – splicing and connections.

Design of Gantry Girder: impact factors - longitudinal forces, Design of Gantry girders.

Introduction

Steel structures :-

The structures which are constructed with structural steel are called as steel structures.

Advantages :

1. Better quality control when compared to R.C.C structures.
2. Different components in steel structures are fastened together by simple connecting techniques such as welding, bolting and riveting.
3. Pre-fabricated steel structures results in proper planning of construction, saving of time result in speed erection & economic of structure.
4. Steel structural members can readily dis-assembled at the end of the useful life which results in environmental & economical advantages.
5. The scrap value of steel structure is high.
6. Repairing of retrofitting of steel structures are very simple.

Disadvantages :-

1. steel rusts easily & thus requires a protection from Corrosion this increases the maintanence Cost of steel structures.

2. steal beam is an excellent heat Conductor. steel structures are to be protected by insulating materials thus increases fire proofing Cost.

[Fire proofing material – Apexi painting]

3. Steel structures requires skill personal and very high accuracy is needed desired in fabrication.

4. If the Compression members of steel structures are longer & slender may have Suspendibility to buckled.

Loads :-

1. Dead load.
2. Imposed load (or) live load.
3. Wind load.
4. Snow load.
5. Special loads &
6. Load Combination.

Structural members:-

It is nothing but forces acting on the steel structures.

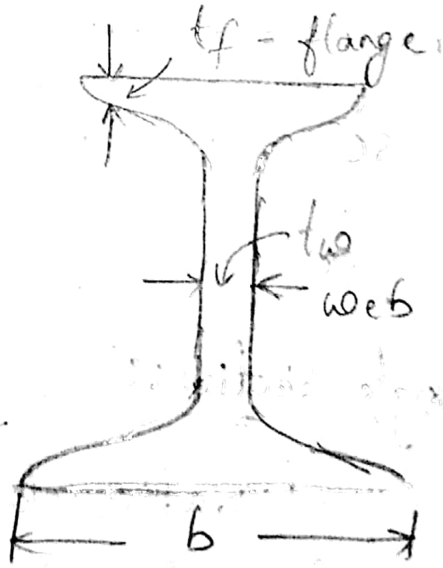
1. Flexural members.
2. Tension member.
3. Compression members.
4. Torsional members.

Standard structural Sections:-

1. I - Section.
2. Channel Section.
3. Angel Section.
4. T-Section.
5. Flats.

I -Section:

$$t_f = \frac{b - t_w}{4}$$



where,

$t_f \rightarrow$ thickness of flange.

$t_w \rightarrow$ thickness of web.

ISJB → Indian standard Junior Beam.

ISLB → " " Lite beam

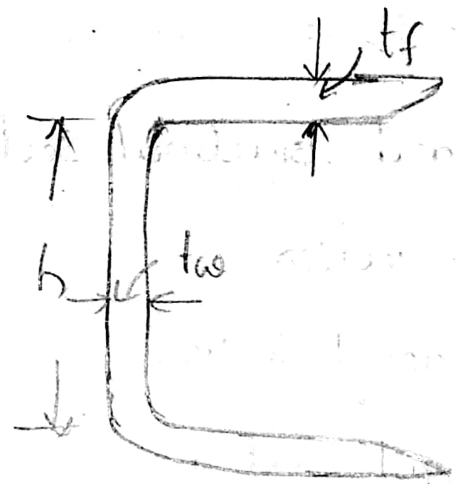
ISWB → " " wide "

ISMB → " " medium "

ISHB → " " Heavy "

Channel Section:

$$t_f = \frac{b - t_w}{2}$$



ISJC → Indian standard Junior channel.

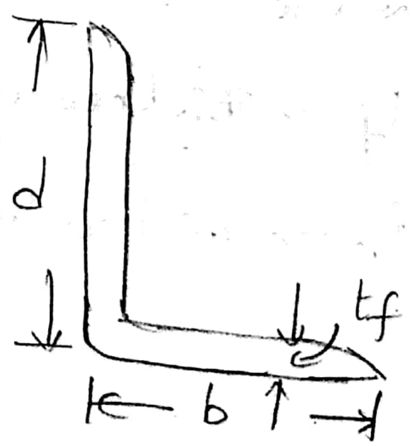
ISMC → " " medium "

ISLC → " " Lite "

ISSC → " " special "

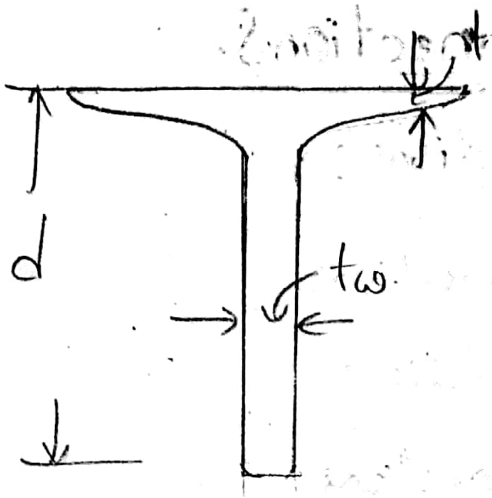
ISMPC → " " Parallel flange "

Angle Sections:-



1. Equal Indian standard Angle [ISA]
2. Unequal ISA

T-sections:



ISJT → Indian standard Junior T-section.

ISNT → " " Normal "

ISHT → " " High " (or)

wide flange T-section

ISSP → " " short-legged "

ISLT → " " long-legged "

Flats:-

ISRO → Indian standard round bar.

ISSQ → " " Square "

UNIT-I

Connections.

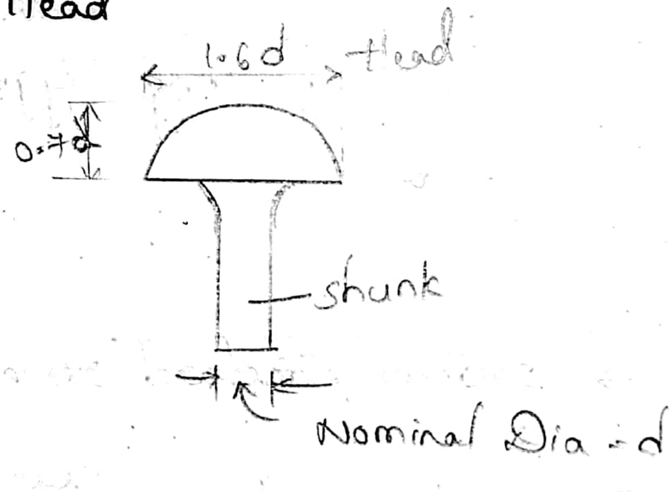
Types of Connections :-

1. Riveted Connections.
2. Bolted Connections.
3. Welded Connections.
4. pinned Connections &
5. Combinations.

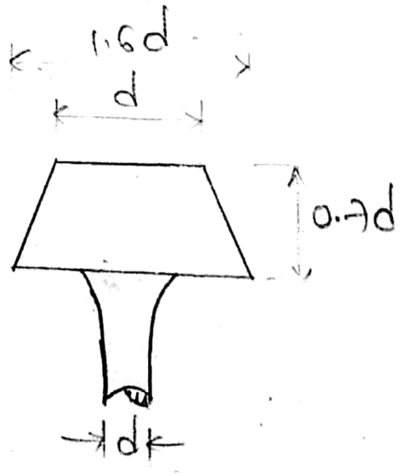
Riveted Connections:-

Types of rivets:

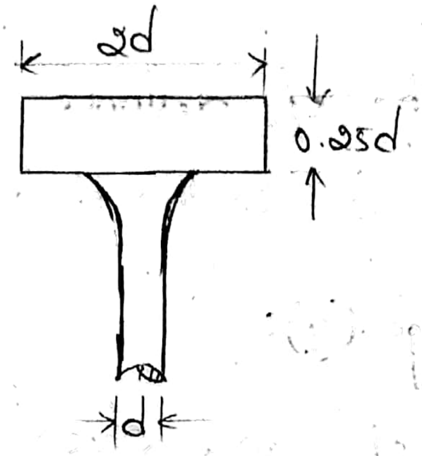
1. Span Head



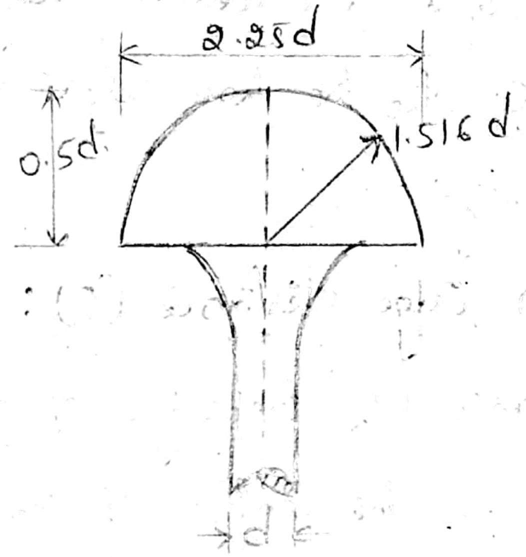
2. pan Head.



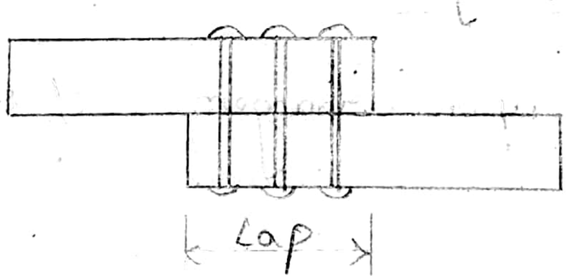
3. Flat head.

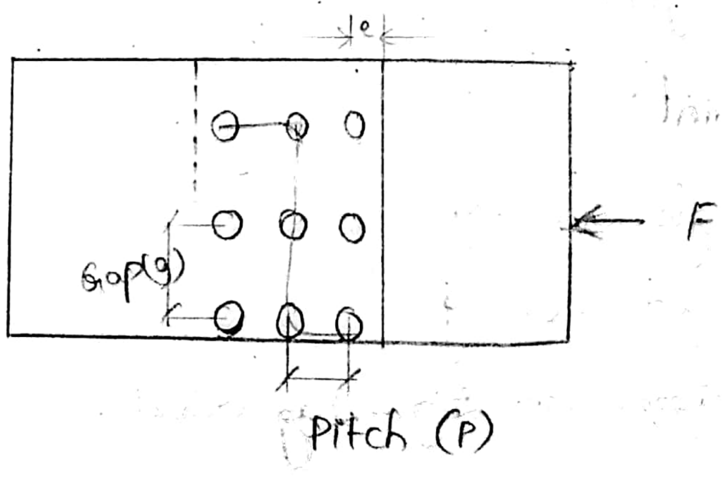


4. Mushroom head.



Terminology :-





Pitch (P):

It is the c/c distance of adjacent rivets (or) bolts measures in the direction of stress

Gap (or) gauge (g):

A row of rivets which is parallel to the direction of stress is called a gauge line. The normal distance b/w the two adjacent gauge lines is called as gauge.

Eccentricity or Edge distance (E):

The distance b/w the edge of a member (or) cover plate from the centre of the nearest rivet bolt is edge distance.

Types of rivet joints:-

(i) Depending upon arrangement of rivets & plates.

1. Lap joint
2. Butt joint.

Lap joint:

a). Single riveting
b). chain riveting.
c). staggered (or) zig-zag rivet.

Buff joint:

a). Single riveting.

b). chain riveting.

c). zig-zag rivet

(ii) Depending upon the mode of load transmission

1. Single shear.
2. Double shear.
3. Multiple shear.
4. Bearing shear.

(iii) Depending upon nature & location of load

1. Direct shear connection.
2. Eccentric connection.
3. pure moment shear connection.
4. Moment shear connection

Welded Connections :-

It is the process of connecting metal pieces by application of heat with (or) without pressure.

Welding process types:

1. Gas welding [The edges are to be joint (or)

welted on oxyactelene gas flame].

2. Froge welding.

3. Thermit welding.

4. Electric Arc welding.

Advantages:-

1. As no hole is require for welding.

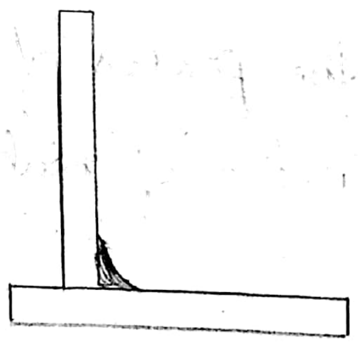
2. No reduction of area.

3. So structural member can easily take the loads.

4. In welding fillet plate & gussted plates aren't used.

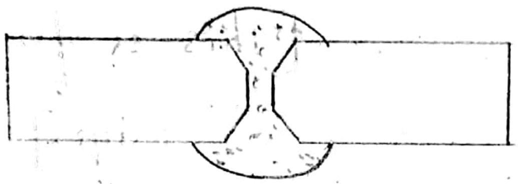
Types of welds:-

1. Fillet weld:

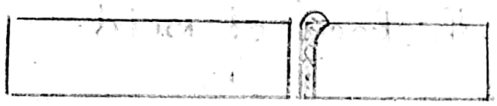


2. Single beam butt joint:

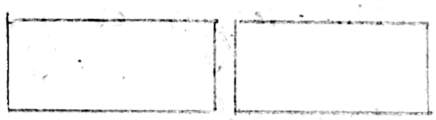
3. Double beam butt joint:



4. Single 'J' - Butt joint:



5. Square butt joint:



Where,

t = throat thickness

S = Size of weld

$t = 0.7S$

*

Thickness of thicker plate	Minimum Size
upto 6mm	3mm
6 to 12mm	4mm
12 to 18mm	6mm
18 to 36mm	8mm

36 to 56mm	10mm
56 to 150mm	12mm
above 150mm	16mm

(i) Max. Size of weld = $t - 1.5$

[This is for only square edges]

(ii) Remaining max. Size of weld = $\frac{3}{4} t$

where, t = thickness of weld.

(iii) Effective length (l_e) = Overall length $- 2S$

→ Minimum effective length is greater than '4' times of size of weld

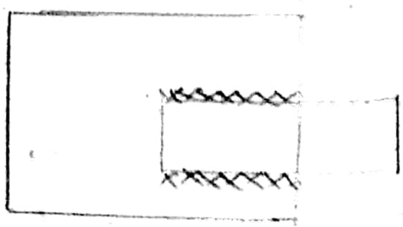
(iv) ∴ $l_{min} > 4S$

(v) Lap length (l) $> 5t$

where t = thickness of thinner plate.

(vi) End returns $> 2S$

where s = size of weld



Design strength in fillet weld:-

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

for,

$$\left[\text{fillet weld } (f_{wd}) = \frac{f_u/\sqrt{3}}{\gamma_{mw}}\right]$$

for,

$$\text{Butt weld } (f_{wd}) = \frac{f_y}{\gamma_{mw}}$$

where,

$f_{wn} = f_u/\sqrt{3}$

$\gamma_{mw} = 1.25$ [for ~~soap~~ shop weld]

$\gamma_{mw} = 1.5$ [for field purpose].

$$\boxed{P_w = f_{wd} \times A_w}$$

$f_u = 410$

$f_y = 250$

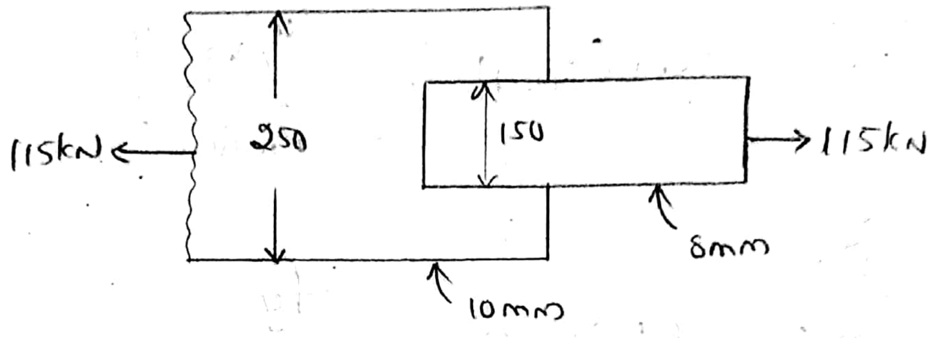
where,

P_w = Design strength in fillet weld

$A_w = l_w \times t$

t = throat thickness

1. Design a suitable fillet weld to connect the two plates as shown in fig.



Sol

Min. Size of weld $S_{min} = 4mm$

Max. Size of weld $= t - 1.5$

$= 8 - 1.5$

$= 6.5mm \simeq 6mm$

$\therefore$ Size of weld $S = 6mm$

$t = 0.7S$

$= 0.7 \times 6$

$\therefore \boxed{t = 4.2mm}$

Design strength:

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = f_u / \sqrt{3}$$

$$= \frac{410}{\sqrt{3}}$$

$$f_{wn} = 236.7 \ N/mm^2$$

$$\therefore f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$= \frac{236.7}{1.5}$$

$$\therefore \boxed{f_{wd} = 157.81 \ N/mm^2}$$

$$P_w = f_{wd} \times A_w$$

$$115 \times 10^3 = 157.81 A_w$$

$$157.81 A_w = 115 \times 10^3$$

$$A_w = \frac{115 \times 10^3}{157.81}$$

$$\boxed{A_w = 728.72 \ mm^2}$$

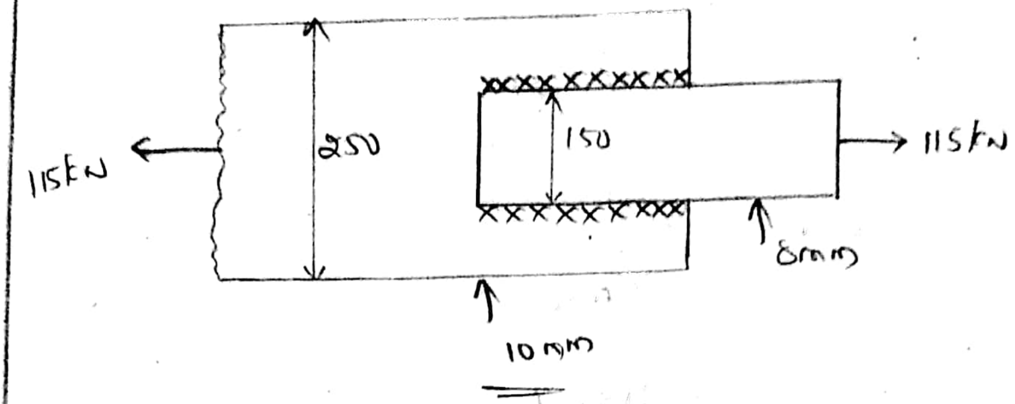
where,

$$A_w = l_w \times t$$

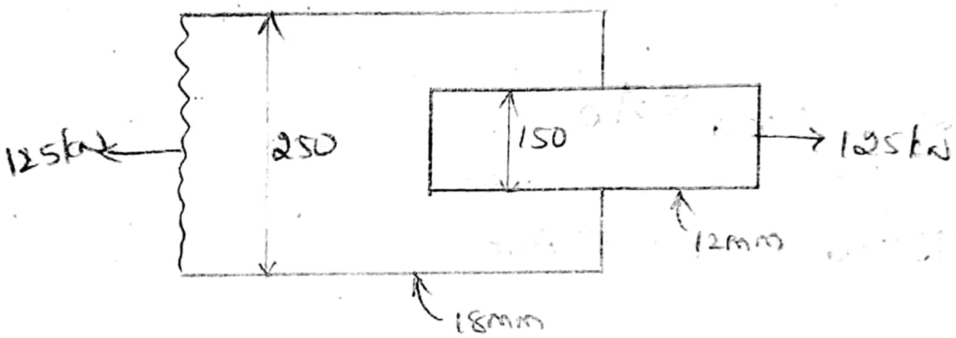
$$t = 4.2$$

$$728.72 = 4.2 l_w$$

$$\therefore \boxed{l_w = 173.5 mm}$$



(2)



Sol

Min. Size of weld = 6mm

Max. Size of weld = $t - 1.5$

$= 12 - 1.5$

$= 10.5\text{mm} \simeq 10\text{mm}$

$\therefore$ Size of weld $S = 10\text{mm}$

$t = 0.7S$

$= 0.7 \times 10$

$\therefore$ $\boxed{t = 7\text{mm}}$

Design Strength:

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = f_u/\sqrt{3}$$

$$= \frac{410}{\sqrt{3}}$$

$$\therefore \boxed{f_{wn} = 236.7\, N/mm^2}$$

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$= \frac{236.7}{1.5}$$

$$\therefore \boxed{f_{wd} = 157.81\, N/mm^2}$$

$$P_w = f_{wd} \times A_w$$

$$125 \times 10^3 = 157.81 \cdot A_w$$

$$A_w = \frac{125 \times 10^3}{157.81}$$

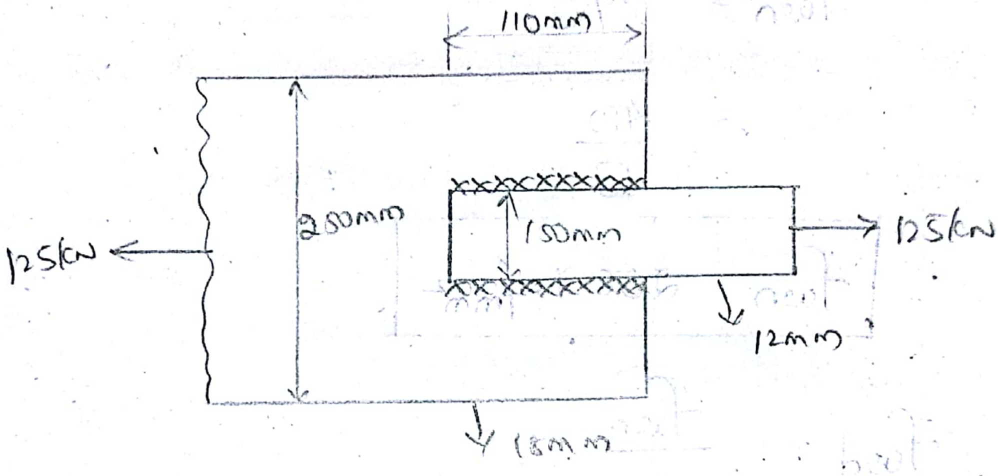
$$\boxed{A_w = 792.1\, mm^2}$$

where,

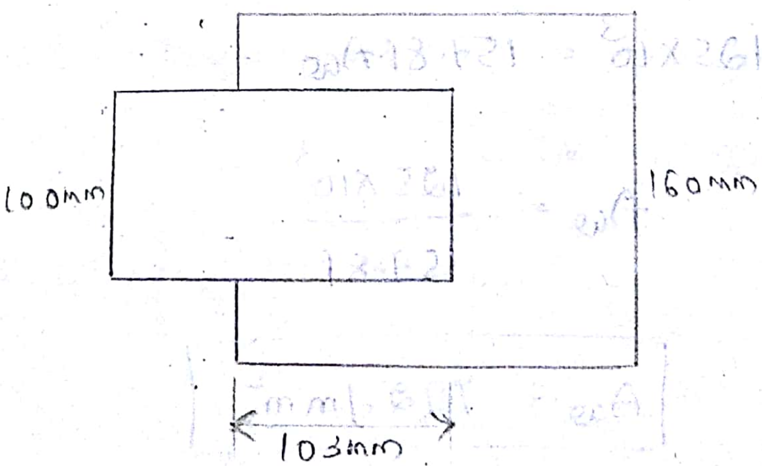
$$A_w = l_w \times t$$

$$l_w = \frac{792.1}{7}$$

$$\therefore \boxed{l_w = 113.15mm} \simeq 110mm$$



3. Design a suitable longitudinal fillet weld to connect the plates as shown in fig. to transmit a pull equal to the full strength of small plate. plates are 12mm thickness. Assume shop welding.



Sol

Thickness of weld = 12mm

Max. Size of weld = $t - 1.5$

$= 12 - 1.5$

$= 10.5mm \simeq 10mm$

$\boxed{S = 10mm}$

Min. Size of weld = 4mm [Assume]

Throat thickness $t = 0.7S$

$= 0.7 \times 10$

$= 7mm$

$\boxed{t = 7mm}$

Small plate Area $= 100 \times 12$

$A = 1200 mm^2$

Small plate strength,

$\Rightarrow A.f_y / \gamma_{mo}$

where,

$\gamma_{mo} = 1.1$

$f_y = 250$

$\therefore$ Small plate strength $= \frac{1200 \times 250}{1.1}$

$= 272.72 \times 10^3 N$

$= 272.72 kN$

$\boxed{P = 272.72 kN}$

<u>Design strength</u> :-

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = \frac{f_u}{\sqrt{3}}$$

$$= \frac{410}{\sqrt{3}}$$

$$\therefore \boxed{f_{wn} = 236.7 \, N/mm^2}$$

$$f_{wd} = \frac{236.7}{1.25}$$

$$\boxed{f_{wd} = 189.36 \, N/mm^2}$$

$$P_w = f_{wd} \times A_w$$

$$272.72 \times 10^3 = 189.36 A_w$$

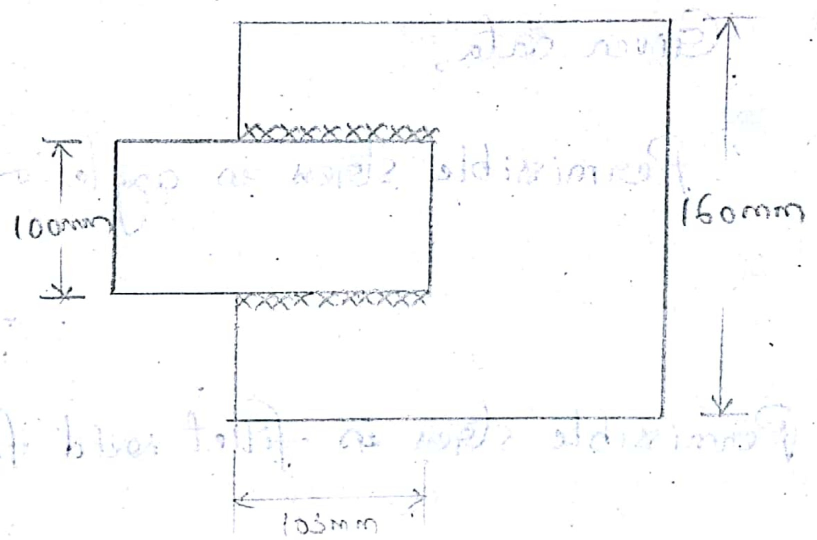
$$\boxed{A_w = 1440.22 mm^2}$$

where,

$$A_w = l_w \times t$$

$$l_w = \frac{1440.22}{7}$$

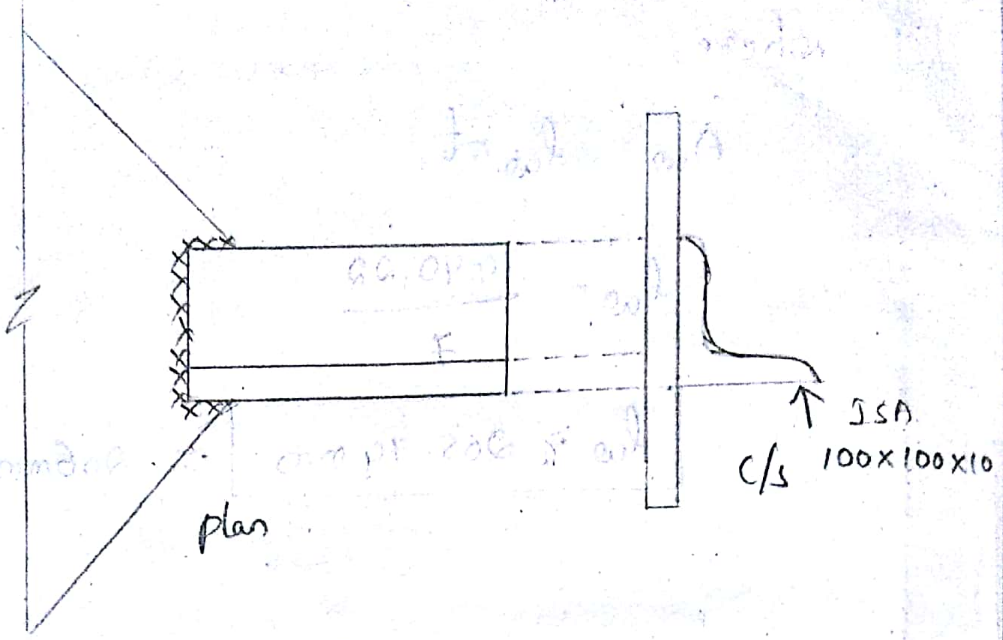
$$\boxed{l_w = 205.74\,mm} \simeq 206mm$$



$\therefore$ length of weld on each side $= \frac{206}{2}$

$= 103mm$

4. Design the welded joint as shown in below. Assuming permissible stresses in angles and fillet welds those are 150MPa & 108MPa.



Sol

Given data,

Permissible stress in angle $\sigma = \left(f_y / \gamma_{mo}\right)$

$= 150 N/mm^2$

Permissible stress in fillet weld $f_{wd} = 108 N/mm^2$

Min. Size of weld = 4mm

Max. Size of weld (Angle) = $\cancel{t-1.5}$ $\frac{3}{4} t$

$= \frac{3}{4} \times 10$

$= 7.5mm \simeq 7mm$

$\boxed{S = 7mm}$

Max. Size of weld (plate) = $t - 1.5$

$= 10 - 1.5$

$= 8.5mm \simeq 8mm$

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$\boxed{S = 8mm}$

150 MPa

$150 \times \frac{10^6}{10^9} \times 10^3 \, N/mm^2$

$= 150 \, N/mm^2$

Throat thickness = 0.7S

$= 0.7(7)$. [Consider Angle max. weld]

$= 4.9mm$

$\boxed{t = 4.9mm}$

Area of angle = 19.03 Cm^2

$= 1903 mm^2$ [from steel tables]

Strength of Angle = $Ag \times \sigma$ ($mm^2 \times N/mm^2$)

$= 1903 \times 150$

$= 285.45 \times 10^3 N$

$P = 285450$ kN

load $p = A_w \cdot f_{wd}$

$285.45 \times 10^3 = 108 \times A_w$

$$A_w = \frac{285.45 \times 10^3}{108}$$

$\boxed{A_w = 2643.05 mm^2}$

where,

$A_w = l_w \times t$

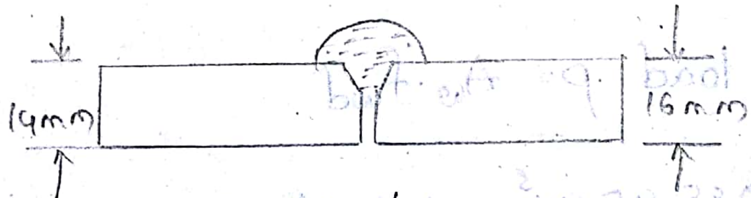
$$l_w = \frac{2643.05}{4.9}$$

$$= 539.39 mm \simeq 530 mm$$

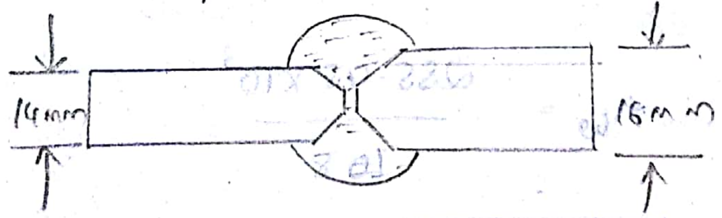
$$\boxed{l_w = 530mm}$$

5. Two plates of 16mm, 14mm thickness are to be joint by a butt weld. The joint is subjected to a factored tensile force of 430 kN. Due to some reasons effective length of weld could be provided 175mm only. check the safety of the joint. If (a) Single V-butt weld is provided (b) Double VV butt welded

a). If ~~single~~ 16mm of single 'v'



b). If 16mm of Double 'v'



Single = $\frac{5}{8}t$

Double = t

sol a) As per specification of IS throat thickness,

$$t_w = \frac{5}{8}t$$

t = thickness of thinner plate.

$t_w = \frac{5}{8} \times 14$

$= 8.75 mm$

length of weld $l_w = 175 mm$

load $p = 430 kN$

Strength of weld $= \cancel{Ag}\ A_{wd} \times f_{wd}$

$\left[f_{wd} = \frac{f_y}{\gamma_{m}} \right]$

$f_{wd} = \frac{250}{1.25}$

$P = 175 \times 8.75 \times \frac{250}{1.25}$

$= 306.25 \times 10^3 N$

$430 > 306.25$

$\therefore$ Hence ~~safe~~ unsafe.

b) As per specification of IS. throat thickness = t

$t_w = 14 mm$

length of weld $l_w = 175 mm$

load $P = 430 kN$.

Strength of weld $P = A_{wd} \times f_{wd}$

$P = 175 \times 14 \times \frac{250}{1.25}$

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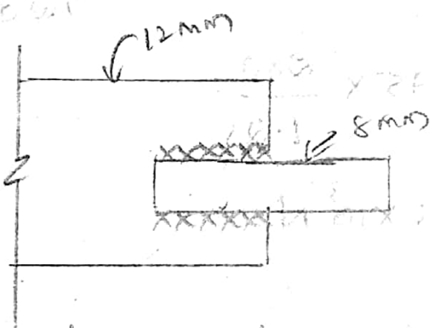
$$\boxed{P = 490\,kN}$$

$430 < 490\,kN$

$\therefore$ Hence safe.

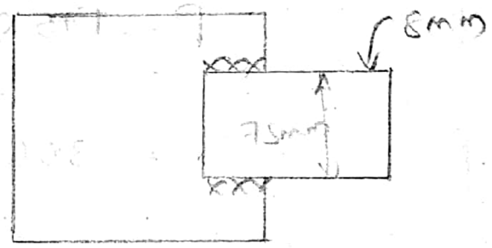
6\. A 75 x 8mm tie member is to transmit a factored load of 145kN. Design fillet weld & neccessary overlap for the cases shown in fig. Assume gusset plate to be 12mm thickness

(a)



Shop welding

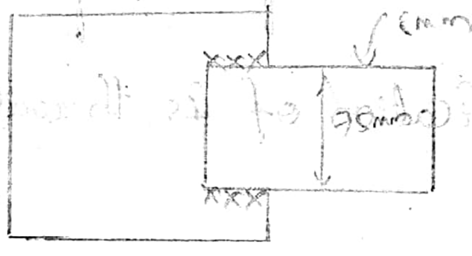
(b)



Shop welding

Overlap = 53.65
≃ 54

(c)



Field welding

sol (a) Factor of safety for shop welding,

$\gamma_{mw} = \cancel{145kN}\ 1.25$

Min. Size of weld = 4mm

Max. Size of weld = $t - 1.5$

$= \cancel{12}\ 8 - 1.5$

$= 6.5\,mm \simeq 6\,mm$

$\boxed{S = 6\,mm}$

Throat thickness $t = 0.7S$

$= 0.7 \times 6$

$= 4.2\,mm$

$\boxed{t = 4.2\,mm}$

load $P = 145\,kN$

$= 145 \times 10^3\,N$

Design strength:

$$f_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \frac{410/\sqrt{3}}{1.25}$$

$\boxed{f_{wd} = 189.37\,N/mm^2}$

$P = f_{wd} \times A_w$

$145 \times 10^3 = 189.37 A_w$

$189.37 A_w = 145 \times 10^3$

$$A_w = \frac{145 \times 10^3}{189.37}$$

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$$\boxed{A_w = 765.69 mm^2}$$

where,

$A_w = l_w \times t$

$765.69 = 4.2\, l_w$

$4.2 l_w = 765.69$

$l_w = \frac{765.69}{4.2}$

$$\boxed{l_w = 182.31 mm}$$

length of welding on each side $= \frac{182.31}{2}$

$= 91.15 \simeq 92$

$= 92mm$

<u>End returns:</u>

End returns $= 2S$

$= 2 \times 6$

$= 12mm$

Overall length of weld $= 92+92+12+12$

$= 208mm$

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(b) $2 \times \text{Overlap} + 75 = \text{length of plate}$

$$2 \times \text{Overlap} + 75 = 182.31$$

$$2 \times \text{Overlap} = 182.31 - 75$$

$$= 107.31$$

$$\text{Overlap} = \frac{107.31}{2}$$

$$= 53.65\,mm \simeq 54\,mm$$

$$\boxed{\text{Overlap} = 54\,mm}$$

End returns $= 2S$

$$= 2 \times 6$$

$$= 12\,mm$$

Overall length of weld $= 54 + 54 + 12 + 12 + 75$

$$= \cancel{132\,mm}\ 207\,mm$$

(c) Factor of Safety for field welding,

$$\gamma_{mw} = 1.5$$

Min. Size of weld $= 4\,mm$

Max. Size of weld $= t - 1.5$

$$= 8 - 1.5$$

$$= 6.5\,mm \simeq 6\,mm$$

$$\boxed{S = 6\,mm}$$

Throat thickness = 0.7S

$= 0.7 \times 6$

$= 4.2mm$

$\boxed{t = 4.2mm}$

load $P = 145 \times 10^3 N$

Design strength:

$$f_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \frac{410/\sqrt{3}}{1.5}$$

$$\boxed{f_{wd} = 157.81 N/mm^2}$$

$P = f_{wd} \times A_w$

$145 \times 10^3 = 157.81 \times l_w \times 4.2$

$662.79 l_w = 145 \times 10^3$

$$\boxed{l_w = 218.76mm}$$

length of welding on each side $= \frac{218.76}{2}$

$= 109.38mm \simeq$

$\simeq 110mm$

End returns = 2S

= 2 × 6

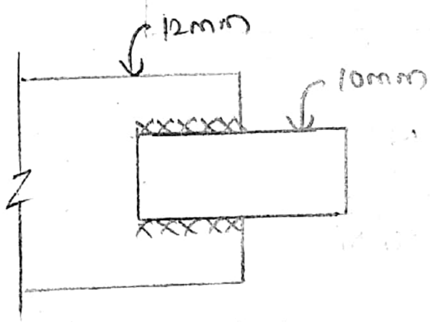
= 12 mm

Overall length of weld = 110 + 110 + 12 + 12

= 244 mm

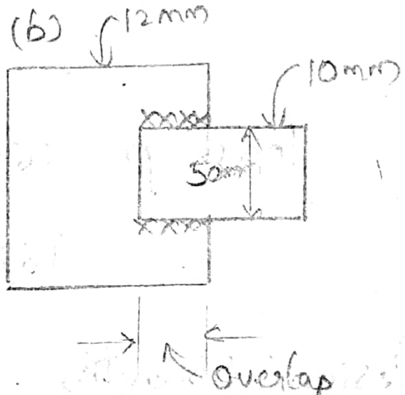
7. A 50 × 10 mm tie member is to transmit a factored load of 162 kN. Design fillet weld & necessary overlap for the cases shown in fig. Assume gusset plate to be 12mm thickness.

(a)



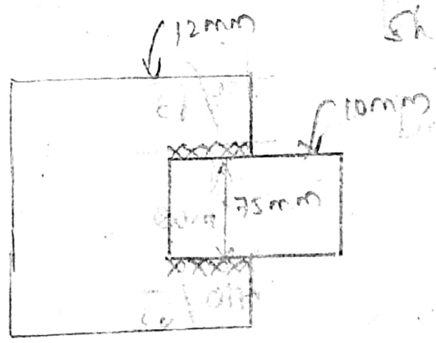
shop weld

(b)



shop weld.

(c)



field weld.

Sol (a) Factor of safety for shop weld,

$\gamma_{mw} = 1.25$

Min. Size of weld = 4mm

Max. Size of weld = $t - 1.5$

$= 10 - 1.5$

$= 8.5mm \simeq 8mm$

$\boxed{S = 8mm}$

Throat thickness $t = 0.7S$

$= 0.7 \times 8$

$\boxed{t = 5.6mm}$

load $P = 162kN$

$= 162 \times 10^3 N$

Design strength,

$$f_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \frac{410/\sqrt{3}}{1.25}$$

$$\therefore \boxed{f_{wd} = 189.37 \, N/mm^2}$$

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$P = f_{wd} \times A_w$

$162 \times 10^3 = 189.37 \times l_w \times 5.6$

$1060.472 l_w = 162 \times 10^3$

$l_w = 152.76 mm$

length of welding on each side $= \frac{152.76}{2}$

$= 76.38 mm \simeq$

$\simeq 77mm$

End returns $= 2S$

$= 2 \times 8$

$= 16 mm$

Overall length of weld $= 77 + 77 + 16 + 16$

$= 186 mm$

(b) $2 \times$ Overlap $+ 50 =$ length of weld

$2 \times$ Overlap $+ 50 = 152.76$

$2 \times$ Overlap $= \cancel{77-50}\ 152.76 - 50$

$= 102.76 mm$

Overlap $= \cancel{12.5 mm}$

$= \frac{102.76}{2} \cancel{\simeq 103 mm}$

$= 51.38 \simeq 52 mm$

$\therefore$ $\boxed{\text{Overlap} = 52mm}$

(c). Factor of safety for field welding,

$$\gamma_{mw} = 1.5$$

Min. Size of weld = 4mm

Max. Size of weld = $t - 1.5 \Rightarrow 10 - 1.5$

$= 8.5 \simeq 8mm$

$\boxed{S = 8mm}$

Throat thickness = 0.7S

= 0.7×8

= 5.6mm

$\boxed{t = 5.6mm}$

load $P = 162 \times 10^3$ N

Design strength:

$$f_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \frac{410/\sqrt{3}}{1.5}$$

$\boxed{f_{wd} = 157.81 \text{ N/mm}^2}$

$P = f_{awd} \times A_w$

$162 \times 10^3 = 157.81 \times l_w \times 5.6$

$883.73 \, l_w = 162 \times 10^3$

$$\boxed{l_w = 183.31\,mm}$$

length of welding on each side $= \frac{183.31}{2}$

$= 91.65 \simeq 92\,mm$

End returns $= 2S$

$= 2 \times 8$

$= 16\,mm$

Overall length of weld $= 92 + 92 + 16 + 16$

$= 216\,mm$

(b) End returns $= 2S \Rightarrow 2 \times 8$

$= 16\,mm$

Overall length of weld $= 52 + 52 + 16 + 16 + 50$

$= 186\,mm$

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UNIT - II

Beams.

Beam :-

It is one of the structural members subjected to the loads perpendicular to the axis of the member

Types of Beams :-

1. Simply supported Beams.
2. Cantilever beam.
3. Propped Cantilever beams.
4. Over hanged beams.

Joist :-

A closely spaced beams supporting floor or roofs of a building but not supporting to other beams.

Girder :-

Large beams are used for supporting a no. of joists.

Purlins & Rafters :-

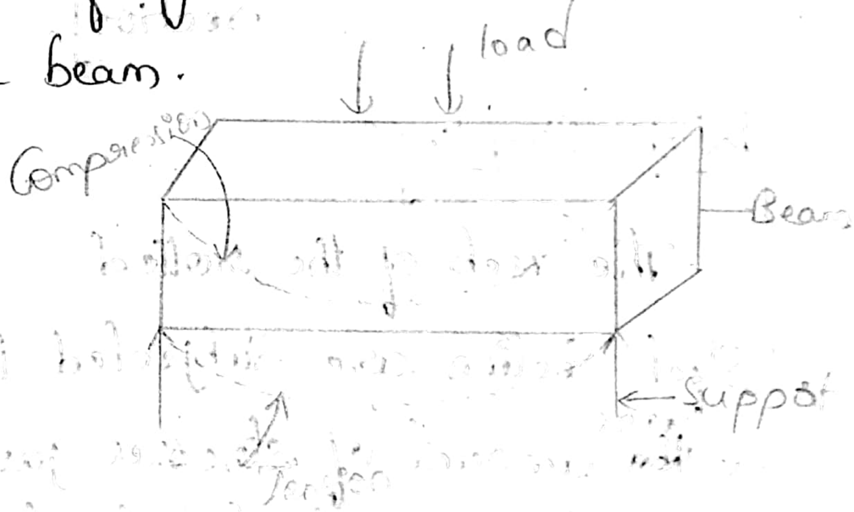
Beams are also used to carry roof

loads in trusses. These beams are called Purlins.

Modes of failures & Causes :-

1. Bending failure:

It generally occurred due to crushing of compressive flange or fracture of tension flange of the beam.



2. Shear failure:

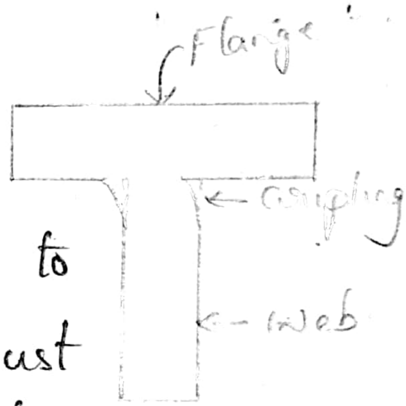
It occurs due to buckling of web of the beam near location of high shear force. The beam can fail locally due to crushing (or) buckling of web. Near the reaction of the concentrated load.

$$\text{Deflection failure } \delta = \frac{5wl^4}{384EI}$$

Types of Sections:-

1. Angle Section
2. T-Section.
3. channel Section.
4. Built up Section.
5. I-Section [Most economical and efficient Section].

Web Cripling:-



The web of the rolled Steel Section are Subjected to a large amount of stresses just below the Concentrated load & above the Supports.

Large bearing stresses are developed below the Concentrated loads to keep the beared stresses within the permissible limits. The Concentrated loads should be transferred from flanges to the web on sufficiently large bearing areas.

$$\boxed{f_w = (b_1 + n_2) t_w \cdot f_{yw}/\gamma_{mo}}$$

Design procedure of a Beam:-

1. Step 1:-

Total ~~tad~~ load,

Factored load (or) design load (w_u) = 1.5 × load

Design bending moment = $\frac{w_u l_e^2}{8}$

Shear force (V_u) = $\frac{w_u l}{2}$

Step 2:- (Selection of section)

$$Z_p = \frac{M}{f_y} \times \gamma_{m0}$$

Step 3:- (Sectional details)

like; Area, depth of section, b_f, t_f, t_w & I_{xx}

Step 4:- (Selection classification)

$$\varepsilon = \sqrt{\frac{250}{f_y}} \text{ (or) } \varepsilon = \left(\frac{250}{f_y}\right)^{1/2}$$

⇒ b/t_f

⇒ d/t_w

Step 5:- (Check for shear)

$$V_d = \frac{V_n}{\gamma_{m0}}$$

$$V_n = V_p = \frac{A_v f_y}{\sqrt{3}}$$

$$\boxed{V_u < V_d}$$

Step 6 :- (check for moment).

$M = ?$

$V = ?$

$V_d = ?$

$0.6 \times V_d =$

$$M_d = \frac{\beta_p Z_p f_y}{\gamma_{mo}}$$

$$\beta_p = Z_e / Z_p$$

$$\boxed{M_d < M}$$

Step 7 :- (check for Deflection)

Max deflection, $\delta_{max} = \frac{5wl^4}{384EI}$

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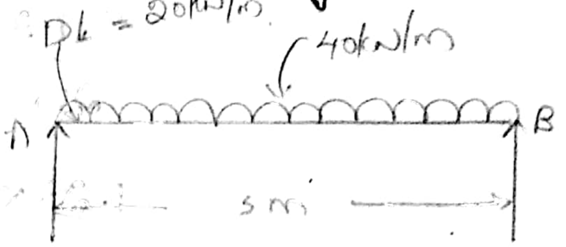
Design a suitable I-section beam for a S.S span of 5m & carrying a dead load of 20kN/m & imposed load of 40kN/m. Take $f_y = 250 N/mm^2$.

Given data,



Span of beam = 5m

Imposed load (U.d.L) = 40kN/m

Dead load = 20kN/m

$f_y = 250 N/mm^2$

<u>Step 1</u>:- (Total load) = 40 + 20 => 60kN/m

Factored load = 1.5 x Total load

= 1.5 x 60

$\boxed{W_u = 90 kN/m}$

Design Bending moment;

$\Rightarrow \frac{W_u l^2}{8}$

$\Rightarrow \frac{90 \times 5^2}{8}$

$\boxed{M_u = 281.25 kN\text{-}m}$

Shear force $V_u = \frac{W_u l}{2}$

$= \frac{90 \times 5}{2}$

$\boxed{V_u = 225 kN}$

Step 2:- (Selection of Section)

$$Z_p = \frac{M}{f_y} \times \gamma_{mo}$$

$$= \frac{281.25 \times 10^6}{250} \times 1.1$$

$$= 1.23 \times 10^6 / 10^3 \ cm^3$$

$$\therefore \boxed{Z_p = 1237.5 \ cm^3}$$

Trial Section, [from steel code book].

ISHB 350 @ 72.4 kg/m ;

$Z_p = 1268.69 \ cm^3$

(or)

ISWB 400 @ 66.7 kg/m ;

$Z_p = 1290.19 \ cm^3$

Let us trial with ISHB 350 @ 72.4 kg/m ;

$Z_p = 1268.69 \ cm^3$

Step 3:- ~~Sede~~ Sectional details [from steel tables]

Area = $92.21 \ cm^2$

Depth of section ; height = 350mm

Breadth of flange (bf) = 250mm

width

Thickness of flange (t_f) = 11.6mm

Thickness of web (t_w) = 10.1mm

Modulus I_{xx} = $19802.8 \times 10^4 \, mm^4$

$r_1 = 12mm$

Step 4:- (Selection classification).

$$\varepsilon = \sqrt{\frac{250}{f_y}} \Rightarrow \sqrt{\frac{250}{250}} = 1$$

$$\Rightarrow b/t_f = b/2 / t_f \Rightarrow \frac{250/2}{11.6}$$

$$\Rightarrow \cancel{\frac{250}{11.6}} = 10.77mm$$

$$= \cancel{21.55mm}$$

$$\Rightarrow d/t_w \Rightarrow \frac{(h-2t_f+r_1)}{t_w}$$

$$= \cancel{350/10.1} \Rightarrow \frac{350 - 2(11.6)+12}{10.1}$$

$$= \cancel{34.65mm} \qquad = 33.54mm$$

Step 5:- check for shear

$$V_d = \frac{V_n}{\gamma_{m0}}$$

Here,

$$V_n = V_p = \frac{A_v \cdot f_y}{\sqrt{3}}$$

$V_u = 225 kN$

$A_v = h \times t_w$

$= 350 \times 10.1$

$= 3535$

$\therefore V_n = V_p = \frac{A_v \cdot f_y}{\sqrt{3}}$

$= \frac{3535 \times 250}{\sqrt{3}}$

$V_n = 510.23 \times 10^3 N \Rightarrow 510.23 kN$

$\therefore V_d = \frac{510.23}{1.10}$

$= \cancel{463848}$

$$\boxed{V_d = 463.84 kN}$$

$$\boxed{V_u < V_d}$$

$\therefore$ Hence safe.

Step 6:- check for moment,

$M_u = 281.25 kN\text{-}m$

$V_u = 225 kN$

$V_d = 463.84 kN$

$0.6 \times V_d = 0.6 \times 463.84$

$= 278.304$

$$M_d = \frac{B_p \cdot Z_p f_y}{\gamma_{m0}} < \frac{1.2 \times Z_e \times f_y}{\gamma_{m0}}$$

$$B_p = \frac{Z_e}{Z_p}$$

$Z_e = 1131.6$ (section modulus)

$Z_p = 1268.69$ (plastic modulus)

$$B_p = \frac{1131.6}{1268.69}$$

$$\boxed{B_p = 0.89}$$

$$M_d = \frac{0.89 \times 1268.69 \times 250}{1.10} < \frac{1.2 \times 1131.6 \times 250}{1.10}$$

$= 256.62 < 308.61$ kN-m

$$\boxed{M_d < M}$$

∴ Hence Safe.

Step 7:- Check for deflection,

$$\text{Max. deflection, } \delta_{max} = \frac{5wl^4}{384EI}$$

$$= \frac{5 \times 60 \times (5000)^4}{384 \times 2 \times 10^5 \times 19802.8 \times 10^4}$$

$$\therefore \boxed{\delta_{max} = 12.32 mm}$$

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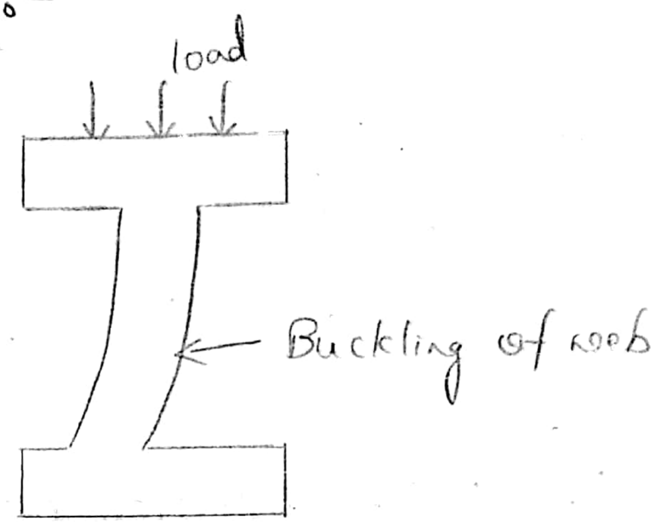
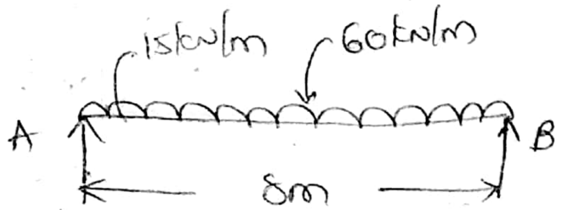
Web Buckling:-



Certain portion of the beam at supports act as Column to transfer the load. This Compressive force on the web may buckle on the beam. The load dispersion angle may be taken as 45°. Hence there is no need to check for web buckling

2. Design a Suitable I-Section beam for a S.S Span of 8m & Carrying a dead load of 15kN/m & imposed load of 60kN/m. Take $f_y = 250\,N/mm^2$, $E = 2\times10^5\,N/mm^2$



Sol Given data,

Span of beam = 5m
= 5000mm

Imposed load (U.d.l) = 60kN/m

Dead load = 15kN/m

$f_y = 250\,N/mm^2$

$E = 2 \times 10^5 \, N/mm^2$

Step_1:-

Total load = 60 + 15

Total load = 75 kN/m

$\boxed{w = 75 \, kN/m}$

Factored load $W_u = 1.5 \times 75$

$= 112.5 \, kN/m$

$\boxed{W_u = 112.5 \, kN/m}$

Design Bending moment,

$$M_u = \frac{w_u l^2}{8}$$

$$= \frac{112.5 \times 8^2}{8}$$

$\boxed{M_u = 900 \, kN\text{-}m}$

Shear force,

$$V_u = \frac{w_u l}{2}$$

$$= \frac{112.5 \times 8}{2}$$

$\boxed{V_u = 450 \, kN}$

Step_2:- (Selection of Section)

$$Z_p = \frac{M}{f_y} \times \gamma_{m0}$$

$$= \frac{900 \times 10^6}{250} \times 1.10$$

$$= \frac{3.96 \times 10^6}{10^3} \, cm^3$$

$$\boxed{Z_p = 3960 \, cm^3}$$

Trial Section [from steel Code book]

ISWB 600 @ 133.7 kg/m ; $Z_p = 3986.66 \, cm^3$

Step 3:- Sectional Details [from steel tables]

Area = $170.38 \, cm^2$

Depth of section, h = 600mm

width of flange, b_f = 250mm

Thickness of flange, t_f = 21.3mm

Thickness of web, t_w = 11.2mm

Modulus, I_{xx} = $106198.5 \, cm^4$

I_x = $106198.5 \times 10^4 \, mm^4$

r_1 = 17mm

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Step 4:- (Section classification)

$$\varepsilon = \sqrt{\frac{250}{fy}}$$

$$= \sqrt{\frac{250}{250}} = 1$$

$$\Rightarrow b/t_f = \frac{b/2}{t_f}$$

$$= \frac{250/2}{21.3}$$

$$\therefore \boxed{b/t_f = 5.86\,mm}$$

$$\Rightarrow d/t_w \Rightarrow \frac{h - 2t_f + r_1}{t_w}$$

$$= \frac{600 - 2(21.3) + 17}{11.2}$$

$$\therefore \boxed{d/t_w = 51.28\,mm}$$

Step 5:- check for shear,

$$V_d = \frac{V_n}{\gamma_{mo}}$$

where,

$$V_n = V_d = \frac{A_v \cdot fy}{\sqrt{3}}$$

$$A_v = h \times t_w$$

$$= 600 \times 11.2 \Rightarrow 6720\,mm^2$$

$$\boxed{A_v = 6720 mm^2}$$

$$V_n = V_p = \frac{A_v \cdot f_y}{\sqrt{3}}$$

$$= \frac{6720 \times 250}{\sqrt{3}}$$

$$= 969.94 \times 10^3 N$$

$$\boxed{V_n = 969.94 kN}$$

$$\therefore \quad V_d = \frac{969.94}{1.10}$$

$$= 881.76$$

$\boxed{V_d = 881.76 kN}$; $V_u = 450 kN$

$$\boxed{V_u < V_d}$$

$\therefore$ Hence Safe

Step 6:- check for moment,

$$M_u = 900 kN\text{-}m$$

$$V_u = 450 kN$$

$$V_d = 881.76 kN$$

$$0.6 V_d = 0.6 \times 881.76$$

$$= 529.056$$

$$M_d = \frac{\beta_p \cdot z_p \cdot f_y}{\gamma_{mo}} < \frac{1.2 \cdot z_e \cdot f_y}{\gamma_{mo}}$$

$$\beta_p = \frac{z_e}{z_p}$$

Section modulus (z_e) = 3540

Plastic modulus (z_p) = 3986.66

$$\therefore \beta_p = \frac{3540}{3986.66}$$

$$\boxed{\beta_p = 0.88}$$

$$M_d = \frac{0.88 \times 3986.66 \times 250}{1.10} < \frac{1.2 \times 3540 \times 250}{1.10}$$

$$M_d = 797.332 \times 10^3 < 965.45 \times 10^3$$

$$\boxed{M_d < M}$$

$\therefore$ Hence safe

Step 7:- Check for deflection.

$$\delta_{max} = \frac{5wl^4}{384EI}$$

$$\Rightarrow \frac{5 \times 75 \times (8000)^4}{384 \times 2 \times 10^5 \times 1.06198.5 \times 10^4}$$

$$\therefore \boxed{\delta_{max} = 18.83mm}$$

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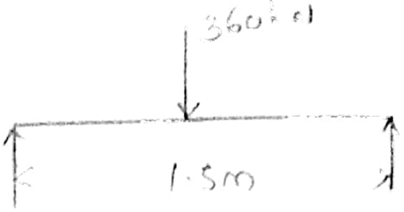
3. Design a S.S beam of effective span 1.5m Carrying a Concentrated load of 360kN at mid Point. $f_y = 250 N/mm^2$



Sol

Given data,

Effective span = 1.5m

load = 360kN

$f_y = 250 N/mm^2$

Step 1: Design BM & SF

$$M_u = \frac{wl}{4}$$

$$= \frac{360 \times 1.5}{4}$$

$$\boxed{M_u = \cancel{270}\,135 \text{ kN-m}}$$

$$V_u = \frac{wl}{2} \Rightarrow \frac{360}{2}$$

$$\boxed{V_u = 180 kN}$$

Step 2: Selection of section

$$Z_p = \frac{M}{f_y} \times \gamma_{mo}$$

$$= \frac{\cancel{270}\,135 \times 10^6}{250} \times 1.1$$

$= \cancel{1188}$

$= \dfrac{594 \times 10^3}{10^3}\ cm^3$

$\boxed{Z_p = 594\ cm^3}$

Trial Section [from steel Code book]

ISMB 300 @ 44.2 kg/m ; $Z_p = 651.74\ cm^3$

Step 3: Sectional Details [from steel tables]

Area = $5626\ mm^2$

Depth of section, h = 300 mm

Width of flange, b_f = 140 mm

Thickness of flange, t_f = 12.4 mm

Thickness of web, t_w = 7.5 mm

Modulus, $I_{xx} = 8603.6 \times 10^4\ mm^4$

$r_1 = 14\ mm$

Step 4: (Section classification)

$\varepsilon = \sqrt{\dfrac{250}{f_y}}$

$= \dfrac{250}{250}$

$= 1$

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$\Rightarrow b/t_f = \dfrac{b/2}{t_f}$

$= \dfrac{\frac{140}{2}}{12.4}$

$= 5.645 mm$

$\Rightarrow d/t_w = \dfrac{h - 2t_f + r_1}{t_w}$

$= \dfrac{300 - 2(12.4) + 14}{7.5}$

$= 38.56 mm$

Step 5: Check for shear,

Additional moment due to dead load,

$W = 44.2 kg/m$

$= 44.2 \times 10 N/m$

$= 442 N/m$ $[\because 1kg = 10N]$

$W = 0.442 kN/m$

$M_2 = \dfrac{wl^2}{8} \Rightarrow \dfrac{0.442 \times 1.5^2}{8}$

$\boxed{M_{u2} = 0.124 kN\text{-}m}$

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Additional shear force,

$$V_{u2} = \frac{wl}{2}$$

$$= \frac{0.442 \times 1.5}{2}$$

$$\boxed{V_{u2} = 0.33\ kN}$$

Total moment $M = M_{u1} + M_{u2}$

$$= 135 + 0.124$$

$$\boxed{M_u = 135.12\ kN\text{-}m}$$

Total shear $V = V_{u1} + V_{u2}$

$$= 180 + 0.33$$

$$\boxed{V_u = 180.33\ kN}$$

Check for shear:

$$V_d = \frac{V_n}{\gamma_{m0}}$$

where,

$$V_n = V_d = \frac{A_v \cdot f_y}{\sqrt{3}}$$

$$A_v = h \times t_w$$

$$= 300 \times 7.5$$

$$\boxed{A_v = 2250\ mm^2}$$

$$V_n = V_p = \frac{A_v \cdot f_y}{\sqrt{3}}$$

$$= \frac{2250 \times 250}{\sqrt{3}}$$

$$V_n = 324.75 \times 10^3 \, N$$

$$\boxed{V_n = 324.75 \, kN}$$

$$V_d = \frac{324.75}{1.10}$$

$$\boxed{V_d = 295.22 \, kN}$$ $V_u = 180.33 \, kN$

$$\boxed{V_u < V_d}$$

∴ Hence safe

Step 6: Check for moment

$M_u = 135.12 \, kN\text{-}m$

$V_u = 180.33 \, kN$

$V_d = 295.22 \, kN$

$0.6 V_d = 0.6 \times 295.22$

$= 177.13$

$$M_d = \frac{\beta_p \cdot Z_p \cdot f_y}{\gamma_{m0}} < \frac{1.2 \cdot Z_e \cdot f_y}{\gamma_{m0}}$$

$$\beta_p = \frac{Z_e}{Z_p}$$

$Z_e = 573.6$ (section modulus)

$Z_p = 651.74$ (plastic modulus).

$$\therefore \beta_p = \frac{573.6}{651.74}$$

$$\boxed{\beta_p = 0.88}$$

$$M_d = \frac{0.88 \times 651.74 \times 250}{1.10} < \frac{1.2 \times 573.6 \times 250}{1.10}$$

$$= 130.34 \times 10^3 \text{ kN-m} < 156.43 \times 10^3 \text{ kN-m}$$

$$\boxed{M_d < M}$$

$\therefore$ Hence safe.

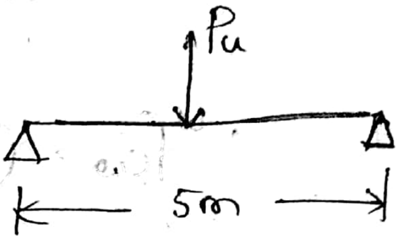
Step 7: Check for deflection,

$$\boxed{\delta_{max} = \frac{5wl^4}{384EI}}$$

$$= \frac{5 \times 360 \times 1500^4}{384 \times 2 \times 10^5 \times 8603.6 \times 10^4}$$

$$\therefore \boxed{\delta_{max} = 1.379 \text{ mm}}$$

4. Determine the Center point load Carrying a Capacity of **ISMB 300** when it is used as a Simply supported of 5m effective span. check it for shear, web buckling, deflection & web Crippling.



Sol let us Consider Central point load Pu beam.

ISMB 300

effective span = 5m

$f_y = 250 N/mm^2$

Step 1:- Sectional details

Area = $5626 mm^2$

$h = 300mm$

$b_f = 140mm$

$t_f = 12.4mm$

$t_w = 7.5mm$

$Z_e = 573.6 \times 10^3 mm^3$

$Z_p = 651.74 \times 10^3 mm^3$

$I_{xx} = 8603.6 \times 10^4 mm^3$

$r_1 = 14mm$

Step 2:- Section classification:

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{140/2}{12.4} \Rightarrow 5.64 < 9.4\,\varepsilon \text{ (Semi Compact)}$$

$$\frac{d}{t_w} = \frac{h-2(t_f+r_1)}{t_w} \qquad \left[\because d = (h-2(t_f+r_1))\right]$$

$$= \frac{300-2(12.4+14)}{7.5}$$

$$= 32.96 < 84\,\varepsilon \text{ (plastic section)}$$

Step 3:- Moment Carrying Capacity of the beam

$$M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{m0}} < \frac{1.2 \cdot Z_e \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.88 \times 651.74 \times 10^3 \times 250}{1.10} < \frac{1.2 \times 573.6 \times 10^3 \times 250}{1.10}$$

$$= 148.122 \times 10^6 < 156.436 \times 10^6$$

$$= 148.122 \text{ kN-m} < 156.43 \text{ kN-m}$$

$$\therefore \boxed{M_d = 148.122 \text{ kN-m}}$$

B.M due to Central point load:-

$$M = \frac{wl}{4} = \frac{P_u \times 5}{4}$$

$$M = 1.25 p_u \text{ kN-m}$$

Limiting Case:

$$M = M_d$$

$$1.25 p_u = M_d$$

$$P_u = \frac{148.122}{1.25}$$

$$= 118.497 \text{ kN}$$

$$\boxed{P_u \simeq 118.50 \text{ kN}}$$

Step 4:- Check for shear ($\because w = P_u$)

$$V = \frac{w}{2} = \frac{118.50}{2} = 59.25 \text{ kN}$$

$$V < V_d$$

$$V_d = \frac{V_n}{\gamma_{m0}} \Rightarrow \frac{A_w \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} \quad \left[\because V_n = \frac{A_w \cdot f_y}{\sqrt{3}}\right]$$

$$= \frac{h \times t_w \times f_y}{\sqrt{3} \cdot \gamma_{m0}} \quad [\because A_w = h \times t_w]$$

$$= 295.23 \text{ kN}$$

$$\therefore \boxed{V < V_d}$$

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Hence check for shear is safe.

Step 5:- check for web buckling

$$\text{Slenderness ratio} = \frac{kL}{r}$$

where $k = 2.5$

$$\frac{L}{r} = \frac{h}{t_w}$$

$$\therefore \text{slenderness ratio} = 2.5 \times \frac{300}{7.5}$$

$$= 100$$

from table 9(c) IS 800:2007

$$f_c = 107 \ N/mm^2$$

web buckling strength: $F_{cw} = (b_1 + n_1) t_w f_c$

$$n_1 = h/2 = \frac{300}{2} = 150 mm$$

$$b_1 = 100$$

$$F_{cw} = (100 + 150) 7.5 \times 107$$

$$= 200.62 \ kN > V (259.25)$$

Step 6:- check for web crippling,

$$F_w = (b_1 + n_2) t_w \cdot \frac{f_y}{\gamma_{m0}}$$

where,

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$n_2 = (2.5 (t_f + r_1))$

$= (2.5 (12.4 + 14))$

$= 66$

$f_w = (100 + 66) \times 7.5 \times \frac{250}{1.10}$

$= 282.95 \text{ kN}$

Step 7:- check for deflection.

$$\boxed{\text{Max. deflection } \delta_{max} = \frac{wl^3}{48EI}}$$

$w = \frac{118.5}{1.5}$

$= 79$

1.5 is a factor of safety

$$\delta_{max} = \frac{79 \times 10^3 \times 5000^3}{48 \times 2 \times 10^5 \times 8603.6 \times 10^4}$$

$= 11.95$

From IS 800: 2007 $\left\{ \frac{L}{300} = \frac{5000}{300} \right.$

$= 16.67$

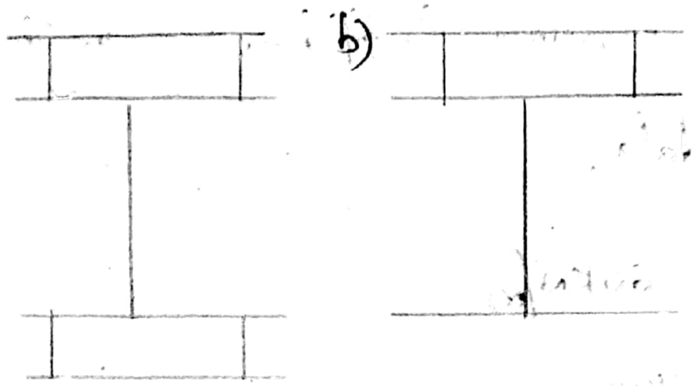
∴ Hence safe.

Built up beams :-

Types :

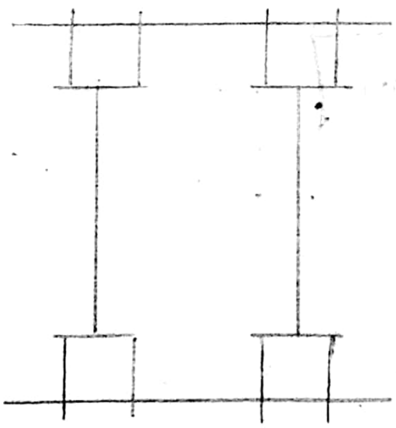
1. plated type built up beam

a) b)



2. Compound beams

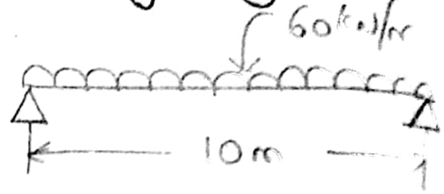
3. Combinations



6. Design a S.S beam of 10m span carrying a factored load of 60kN/m. The depth of the beam shouldn't exceed 500mm. The compression flange of the beam is laterally supported by floor construction. Assume stiff bearing length 75mm

Sol

Given data,



load $(w) = 60kN/m$

$l = 10m$

$f_y = 250N/mm^2$

$b_1 = 75mm$

Step 1:- Cal. of BM & SF

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

$$= \frac{60\times10^2}{8}$$

$$\boxed{M_u = 750kN\text{-}m}$$

$$V = \frac{wl}{2}$$

$$= \frac{60\times10}{2}$$

$$\boxed{V_u = 300kN}$$

Step 2:- Selection of Section.

$$Z_p = \frac{M}{f_y} \times \gamma_{m0}$$

$$= \frac{750 \times 10^6}{250} \times 1.10$$

$\frac{kN\cdot m}{N/mm^2}$

$$= 3.3 \times 10^6 \ mm^3$$

$$= \frac{3.3 \times 10^6}{10^3} \ cm^3$$

$$\boxed{Z_p = 3300 \ cm^3}$$

~~Trial Section~~ [~~from~~ Steel ~~Code book~~]

Here depth of beam restricted to 500mm

∴ Selecting ISMB 450 @ 72.4 kg/m

with suitable cover plates. [∵ M, H, W, L]

$$Z_p = 1533.36 \ cm^3$$

~~Section modulus~~

$$Z_p \text{ for plates} = 3300 - 1533.36$$

$$= 1766.64 \ cm^3$$

$$\therefore \boxed{Z_p = 1766.64 \ cm^3}$$

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Let A_p = Area of each plate

d = c/c distance b/w plates

t_p = thickness of plate

Z_p for plates $(Z_p) = \frac{A_p \times d}{\gamma_{m0}}$

$1766.64 = \frac{A_p \cdot d}{\gamma_{m0}}$

$A_p = \frac{1766.64 \times 1.1}{450}$

$\therefore \boxed{A_p = 4.32\,cm^2}$

$= 4320\,mm^2$

Let us selecting 20mm thickness of plate

$b t_p = 4320$

$b = \frac{4320}{20}$

$= 216 \simeq 220\,mm$

$\therefore \boxed{b = 220\,mm}$

$\therefore$ 220 x 20mm size plate with ISMB 450

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as a beam member.

Step 3:- Sectional details (using steel code & steel tables)

ISMB 450;

$W = 72.4$

$A = 92.27\,cm^2 \Rightarrow 9227\,mm^2$

$h = 450\,mm$

$b_f = 150\,mm$

$t_f = 17.4\,mm$

$t_w = 9.4\,mm$

$I_{xx} = 30390.8\,cm^4$

$\Rightarrow 30390.8 \times 10^4\,mm^4$

$r_1 = 15.00\,mm$

$Z_e = 1350.7\,cm^3 \Rightarrow 1350.7 \times 10^3\,mm^3$

$Z_p = 1533.36\,cm^3 \Rightarrow 1533.36 \times 10^3\,mm^3$.

Step 4:- Section classification

$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

$$= \sqrt{\frac{250}{250}}$$

$$= 1$$

$b/t_f = \frac{b/2}{t_f} \Rightarrow \frac{150/2}{17.4} \Rightarrow 4.3 < 9.4\varepsilon$

$$b/t_w \rightarrow \frac{h-2(t_f+r_1)}{t_w}$$

$$= \frac{450-2(17.4+15)}{9.4}$$

$$= 40.97 < 84\varepsilon$$ [∵ Pg: 18 from Steel code]

Step 5:- check for shear,

$$V_d = \frac{V_n}{\gamma_{mo}} \Rightarrow \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

$$\Rightarrow \frac{(h \cdot t_w) f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

$$= \frac{(450 \times 9.4)\, 250}{\sqrt{3} \times 1.10}$$

$$= 555.04 \times 10^3 \, N$$

$$\boxed{V_d = 555.04 \, kN}$$

$$\boxed{V_u < V_d}$$

∴ Hence Safe

Step 6:- Check for moment

$M_u = 780 \, kN\text{-}m$

$V_u = 300 kN$

$V_d = 555.04 kN$

$0.6 V_d = 0.6 \times 555.04$

$= 333.024$

20

450

$M_d = \frac{\beta_p \cdot Z_p \cdot f_y}{\gamma_{m0}} < \frac{1.2 \cdot Z_e \cdot f_y}{\gamma_{m0}}$

$\cancel{\beta_p = \frac{Z_e}{Z_p}}$

$\cancel{= \frac{1350.7 \times 10^3}{1533.36 \times 10^3}}$

$\cancel{= 0.88}$

$\cancel{M_d = 0.88 \times 1533.36 \times 10^3}$

Z_p = Z_p of I-section + Z_p of plate.

$= 1533.36 \times 10^3 + (A_p \times d)$

$= 1533.36 \times 10^3 + ((220 \times 20)(450 + 20))$

$\boxed{Z_p = 3601.36 \times 10^3 mm^3}$

$Z_e = I/y$

$\left(\frac{450}{2} + \frac{20}{2}\right)$

I = M.O.I. of I_{xx} + M.O.I of plate.

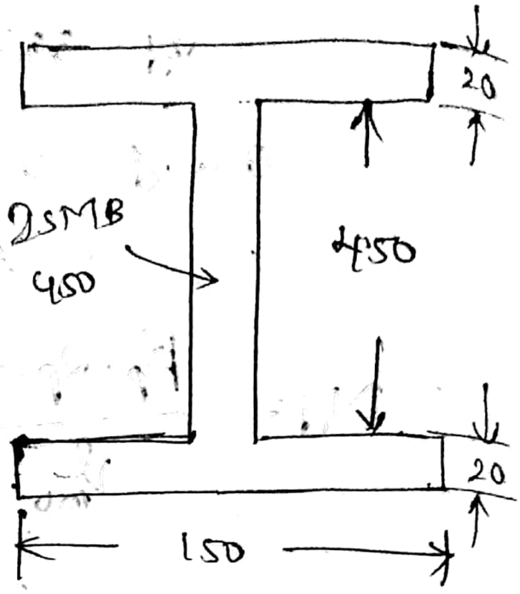
$= 30390.8 \times 10^4 + 2\left[\frac{220 \times 20^3}{12} + 220 \times 20 \times (235)^2\right]$

$= 79018.13 \times 10^4 \, mm^4$

$Z_e = I/y$

$= \frac{79018.13 \times 10^4 \, mm^4}{490/2}$

$= 3.225 \times 10^6 \, mm^3$



$\beta_p = \frac{3.225 \times 10^6}{3601.36 \times 10^3}$

$\boxed{\beta_p = 0.88}$

$\therefore M_d = \frac{0.88 \times 3601.36 \times 10^3 \times 250}{1.10} < \frac{1.2 \times 3.225 \times 10^6 \times 250}{1.10}$

$= 720.27 \times 10^6 < 879.54 \times 10^6 \, kN\text{-}m$

$\therefore \boxed{M_d < M}$

$\therefore$ Hence Safe.

Step 7:- Check for ~~deflection~~ web buckling,

Slenderness ratio $= \frac{kl}{r}$

where $k = 2.5$

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$$\frac{L}{r} = \frac{h}{t_w}$$

$$= \frac{450}{9.4}$$

$$= 47.87$$

$\therefore$ slenderness ratio = 2.5 × 47.87

= 119.675

from table No: 9(c) & of pg: 42

KL/r	f_y
↓	↓
x_1 – 110 →	94.6 – y_1
x_2 120 →	83.7 – y_2
119.675 →	? – z_1

$$= \cancel{83.7} + \cancel{\left(\frac{94.6 - 83.7}{120 - 110}\right) \times (119.675 - 110)}$$

$$= y_1 + \frac{y_2 - y_1}{x_2 - x_1}(z_1 - x_1)$$

$$= 94.6 + \frac{83.7 - 94.6}{120 - 110} \times (119.675 - 110)$$

$$\boxed{f_c = 84.05 \ N/mm^2}$$

web buckling strength,

$$f_{cw} = (b_1 + n_1) t_w \cdot f_c$$

$$= \left(75 + \frac{450}{2}\right) 9.4 \times 84.05$$

$$= 237.02 \times 10^3 \ N$$

$$\boxed{f_{cw} = 237.02 \ kN}$$

~~Step 8:-~~

Step 8 :- web crippling,

$$f_w = (b_1 + n_2) t_w \cdot f_y / \gamma_{m0}$$

where,

$$n_2 = [2.5 (t_f + r_1)]$$

$$= 2.5 [17.4 + 15]$$

$$= 81$$

$$\therefore f_w = (75 + 81) 9.4 \times \frac{250}{1.10}$$

$$= 333.27 \times 10^3 \ N$$

$$\therefore \boxed{f_w = 333.27 \ kN}$$

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Step 9:- check for deflection.

$$\delta_{max} = \frac{5wl^4}{384EI}$$

$$= \frac{5 \times 60 \times (10000)^4}{384 \times 2 \times 10^5 \times 50390.8 \times 10^4}$$

79016.13×10^4

$$\therefore \boxed{\delta_{max} = 49.43mm}$$

As per IS Code Max. deflection $= \frac{Span}{300}$

$$= \frac{10000}{300}$$

$$= 33.33$$

$$49.43 > 33.33$$

$\therefore$ Hence it is unsafe.

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UNIT - 3

Tension ~~[illegible]~~ members & Compression members

Compression member:-

A structural member carrying axial compressive force is known as compression member

Ex:- struts.

Mode of failures:-

1. Crushing failure:

A very short length compression member & the compressive force.

2. Buckling failure:

A very long length compression under compressive force

Ex:- Fan

Mixed mode:-

The above two failures occurs in same case

Slenderness ratio :-

$$\lambda = \frac{\text{Effective length}}{r}$$

Col = ISHB

$$\lambda_{max} = \frac{l_{eff}}{r_{min}}$$

If slenderness ratio increases the strength of the Column decreases

10 Determine the design axial load carrying capacity of the Column ISHB 300 @ 58.8 kg/m. If the length of the Column is 3m & its both ends are pinned

sol Given data,

ISHB 300 @ 58.8 kg/m

length of Column = 3m

l_{eff} = 3m

= 3000mm ⇒ KL

load carrying = ?

Buckling class Sectional Details:

ISHB 300 @ 58.8 kg/m

Area = $74.85 cm^2$ ⇒ $7485 mm^2$

$h = 300mm$

$b_f = 250mm$

$t_f = 10.6mm$

$t_w = \cancel{9.4}\ 7.6 mm$

$r_{xx} = 12.95 cm$

$= 129.5 mm$

$r_{yy} = 5.41 cm$

$= 54.1 mm$

$$\frac{h}{b_f} = \frac{300}{250} \rightarrow 1.2mm$$

$t_f = 10.6mm < 100mm$

Buckling class 'b' about 'xx'

Buckling class 'c' about 'yy'

Design strength of Column for buckling class 'b'.

Imperfection factor, $\alpha = 0.34$ [from table 7 Pg: 35]

$$f_{cc} = \frac{\pi^2 E}{(KL/r)^2} \quad [Pg:- 34]$$

$$\frac{KL}{r} = \frac{3000}{54.1} \Rightarrow 55.45$$

where,

fcc - Euler buckling stress fro

$$\therefore f_{cc} = \frac{\pi^2 \times 2 \times 10^5}{(55.45)^2}$$

$$\boxed{f_{cc} = 641.98\ N/mm^2}$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} \Rightarrow [Pg: 34].$$

$$= \sqrt{\frac{250}{641.98}}$$

$$\boxed{\lambda = 0.62}$$

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

$$= 0.5\left[1 + 0.34(0.62 - 0.2) + (0.62)^2\right]$$

$$\boxed{\phi = 0.76}$$

Design Compressive stress, fcd

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} < f_y/\gamma_{m0}$$

$$= \frac{250/1.10}{0.76 + \left[0.76^2 - 0.62^2\right]^{0.5}} < 227.27$$

$$= \frac{227.27}{1.199} < 227.27$$

$$\boxed{f_{cd} = 189.55\ N/mm^2} < 227.27\ N/mm^2$$

load, $P_d = A \cdot f_{cd}$ — [$\because$ pg: 34] from code

$$= 7485 \times 189.55$$

$$= 1.418 \times 10^6\ N$$

$$\boxed{P_d = 1418.7\ kN}$$

Design strength of column for buckling class 'c'. Imperfection factor, $\alpha = 0.49$

$$f_{cc} = \frac{\pi^2 E}{(KL/r)^2}$$

$$\frac{KL}{r} = \frac{3000}{54.1} \Rightarrow 55.45$$

$$\therefore f_{cc} = \frac{\pi^2 \times 2 \times 10^5}{(55.45)^2}$$

$$\boxed{f_{cc} = 641.98\ N/mm^2}$$

$$\lambda = \sqrt{f_y / f_{cc}} = \sqrt{\frac{250}{641.98}}$$

$$\boxed{\lambda = 0.62}$$

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

$$= 0.5\left[1+0.49(0.62-0.2)+0.62^2\right]$$

$$\boxed{\phi = 0.79}$$

Design Compression stress, fcd

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi+\left[\phi^2-\lambda^2\right]^{0.5}}$$

$$= \frac{250/1.10}{0.79+\left[0.79^2-0.62^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 177.61\ N/mm^2}$$

load, $P_d = A \cdot f_{cd}$

$$= 7485 \times 177.61$$

$$= 1.329 \times 10^6\ N$$

$$\boxed{P_d = 1329\ kN}$$

9. Design a Column of I-section with a length of 3m to carry an axial Compressive force of 300kN. The Column is effectively held in position at both ends but n't restrained against rotation

sol

Given data,

Compressive force = 300kN

length of Column = 3m => 3000mm

end Conditions : (Not restrained against rotation)

Effective length $l_{eff} = 1.0L$

$= 1(3)m$

$KL = 3000mm$

factored load $(P_d) = 1.5 \times 3000$

$= 450kN$

Assume $f_{cd} = 90 N/mm^2$ (90 to $130 N/mm^2$)

Step 1: (Selection of Section).

$P_d = A_c \cdot f_{cd}$

$$A_c = \frac{450 \times 10^3}{90}$$

$= 5000 mm^2$

$$\boxed{A_c = 5000\ mm^2}$$

let trial sections be find out

ISMB 300 ; Area, $A = 56.26\ cm^2$

$= 5626 mm^2$

ISWB 250 ; Area $A = 52.05\ cm^2$

$= 5205\ mm^2$

ISHB 200 ; Area $A = 50.94\ cm^2$

$= 5094 mm^2 \simeq 5000 mm^2$ ✓

let us try with ISHB 200, $A = 5094 mm^2$

Step 2:- (Buckling class)

$h = 200 mm$

$b = 200 mm$

$t_f = 9.0 mm$

$t_w = 7.8 mm$

$I_{xx} = 3721.8\ cm^4$

$= 3721.8 \times 10^4\ mm^4$

$r_r = 9 mm$

$r_{xx} = 8.55\ cm^4$; $r_{yy} = 4.42\ cm^4$

$= 8.55 \times 10 mm$ $= 4.42 \times 10 mm$

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About xx - buckling class - b

yy - buckling class - c

Step 3:-

Design strength of Column for buckling

Class 'c';

$$\alpha = 0.49$$

$$f_{cc} = \frac{\pi^2 E}{(KL/r)^2}$$

$$= \frac{\pi^2 \times 2\times10^5}{\left(\frac{3000}{44.2}\right)^2}$$

$$\boxed{f_{cc} = 428.48 \ N/mm^2}$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} \Rightarrow \sqrt{\frac{250}{428.48}}$$

$$\boxed{\lambda = 0.76}$$

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

$$= 0.5\left[1+0.49(0.76-0.2)+0.76^2\right]$$

$$\phi = 0.926$$

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Design Compressive stress, f_{cd}

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

$$= \frac{250/1.10}{0.926 + [0.926^2 - 0.76^2]^{0.5}}$$

$= 156.19 N/mm^2$

load, $P_d = A \cdot f_{cd}$

$= 5094 \times 156.19$

795.63

$= \cancel{780.45} \times 10^3 N$

$\boxed{P_d = \cancel{795.95 kN}}$ 795.63 kN

$\therefore P < P_d$ satisfied

Hence ok.

3. A Column 4m long has to support a factored load of 6000 kN. The Column is effectively held in position at both ends & restrained in direction at one of the end. Design the Column using beam section & plates

Sol

Given data,

Factored load $P = 6000\,kN$

length of the Column = 4m

Effective length $(KL) = 0.8L$

$= 0.8 \times 4$

$= 3.2m$

~~$P = A_c f_{cd}$~~

Step 1:- (selection of section)

~~Step 1:- (Selection of section)~~

$P = A_c \cdot f_{cd}$

$$A_c = \frac{6000 \times 10^3}{200}$$

$= 30 \times 10^3\,mm^2$

$= 30000\,mm^2$

ISHB 450 @ $117.89\,cm^2$

$A_c = 117.89\,cm^2$

$= 11789\,mm^2$

Area of plates = 30,000 − 11789

$= 18211\,mm^2$

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Providing 2 plates on both sides on I-section

$2 \times b \times t = 18211$

$b \times t = 18211/2$

$b \times t = 9105.5$

ISHB
ISMB
ISWB

Selecting 20mm of thickness

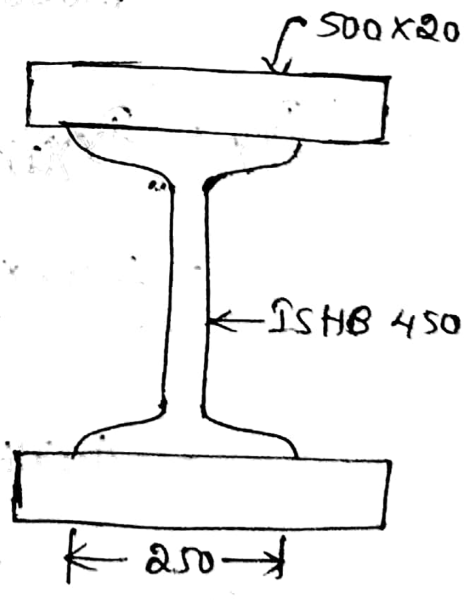
$b \times 20 = 9105.5$

$20b = 9105.5$

$b = \frac{9105.5}{20}$

$= 455.27mm \simeq 500mm$

Selecting 500x20 mm on both sides of ISHB 450 as a built up member.



Over hanging $= \frac{500-250}{2}$

$= 125mm$

Conditions:-

$125 < 12t$ (or) 200
(which ever is high)

$125 < 240mm$ (Hence ok).

Step 2:- Sectional details,

$$I_{xx} = 40349.9 \times 10^4 \, mm^4$$

$$I_{yy} = 3045.0 \times 10^4 \, mm^4$$

$$A = 11789 \, mm^2$$

Sectional properties of built-up member,

$$A = A_c + A_p$$

$$= 11789 + 2(500 \times 20)$$

$$A = 31789 \, mm^2$$

$$I_{xx} = I_{xx(I)} + I_{xx(P)}$$

$$= 40349.9 \times 10^4 + 2\left[\frac{500 \times 20^3}{12} + (500 \times 20) \times 235^2\right]$$

$$= 150866.57 \times 10^4 \, mm^4$$

$$I_{yy} = I_{yy(I)} + I_{yy(P)}$$

$$= 3045 \times 10^4 + 2\left[\frac{20 \times 500^3}{12}\right]$$

$$= 44711.6 \times 10^4 \, mm^4$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}}$$

$$= \sqrt{\frac{44711.6 \times 10^4}{31789}}$$

$\therefore r_{yy} = 118.59 mm$

Slenderness ratio, $\frac{kL}{r_{yy}}$

$$= \frac{3200}{118.59}$$

$$= 26.98$$

Buckling class 9(c) for built up members from table 9(c)

$x_1 - 20 \rightarrow 224 - y_1$

$x_2 - 30 \rightarrow 211 - y_2$

$z_1 - 26.98 \rightarrow ?$

$$f_c = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(z_1 - x_1)$$

$$f_c = 224 + \frac{211 - 224}{30 - 20}(26.98 - 20)$$

$$\boxed{f_c = 214.92\ N/mm^2}$$

$\therefore P_d = A \times f_c$

$= 31789 \times 214.92$

$= 6.83 \times 10^6\ N$

$P_d = \cancel{6.83 \times 10^3 N}$ 6832 kN > 6000 kN

∴ Hence ok

Tension members:-

Some elements of steel structures are structural members are subjected to two pulling forces applied at both sides. Such structural members that are intended to resist tensile load are termed as tension members or 'tie' members.

A member carrying direct tension is called as tie member

Types of tension members:-

1. Wires & Cabels
2. Rods & wires
3. Single ~~sturut~~ structural shapes & plates
4. Built-up sections

Various forms of tension member:-

1. Circular
2. Rectangular
3. Angle [Equal & unequal]

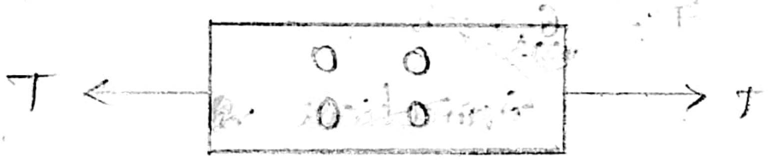
4. Double Angle

5. Stag Angle

Net Area:-

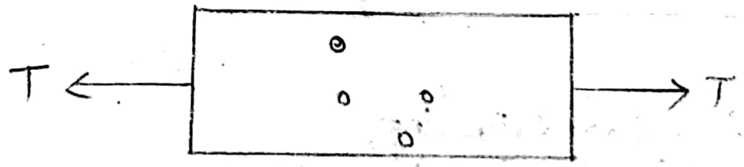
When a tension member is joined to any other member by rivets or hole which gross sectional area is reduced by the holes. Hence the tension members are designed for its net sectional area.

For chain riveting:-



$$A_n = (b - nd)t$$

For zig-zag riveting:-



$$A_n = \left(b - nd + \Sigma \frac{p^2}{4g}\right)t$$

4. Double Angle

5. Stag Angle

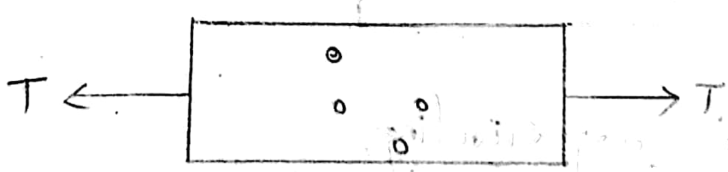
Net Area:-

When a tension member is joined to any other member by rivets or hole which gross sectional area is reduced by the holes. Hence the tension members are designed for its net sectional area.

For chain riveting:-

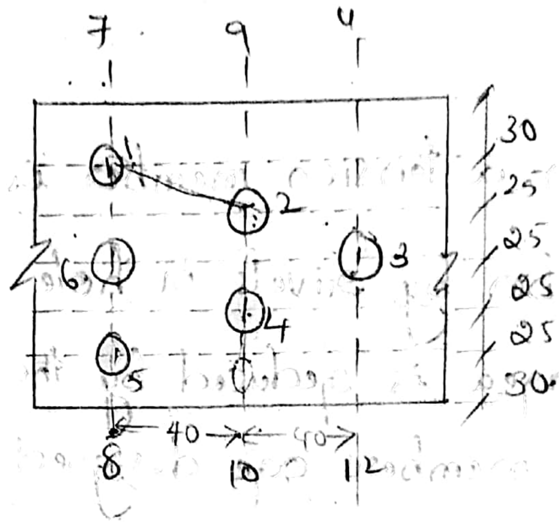
$$A_n = (b - nd)t$$

For zig-zag riveting:-



$$A_n = \left(b - nd + \Sigma \frac{p^2}{4g}\right)t$$

4. Determine the design tensile strength of 160 x 8mm plate with holes of 18mm ϕ as shown in fig.



Sol

Dia of holes = 18mm

<u>For chain riveting;</u>

no = 3

7 - 1 - 6 - 5 - 8

~~Connections = 2~~

1, 6, 5 - no's = 3

$A_n = (b - nd)t$

$= 160 - 3(18) \times 8$

$\boxed{A_n = 848\,mm^2}$

<u>For zig-zag riveting;</u>

no = 5

7 - 1 - 2 - 3 - 4 - 5 - 8

Connections - 4

no; 1, 2, 3, 4, 5 = 5

$$A_n = \left(b - nd + \frac{\Sigma p^2}{4g}\right)t$$

$$= 160 - 5\times18 + \frac{4\times40^2}{4\times25}\times8$$

$$\boxed{A_n = \cancel{976}\,1072\,mm^2}$$

~~7-1-2-4-10~~

For zig-zag riveting,

7-1-2-4-10

$$A_n = \left(b - nd + \frac{\Sigma p^2}{4g}\right)t$$

$$= \left(160 - 3\times18 + \frac{1(40)^2}{4\times25}\right)\times8$$

$$\boxed{A_n = 976\,mm^2}$$

5. Design tensile strength is the min. of the followings:

1. Design strength due to yielding of ~~g/s~~ gross section (T_{dg})
2. Design strength due to rupture of critical section (T_{dn})

$$T_{dg} = A_g \cdot f_y / \gamma_{m0}$$

$$= (160 \times 8)\, 250 / 1.10$$

$$= 290.9\,kN$$

$$T_{dn} = 0.9 A_n \cdot f_u / \gamma_{m1}$$

$$= \frac{0.9(848) \times 410}{1.25}$$

$$= 250.3\,kN$$

5).

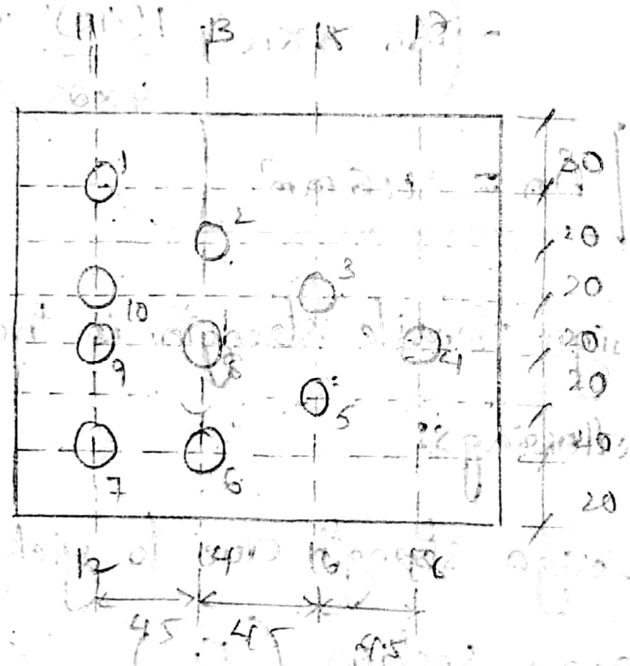


Plate - 140x12mm

hole ϕ = 20mm

For chain riveting:

11 – 1 – 10 – 9 – 7 – 12

$A_n = (b - nd)t$

$= 140 - 4(20) \times 12$

$$\boxed{A_n = 720 mm^2}$$

For zig-zag riveting:

11 – 1 – 2 – 3 – 4 – 5 – 6 – 14

$A_n = \left(b - nd + \frac{\varepsilon p^2}{4g}\right) \times t$

$= \left(140 - 6(20) + \frac{5 \times 45^2}{4 \times 20}\right) \times 12$

$$\boxed{A_n = 1758.75 mm^2}$$

zig-zag riveting:

11 – 1 – 2 – 8 – 6 – 14

$A_n = \left(b - nd + \frac{\varepsilon p^2}{4g}\right) \times t$

$= \left(140 - 4(20) + \frac{2 \times 45^2}{4 \times 20}\right) \times 12$

$= 1327.5 mm^2$

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Design tensile strength is the min. of the following:

1. Design strength due to yielding of gross section (T_{dg})

2. Design strength due to rupture of critical section (T_{dn})

$$T_{dg} = A_g \cdot f_y / \gamma_{m0}$$

$$= \frac{(190 \times 12) 250}{1.10}$$

$$= 381.81 \text{ kN}$$

$$T_{dn} = 0.9 \cdot A_n \cdot f_u / \gamma_{m1}$$

$$= \frac{0.9 \times 720 \times 410}{1.25}$$

$$= 212.54 \text{ kN}$$

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6)

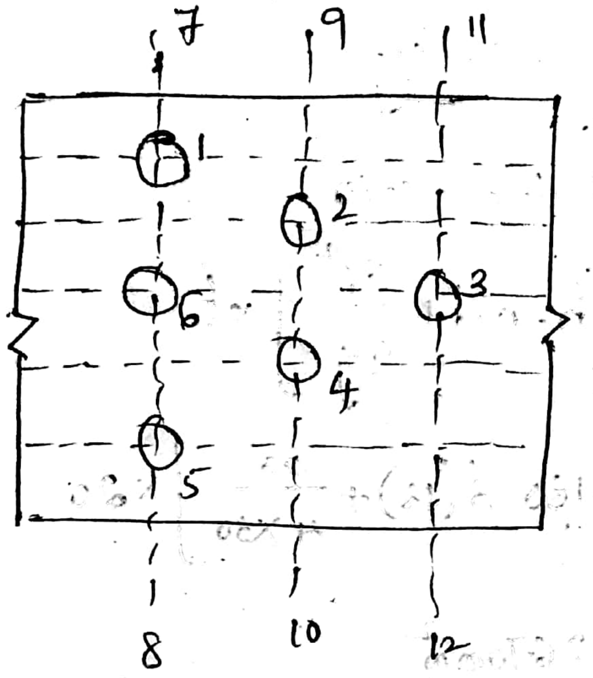


Plate = 180 X 20 mm

d = 18 mm

Sol For chain riveting:

7 - 1 - 6 - 5 - 8

$A_n = (b - dn)t$

$= [180 - 18(3)] \times 20$

$= 2520 \text{ mm}^2$

for zig-zag riveting:

7 - 1 - 2 - 3 - 4 - 5 - 8

$$A_n = \left[b - nd + \frac{\Sigma p^2}{4g}\right] \times t$$

$$= \left[180 - (5 \times 18) + \frac{4 \times 30^2}{4 \times 30}\right] \times 20$$

$= 2400 \text{ mm}^2$

for zig-zag riveting:-

7-1-2-4-10

$$A_n = \left[b - nd + \frac{\Sigma p^2}{4g}\right] \times t$$

$$= \left[180 - 3(18) + \frac{30^2}{4 \times 30}\right] \times 20$$

$$= 2670 mm^2$$

$$T_{dg} = A_g \cdot f_y / \gamma_{m0}$$

$$= \frac{(180 \times 20) \times 250}{1.10}$$

$$= 818.18 \times 10^3 N$$

$$\boxed{T_{dg} = 818.18 kN}$$

$$T_{dn} = 0.9 A_n \cdot f_u / \gamma_{m1}$$

$$= \frac{0.9 \times 250 \times 410}{1.25}$$

$$\boxed{T_{dn} = 743.9 kN}$$

7).

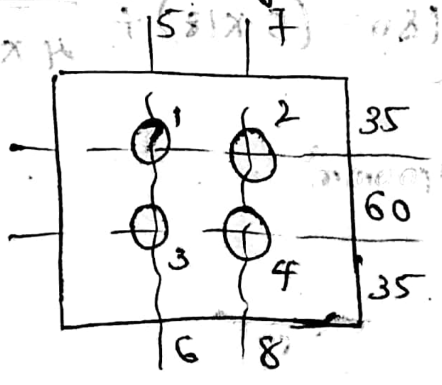


Plate = 130 x 12mm

d = 18mm

sol i) For chain riveting:

5-1-3-6

$A_n = (b - d_n)t \Rightarrow [130 - 18(2)] \times 12$

$\boxed{A_n = 1128 mm^2}$

(ii) 7-2-4-8

$A_n = (b - d_n)t \Rightarrow [130 - 18(2)] \times 12$

$\boxed{A_n = 1128 mm^2}$

$T_{dg} = A_g \cdot f_y / \gamma_{m0}$

$= \frac{(130 \times 12) 250}{1.10}$

$= 354.5 kN$

$T_{dn} = 0.9 A_n \cdot f_u / \gamma_{m1}$

$= \frac{0.9 \times 1128 \times 410}{1.25}$

$= 332.98 kN$

14/2/18

To determine the design tensile strength of the Angles:-

i) The gusset is Connected to 90mm length.

2). If the gusset is Connected to 60mm length

then $g = 50mm$ for 90mm length

$g = 30mm$ for 60mm length

The design tensile strength is least of T_{dg}, T_{dn}

& T_{db}

where; T_{dg} - Design strength due to yield of g/s

T_{dn} - due to rupture of critical section

T_{db} - due to block shear

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}}$$

$$T_{dn} = 0.9 A_{nc} \cdot f_u / \gamma_{m1} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

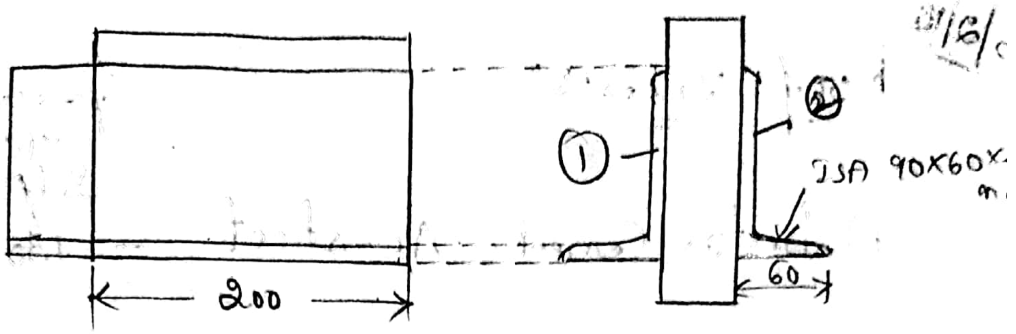
$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9 A_{tn} \cdot f_u}{\gamma_{m1}}$$

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

$b_s = w + w_1 - t$

w = Out stand leg width = 60mm

8. Determine the design tensile strength of roof member ISA 90x60x6mm connect to a gusset plate of 8mm thickness by 4mm welding the effective length of weld is 200mm



sol Given data;

Two ISA 90×60×6mm

length of weld = 200mm

design strength due to yielding of g ls;

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} ; f_y = 250, \gamma_{m0} = 1.1$$

$A_g = 1137 mm^2$ from steel table (90×60×8)

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} \Rightarrow \frac{1137 \times 250}{1.1} \Rightarrow 258.40 kN$$

Design strength due to rupture of critical Section;

$$T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

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

where, $w = 60mm$, $L_c = 200mm$, $b_s = 60mm$, $t = 6mm$

$$\beta = 1.4 - 0.076 \left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\left(\frac{60}{200}\right) \Rightarrow 1.26$$

$$\beta < \frac{f_u \cdot \gamma_{m0}}{f_y \cdot \gamma_{m1}} \Rightarrow \frac{410 \times 1.10}{250 \times 1.25} \Rightarrow 1.26 < 1.44$$

$A_{nc} = 2(90 - 6/2)6 = 1044 mm^2$

$A_{go} = 2(60 - 6/2)6 = 684 mm^2$

$$T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

$$= 504.05 kN$$

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Roof Trusses:-

Sp. gravity (or) Self wt.

Asbestos Cement; AC-sheet - 130 N/mm²

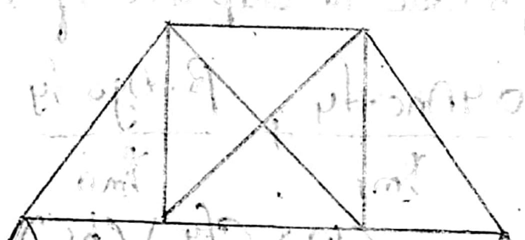
Galvanised Iron; GI sheet → 85 N/mm²

Types of roof trusses:

1. King post:-

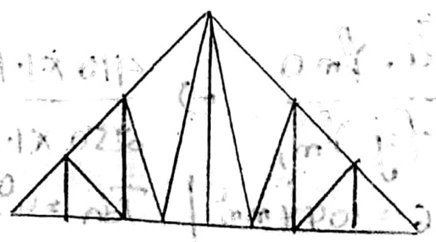
→ upto 6m span

2. Queen post:-



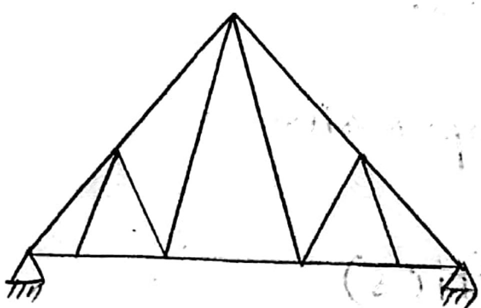
-upto 6-9m

3. Howe post truss (or) Howe triangular:-

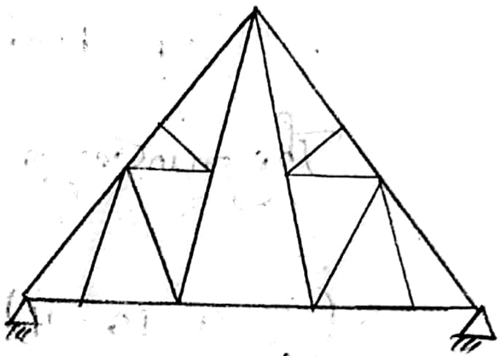


upto 9-15m span.

4. French type trusses:-

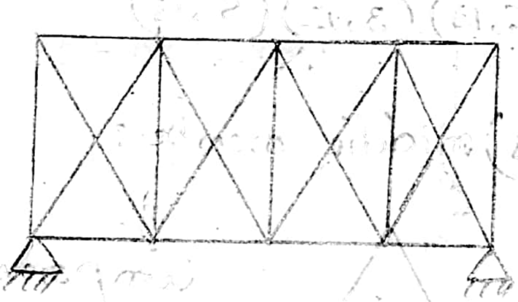


upto 9m



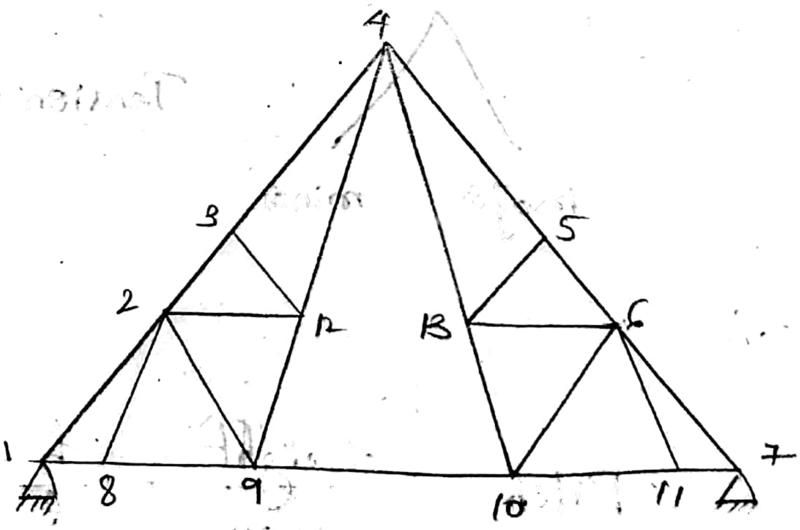
For span b/w 9-12m

5. pratt type truss:-



Span b/w 16-30m

Truss Components:-



→ (1-2-3-4-5-6-7) — Rafters

Top chord members,

They undergo Compression.

→ (8-9-10-11) - (TIES)

Bottom chord members,

They undergo tension.

→ (2, 12), (6, 13), (3, 12) (5, 13)

(Struts) middle members.

Compressive force

(strings) middle members.

Tension develops.

Pitch:-

$$\text{Pitch} = \frac{\text{Height}}{\text{span}} = \frac{h}{l}$$

For G.I sheet which pitch $p = \frac{1}{6}$

for AC sheet $p = \frac{1}{12}$

Spacing of trusses:-

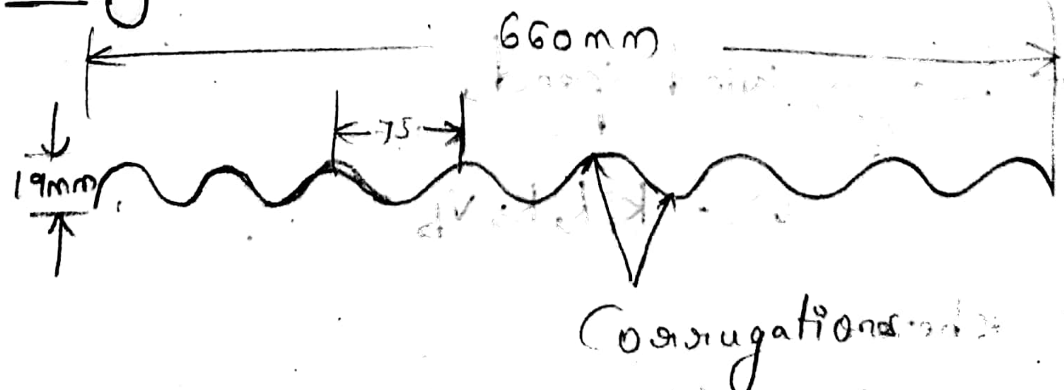
1. 3m to 4.5m - upto 15m span
2. 4.5 to 6m - upto 15-30m span

Purlin:-

It should be located on panel point of top chord members. Generally the spacing of purlin varies from 1.35m to 2m.

Angle purlins are used for smaller value of spacing [spacing of trusses 3 to 4m], for medium sizes 4 to 5m channel sections are used.

Sheeting:-



→ For 8 Corragulations Overall width will be 660mm.

→ For 10 Corragulations Overall width will be 810mm

→ In general the roof Cover will be is Covering weight including lap Connecter 100 to 150 N/mm². This is for G.I sheet. For AC sheet 170 to 200 N/mm².

→ Weight of purlin = 100 to 120 N/mm²

Imposed or Live load :- [IS :875]

→ upto 10° slope will be 0.75 kN/m²

→ For more than 10° slope will be 0.75 to 0.02

→ However minimum slope 0.4kN/m²

Wind load [IS :875 part 3] :

→ Design wind speed,

$$\omega_x = k_1 k_2 k_3 V_b$$

where,

$k_1 \rightarrow$ Risk Coefficient

$k_2 \rightarrow$ Terrain height & structure size factor.

$k_3 \rightarrow$ Topography factor.

Load Combinations :-

Dead load + Live load

Dead load + wind load

Dead load + sheet load.

9. A symmetrical truss of span 20m & height 5m are spaced at 4.5 c/c. Design channel section purlin to be placed at suitable distance to resist the following loads.

wt. of sheeting including bolts = 171 N/m²

Live load = 0.4 kN/m²

wind load = 1.2 kN/m²

Spacing of purlins = 1.4 m

Sol: Given data,

Span of truss (l) = 20m

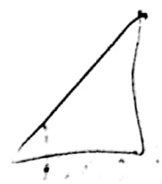
Height (h) = 5m

Spacing of truss (S) = 4.5 m c/c

Live load = $0.4\ kN/m^2$

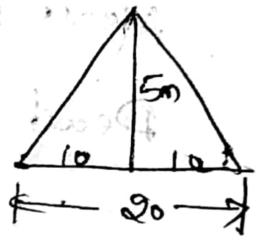
Wind load = $1.2\ kN/m^2$

Dead load = $171\ kN/m^2$



Spacing of purlin = 1.4 m

Inclination, $\tan\theta = \frac{5}{10}$



$\theta = \tan^{-1}\frac{5}{10}$

$\therefore \boxed{\theta = 26^\circ}$

Design for DL + LL,

DL for sheeting = $171\ N/mm^2$

DL for purlin = $125\ N/mm^2$

Total DL = $296\ N/m^2$

$= 0.296\ kN/m^2$

L.L = $0.4\ kN/m^2$

Total load = D.L + L.L

$= 0.296 + 0.4$

$= 0.696\ kN/m^2$

∴ factored load $w_u = 1.5 \times 0.696$

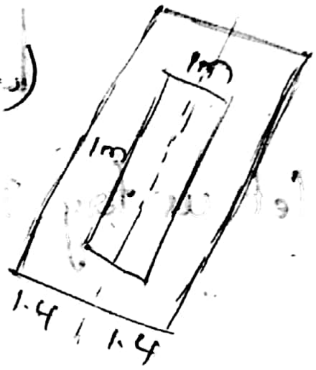
$= 1.044\ kN/m^2$

20/2/18

So, per unit length meter,

$udl\ (w) = 1.4 \times \overset{(w_u)}{1.044}$ [∴ spacing of Purlin × w_u]

$= 1.4616\ kN/m$



Normal load to sheeting (i.e.,)

vertical loads $= 1.46 \cos\theta$

Parallel load to sheeting (i.e.,)

Horizontal loads $= 1.46 \sin\theta$

$w_x = 1.46 \cos 26°$

$= 1.312$

$w_y = 1.46 \sin 26°$

$= 0.64$

Bending moment,

$M_x = \dfrac{w_x l^2}{8} \Rightarrow \dfrac{1.312(4.5)^2}{8}$

$\boxed{M_x = 3.321\ kN\text{-}m}$

$M_y = \dfrac{w_y l^2}{8} \rightarrow \dfrac{0.64(4.5)^2}{8}$

$\boxed{M_y = 1.62\ kN\text{-}m}$

Shear force, $V_x = \frac{w_x l}{2} \Rightarrow \frac{1.312 \times 4.5}{2}$

$$\boxed{V_x = 2.95 kN}$$

$V_y = \frac{w_y l}{2} \Rightarrow \frac{0.64 \times 4.5}{2}$

$$\boxed{V_y = 1.44 kN}$$

let us try ISMC 125 [channel sections are used for trusses & the purlins].

$$\boxed{Z_{px} \text{ required} = \left(\frac{M_x}{f_y} \times \gamma_{m0}\right) + 2.5 \frac{d}{b} \cdot \frac{M_y}{f_y} \cdot \gamma_{m0}}$$

$$Z_{px} = \left(\frac{3.321}{250} \times \cancel{1.10}\right) + 2.5 \cancel{\frac{65}{125}} \times \cancel{\frac{1.62}{250} \times 1.10}$$

where,

$d = h - 2(t_f + r_1)$

$$\boxed{d = 89.8 mm}$$

$$Z_{px} = \left(\frac{3.32 \times 10^6}{250} \times 1.10\right) + 2.5\left(\frac{89.8}{65}\right) \times \frac{1.62 \times 10^6}{250} \times 1.10$$

$= 14608 + 3.45 \times 7128$

$= 39.19 \times 10^3$ N-mm

$\Rightarrow 39.19\ kN\text{-}m$

Z_p for ISMC - 125 = 77.15 cm^3

Check for moment :-

$$M_{dx} = \frac{\beta_p \cdot Z_{px} \cdot f_y}{\gamma_{m0}}$$

$(\because \beta_p = 1)$

$$\frac{b}{t_f} = \frac{65}{8.1} = 8.02 < 9.4\epsilon$$

$$\therefore M_{dx} = \frac{1 \times 77.15 \times 10^3 \times 250}{1.10}$$

$$\therefore \boxed{M_{dx} = 17.53\ kN\text{-}m}$$

$$M_{dy} = \frac{\beta_p \cdot Z_{py} \cdot f_y}{\gamma_{m0}}$$

$$Z_{py} = \frac{b^2 \cdot t_f}{2} \Rightarrow \frac{65^2 \times 8.1}{2}$$

$$Z_{py} = 17111.25\ mm^3$$

$$\therefore M_{dy} = \frac{1 \times 17111.25 \times 250}{1.10}$$

$$= 3.88\ kN\text{-}m$$

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} \Rightarrow \frac{3.32}{17.53} + \frac{1.62}{3.88}$$

$$= 0.61 < 1$$

$\therefore$ Hence ok

Check for DL & WL action:-

$DL = 0.296\ kN/m^2$

$= 0.296 \times 1.4$

$= 0.4144\ kN/m$

factored load $= 1.5 \times 0.4144$

$= 0.6216\ kN/m$.

$WL = 1.2\ kN/m^2$

$= 1.2 \times 1.4\ kN/m$.

$= 1.68\ kN/m$

factored load $= 1.5 \times 1.68$

$= 2.52\ kN/m$

Factored D.L normal to sheeting:-

Factored D.L $= 0.6216 \cos\theta$

$= 0.6216 \times \cos 26^{\circ}$

$= 0.558 \, kN/m$

$\omega_x = 0.55 -$ factored load

$= 0.55 - 2.52$

$\boxed{\omega_x = -1.96 \, kN/m}$ (– indicates section)

$\omega_y = 0.6216 \sin\theta$

$= 0.6216 (\sin 26^\circ)$

$= 0.27 \, kN/m$

Bending moment :

$M_x = \frac{\omega_x l^2}{8} \Rightarrow \frac{1.96 \times 4.5^2}{8}$

$= 4.96 \, kN\text{-}m$

$M_y = \frac{\omega_y l^2}{8} \Rightarrow \frac{0.27 \times 4.5^2}{8}$

$= 0.68 \, kN\text{-}m$

From table (14)

$\Rightarrow \frac{h}{t_f}$

$\Rightarrow \frac{125}{8.1}$

$= 15.43$

$KL/r_y = \frac{1.2 \times 4500}{19.2} = 281.25$

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$h/t_f = 15.43$

~~14~~

	14	16
280 –	126.9	111.9
290 –	122.3	107.8

$$\Rightarrow 126.9 + \left(\frac{111.90 - 126.90}{16-14}\right) \times (15.43 - 14)$$

$$\boxed{x = 116.17}$$

$$\Rightarrow 122.3 + \left(\frac{107.8 - 122.3}{16-14}\right) \times (15.43 - 14)$$

$$\boxed{y = 111.93}$$

$f_{bd} \Rightarrow 280 - 116.17$

290 – 111.93

281.95 – ?

~~fbd from table 13(a):~~ (~~pg:~~

$$f_{bd} = 116.17 + \left(\frac{111.93 - 116.17}{290 - 280}\right) \times (281.25 - 280)$$

$= 115.64$

f_{bd} from table 13(a) :- pg (55)

100 - 77.3

150 - 106.8

115.64 - ?

$$\Rightarrow 77.3 + \left(\frac{106.8 - 77.3}{150 - 100}\right) \times (115.64 - 100)$$

$= 86.52$

$M_{dx} = \beta_p \cdot Z_p \cdot f_{bd}$

$= 1 \times 77.15 \times 10^3 \times 86.52$

$= 6.672$ kN-m

Combined Section:-

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} \Rightarrow \frac{4.96}{6.672} + \frac{0.68}{3.88}$$

$= 0.91 < 1$

$\therefore$ Hence ok