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PERAMBALUR-621212, TAMILNADU, INDIA.
Website: www.dsengg.ac.in

DEPARTMENT OF CIVIL ENGINEERING

U23CET62 – DESIGN OF STEEL STRUCTURES

HAND WRITTEN NOTES

PREPARED BY

MRS. R.JEEVIKA

ASSISTANT PROFESSOR

UNIT I SECTIONS AND JOINTS

Types of steel structures – Properties of rolled steel sections and Light gauge steel sections – Allowable Stresses as per IS code - Riveted and bolted connections – Failures of joints – Single and multiple riveted lap and butt joints under axial and eccentric loading – Strength of fillet and butt welded joints –Design of riveted and welded joints.

UNIT II TENSION MEMBERS

Types of sections – Net area – Net effective sections for angles and Tee in tension – Design of connections in tension members – Use of lug angles – Design of tension splice – Concept of shear lag.

UNIT III COMPRESSION MEMBERS

Types of compression members – Theory of columns – Basis of current code provision for compression member design – Slenderness ratio – Design of single section and compound section compression members Design of laced and battened type columns – Design of column bases Gusseted base.

UNIT IV BEAMS

Design of laterally supported and unsupported beams – Built up beams – Beams subjected to uniaxial and biaxial bending – Design of plate girders - Intermediate and bearing stiffeners – Flange and web splices.

UNIT V ROOF TRUSSES AND INDUSTRIAL STRUCTURES

Roof trusses – Roof and side coverings – Design of purlin and elements of truss; end bearing Design of gantry girder

Design of Steel Structure

Unit - 1 Sections and joints.

Introduction to steel & steel structure:-

Structural steel is one of the materials which used for any kind steel construction, It is formed with a specific shape

These steel materials are of certain standards of chemical composition & proper strength.

All across the world, There is an increasing demand for steel structures.

There is a big advantage of steel over the concrete in terms of its ability to bear better tension as well as compression, which resulted in lighter construction.

Advances :-

Currently undergoing a major evolution, driven by the dual pressures of environmental sustainability & need for rapid, efficient construction.

Advantage :-

1) high strength.

2) Element size - small / saving space

3) Quality, high durability.

4) speed of construction.

5) Welding - add - strengthening the structural.

6) bolt connection - dismandle / replace

7) Re-usable.

Dis - Advantage :-

1) Fire accident / deformed at high temp.

2) corrosion

3) Maintenance cost / Paint

4) Costly.

Types :-

1) Carbon + iron.

2) chemicals added -> Variety of steel Produced

-> high Carbon - high strength.

-> high sulphur Phosphorus (0.06) beyound - brittleness.

-> chrome + nickel - Corrosion resistance

-> high copper - higher corrosion resistance

Propertier of steel :-

1) Density : mass per unit volume.

Structural steel has density of 7750-2050kg/m3.

2) Elastic Modulus :- Measurement of tendency of an object to be deformed when force or stress is applied on it.

Elastic modulus of all steel classes is same and equal to 2×10^5 MPa.

3) Poisson's Ratio : It is the ratio of contraction and elongation of the material Lower the value, lesser the object will shrink in thickness when stretched.

Acceptable value for structural steel are - 0.27 to 0.3.

4) Tensile strength : It is the determination of the limit of the object can be stretched without breaking.

Fracture Point - is the point at which the object breakdown after application of stress.

High tensile - steel.

5) Melting Point: steel often melt at 1370°c.

6) Hardness: It offers resistance to the indentation and scratching. testing:- Brinnell, Rockwell

7) Ductility: Permanent deformation without loss of strength under the application of tensile load.

8) Malleability: Their ability to be hammered pressed or rolled into thin sheets without breaking.
(Deform under compression)

9) Fatigue: Damage caused by the repeated fluctuation of stresses leads to the progressive cracking of the structural element. (cyclic loading).

10) Toughness: offers resistance to fracture under the action of impact loading.

Classification of steel :-

1) mild steel.

2) High tensile steel.

Steel

- Carbon steel
  - Low carbon / mild steel
  - Medium carbon steel.
  - High carbon steel.
- Alloy steel
- Stainless steel
  - Ferritic
  - Martensitic
  - Austenitic
  - Duplex
  - Precipitation hardening.
- Tool steel.

Carbon steel! Defined primarily by its carbon content. with only trace amount of other elements.

Low carbon - 0.04% to 0.30% carbon
car body Panels, ductile nature

Medium Carbon - 0.31% to 0.60%
gears, axles & railway tracks

High Carbon - 0.61% to 1.50%
hard + wear resistant
springs, blades, wires.

Alloy steel :- Contains additional elements, like nickel, chromium or molybdenum (50% by weight). toughness & corrosion resistance.

Stainless steel : Contains at least 10.5% chromium. It forms a protective layer) oxides that makes it highly resistant to rust.

- → Austenitic (non-magnetic)
- → Ferritic (magnetic)
- → Martensitic (hardenable).

Tool steel : Formulated with tungsten, molybdenum, and cobalt for extreme hardness & heat resistance.

Cutting, drilling & machining tools.

The structural steel :

It is the steel used for the manufacturing of rolled structural steel sections, fastening and other elements for use in structural steel works.

Alloy of iron >98% + Carbon (0.1% to 1.1%) + Sulphur, Phosphorus, silicon, manganese, Nickel → Steel.

Mild steel is used for the manufacture of rolled structure steel sections, rivets + bolts.

⇒ Cutting, ⇒ Punching, ⇒ Drilling.
⇒ Machining, ⇒ welding, → forging when heated, ~~&~~

standards.

IS: 226-1975	structural steel	(std quality)
IS: 1977-1975	structural steel	(ord. quality)
IS: 2062-1984	Weldable structural steel.	
IS: 961-1975	structural steel (high tensile)	
IS: 8500-1977	Weldable structural steel. (medium & high strength qualities)	

⇓

Fe-440, Fe-540, Fe-510, Fe-590, Fe-640.

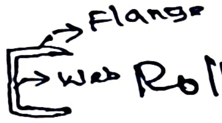
Rolled structural steel section:-

steel section manufactured in rolling mills and used as structural members are known as Rolled structural steel sections.

The steel sections are named according to their cross-sectional shapes.

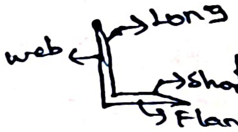
"ISI Handbooks for structural Engineers" given nominal dimensions, weights and geometrical properties of various rolled structural steel sections.

Types! Rolled steel I-section

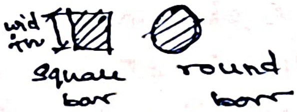


Rolled steel channel section.

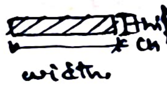

Rolled steel Tee section



Rolled steel Angles section

Rolled steel bars



Rolled steel Tubes.



Rolled steel Flats.

Rolled steel sheets + strips

Rolled steel plates

Indian Standards !-

ISJB - Joist Beam.

ISLB - Light Beam.

ISMB - Medium Weight Beam

ISWB - Wide flange Beam.

ISHB - Heavy Weight Beam.

Loads on structures :-

The structure + structural member are designed to meet the functional + structural aspects. Both aspects are Interrelate

The functional aspect takes in to consideration the purpose for which the building is designed.

Types of load :-

1) Dead Loads, (IS 875-1987-I)
2) Live or imposed Load. (IS 875-1987-II)
3) Wind Load, (IS 875-1987-III)
4) Snow Load, ($25 N/m^2$)
5) Seismic Load. (IS 1893-1962)
6) Temperature effects.

The following forces + effects are also considered While designing.

1) Vibration,
2) fatigue,
3) Impact,
4) Erection Load,
5) stress concentration
6) soil + fluid Pressure,
7) Foundation movement.
8) Elastic axial shortening.

Load Combination:-

1) D.L + L.L ,
2) D.L + L.L + Wind or Earth quake Load.
3) Dead Load + wind or Earthquake Load.

Steel → Tension ≃ compression.

Concrete → Tension = $\frac{1}{10}^{th}$ of compression.

Factor of safety: Indicating how much ~~stronger~~ the structure is than required for expected loads, providing a safety margin against unexpected loads.

$$FOS = \frac{\text{Ultimate strength (yield)}}{\text{Allowable stress (Permissible)}}$$

greater FOS → Larger C/S member.

small FOS → appreciable saving in the material.

Working stress method → 1.7 to 1.8

Limit State method → 1.15.

Stress :-

When a structural member is loaded, deformation of the member takes place and resistance set up against deformation.

This resistance to deformation is known as stress.

$$\sigma = F/A$$

Nature of stress developed depends on nature of loading.

Types :-

1) Axial stress (direct stress)
   i) Tensile stress
   ii) Compressive stress.

2) Bearing stress.

3) Bending stress.

4) Shear stress.

A member may be subjected to combine axial + Bending stress, such stress is known as combined stress.

Tensile (+), compressive (−).

% Elongation :-

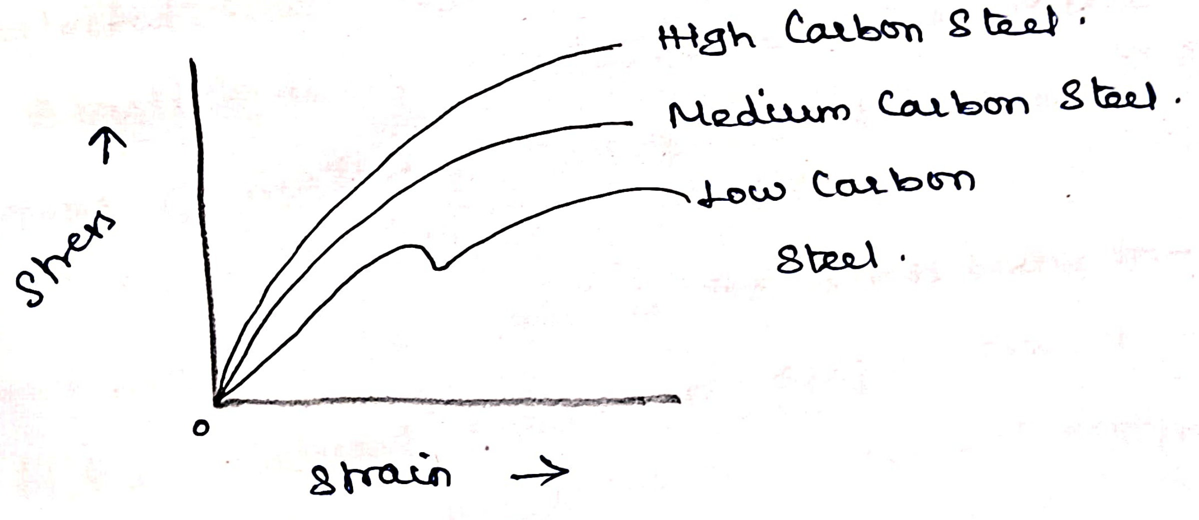
$$\% \text{ Elongation} = \frac{\delta L}{L} \times 100.$$

elongation ⇒ 15 % ductile material.

⇒ 5-15 % Intermediate material

→ less than 5% Brittle material.

Stress & strain curve



When carbon content increase.

↑ { Yield strength (fy) increase
Ultimate strength (fu) increase
Hardness of steel increases.

↓ { Ductility decrease
Toughness of steel decrease

Design Methods

- Working Stress Method
- Ultimate Strength Method
- Limit State Method.
  - Limit State of Strength
  - Limit State of Serviceability.

Working Stress Method:

⇒ Traditional Method, Structural material behaves in a elastic Manner.

⇒ Limitations due to linearity + buckling are neglected.

⇒ Elastic design: Which assumes elastic behaviour of material throughout the service Load range

⇒ Elastic Limit: It is the stress below which the material regain its original size + shape when load is removed. In steel design, It is taken as yield stress.

⇒ Stress at which the material starts to yield is taken as Permissible stress of the section. All sections are designed not to exceed the Permissible stress. $= \dfrac{\text{Yield stress}}{\text{FOS}}$.

Limitations:

Material - non linearity

No allowance for redistribution of loads in statically indeterminate structure

Ultimate load Method:

Permissible load is a load when all the fiber in the steel is yielded

This method does not ensure serviceability

Limit state Method. (IS 800-2007 - Page 27)

In the limit state method, the structure could be designed to withstand safely all loads likely to act on it throughout its life.

The acceptable limit for the safety & serviceability requirements before failure occurs is called limit state.

It uses a multiple safety factors, that attempts to produce adequate safety at ultimate load as well as adequate serviceable at service loads.

1) Limit state of strength :

⇒ Limit state is prescribed to avoid the collapse of the structure which may endanger the safety of life & includes.

⇒ Loss of equilibrium of the structure

→ Loss of stability

⇒ Failure by excessive deform.

⇒ Fracture due to fatigue

→ Brittle fracture these are maintain.

Limit state of strength found FOS members in tension, compression, flexure & shear.

Limit state of serviceable :-

It includes.

⇒ The deformation & deflection adversely affecting the appearance or effective use of the structure

⇒ Vibration in structure or any part of its component limiting its functional effectiveness.

⇒ Repairable damage or crack due to fatigue

→ corrosion, → fire.

2 main Partial safety factors are there

For loading → γ_p (table 4)

For material → γ_m (table 5).

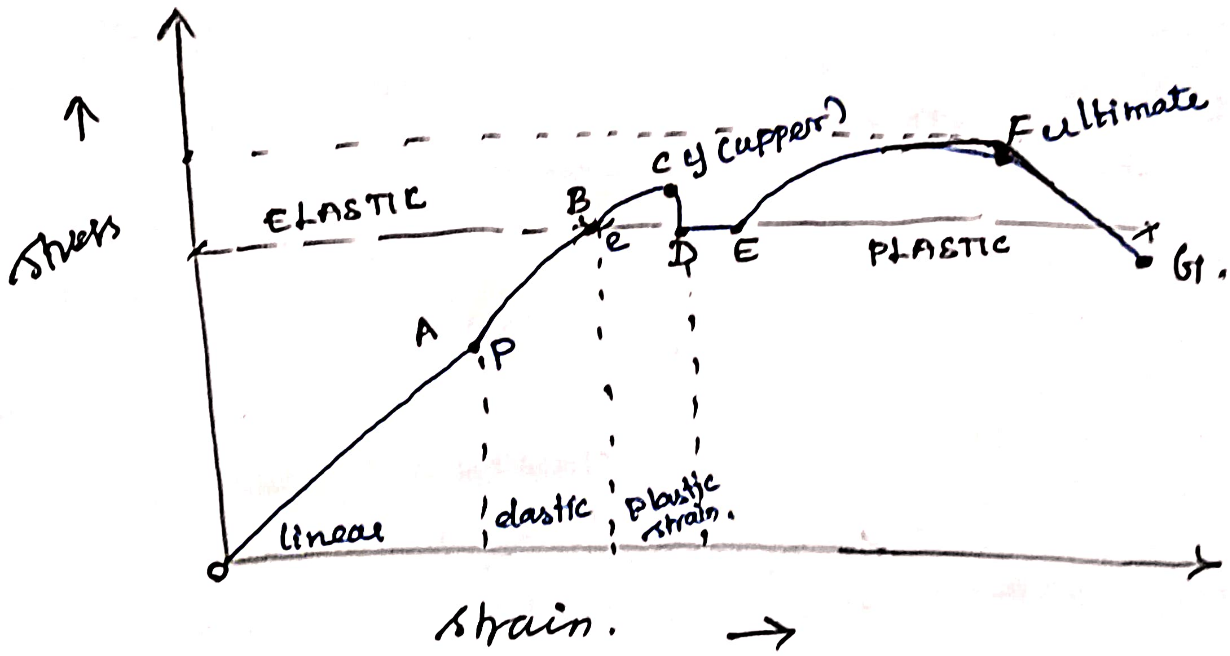
$$\text{Design strength} = \frac{\text{Ultimate strength}}{\gamma_m}.$$

Based on analysis :-

Design force - Axial + Shear force

Design moment - Bending + twisting.

Stress - strain curve for mild steel:



A - Proportionality limit

A-B - Elastic limit.

C - Upper yield limit

D-E - Lower yield limit

F - Ultimate stress point.

G - Breaking stress point.

CONNECTIONS.

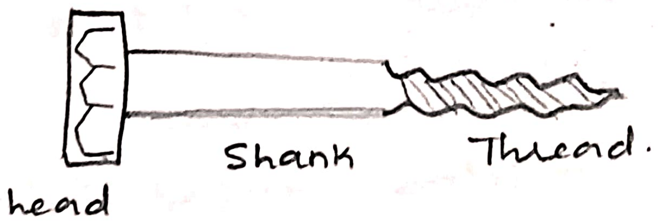
1) Bolt connection, 2) Weld connection.

3) Rivet connection.

Bolt Connection:

A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut.

Bolts are used for joining pieces of metals by inserting them through holes in the metal and tightening the nut at the threaded ends.



Shank dia > Thread dia.

Classification:

Unfinished (Black) bolt.
Finished (turned) bolt.
High strength Friction grip (HSFG) Bolt.

Unfinished Bolt:

Made up of mild steel rod with square or hexagonal head. The shank is left unfinished (ie) rough as rolled.

Joints remain quite loose resulting into large deflection.

It used for light structure under static load such as truss, bracings

Finished bolt!

This is also made up of mild steel hexagonal rod, which are finished by turning to a circular shape.

Actual dia of bolts are kept 1.2 to 1.3mm larger than the nominal dia. As usual hole is kept 1.5mm larger than the nominal dia.

Need special methods to align bolt holes before bolting.

connection is more tight, results much better bearing contact b/w bolts + holes.

It is used in special jobs like connecting machine parts subjected to dynamic loading.

High strength bolt!

⇒ made up of high strength steel rod.

⇒ These bolt are tightened to proof load using calibrated wrenches. Hence they grip the member tightly.

⇒ In addition nuts are provided by using clamping devices. If joints is subjected to shearing load it is primarily resisted by frictional force b/w the member & washer.

⇒ shank of bolt is not subjected to any shearing, results into no-slippage in joint.

⇒ Hence bolts can be used to connect member subjected to dynamic loads.

1) Pitch of bolts (P)

c-c spacing of the bolt in a row, measured along the direction of the loads.

2) gauge distance (g)

distance b/w the 2 consecutive bolts of adjascent rows and measured in right angle to the direction of load.

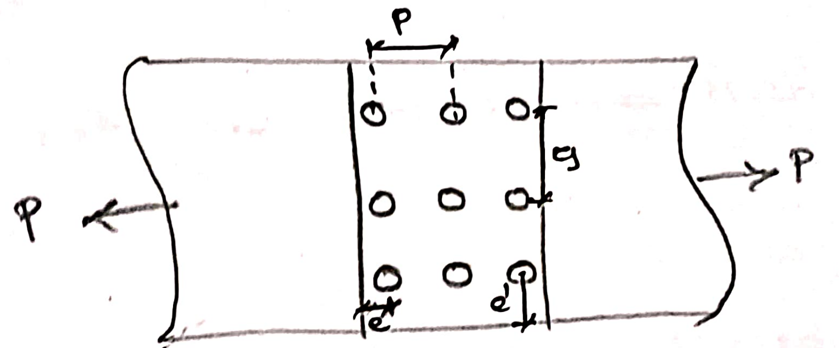
3) Edge distance :- (e)

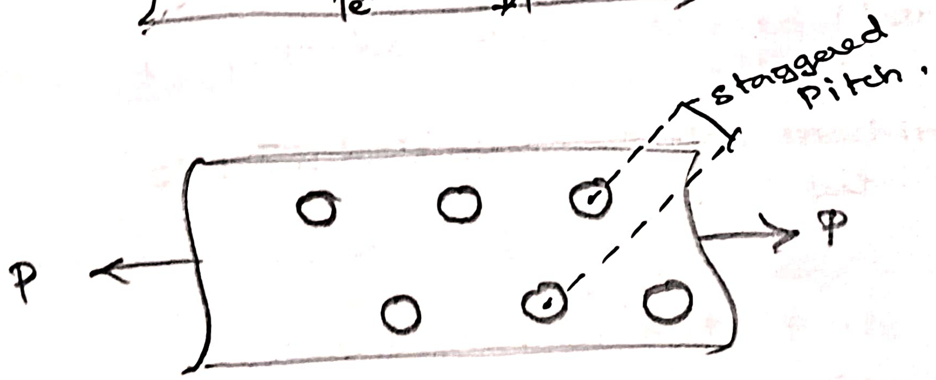
distance of the centre of bolt hole from the adjacent edge of the plate.

4) End distance : (e') distance of nearest bolt hole from the end of the plate.

5) Staggered distance :-

It is the c-c distance of staggered bolts measured obliquely on the member





As per IS - 800 - 2007 pitch + edge distance of bolt : (73-page).

Pitch : (P) shall not be less than 2.5 xd.

d - nominal dia. of bolt.

'p' not more than 16t / 200mm, which ever is less in case of tension member.

12t / 200mm, whichever is less, - Compression member

In case of staggered pitch, Pitch may be increased by 50% of value specified above provided the gauge distance is less than 75mm.

But joint max - Pitch restricted → 4.5d

Gauge:

Length should not be more than 100+4t / 200mm.

Edge:

min : 1) 1.7 x hole dia - Sheared / hand flame cut edges.

2) 1.5 x hole dia - rolled machine flame cut.

max : 1) $12t\varepsilon$, $\varepsilon = \sqrt{\frac{250}{fy}}$

t → thinner outer plate.

(2) 40+4t, t - Thinner plate

Fasteners:

→ Additional bolts provided.

max - 32t / 300mm.

- 16t / 200mm.

Joints:

Members made up of a flats, angles / tees or channels rivets to be provided along the length to connect

i) No exceed 1000mm, if it is a tension member

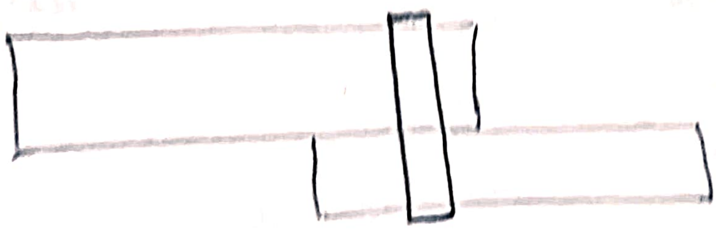
ii) No exceed 600mm, If it is a compression member.

Types: 1) Lap joint

2) Butt Joint.

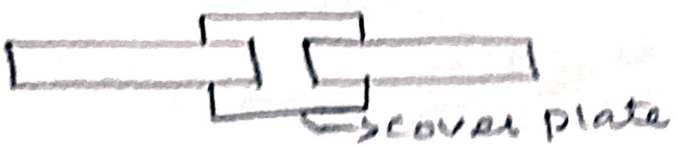
Lap joint :

It is the simplest types of joint, Plate to be connected overlap one another.



Butt joint :

Two main plates connect against each other, it is made by providing a single / double cover plate connected to the main plates.



Design Procedure for bolt Connection:-

(1) Shear
(2) Bearing Capacity } Least strength

Shear Capacity:-

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

V_{nsb} = nominal shear Capacity of bolt

γ_{mb} - Partial factor of material of bolt

V_{dsb} - design strength of bolt.

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

f_{ub} = Ultimate tensile strength of bolt

n_n - No. of shear planes with threads intercepting.

n_s - No. of shear planes without thread intercepting.

A_{sb} - Nominal shank area of bolt

A_{nb} - net shear area of bolt at threads.

$$A_{nb} = \pi/4 (d - 0.9382 P)^2.$$

$$A_{nb} = 0.78 \pi/4 d^2.$$

Reduction factor for shear capacity :- (RF)

The code suggest the use of reduction factor for shear capacity.

1) If the joint is too long.

2) grip length is large.

3) If the packing plates of thickness

1) RF for long joint :- (B_{lj})

If distance b/w First & last (l_j) exceeds 15d then reduction applied.

$[0.75 \leq B_{lj} \leq 1.0]$

$$B_{lj} = 1.075 - 0.005 \frac{l_j}{d}$$

2) Grip length : (B_{lg})

Thick of plate > 5 x d ; then reduced by

$$B_{lg} = \frac{8d}{3d + l_g}$$

(l_g = grip length

$l_g \ngtr 8d$

3) If paking path (B_{pk})

If plate thick more than 6mm, Then shear capacity is reduced by

$$B_{pk} = 1 - 0.0125 + pk$$

Finally then the formula $\frac{f_u}{\sqrt{3}}(n_n \times A_{nb} + n_s A_{sb}) B_{lj}\ B_{lg}\ B_{pk}$.

Bearing Capacity:

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

V_{dpb} - Bearing strength

V_{npb} - Nominal Bearing strength.

γ_{mb} - Partial safety factor of material.

$$V_{npb} = 2.5\, k_b\, d\, t\, f_u.$$

k_b, i) $e/3d_o$

ii) $P/3d_o - 0.25$

iii) f_{ub}/f_y

iv) 1.0

Dia of bolt hole:

Nominal dia of bolt (mm) d	12	14	16	20	22	24	30	36
Dia of bolt hole (mm) d_o	13	15	18	22	24	26	33	39
Outer dia of washer in (mm)	-	-	30	37	-	44	56	60.

bolt dia	hole dia.
< 14	+1
14 - 24	+2
> 24	+3.

Area !

A_{sb} = Area of shear bolt at shank $\frac{\pi}{4} d^2$

A_n = Area of bolt A_{nb} = $0.78 A_{sb}$.

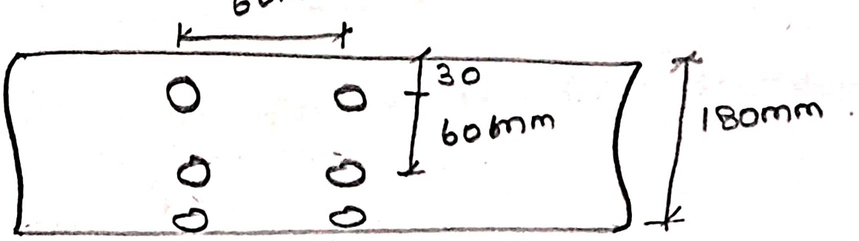
Properties!

Grade 4.6	f_{yb} = 240mpa	f_{ub} = 400Mpa
4.8	320Mpa	420Mpa
5.6	300Mpa	500Mpa
5.8	400Mpa	520Mpa.

$$\eta = \frac{\text{Strength of joint}}{\text{strength of solid plate}} \times 100.$$

Problem

1) Find the efficiency of Lap joint shown in fig. M20 bolt of grade 4.6 + Fe410 Plate are used!



Given!

for M20, grade 4.6 $d = 20mm$, $d_o = 22mm$, $f_{ub} = 400MPa$

$\gamma_{mb} = 1.25$; $b = 180mm$. $e = 30mm$, $P = 60mm$.

$g = 60mm$.

1) strength of Plate :-

$$T_{dn} = \frac{0.9 A_n f_y}{\gamma_{m1}}.$$

Gross Area = Total area of plate

Net area - After provided hole, the area of plate.

$$A_n = (b - n d_o) \times t$$

$$= 180 - (3 \times 22) \times 20$$

$$= 2280 \text{ mm}^2.$$

$$T_{dn} = \frac{0.9 \times 2280 \times 410}{1.25}$$

$$= 673056 \text{ N} \Rightarrow 673.06 \text{ kN}$$

2) shear strength of bolt:

For lap joint $n_s A_{sb} = 0$.

$n_n = 1$, $n_s = 0$, $\boxed{n_n = 1 \times 6 = 6}$ → [total no. of bolts]

Here no-reduction factor ie) $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$.

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

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

$$= \frac{400}{\sqrt{3}} [6 \times 0.78 \pi/4 \times 20^2]$$

($A_{nb} = 0.78 \pi/4 \, d^2$)

$$\boxed{V_{dsb} = 339.4 \text{ kN}}$$

$$V_{dsb} = \frac{339.48}{1.25} = 271.586\,kN.$$

③ Bearing Capacity of bolt:-

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

$$V_{npb} = 2.5\,k_b\,d\,t\,f_u.$$

k_b, i) $e/3d_0 = 30/3\times22 = 0.45$

ii) $P/3d_0 - 0.25 = 60/3\times22 - 0.25 = 0.65$

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

v) 1

$\therefore$ $\boxed{k_b = 0.45}$

$$V_{npb} = 2.5\times0.45\times20\times20\times410$$

$$= 186345\,N = 186.3\,kN.$$

$$V_{dpb} = \frac{186.3\,kN}{1.25} = 149.07\,kN.$$

design of strength of joint $= 6\times149.07 = 894.56\,kN.$

Least value of strength $= 271\,kN. < T_{dn}.$

④ Efficiency:

$$\text{solid plate} = \frac{f_y A_g}{\gamma_{mb}} = \frac{250\times180\times20}{1.1}$$

$$= 818.18\,kN$$

$$\eta = \frac{\text{Shear strength}}{\text{plate}}\times100. = \frac{271}{818.18}\times100$$

$$\boxed{\eta = 33.19\,\%}$$

2) Find the efficiency of joint, above problem instead of lap, butt joint is made using 2 cover plate size 12mm, & 6 No. of bolts on each side.

Sol:

$n_s = 6$.

Thickness of plate ⇒

1) On each side cover plate $= 2 \times 12 = 24$ mm.

2) Thickness of plate $= 20$mm.

Take min. value $= 20$mm.

① Tensile:

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

$$A_n = (b - n d_o) t$$

$$= (180 - 6 \times 22) 20$$

$$= 2280 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 \times 2280 \times 410}{1.25} = 673.06 \text{ kN.}$$

② Shear:

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

$$V_{nsb} = \frac{f_u}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}]$$

$$A_{sb} = \pi/4 \times d^2 = \pi/4 \times 20^2 = 314.16 \text{ mm}^2$$

$$A_{nb} = 0.78 \pi/4 \, d^2 = 0.78 \times \pi/4 \times 20^2 = 245 \text{ mm}^2$$

$$= \frac{400}{\sqrt{3}} [6 \times 245 + 6 \times 314.16]$$

$$V_{nsb} = 774.79.$$

$$V_{dsb} = \frac{774.79}{1.25} = 619.38 \text{ kN.}$$

③ Bearing : $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$

Same as Previous problem.

$V_{dpb} = 894.456 kN.$

Efficiency :-

$$\eta = \frac{619.38}{818.18} \times 100$$

$\eta = 75.76\%$

3) A boiler shell is made up of 14mm thick Fe415 plates. If the joint is double bolted lap joint with M16 bolts of grade 4.6 at distance of 50mm. determine the design strength of joint per pitch width. Is it a safe design. if the internal dia of bolt is 1m & steam Pressure is 1.2MPa?

Sol:

$d = 16 mm, \quad t = 14 mm, \quad p = 50mm,$

Pressure = 1.2MPa, internal dia = 1m.

$d = 16mm, \quad d_0 = 18mm, \quad f_u = 410Mpa.$

No. of bolt in double bolted joint per 50mm width. $n_n = 1$

Design strength:

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

$$A_n = (50 - 1\times 18)\times 14 = 448\,mm^2$$

$$T_{dn} = \frac{448 \times 0.9 \times 410}{1.25}$$

$$\boxed{T_{dn} = 132.25\ KN}$$

The lap joint, shear plane at shank $\boxed{n_s = 0}$

Two plate per pitch width $\boxed{n_n = 2}$.

Shear strength:

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

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right].$$

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

$$= 400/\sqrt{3}\left[2\times 156.89\right] = 72436N.$$

$$V_{dsb} = \frac{72436}{1.25} = 57.95\ KN.$$

Bearing:

(a) = $e/3d_o$ =

(b) : $p/3d_o - 0.25 = 50/3\times18 - 0.25 = 0.67$

(c) $f_{ub}/f_u = 400/410 = 0.97$

(d) 1

$$\boxed{K_b = 0.67}$$

$V_{npb} = 2.5\, k_B\, d\, t\, f_u$.

$= 2.5 \times 0.67 \times 14 \times 16 \times 410$

$\boxed{V_{npb} = 155.187\, kN}$

For 2 bolts $2 \times 155.187 = 310.37\, kN > V_{dsb}$

Strength = lowest value of above

$= 57.94\, kN$.

strength of bolt $<$ strength of plate.

Applied force $= \dfrac{P_r D}{2t}$ P_r = applied pressure, D = dia of boiler

$= \dfrac{1.2 \times 1000 \,(1m)}{2 \times 14}$

$= 42.85\, N/mm^2$

force per unit 50mm length = $42.85 \times 50 \times 14$

$= 30\, kN$.

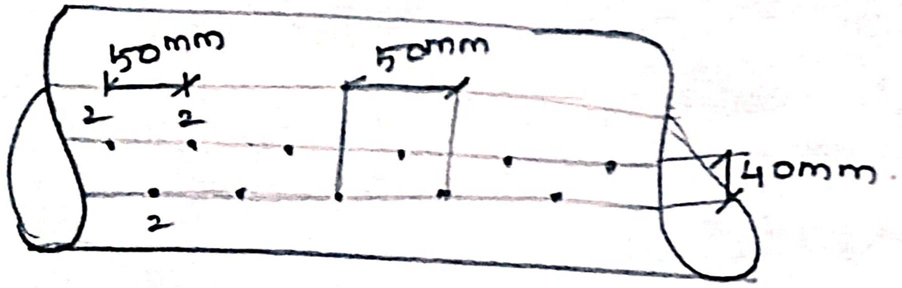
factor design action $= 1.5 \times 30$

$= 45\, kN$.

$57.94 > 45$

∴ safe design.

4. Check the safety of joint, example 3.3 zig zag bolting is used shown in fig.



Sol:

$d = 16mm$, $d_0 = 18mm$, $t = 14mm$,

no. of bolts = 2

Along section 1-1 & 2-2.

a) section (1)-(1) = $A_{n_1} = (b - n d_0)t$

$= [50 - (1 \times 18)] \times 14$

$= 448 \ mm^2$.

(b) section (2)-(2)

$A_{n_2} = (b - n d_0)t + \Sigma \frac{P s_i^2}{4 g_r}$

$= (50 - 2 \times 18)14 + \left[\frac{2 \times 40^2}{4 \times 25}\right] \times 14$

$= 644 mm^2$

Section (1)-(1) is weaker

$$T_{dn} = \frac{0.9 \times 448 \times 410}{1.25} = 132.25 kN.$$

Strength of bolt at 50mm width.

(1) Shear - 57.94kN

(2) Bearing - 155.18 kN

(3) design action = 45kN

57.94 > 45

Design is safe

5) Design the lap joint b/w the two plates each of width 120mm, If the thickness of one plate is 16mm other is 12mm. The joint has to transfer a design load 160kN. The plates are of Fe 410 grade. Use bearing type bolts.

Sol:

$d = 16mm$, $d_0 = 18mm$, $f_{ub} = 400 N/mm^2$, $f_y = 410$

Shear :- $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$, $V_{nsb} = \frac{f_{ub}}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]$

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

$V_{nsb} = 36.218 N.$

$V_{dsb} = \frac{36.218}{1.25} = 28.97 kN.$

min edge distance $e = 1.5 \times 18 = 27mm \simeq 30mm$.

min Pitch $P = 2.5 \times 16 = 40mm$.

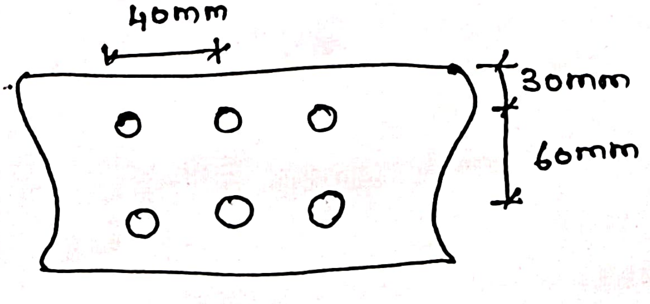
Bearing : $k_b = 0.49$.

$V_{npb} = 2.5 \times 0.49 \times 16 \times 12 \times 400$

$= 94.22 kN$

$V_{dpb} = \frac{94.22 kN}{1.25} = 75.38 kN.$

No. of bolts $= \frac{160}{75.38} = 5.5 \simeq 6$



$$T_{dn} = \frac{0.9 \times A_n \cdot f_y}{\gamma_m}$$

$$= \frac{0.9 \times (120 - 2\times18) \times 12 \times 410}{1.25}$$

$$T_{dn} = 297.56 \text{ kN} > 160 \text{ kN}.$$

Safe design.

6) Design a single bolt double cover butt joint to connect boiler plate of thickness 12mm for max efficiency. M16 bolts grade 4.6, Plates are Fe410 grade. n.l.

Shear :- $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

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

$d = 16mm$

$$= 82.65 \text{ kN}$$

$$V_{dsb} = \frac{82.65}{1.25} = 66.121 \text{ kN} \rightarrow (1)$$

Design stg of plate per pitch.

$$T_{dn} = \frac{0.9\, A_n f_y}{\gamma_{mb}}$$

$$= \frac{0.9 \times 410 \times 12 \times (P - 18)}{1.25}$$

$$= 3542.4\,(P - 18) \rightarrow (2)$$

equating (1) & (2)

$$3542.4(P - 18) = 66121$$

$$P = 36.67 \text{ mm}.$$

min Pitch $= 2.5d \Rightarrow 2.5 \times 16 = 40mm$.

Bearing:

$e/3d_0$, $P/3d_0 - 0.25$, f_{ub}/f_u, 1.0

$k_b = 0.49$

$$= \frac{2.5 \times 0.49 \times 16 \times 12 \times 400}{1.25} = 75.37 > 66.11$$

solid plate as per 40mm width

$$= \frac{250 \times 40 \times 12}{1.1} = 109.09 kN,$$

$$\eta = \frac{66.121}{109.09} \times 100 = 60.61\%.$$

Procedure for ecentric loading:

1) Line of action of ecentric load is in the Plane of group of bolts.

2) Line of action of the ecentric load is in the plane Perpendicular to plane of group of bolt.

3) Load acting some ecentric distance then ten moment will create.

$$M = P \times e$$

4) Calculate Resultant force:

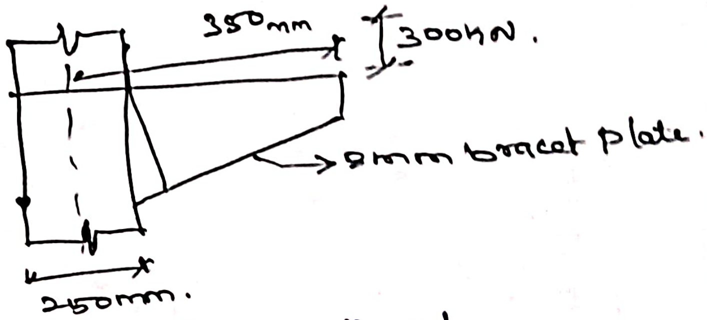
$$F = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}.$$

Where $F_1 = P/A$, $F_2 = K r_{max}$, $K = \frac{P \cdot e}{\Sigma r^2}$.

It may noted that fastenest bolt is subjected max force

Problem :-

1) A bracet is bolted to flange of column shown in fig. 8mm thick Plate. M20 bolt, grade 4.6 design connection.



shear:

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

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

$n_n = 1$

$n_s = 0$

$$V_{nsb} = \frac{400}{\sqrt{3}} [1 \times 0.78 \, \pi/4 \, 20^2]$$

$$= 56.59 \text{ kN}$$

$$V_{dsb} = \frac{56.59}{1.25} = 45.27 \text{kN}.$$

Bearing:

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

$$V_{npb} = 2.5 \, k_B \, d t f_u$$

$k_B = 0.5$

$$= 2.5 \times 0.5 \times 20 \times 8 \times 410$$

$$V_{dpb} = \frac{81.2 \text{kN}}{1.25} = 64.96 \text{kN}$$

Least value $= 45.27$ kN

$k_b \Rightarrow e/3d_0 = 50/3\times 22 = 0.83$

$p/3d_0 - 0.25 = 0.50$

$\frac{f_{ub}}{f_y} = \frac{400}{410} = 0.97$

1.0

$e = 1.5 d_0 = 1.5 \times 22 = 33 = 50$mm

$p = 2.5 d = 2.5 \times 20 = 50$mm

No. of bolts :- $n = \sqrt{\frac{6m}{VP}}$ $\boxed{m = P \times e}$

Assume arranged in rows.

$V \to$ strength value

$P \to$ Pitch.

$$n = \sqrt{\frac{6 \times 320 \times 10^3 \times 350}{2 \times 45.25 \times 50 \times 10^3}}$$

$$\boxed{n = 12}$$

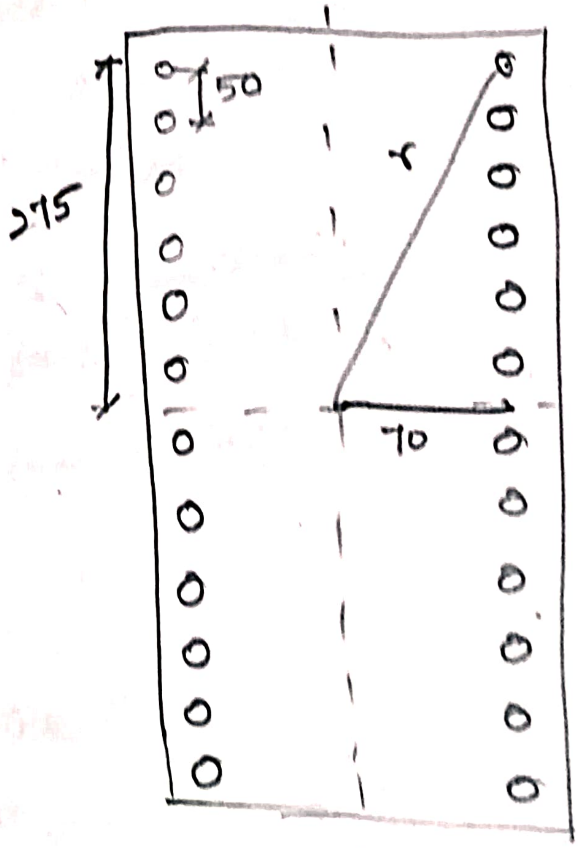
Check for strength

$$F = \sqrt{F_1 + F_2 + 2F_1F_2\cos\theta}$$

(2 rows of bolt)

$$F_1 = P/n = \frac{300}{24}$$

$$F_2 = kr$$



$r_1 = r_{12} = r_{13} = r_{24}$

$r_2 = r_{11} = r_{14} = r_{23}$.

$r_3 = r_{10} = r_{15} = r_{22}$

$r_4 = r_9 = r_{16} = r_{21}$

$r_5 = r_8 = r_{17} = r_{20}$

$r_6 = r_7 = r_{18} = r_{19}$

For example:-

$r_1^2 = 275^2 + 70^2$

$r_2^2 = 225^2 + 70^2$

$r_3^2 = 175^2 + 70^2$

$r_4^2 = 125^2 + 70^2$

$r_5^2 = 75^2 + 70^2$

$r_6^2 = 25^2 + 70^2$.

$\Sigma r_i^2 = 4(25^2+70^2) + 4(75^2+70^2) + 4(125^2+70^2) + 4(175^2+70^2) + 4(225^2+70^2) + 4(275^2+70^2)$.

$r_{max} = \sqrt{70^2+275^2} = 283.77$ mm

$\cos\theta = 70/283.72 = 0.24668$

$$k = \frac{300 \times 10^3 \times 350}{832600}$$

$F_2 = kr = 35786.5N$

$F_R = \sqrt{12.5^2 + 35.78^2 + 2(12.5 \times 35.78 \times 0.246}$

$F = 40.714 \, kN < 45.27 \, kN.$

Resultant force < strength.

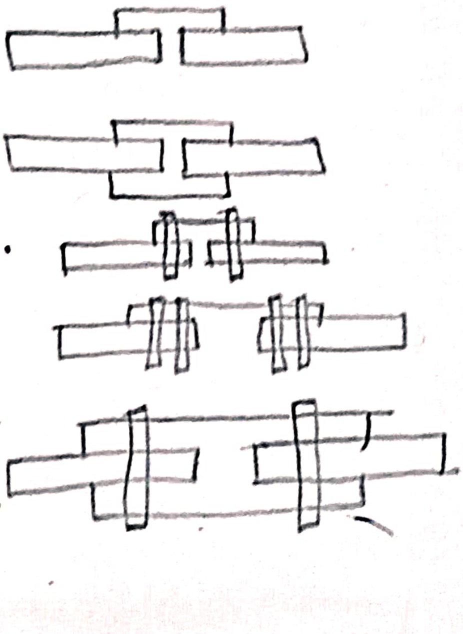
Hence the design safe.

Failure of Bolt connection!

1) Shear failure of bolt
2) Bearing failure of bolt
3) Bearing failure of plate
4) Tension failure of bolts
5) Tension / Tearing failure of Plates.
6) Block shear failure of bolt.

Types!

(1) Single cover Butt joint
(2) double cover Butt joint.
(3) single cover, single bolt joint.
(4) single cover, double bolt
(5) double cover, single bolt
(6) double cover, double bolt



Weld connection :

Welding is the method locally melting the metals with intensive heating along with a filler metal or without it and allowing cooling them to form a connect mass, thus creating a joint.

Advantage :

(1) Lesser weight,

(2) Improve strength

(3) Efficiency high

(4) Neat appearance

(5) More regid

(6) Contineuous structure used.

(7) Any shape of joint may make. (corner connection)

Disadvantage :

(1) Less fatigue strength

(2) Skilled labour

(3) unequal heating & cooling.

(4) No expansion & contraction space

Types :

(1) Butt / groove weld

(2) Fillet / Lap joint.

(3) Slot & plug weld.

Butl joint :

Depending upon the shape of the groove made for welding.

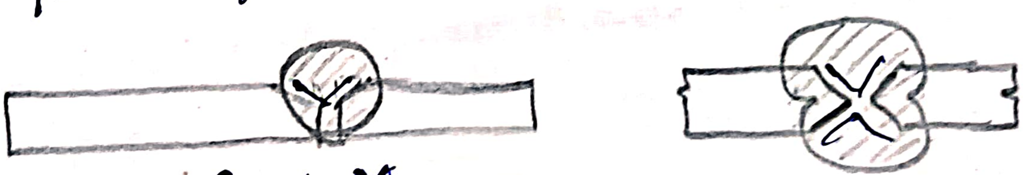
Types!

(1) Square Butt weld!

The square butt weld is a weld in the Preparation of which the fusion faces lie approximately at right angle to the surface of component to be joined and are substantially parallel to one another.

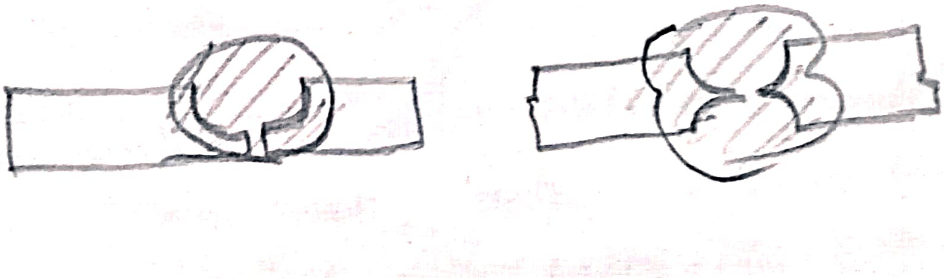
(2) Single V Butt weld!

A single V-butt weld is a weld in the preparation of which the edge of both Components are prepared so that in the c/s the fuction faces from a "V" as shown in fig.

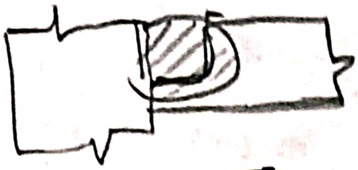


Single V

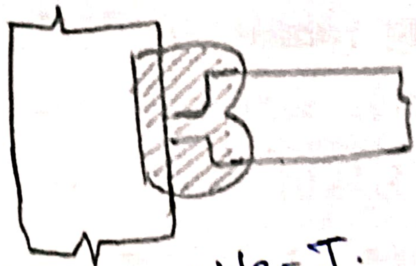
(3) U - Butt



J - Butt !



Single J.

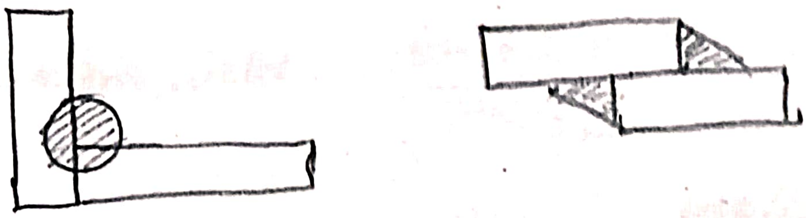


Double - J.

Fillet weld :

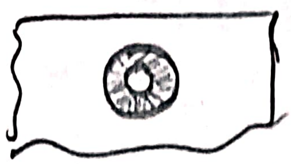
This type of weld is used when the member to be connected overlap each other.

It is approximately △ (triangular) cross section joining two surface approximately at right angle to each other.



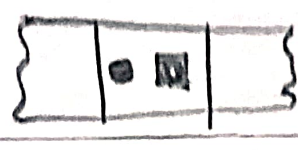
Slot

slot weld in which a plate with circular hole is kept with another plate to be joined & then fillet welding is made along the Periphery of the hole



Plug :-

Plug weld is small hole are made in one plate + kept over another plate to be connected and then "entire hole" is filled with filler material.



① A 18mm thick plate is joined to a 16mm plate by 200mm long butt weld. Determine the strength of joint if (i) a double V butt weld is used. (ii) a single V butt weld is used. Assume Fe 410 grade plate + shop weld are used.

Sol:

1) Double V - Butt:

$t = 16mm$, $L_w = 200mm$, $f_u = 410 N/mm^2$,

$\gamma_{mw} = 1.25 \rightarrow$ (Pg.No-30)

Effective area of weld = thickness × Length.

Design weld $= \frac{L_w \times t \times f_u/\sqrt{3}}{\gamma_m}$ (Pg No-79)

$= \frac{200 \times 16 \times 410/\sqrt{3}}{1.25}$

$= 605.98 KN$

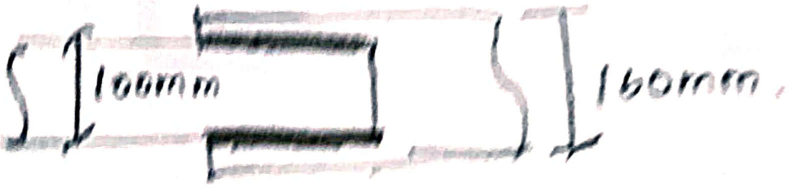
2) Single V Butt:-

$t' = 5/8 \times 16 = 10mm.$

$= \frac{L_w \times t' \times f_u/\sqrt{3}}{\gamma_{mw}} = \frac{200 \times 10 \times 410/\sqrt{3}}{1.25}$

$= 378.74 KN.$

4) Design a suitable longitudinal fillet weld to connect the plate shown in fig. to transmit a pull equal to full strength of small plates. plates are 12mm thick. grade of plate Fe410. welding to be made in workshop.



Sol:

min size = 5 mm (As per table -5), (Pg.No: 78, 79)

max = 12 - 1.5 = 10.5mm.

Use s = 10mm.

Thick of plate = 12mm, b = 100mm.

Design strength = $\frac{A_g f_y}{\gamma_{m0}}$ (table -21)

$= \frac{12 \times 100 \times 250}{1.1} = 272727N.$

$t = 0.7 \times 10 = 7mm.$

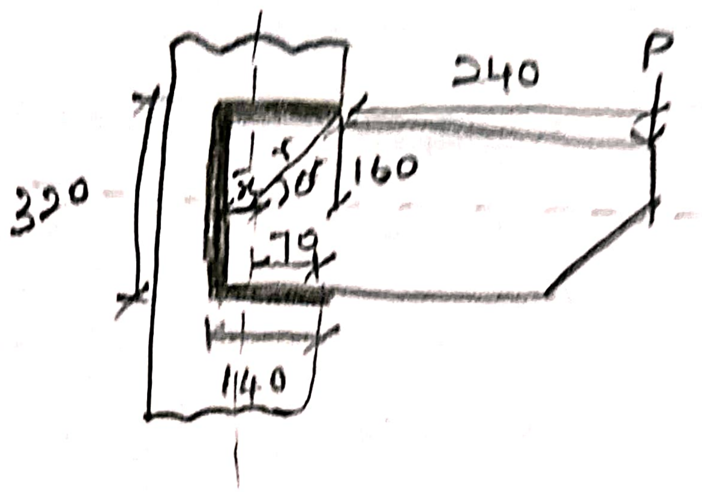
Design of weld $= l_w \times t \times f_y/\sqrt{3} \times 1/1.25$ $\left[\frac{l_w t f_u/\sqrt{3}}{\gamma_{mw}}\right]$

$l_w \times 7 \times \frac{400}{\sqrt{3}} \times 1/1.25 = 272727$

$l_w = 205.7mm.$

Provide effective length 205mm on each side.

Determine the max load that can be resisted by bracket by fillet weld of size 6 mm, if it is shop welding.



Sol:

Size of weld = 6 mm.

throat thick = 0.7 × 6 = 4.2 mm

Area of weld = (140 × 4.2 × 2) + (320 × 4.2)

= 600 × 4.2 mm^2

$$\bar{x} = \frac{140 \times 4.2 \times 70 \times 2}{600 \times 4.2} = 32.67 mm$$

$$\left[\frac{140 \times t \times \frac{1}{2} 140 \times 2}{\text{area of weld}}\right]$$

$$I_{xx} = \frac{bd^3}{12} + Ay^2$$

$$I_{xx} = \frac{4.2 \times 320^3}{12} + (140 \times 4.2)(160^2) \times 2$$

$$I_{xx} = 41574400 mm^4$$

$$I_{yy} = A\bar{x}^2 + 2\left[\frac{bd^3}{12} + A(\bar{x}_w - \bar{x}_p)^2\right]$$

$$I_{yy} = (320 \times 4.2) \times (36.27)^2 + 2\left[\frac{140^3 \times 4.2}{12} + [140(70 - 32.67)^2]\right]$$

I_{yy} = 4994080 mm^4.

$I_{zz} = I_{xx} + I_{yy}$.

$r_{max} = \sqrt{160^2 + (140-32.07)^2}$

$= 192.66 mm$.

$\tan\theta = 160/140-32.67$

$\theta = 56.15°$.

eccentricity $e = 240 + 140 - 32.67$.

$= 347.33$.

$F_1 = P/A = \dfrac{P \times 10^3}{600 \times 4.2} = 0.39 P \ N/mm^2$

$F_2 = \dfrac{P \times e \times r_{max}}{I_{zz}}$

$= \dfrac{P \times 10^3 \times 347.33 \times 192.66}{46568480}$

$= 1.43 P \ N/mm^2$

Force $= \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos\theta}$

$= P\sqrt{0.34^2 + 1.43^2 + 2 \times 0.39 \times 1.43 \times \cos 56°15'}$

$= 1.69 P$

Weld strength $= f_y/\sqrt{3} \times 1/1.25$.

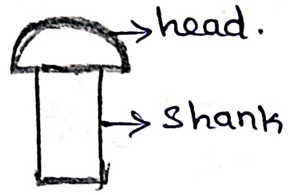
$= \dfrac{410}{\sqrt{3} \times 1.25} = 189.3 \ N/mm^2$

$1.69P = 189.3$

The max load is $\boxed{P = 112 kN}$

Rivet Connection

Riveting is a method of joining together pieces of metal by inserting ductile metal pin called rivet into hole of pieces to be connected and forming a head at the end of the rivet to prevent each metal piece from coming out



* head in to ↑ temp
* $1000^\circ F$ to $1850^\circ F$ before driving.

Disadvantage!

(1) High noise pollution.
(2) Need heating the rivet to red hot
(3) Skilled Labour
(4) Removing of rivet is costly.
(5) Labour cost is high

Problem

1) A double riveted lap joint with zig zag riveting is to be designed for 13 mm thick plate. Assume $\sigma_{at} = 80 MPa$, $\tau_{cf} = 60 MPa$, $\sigma_p = 120 MPa$, state how the joint will fail + find η of joint.

Sol:

1) $d_o = 6\sqrt{t} \Rightarrow 6 \times \sqrt{13}$
$= 21.03 \simeq 22 mm$

from dia = $\phi + 1.5$ mm (when $\phi \le 25$mm)

= $\phi + 2$ mm ($\phi > 25$mm).

gross dia = 22+1.5 = 23.5mm.

2) Pitch of rivet

tearing strength of bolt = shear strength of rivet

$\sigma_{at}(P-d)t = \tau_{vf} \times \pi/4 \, d^2$

$80(P-23.5)13 = 2 \times 60 \times \pi/4 \times 23.5^2$

$25P - 12222 = 2602.41$

$52P = 3804.4$

$P = 73.54mm \simeq 74mm$.

3) Check ! IS code (Pg. NO: 73)

min = 2.5d < P

max = 32t / 300 mm.

min ⇒ 2.5×23.5 = 58.75mm < Pitch.

max ⇒ 32×13 = 410 or 300 mm > Pitch.

Hence ok.

4) Failure of joint :

Tensile strength of Plate = $\sigma_{at} \times (P-d) \times t$

$= 60 \times (74-23.5) \times 13$

= 52520 N.

2) Shearing strength of rivet = $2 \tau_{vf} \pi/4 \, d^2$

$= 2 \times 60 \times \pi/4 \times 23.5^2$

$= 52.48 \, N$

3) Bearing $= 2 \sigma_p \, dt$

$= 2 \times 120 \times 23.5 \times 13$

$= 73320 \, N.$

∴ the joint will fail under shear, the least value

5) Efficiency :

$$\eta = \frac{\text{strength of joint}}{\text{strength of plate Area}} \times 100$$

$$\left[\frac{52048}{\sigma_{ap} Pt} \times 100 \right]$$

$$= \frac{52048}{80 \times 74 \times 13} \times 100$$

$$\boxed{\eta = 67.62\%}$$

2) A 16mm thick plate joint by double was butt joint using 10mm thick cover plate. The permissible tensile strength of plate is 150MPa, Determine the strength & efficiency of joint $p = 9$ cm, $d = 20$mm, shop rivet used $\sigma_{pt} = 300$, $\tau_{vf} = 100$MPa.

sol:

$\sigma_{at} = 150 \, N/mm^2$, $\sigma_{pt} = 300$, $\tau_{vf} = 100$

1) $d + 1.5 = 21.5$mm.

2) shear $= 2n \, \tau_{vf} \times \pi/4 \, d^2 = 2 \times 1 \times 100 \times \pi/4 \times 21.5^2$

$= 72610 \, N.$

3) Bearing :

$= n\,\sigma_{pt} \times d \times t$

$= 1 \times 300 \times 21.5 \times 16$

$= 103200\,N$

$$\eta = \frac{72610}{150 \times 90 \times 16} = 33.61\%$$

Unit - II
Tension Members

Tension !

Equal force applied in one plate at same time, while tension occurs.

IS 800-1984 – WSM

800 - 2007 – LSD

SP (6) – steel table

bolt dia	dia of hole
<14	+1
14-24	+2
>24	+3

gross area = Total area.

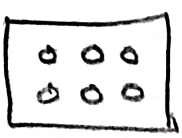
Net area = $(b - n d_o) t$ (After providing hole to plate)

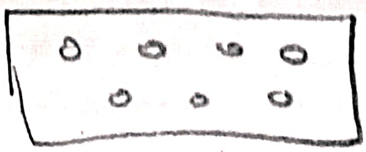
Pitch = along loading direction.

gauge = across loading direction.

Type!

1) chain, 2) zig-zag / staggered pitch.





$A_{ne} = \alpha A_n$

where, $\alpha = 0.6$ when no. of bolts ≤ 2

$= 0.7$ " " ≤ 3

$= 0.8$ " " ≥ 4

$= 0.8$ for welds

Tension member are linear member in which axial force act so as to elongate the member

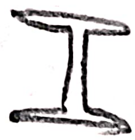
Tension member carry loads most efficiently, since the entire c/s is subjected to uniform stress. It is also known as tie member

Types of section !

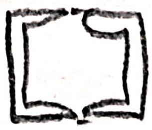
1) Round / Rectangular bar. 

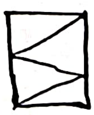
2) Cables composed of many small wire 

3) single + double angle. 

4) rolled W - & S-section. — 

5) structural Tee — 

6) Built-up box section 

* tension member are structural elements that are subjected to axial tensile force. 

Example! bracing for building + bridges, truss member wire, cable, etc..

Type!

1) wire + cable
2) Round + square rod
3) angle + Tee section
4) builtup section

Design Procedure:

1) Design strength due to yielding! (Pg.No: 32, 33)

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

$A_g \rightarrow$ gross area of C/S

$f_y \rightarrow$ yield stress

$\gamma_{mo} \Rightarrow$ Partial safety factor

2) Design strength due to Rupture

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

γ_{ml} = Partial safety factor

$A_n \rightarrow$ Net area

$f_u \rightarrow$ ultimate stress

$$A_n = (b - n d_o) t$$

If staggered pitch

$$A_n = \left(b - n d_o + \sum_i^m \frac{p_{si}^2}{4 g_i}\right) t$$

If single angles :-

$$T_{dn} = \frac{0.9 A_{ne} f_u}{\gamma_{ml}} + \frac{\beta A_g f_y}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076 \left(\omega / t\right)\left(f_y / f_u\right)\left(\frac{b_s}{L_c}\right) \leq 0.7$$

3) Block shear!

$$T_{db} = \left[A_{vg} f_y / \sqrt{3}\,\gamma_{mo} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}\right]$$

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{ml}} + \frac{A_{tg} f_u}{\gamma_{mo}}$$

A_{vg} = min gross area in shear.

A_{vn} = min net area in shear

A_{tg} = min gross area in tension.

A_{tn} = min net area in tension.

Determine the design tensile strength of plate 80mm × 12mm with holes for 16mm diameter bolt steel used in 415 grade

Sol:

$f_y = 250\ N/mm^2$, $f_u = 410\ N/mm^2$

1) Yielding : Pg. No : 32

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} \qquad A_g = 130 \times 12 = 1560 mm^2$$

$$T_{dg} = \frac{1560 \times 250}{1.1} = 354.54 kN.$$

2) Rupture :

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} , \qquad d = 16mm, \quad d_o = 18mm.$$

$$A_n = (130 - 2 \times 18) \times 12 = 1128 mm^2$$

$$T_{dn} = \frac{0.9 \times 1128 \times 410}{1.25} = 332.98 kN.$$

3) Block shear :

$A_{vg} = 2(e + p)t = 2(35 + 60) \times 12 = 2280 mm^2$

$A_{vn} = 2(e + p - 1.5 d_o)t = (35 + 60 - 1.5 \times 18) \times 10 \times 2$
$= 1632 mm^2$

$A_{tg} = e \times t = 60 \times 12 = 720 mm^2$

$A_{tn} = (g - d_o)t = (60 - 18) \times 12 = 504 mm^2$

$$T_{db} = \frac{2280 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 504 \times 410}{1.25} = 447.5 kN$$

$$T_{db} = \frac{0.9 \times 1632 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1} = 441.78 kN$$

strength of Plate = 332.98 kN (Least value considered)

Q) Determine the strength design tension strength of plate 160×8mm with holes for 16mm bolts. grade 415.

Sol:

(i) Yielding : $T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$

$= \frac{160 \times 8 \times 250}{1.1} = 290 kN$

(ii) Rupture, zig-zag connection } $A_n = [b - nd_0 + \sum \frac{p_s^2}{4g}] \times t$.

$b = 160$, $d_0 = 18mm$, $P = 40mm$.

$g = 25mm$.

(i) consider section (1-1) $A_n = (160 - 3 \times 18) \times 8$

$= 848 mm^2$

(2) Section 1-2-2-1

$A_n = [160 - 4 \times 18 + \frac{4 \times 40^2}{4 \times 25}] \times 8$ → no. of bolts.

$= 960 mm^2$.

(3) section 1-2-3-2-1

$A_n = [160 - 5 \times 18 + \frac{4 \times 40^2}{4 \times 25}] \times 8$

$= 1072 mm$

Take least area $A_n = 848 mm^2$

$T_{dn} = \frac{0.9 \times 848 \times 410}{1.25} = 250.3 kN$

∴ The strength of plate } = 250 kN.

Design of tension member (factored load given)

(1) Find gross area required $A_g = \frac{1.1 T_u}{f_y}$.

(2) Select suitable section depending upon types of structure

A_g = 25 to 40 %. (Extra of average required)

A_g = 1.25 to 1.4 times A_{reg} required.

(3) Determine the no. of bolts.

(4) Find the strength considering

(1) Strength in yielding (T_{dg})

(2) strength in rupture (T_{dn})

(3) strength in block shear (T_{db})

(5) Design tensile load > given factor load.

(6) Check slenderness ratio (Pg. No: 20)

3) Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225 kN. The member is subjected to the possible reversible of stress due to the action of wind. The effective length of member is 3 m (20 mm, 4.6 grade

Sol:

$T_u = 225$ kN, $d = 20$ mm, $f_y = 240$

$d_o = 22$ mm, $f_{ub} = 400$

1) $A_g = \frac{1.1 \times 225 \times 10^3}{240}$ $\left[\frac{1.1 \times T_u}{f_y}\right]$

$= 1031.2 \text{ mm}^2$.

2) Extra → 25%.

$\Rightarrow 1.25 \times 1031$

$= 1288\, mm^2$

Select section ISA 25×95×6mm.

$A_g = 1286\, mm^2$

3) No. of. Bolts.

Shear $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_y/\sqrt{3}\,[n_n A_{nb} + n_s A_{sb}]}{\gamma_{mb}}$

$V_{dsb} = \frac{56.59}{1.25} = 40.27\, KN.$

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

$V_{npb} = 2.5\, K_b\, d\, t\, f_u$

$K_b \Rightarrow e/3d_0 = 40/_{3\times22} = 0.6$

$\Rightarrow p/3d_0 - 0.25 = 50/_{3\times22} - 0.25 = 0.50$

$\rightarrow \frac{f_{ub}}{f_u} = \frac{400}{415} = 0.94$

$\Rightarrow 1$

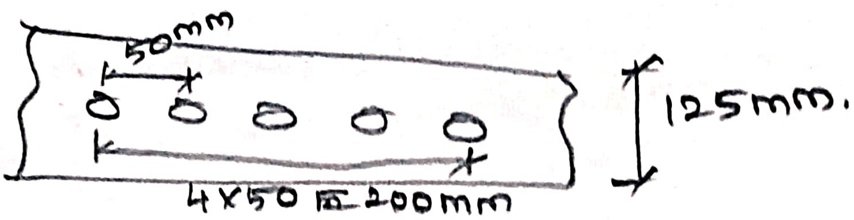
$K_b = 0.50$

$V_{npb} = 2.5 \times 0.50 \times 20 \times 6 \times 400$

$= 60.961\, KN$

$V_{dpb} = \frac{60.961}{1.25} = 48.76\, KN.$

No. of bolts $= \frac{225}{45.26} = 4.9 \simeq 5$



4) i) Yielding $T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$

$= \frac{1286 \times 240}{1.1} = 280\,kN$

(ii) Rupture $T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$

$A_{nc} = (L_c - d_o - t/2) \times t = (125 - 22 - 6/2) \times 6$

$= 600\,mm^2$

$A_{go} = (L_o - t/2) \times t = (95 - 6/2)6 = 552\,mm^2.$

$\beta = 1.4 - 0.076\,(w/t)\,(b_s/L_e)\,(f_y/f_u)$

$= 1.4 - 0.076\,(95/6)\,(164/200)\,(240/410).$

$b_s = w + w_1 - t = 95 + 75 - 6 = 164\,mm.$

$\beta = 0.822$

$T_{dn} = \frac{0.9 \times 600 \times 410}{1.25} + \frac{0.822 \times 522 \times 240}{1.1}$

$T_{dn} = 276\,kN.$

(iii) Block shear:

A_{vg} (shear) $= (40 + 4 \times 50) \times 6 = 1440\,mm^2$

$A_{vn} = (40 + (50 \times 4) - 4.5 \times 22) \times 6$

$= 846\,mm^2$

$A_{tg} = 50 \times 6 = 300\,mm^2$

$A_{tn} = (50 - 22/2)6 = 234\,mm^2.$

sub in $T_{db} = 250.4\,kN$

$T_{db} = 209.64\,kN < 225\,kN$

Design not safe, so increased pitch, gauge value, $p = 60mm$, $g = 65mm$.

$A_{tg} = 60 \times 6 = 360mm^2$

$A_{vg} = (40 + (60 \times 4) \times 6) = 1680\ mm^2$

$A_{tn} = (60 - 22/2) \times 6 = 294mm^2$

$A_{vn} = (40 + (60 \times 4) - (4.5 \times 22)) \times 6 = 1086mm^2$

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

$T_{db} = 298\ kN > 22.5\ kN$.

Hence safe.

Tension member splices:

If the single piece of required length is not available tension member are spliced to transfer required tension from one piece to the another. When tension member of different thick are to be connected filled plates are used to bring the member in level. If we are using filler plates while calculating strength in shear multiply the shear strength with Bpk.

$B_{pk} = (1 - 0.0125\ t_{pk})$ where t_{pk} = thickness of packing plate

Design a splice to connect a 300 x 12mm plate with a 300 x 10mm plate. The design load is 500kN, 20mm black bolt fabricated in the shop

Sol:

Let 6mm thick double cover butt joint,

Thick of packing plate = (12 − 10) = 2mm

1) Strength of bolt:

Shear: $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ ⇒ $V_{nsb} = \beta_{pk}[n_n A_{nb} + n_s A_{sb}]$

$\beta_{pk} = 1 - 0.0125 \times 2 = 0.975$

$V_{nsb} = 0.975[1 \times 0.78 \, \pi/4 \times d^2 + \pi/4 \, 20^2] \frac{400}{\sqrt{3}} = 125.9 kN$

$V_{dsb} = \frac{125.9}{1.25} = 100.73 kN$

Bearing:

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$ → $V_{npb} = (2.5 k_B d t f_u)$

Assume

$e = 40mm, \; P = 60mm.$

1) $e/3d_0 = 40/3 \times 22 = 0.6$

2) $P/3d_0 - 0.25 = 0.65$ $\boxed{k_b = 0.6}$

3) 0.975

4) 1.

$V_{npb} = 2.5 \times 0.6 \times 10 \times 20 \times 400)$

$= 121.2 kN.$

$V_{dpb} = \frac{121.2}{1.25} = 96.96 kN.$

Take least value 96.96kN.

2) No. of bolts: $\frac{500}{96} = 6$ Nos.

3) Yielding :-

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{300 \times 10 \times 250}{1.1} = 681.818\,kN > 500000N$$

rupture : $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$

$$= \frac{0.9 \times 2340 \times 400}{1.25} = 690kN > 500kN$$

4) Block shear!

$A_{vg} = (e+P) \times t = (40+60) \times 10 = 1000 mm^2$

$A_{vn} = (40+60 - 1.5 \times 22) \times 10 = 670 mm^2$

$A_{tg} = 220 \times 10 = 2200 mm^2$

$A_{tn} = (220 - 2 \times 22) \times 10 = 1760 mm^2$.

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{1000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2160 \times 410}{1.25}$$

$= 768.84 kN$.

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}.$$

$$= \frac{0.9 \times 670 \times 400}{\sqrt{3} \times 1.25} + \frac{2600 \times 240}{1.1}$$

$= 750N$

$T_{db} = 705kN > 500kN$

Hence safe.

Lug Angles:

1) Length of the end connector of a heavily loaded tension member may be reduced by using lug angles

2) By using lug angles there will be saving in gusset plate.

3) Lug angles are designed for 1.2 times load.

4) Lug angles are designed for 20% extra load (1.2 times) than the force in outstanding leg

5) Load on connected load is 40% extra

6) Also design connection of main angle with gusset plate + also length + width of gusset plate.

Procedure: (Connections with lug angles)

1) Gross area connected leg = gross area of outstanding leg, total load is sheared by 2 legs.

Load on connected load = Load on outstanding leg = $\frac{\text{Total Load}}{2}$

2) Lug angles are designed for Ag = 1.2 x load on outstanding leg.

$$1.2x = \frac{\text{total Load}}{2}.$$

3) Check the member for rupture

4) Find the no. of bolts required for single shear

5) No. of bolts $= \frac{1.2 \times TL}{2 \times \text{Bolt value}}$

6) Design forced on connected leg $= \frac{1.4 \times T.L}{2}$

7) No. of bolts $= \frac{\text{Total Load} \times 1.4}{2 \times \text{bolt value}}$

8.) Connection of gusset plate with main angle.

No. of bolts $= \frac{T.L}{2 \times BV}$

9) Length of gusset plate = Edge distance + No. of bolts + Pitch

<u>Problem</u>

1) The tension member of roof truss cover a factored axial tension load 430 kN, Design the section + its connection. (Bearing)

1) without using lug angle
2) with lug angles.

sol:

Case: 1 without lug angle.

1) $A_g = \frac{1.1 \times T.L}{f_y} = \frac{1.1 \times 430 \times 10^3}{240} = 1870\,mm^2$

2) Extra 25% Area = $1870 \times 1.25 = 2462\,mm^2$

∴ Provide $125 \times 85 \times 12\,mm$.

$A = 2498\,mm^2$

3) No. of Bolts :-

Shear → $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{400/\sqrt{3}\,(0.78\,\pi/4\,d^2)}{1.25}$

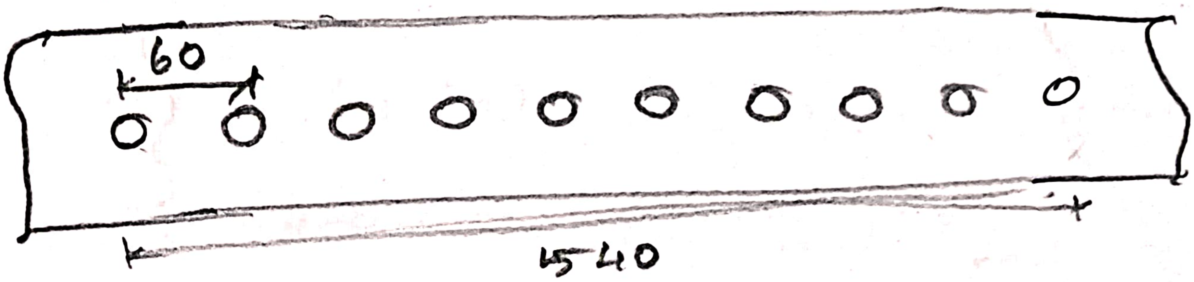
$V_{dsb} = 45.56\,kN.$

Bearing :- $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$ [$k_b = 0.6$] previous problem.

$= \frac{2.5 \times 0.6 \times 20 \times 12 \times 400}{1.25}$

$V_{dpb} = 116.35 kN$

no. of bolts $= \frac{430 \times 10^3}{45 \times 10^3} \simeq 10 Nos.$



$L_e > 15d.$

$L_e = 9 \times 60 = 540mm.$

$15d = 15 \times 20 = 300mm.$

∴ so it is long connections.

$\beta_{lj} = 1.075 - 0.75 (l_j/d)$ (Eq. No: 75)

$\beta_y = 1.075 - 0.0005 (l/d)$

$= 1.075 - 0.005 (540/20) = 0.5625.$

$V_d B_b = 0.56 \times 45.56 = 43.57 kN.$

No. of bolts $= \frac{430}{43.57} = 9.8 \simeq 10.$

So 10 bolts sufficient

4) Yielding !

$$T_d = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{2498 \times 240}{1.1}$$

$$= 545 > 430 KN.$$

Rupture !-

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

$$A_{nc} = (l_e - d_0 - t/2) t = (125 - 22 - 12/2) \times 12$$

$$= 1164 mm^2.$$

$$A_{no} = (l_0 - t/2) \times t = (95 - 12/2) \times 12 = 1068 mm^2$$

$$\beta = 1.4 - 0.077 (w/t)(f_y/f_u)(b_s/L_c).$$

$$\boxed{\beta = 1.297.}$$

$$b_s = w + w_1 - t$$

$$= 95 + 75 - 12$$

$$= 155 mm$$

$$T_{dn} = 640 KN > 430 KN$$

Hence OK

Block shear.

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3} \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$A_{tg} = g \times t = 50 \times 12 = 600 mm^2$$

$$A_{tn} = (50 - 22/2 \times t) = (g - d_0/2) \times t = 468 mm^2$$

$$A_{vg} = (40 + 60 \times 9) \times 12 = 660 mm^2$$

$$A_{vn} = (40 + 60 \times 9) - 9.5 \times 22) \times 12 = 4452 mm^2$$

↳ no. of bolts 10.

$$T_{db} = 1015 KN > 430 KN$$

at centre failure.

$$T_{db} = 889 > 430 KN$$

∴ Hence OK

(ii) Design of lug angles (with lug angle)

(1) Load on connected leg = Load on outstanding leg.

$430/2 = 215 kN$.

(1.2 time of load)

(2) Lug angle load designed $= 1.2 \times 215 = 258 kN$

(3) $A_g = \dfrac{1.1 \times 258 \times 10^3}{240} = 1180 mm^2$.

(4) ∴ Provide 80×50×10mm ISA $A_g = 1202 mm^2$

(5) From Previous calculation the bolt value is 45 kN.

(1.4 times load)

(6) Design force $= 1.4 \times 215 = 301 kN$

no. of bolts $= \dfrac{1.4 \times 215}{45} = 7$ Nos.

(7) main angle to gusset plate -

Force $= 215 kN$

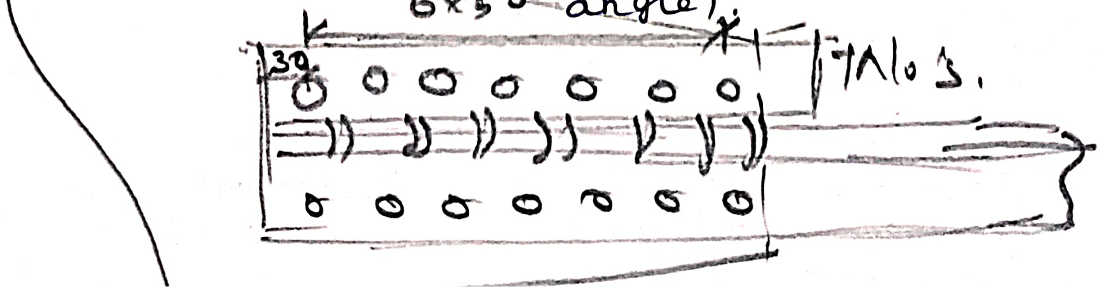
Bolt value $= 45 kN$

no. of bolts $= \dfrac{215 \times 10^3}{45} = 5$ No's.

Length ⇒ $30 + (7-1) 50 = 330 mm$

(540mm required without lug angle).

$6 \times 50 = 300 mm$



Shear Lag :-

(1) The tensile force is transferred from gusset to the Tension member (such as angles, Channel, T-Section) through one leg by bolt or welds

(2) In this process initially the connected leg may be subjected to more Stress than the outstanding leg & finally the stress than the Outstanding leg become uniform over the section away from the connection.

(3) Thus one part lags behind other, this is refered as shear lag.

(4) The non-uniform transfer of stress across the section away from cause non-uniform straining of the section due to which the section cannot be effectively utilized & fails under the min strength across the section instead of average

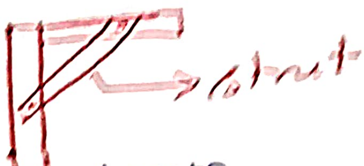
(5) This whole Phenomena is known as shear lag effect

3. Compression Member:-

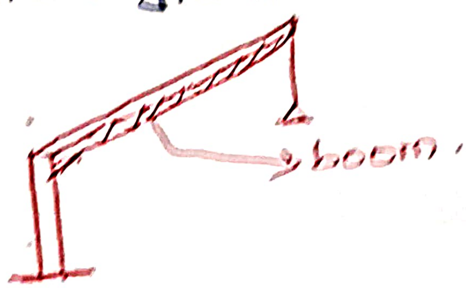
A structural member subjected to axially compression / compressive forces the member is called compression member.

ex :- in a building vertical compression member - Column.

in a roof truss the compression member - strut



in a crane - jib or boom.



failure occur in

1) short column due to crushing
2) Long column due to buckling.
3) both 1 & 2.

Slenderness Ratio:-

It is the ratio of effective length to the corresponding radius of gyration

$$\lambda = \frac{KL}{r_{min}}$$

$KL \rightarrow$ effective length

r_{min} - radius of gyration

Column end condition and Effective Length.

1) Both end hinged — $K = 1$, $l_e = L$

2) Both end fixed — $K = 0.65$, $l_e = 0.65L$

3) One end fixed & other end hinged — $K = 0.8$, $l_e = 0.8L$

4) One end fixed & other end free — $K = 2$, $l_e = 2L$

Short compression member:-

1) $L/r \leq 88.85$, for $F_y = 250$ MPa.

2) Failure to equal yield stress stress

3) No Buckling.

Long column:-

1) Buckle elastically.

2) Axial buckling stress is below Proportional limit.

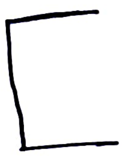
section used for compression member:-

section availability, Connection Problem,

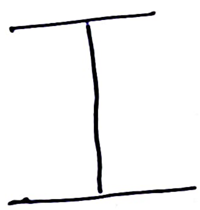
type of structure

single angle, double angle, Tee.

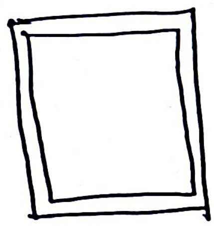
Channel



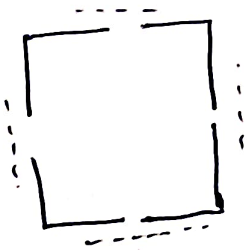
W column



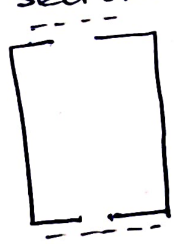
square/Rectangular HSS tubing



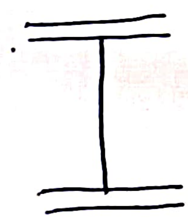
four angle box section



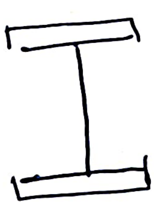
Box section



W with Cover Plate

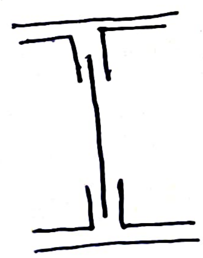


W and channel



Built up

Built up

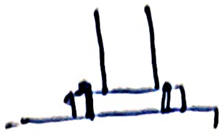


1) Design compressive strength

2) Design of angle strut (inclined compression member)

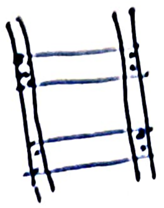
3) Design of compression member.
i) simple, ii) Builtup.

4) Design of slab base.
(1.7 → IS 800)



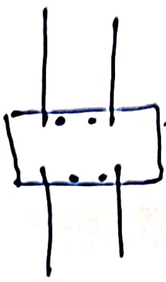
5) Design of laced column.
(49)

6) Design of Batten
(5? - IS 800)



7) Design of gusset base

8) Column splice.



Basic design procedure for compression member.

1) Assuming compressive stress (fcd)

1) Normal rolled steel section, Lacing, Batten } = 135 N/mm²

2) Angle struts = 90 N/mm²

3) Large load simple structure = 200 N/mm²

2) Calculate the required area:-

$P_d = f_{cd} \times A_e$ (Pg. NO: 34 IS800)

3) Selection of trial section using steel table.

ISHB Prefer.

4) Buckling class (use r_{min} - axis)

max - yy direction (Pg. No: 44, T-10)

Ex:- angle - c section

Builtup - c section

✓ calculate slenderness ratio. $\lambda = \frac{KL}{r_{min}}$

K = Value from (T-11, Pg. No: 45)

✓ find f_{cd} using kl/r_{min} - table 9 (b or c or d) based on buckling class.

Ex $f_y = 250$ ✓ if not given ✓

$Le/r_{min} = 29.5$ → $x = 29.5$ $y = ?$

$x_1 = 20$ $y_1 =$ table

$x_2 = 30$ $y_2 =$ table

$$\frac{x - x_1}{x_2 - x_1} = \frac{y - y_1}{y_2 - y_1}$$

⇒ $y = ?$ ⇒ f_{cd}.

first $P_d = f_{cd} \times A_e > P_d$ ✓

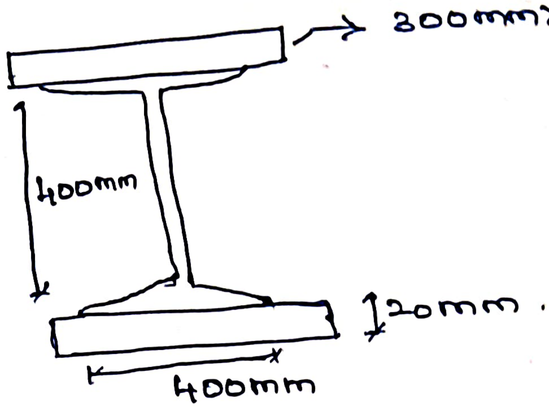
(or)

Having formula Pg. No: 34 (IS 800-2007)

5) Design extra requirement :- for laced or battern

$P_d = A_e f_{cd}$.

1) Determine the load carrying capacity of column section shown in fig. Length 4.5m. One end may be fixed and other end hinged $E = 250 N/mm^2$. $f_y = 250$



Sol:-

ISMB 400.

Refer steel table Pg. NO: 2

Step: 1

Properties of ISMB400.

$A = 1846 mm^2$, $I_{xx} = 20458.4 \times 10^4 mm^4$

$I_{yy} = 622.10 \times 10^4 mm^4$, $r_{max} = 16.15 cm$.

$r_{min} = 2.82 cm$,

Step: 2 Find I_{min}. Er

In the W section usually I_{yy} - is mininum value.

Plate → $I_{yy} = 2 \times \frac{b^3 d}{12}$

$= 2 \times \frac{(300^3 \times 20)}{12}$

$I_{yy} = 90 \times 10^6 mm^4$

I_{yy} for whole section

$I_{yy} = I_{yy}$ of W section + I_{yy} Plates

$= 6.22.10 \times 10^4 + 90 \times 10^6$

$\boxed{I_{yy} = 96.221 \times 10^6 mm^4}$

Step : 3 Slenderness ratio

$$\lambda = \frac{L_e}{r_{min}}$$

$L_e = K \times L \Rightarrow$ for one end fixed and another end hinged $\Rightarrow K = 0.8$

$$l_e = 0.8 \times 4500$$
$$= 3600\ mm.$$

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

A = Area of section + Area of Plate

$$A = 7846 + 2(300 \times 20)$$
$$A = 19846\ mm^2$$

$$r_{min} = \sqrt{\frac{96.22 \times 10^6}{19846}}$$

$$\boxed{r_{min} = 67.63}\ mm$$

$$\lambda = \frac{3600}{r_{min}} = \frac{3600}{67.63} = 51.7$$

Step : 4

Check for buckling claus :-

for section W with plates called builtup

buckling claus Pg. No: 44 (IS 800 - 2007)

c

Step : 5 To find Load :-

f_{cd} at 51.7 = y

$f_y = 250$, $r_{min} = 51.7$

$x = 51.7$, $y = ?$

$x_1 = 50$, $y_1 = 183$

$x_2 = 60$, $y_2 = 168$

$$\Rightarrow \frac{51.7 - 50}{60 - 50} = \frac{y - 183}{168 - 183}$$

$$\frac{1.7}{10} = \frac{y - 183}{-15} \Rightarrow y = 180.45\ N/mm^2$$

$f_{cd} = 180.45 N/mm^2$.

Pg. NO: 34.

$P_d = A \times f_{cd}$

$= 19846 \times 180.45$

$P_d = 3581.21 KN$

Load carrying capacity of column $= \frac{3581.21}{1.5}$

$P = 2387.47 KN$

2) Design a single angle strut connected to a gusset plate to carry 180KN factored load. The length of the strut between centre to centre connection is 3m.

Sol:

$P_d = 180KN$.

step: 1 for angle compressive stress $f_{cd} = 90 N/mm^2$.

step: 2 Effective sectional Area or required

Area:- $A_e = P_d / f_{cd} \rightarrow 180 \times 10^3 / 90$

$A_e = 2 \times 10^3 mm^2$.

Step: 3 Select section, always choose max area.

(Pg: 10 - SP(6)) $A = 2259 mm^2$

$r_{min} = 30.3 mm$.

Step : 4 Effective Length

$l_e \Rightarrow KL$

$= 0.85 \times 3000 = 2550 \text{ mm}.$

Step : 5 Slenderness Ratio.

$$\lambda = \frac{l_e}{r_{min}} = \frac{2550}{30.3}$$

$= 84.6$

Step : 6 Find fcd. (Pg. NO : 44) IS800-2007

$f_y = 250 \text{ N/mm}^2$

$\lambda = 84.6$

$x = 84.6, \quad x_1 = 80, \quad x_2 = 90,$

$y = f_{cd}, \quad y_1 = 133, \quad y_2 = 119.$

$$\frac{x - x_1}{x_2 - x_1} = \frac{y - y_1}{y_2 - y_1} \Rightarrow \frac{84.6 - 80}{90 - 80} = \frac{f_{cd} - 133}{119 - 133}$$

$f_{cd} = 127.176 \text{ N/mm}^2.$

Step : 7 Load ✓

$P_{cd} = A_e \times f_{cd}$

$= 2259 \times 127.176$

$P_{cd} = 287 \text{ kN} > 180 \text{ kN}$

Hence safe

3) A Column 4m Long has to support a factored Load of 6000kN, The Column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam section & Plates.

Sol:

$P_d = 6000 kN$.

Step : 1. for Plated Column of Large Load

Compressive stress $f_{cd} = 200 N/mm^2$.

Step : 2 Required area. (Pg. No : 34)

✓ $P_d = A \times f_{cd}$.

$A = P_d / f_{cd} = \frac{6000 \times 10^3}{200}$

✓ $A = 30 \times 10^3 mm^2$

Step : 3 Selection of section using SP6 (1)

ISHB 450.

Area of Beam } $A = 11789 mm^2$.

Area of Plate $= 30,000 - 11789$

$= 18211 mm^2$.

Assume the thickness of Plates as 20mm. There are 2 plates

$B = \frac{18211}{2 \times 20} = 455.21 \simeq 500mm$ ✓

Size of Plate $= 500 \times 20 mm$ ✓

Properties of section :- ISHB 450

$A = 11789\ mm^2$, $I_{yy} = 3045 \times 10^4\ mm^4$

I_{yy} for a section with 2 plates.

$$I_{yy} = 3045 \times 10^4 + 2 \times \frac{20 \times 500^3}{12} \quad \left(\frac{db^3}{12}\right)$$

$$I_{yy} = 447.2 \times 10^6\ mm^4.$$

$$r_{min} = \sqrt{\frac{I_{yy}}{A_{sp}}}$$

A_{sp} → Area of section + Area of 2 plates.

$$= \sqrt{\frac{447.2 \times 10^6}{11789 + 2(20 \times 500)}}$$

$$= 118.6\ mm$$

Step :4 Slenderness ratio :- Pg.No: 45

$$\lambda = \frac{L_e}{r_{min}}$$

$L_e = KL$ — One end fixed another hinged

$K = 0.8$

$$L_e = 0.8 \times 4000$$

$$\lambda = \frac{3200}{118.6}$$

$$\lambda = 26.98$$

Step : 6 fcd for a section with plates

from → 9 (c) table (IS 800 - 2007)

$x = 26.98$, $x_1 = 20$, $x_2 = 30$.

$y = f_{cd}$, $y_1 = 224$, $y_2 = 211$.

$$\frac{x - x_1}{x_2 - x_1} = \frac{y - y_1}{y_2 - y_1}$$

$$\frac{26.98 - 20}{30 - 20} = \frac{f_{cd} - 224}{211 - 224}$$

$$\frac{6.98}{10} = \frac{f_{cd} - 224}{-13}$$

$$-(0.698 \times 13) = f_{cd} - 224$$

$$f_{cd} = -9.074 + 224$$

$$f_{cd} = 214.93 \text{ N/mm}^2$$

Step : 7 Check for load :-

$P_{cd} = A_e \times f_{cd}$

A_e = section with plates.

$P_{cd} = 31789 \times 214.93$

$P_{cd} = 6832.4 \text{ KN} \checkmark > P_d$

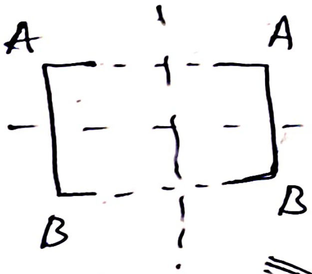
Hence safe. ✓

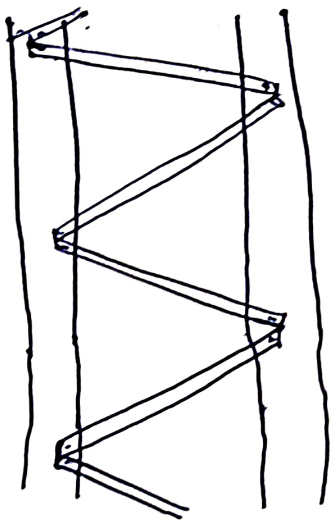
Lacing & Batten (IS 800-2007 - Pg : 48, 49, 50, 51)

To achieve maximum value for minimum radius of gyration, without increasing the area of the section. The number of elements are placed away from the principal axis using suitable lateral system.

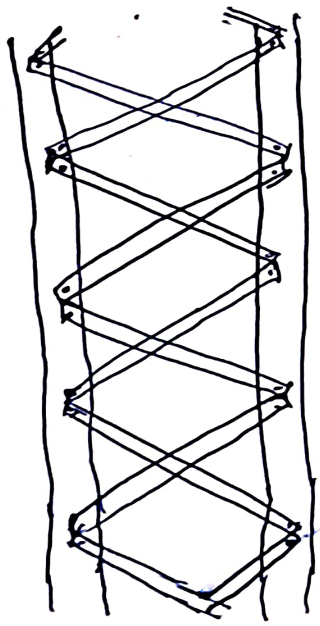
The commonly used lateral system are,

1) Lacing or latticing
2) battening.

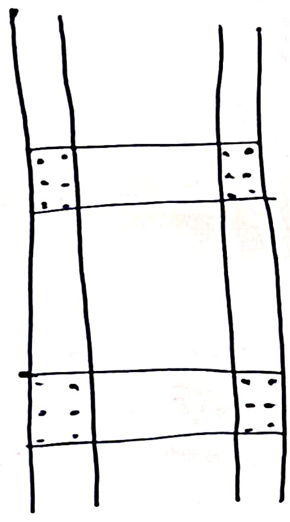




Single Lacing



double Lacing



Battening

✓ can use batten to keep the column member at required distance

Rolled steel flats and angles are used for lacing

Design of Laced Columns:- (IS 800-2007)

1) The Lacing / Latticing system should be uniform.

2) In single laced column the direction of lattice at the opposite face should be shadow of the other, should not be mutually opposite.

3) In a bolted / rivetted construction, The min. width of lacing bar = 3×nominal dia of bolt or rivet

4) Flat lacing bar thickness

for single lacing $> \frac{1}{40} L_e$

for double lacing $> \frac{1}{16} L_e$

5) Lacing bars inclined at ⇒ 40° - 70° to the axis

6) distance b/w the main members $r_{yy} > r_{zz}$, r_{yy} - weaker section axis, r_{zz} = radius of gyration of stronger axis

7) max. spacing of lacing bar in a main member should not greater than 50 or 0.7 times λ.

8) Lacing shall be designed to resist transverse shear (V_t = 2.5% axial force in column).
If there are 2 transverse parallel system then each system has to resist $\frac{V_t}{2}$ shear force

9) If the column subjected to bending also,
V_t = bending shear + 2.5% column force.

10) Effective length of single laced system = length b/w the inner end fasteners.
For welded and double laced system effective length = 0.7 x actual length.

11) Slenderness ratio of lacing bar $\frac{KL}{r} < 145$.

12) Laced compression member shal be provided with end tie plates.

13) Effective slenderness ratio of laced column shall be taken as 1.05 x actual maximum λ, in order to account for shear deformation effect.

1) Design a laced column with 2 channel back to back placed of length 10m, to carry an axial factored load 1400kN. The column may be assumed to have restrained in position and not in direction at both ends.

Sol:

1) $P_d = 1400 kN$.

2) Required area :-

Assume $f_{cd} = 135 N/mm^2$.

$P_d = A_e f_{cd} \Rightarrow A_e = \frac{1400 \times 10^3}{135} = 10370 mm^2$

for single channel $= \frac{10370}{2} = 5185 mm^2$.

Select channel section (steel table)
(or) SP(6) - Pg. No: 6

ISMC - 350 at 413N/m

$A = 5365 mm^2$,

$I_{xx} = 10008 \times 10^4 mm^2$. $r_{xx} = 136.6 mm$

$I_{yy} = 430.6 \times 10^4 mm^2$ $r_{yy} = 28.3 mm$.

$C_{yy} = 24.4 mm$,

✓ $r_{min} = 136.6 mm$

3) Slenderness ratio (λ)

$$\lambda = \frac{l_e}{r_{min}} = \frac{10,000}{136.6}$$

(Pg: No 1&8) $= 73.20$

λ for laced column $= 1.05 \times \lambda$

$= 1.05 \times 73.2$

$\boxed{\lambda = 76.86}$

4) Buckling class.

for channel section Buckling class } = c.

(Pg. No: 44)

from Pg No: 42,

$x = 76.86$, $y = f_{cd}$

$x_1 = 70$, $y_1 = 152$

$x_2 = 80$, $y_2 = 136$.

$$\left[\frac{x - x_1}{x_2 - x_1} = \frac{y - y_1}{y_2 - y_1}\right]$$

$$\frac{76.86 - 70}{80 - 70} = \frac{f_{cd} - 152}{136 - 152}$$

$f_{cd} = 141.02 N/mm^2$.

check for load carrying capacity.

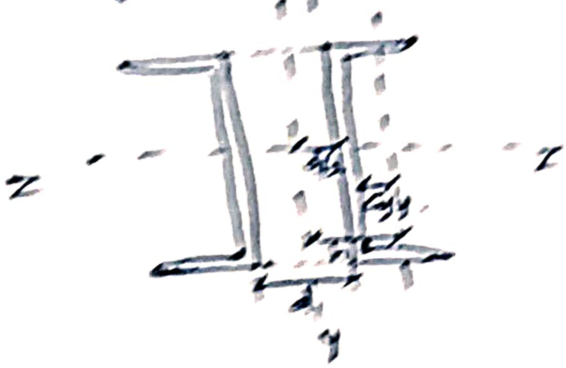
$P_d = A_e f_{cd}$

$P_d = 2 \times 5365 \times 141.02$

$P_d = 1513.42 kN > 1400 kN$.

Hence safe.

5) Spacing b/w channel:-



$I_{xx} / I_{zz} = 2 \times 10008.6 \times 10^4$

$= 20017.2 \times 10^4 \ mm^4.$

$I_{yy} = 2 \left(I_{yy} + A h^2\right]$

$h = d/2 + C_{yy}$

$= 2 \left[430.6 \times 10^4 + 5365 \left[d/2 + 24.4 \right]^2 \right]$

$\boxed{I_{yy} = I_{zz}}$

$20017.2 \times 10^4 = 8612000 + 10722 \left[(d/2 + 24.4)^2 \right]$

$d/2 + 24.4 = 17848.30$

$d = 218.36 \simeq 220 mm$

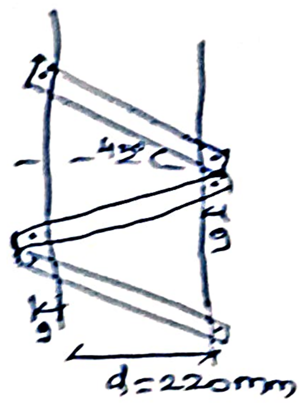
6) Lacing design :- (Pg. No: 50 IS 800)

angle = 45° $\boxed{b = 60}$

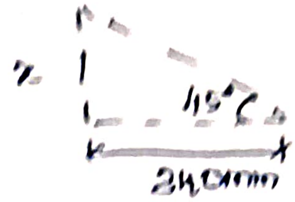
horizontal distance b/w lacing } = d + 2g

$= 220 + 2(60)$

$= 340 mm$



Vertical distance:-



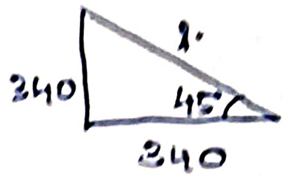
$$\tan 45' = \frac{x}{240}$$

$$x = 240\ (1)$$

[$\tan 45' = 1$]

Horizontal / vertical distance between 2 lacings } = 240×2

= 680mm



Length

$$\cos 45° = \frac{240}{l}$$

$\cos 45° = 1/\sqrt{2}$

$$l = 240/\cos 45'$$

[$l = 4~~8~~0.83mm$] (480.83)

Width:- (IS-800, Pg:-150, class 7-6.2)

min width = 3×d

d = Nominal dia of bolt/rivet

= 3×20

d=20 assume

[b = 60mm]

Thickness [Pg.No: 150 class 7.6.3]

single lacing ⇒ t_L = $1/40 \times L_{eff}$

= $1/40 \times 480.83$

= 12.02 mm ≃ 14mm

Transverse shear to be resist by lacing

V_t (transver shear) = 2.5% of axial force.

$$= \frac{2.5}{100} \times 1400\times10^3$$

[V_t = 35KN] for 2 lacing.

for one lacing $= \frac{35}{2} = \boxed{17500N}$

vii) Bolting :-

In shear, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ (It's like lap $n_s A_{sb} = 0$]

$V_{nsb} = \frac{400 \times [1 \times 0.78 \times \frac{\pi}{4} \times 20^2]}{\sqrt{3} \times 1.25}$ $V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$

$= 45271.07$ $\boxed{V_{dsb} = 45.27 KN}$

In bearing :-

$V_{dpb} = \frac{2.5 \times K_b \times d \times t \times f_{ub}}{\gamma_{mb}}$

$K_b \Rightarrow \frac{e}{3d_0}$ $e = 40$, p_0 60 — 0.606

$\frac{P}{3d_0} - 0.25$

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

1

$V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 14 \times 410}{1.25}$

$V_{dpb} = 103320N$

$\boxed{V_{dpb} = 103.3 KN}$

least value taken as bolt value

$V_{dsb} = 45.27 KN$

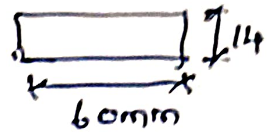
No. of bolts $= \frac{\text{design load}}{\text{bolt value}}$

$= \frac{17500}{45271}$

$= 0.38 \simeq 1$ bolt

8) Slenderness ratio & Pd :-

$S.R = \frac{KL}{r_{min}}$ $\quad KL = 480.83mm$



$r_{min} = \sqrt{I/A}$

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

$= \frac{60 \times 14^3}{12} = 13720 mm^4$

$A = b \times t = 60 \times 14 = 840 mm^2$

$r_{min} = \sqrt{\frac{13720}{840}} = 4.04 mm.$

$\lambda = \frac{480.83}{4.04} = 119.07$ ✓

It is builtup member so,

Buckling class c ✓

from table 9(c)

$x = 119.07, \quad y = f_{cd}$

$x_1 = 110, \quad y_1 = 94.6$

$x_2 = 120, \quad y_2 = 83.7$

$f_{cd} = 84.77 N/mm^2$ ✓

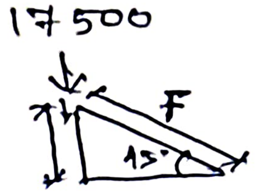
$P_d = A_e f_{cd}$

$= (60 \times 14) \times 84.77$

$P_d = 71.21 kN$ ✓

Check :-

Actual force in lacing



$$\sin 45^\circ = \frac{17500}{F}$$

$$F = \frac{17500}{1/\sqrt{2}} = 24.748\,\text{KN} < 71.21\,\text{KN}$$

hence safe

Batteen Procedure:-

1) Batteen plate should be Provide on similar

2) The No. of Batten should be such that the member is devided into not less than 3 bays.

3) Batten shall be plate, angle, channel (or) I section.

4) r_{yy} & r_{xx}.

5) Slenderness ratio $SR = 1.1 \times \lambda$ column.

6) Vertical spacing of batten, S.R neither greater than 50 nor greater than 0.7 times of whole number

7) Shear force = V_t = 2.5% total actual force

8) transverse shear $V_b = \frac{V_t \times l}{NS}$ ✓

$$M = \frac{V_t \times l}{2N}.$$

l = Distance b/w c/c of batten longitudinal.

N = No. of battern lines

S = Mini. transverse distance b/w fasteners of battern.

9) Efficient of depth of end Batteen (long) shall not be less than distance b/w C.G of main member

11) width ≮ 2 times of width of one member

12) thickness ≮ 1/50 distance b/w innermost connecting line of bolt.

(1) Design built up column for the previous problem using batten instead of lacing system.

Sol:

same as previous problem upto section.

1) slenderness ratio :- (Pg. No: 51, (7.7.1.4))

$S.R = 1.1 \times \lambda$.

$= 1.1 \times 73.2 = 80.52$

Buckling class (Pg. No: 44)

x_1 80 = 136.41

fcd ⇒ x_2 90 = 121.42

fcd = 135.23 N/mm²

Check for Load:-

Pd = Ae × fcd

= 2 × 5365 × 135.2

= 1451.28 kN > 1400 kN ✓

Hence safe.

Design of Batten :- (Pg.No:51, 7.7.3)

1) spacing b/w channel :

$I_{xx} = 2 \times 10008.6 \times 10^4$,

$I_{yy} = 2(I_{yy} + Ah^2) \Rightarrow h = d/2 + C_{yy}$

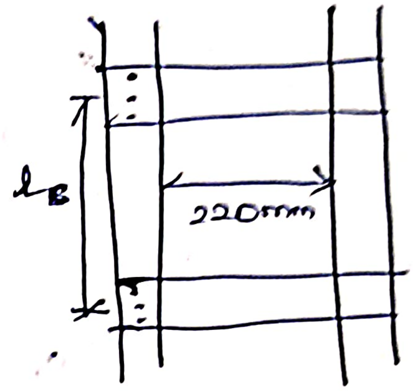
$I_{xx} = I_{yy}$

$200017.2 \times 10^4 = 2[430.6 \times 10^4 + 5365(d/2 + 24.4)^2]$

$d = 218.36 \simeq 220\,mm.$

b/w 2

2) length of batten :-



1) $\frac{KL}{r_{min}} < 50$

hinged $K = 1$

$\frac{L_c}{r_{min}} < 50 \Rightarrow \frac{L}{28.3} < 50$

$L < 50 \times 28.3$

$L < 1415\,mm.$

2) $\frac{KL_c}{r_{min}} < 0.7 \left(\frac{KL}{r_{min}}\right)$ member

$\frac{KL_c}{r_{min}} < 0.7\ (80.52)$

$\frac{L_c}{28.3} < 0.7 \times 80.52$

$L_c < 1595.10$

Assume the value $L_c = 1200\,mm$ ✓

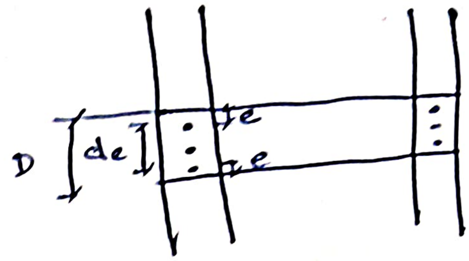
Size of batten :- 51, 7.7.2.3

Effective depth of end Batten

$d_e \nless d + 2 \times C_{yy}$

$\nless 220 + 2 \times 24.4$

$d_e \nless 268.8\,mm \simeq 270$



$e = 1.5 d_o$

$D = d_e + e + e$

$= 270 + 35 + 35$

$D = 340\,mm$

depth of Intermediate batten :-

$d_{ei} \nless 3/4$ end. batten

$d_{ei} \nless 3/4 \times (d + 2 C_{yy})$

$\nless 3/4 \times 268.8$ (67.2)

$d_e \nless 201.6 \simeq 205\,mm.$

$D_i = 205 + 35 + 35 = 275\,mm$

Thickness of battem :- (Pg. NO: 51, 7.7.2.3)

$t_b \nless 1/50 \times S$

$S = d + 2g$

$= 220 + 2 \times 60$

$S = 340 \Rightarrow t_b \nless 1/50 \times 340$ (7)

$t_b \simeq 8\,mm$

Transverse shear force to be resist 51, 7.7.2

V_t = ~~a_g~~ × 2.5 % × axial force

$= 2.5/100 \times 1400 \times 10^3$

$V_t = 35000\ N$

Shear force $V_b = \frac{V_t\ L_0}{N\ S}$ (7.7.2.1)

$= \frac{35000 \times 1200}{2 \times 340}$

$V_b = 61765\ N$

Bending Moment $M = \frac{V_t\ L_0}{2N}$

$= \frac{35000 \times 1200}{2 \times 2}$

$= 105 \times 10^5\ N\ mm$

Bolt connection :-

Pg. No : 75 clause - 10.3.3

use 20mm bolt :-

1) shear capacity $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ — Lap joint $n_s A_{sb}$ ...

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

$= 45.27\ kN.$

2) Bearing $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$ ($V_{npb} = 2.5\ k_b \times d \times t \times f_u$)

e = 35

$= 2.5 \times 0.58 \times 20 \times 8 \times 410$

$V_{dps} = 69.53$ kN

Least value = ~~45.27~~ kN

No. of bolts ⇒ $\frac{61765}{45.27} = 1.4 \simeq 3$ bolts.

Provide 3 bolts to each side

Check for stress in batten plate :-

Shear force $= \frac{V_b}{Dt} = \frac{61765}{340 \times 8}$

$= 22.7$ Mpa < 100

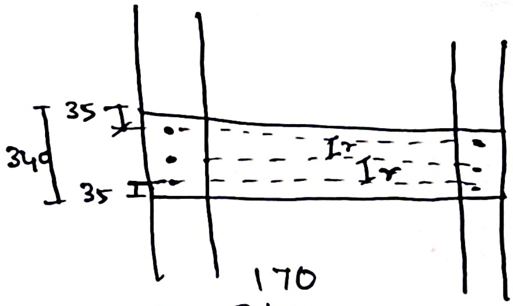
Hence safe.

Bending stress $= \frac{6M}{tD^2} = \frac{6 \times 105 \times 10^5}{8 \times 340^2}$

$= 68.12 < 165$

Hence safe

force due to moment of each bolt:-



$M = \frac{m \times r}{\Sigma r^2}$

$r = \frac{340}{2} - 35$ (170)

$r = 135$

$= \frac{105 \times 10^5 \times 135}{135^2 + 135^2}$

$= 38.89$ kN

force due to shear on each bolt :-

$V = \frac{61765}{3} = 20.589$ kN

Actual force $R = \sqrt{V^2 + M^2}$

$= \sqrt{38.89^2 + 20589^2}$

$= 44002.54N < 45.27kN$ ✓ LOA value

Hence safe

zed
pt.

Unit - IV

Beams

One of the frequently used structural member is a beam, whose main function is to transfer load principally by means of flextural or bending action.

In a structural framework, it forms the main horizontal member spanning between adjacent columns or as a secondary member transmitting floor loading to the main beams.

Design of laterally supported Beam :- (Pg. 53, 70)

1) Assume self weight of the beam as 0.8 kN/m. (If the given load is not including self weight)

2) Factored self weight

3) If the given load is working load find factored load.

4) Find total design load (wu) = given superimposed load + self weight.

5) If the given span is clear span find effective span = clear span + width of support

6) Design moment (Mu) = for simply supported beam $\} = \frac{W_u l^2}{8}$.

for cantilever beam = $\frac{W_u l^2}{2}$.

7) Design of shear force (Vu)

⇒ for simply supported beam = $\frac{W_u l}{2}$

⇒ for cantilever beam = $W_u l$.

8) Plastic section Modulus Required = $\frac{M_u}{f_y} \gamma_{mo}$
(Zp - required)

9) For the required plastic section modular

Select a section (I-section) having 10 to 15% extra plastic modulus from SP6(1) and write down the Properties of the section. (IS-800-2007 · No: 138, 139 & 140)

values of $h, b_f, A, d_f, t_w, I_{xx}, I_{zz}$ and r_1 for SP6(1).

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

$b = \frac{b_f}{2}$.

b - Outstanding leg of compression flange

10) Section classification (Whether plastic, compact or semi compact)

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

b) find (b/t_f)

c) find (d/t_w).

(d) By using table No:2 in IS 800-page 18

classify the section as plastic compact and

semi compact. check out - If >0.8 - then

add self wt

11) Check shear strength IS-800-clause 8.4

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$. (Pg:No:59)

12) check moment capacity IS 800 (53 pg, clause 8.2.1.2)

13) check deflection .. (Pg:No:31 Table-6)

Batten Procedure

1). Batten plate should be provided similarly

2) The no of battens should be such that the member is divided into not less than 3 days.

3). Batten shall be plate, angles, channels (or) 'L' section.

4). $r_{yy} > r_{mn}$

5). S.R => 1.1 × S.R of column

6). Vertical spacing of battens. SR neither greater than 50 nor greater than 0.7 times of whole number.

7). S.F = V_t = 2.5% of T.A.F where V_t → transverse S.F

8). $V_b = \frac{V_t \cdot C}{Ns}$; $M = \frac{V_t \cdot C}{2N}$,

C → Distance b/w c/c of batten longitudinal

N → Number of parallel lines

S → Mini transverse distance b/w fasteners of battens.

9) Efficiency depth of end battens (long) shall not be less than distance b/w C.G of main member

10). Efficiency depth of intermediate battens shall not be less than 3/4 of above distance

11). Width ≮ 2x width of one number

12). Thickness 4 1/50 distance b/w inner most connecting line of bolt.

a = kg-ms

b = kg-m/s

c = kg-m/s²

d =

N →

5). Design a built-up column for the previous problem using batten instead of lacing system.

Sol:

Same previous problem upto section selection.

1) S.R: (Pg:No: 51). (7.7.1.4)

$S.R = 1.1 \times 73.2 = 80.52$

Buckling class - c (Pg:No: 44)

(Pg:No:42)

fcd.

80 → 136

90 → 121.

$f_{cd} = 136 - \frac{15}{10} \times 0.52$.

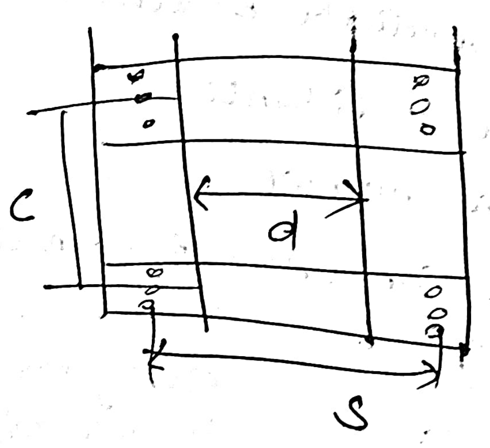
$f_{cd} = 135.23 N/mm^2$

$P_d = A_e \times f_{cd}$.

$= 2 \times 8365 \times 135.2 = 1451.28 kN > 1400 kN$

∴ Hence safe.

Design of Battens. (Pg: No: 51 7.7.3)



$r_{min} = 28.3$

1) $\frac{KL}{r_{min}} < 50$

$\frac{c}{28.3} < 50$

$c < 50 \times 28.3$

$c < 1415$

2) $\frac{KC}{r_{min}} < 0.7 \left(\frac{KL}{r}\right)_{member}$

$c/28.3 < 0.7 \times 80.52$

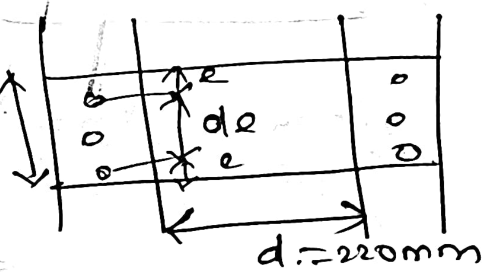
$C < 0.7 \times 80.52 \times 28.3$

$C < 1595.10$

Assume $C = 1200\,mm$

Size of battens (PG: No: 51) clause 7.7.2.3).

Effective depth of end battens,



$de \nless d + 2 \times C_{44}$.

$\nless 220 + 2 \times 24.4$

$\nless 268.8$.

$e = 1.5 d_0$

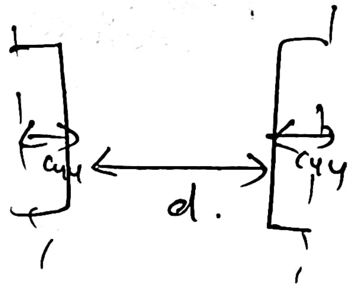
$= 1.5 \times 22$

$= 35$,

$D = de + e + e$

$= 270 + 35 + 35 = 340\,mm$.

Intermediate battens PG: NO: 51 (7.7.2)



$de \nless 3/4 \times (d + 2 \times C_{44}))$

$\nless 3/4 \times (220 + 2 \times 24.4)$

$\nless 3/4 \; 268.8$

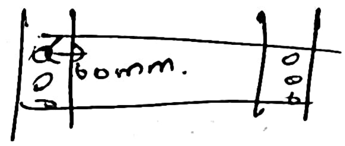
$\nless 201.6$

take $d = 205\,mm$

$D = 205 + e + e = 205 + 35 + 35 = 275\,mm$.

Thickness of batten (PG: No: 51 7.7.2.3)

$t \nless 1/50\,(S)$.



$S = d + 60 + 60$.

$S = 220 + 60 + 60 = 340\,mm$

$t \nless 1/50 \times 340$

$\nless 6.8$.

So take $t = 8\,mm$

Transverse shear to be resist : pg:57 (7.7.2)

$V_A = 2.5\%$ of axial force

$= \frac{2.5}{100} \times 1400 \times 10^3$

$V_t = 35000N$

Shear force $V_b = \frac{V_t C}{NS}$ (7.7.2.1)

$= \frac{35000 \times 1200}{2 \times 340}$

N = no of parallel plane of batten.

$V_b = 61765N$ ✓

Bending moment $M = \frac{V_t C}{2N}$

$= \frac{35000 \times 1200}{2 \times 2}$

$= 10500000$ N-mm $\simeq 105 \times 10^5$ N-mm

Bolt Connection:

Use 20mm bolts, PS: No: 75 clause 10.3.3.

Shearing

$V_{dsb} = \frac{410}{\sqrt{3}} (1 \times 0.78 \times \pi/4 \times 20^2)$

$V_{dsb} = 46404.24N$

$V_{dsb} = 46.40kN$

Assume $e = 35mm$

Bearing

$V_{dsb} = \frac{2.5 k_b d t f_u}{1.25}$

$= \frac{2.5 \times 0.53 \times 8 \times 410 \times 20}{1.25}$

$= 69536N$

Least value, 46.4 kN

$$\text{No of bolts} = \frac{\text{Design force}}{\text{Bolt value}}$$

$$= \frac{61765}{46.41kN} = 1.33 \approx 2 \text{ bolts}$$

∴ provide 3 bolts to take into account bending stress also.

Check for stress in batten plates.

$$\text{Shear stress} = \frac{V_b}{Dt} = \frac{61765}{340 \times 8} = 22.7 \text{ MPa} < 100$$

Hence safe.

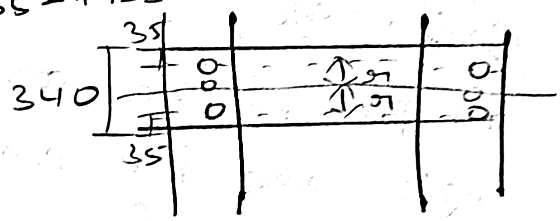
$$\text{Bending Stress} = \frac{6M}{tD^2} = \frac{6 \times 105 \times 10^5}{8 \times (340)^2} = 68.12 < 165$$

∴ Hence safe.

force due to moment of each bolt:

$$M = \frac{Mr}{\Sigma r^2} = \frac{105 \times 10^5 \times 135}{135^2 + 135^2}$$

$$= 38888.88 N$$



$= 340 - (35 \times 2)$

$2r = 270/2$

$r = 135$

$$\text{force due to shear in each bolt } V = \frac{61765}{3 \text{ (no of bolts)}}$$

$$= 20588.33 N.$$

$$\text{Actual force } R = \sqrt{V^2 + M^2}$$

$$= \sqrt{38888.88^2 + 20588.33^2}$$

$$= 44002.54 N < \text{Bolt value } (46.41kN).$$

Column bases:

Introduction

The columns are supported on the column bases. The column bases transmit the column load to the concrete or masonry foundation blocks. The column load is spread over large area on concrete or masonry blocks. The intensity of bearing pressure on concrete or masonry is kept within the max permissible bearing pressure.

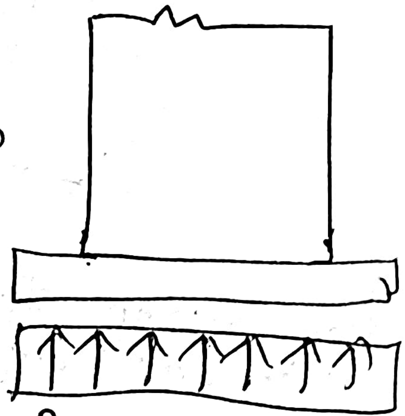
The column bases are of two types,

(1) Slab base
(2) ~~Gestated~~ Gusseted bases.

Slab-base.

1) The slab base consists of cleat angles and base plate. The column end is faced for bearing over the whole area.

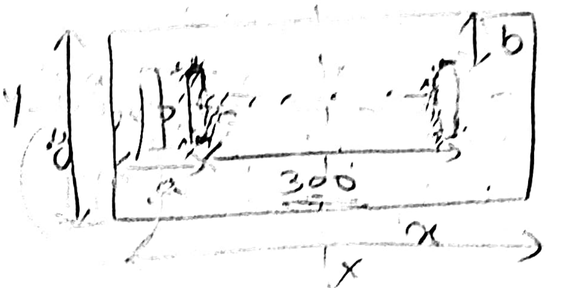
2). The load is transferred to the base partly to bearing ~~and~~ and partly to gusset. Design of slab base (or) simple base.



Pressure distribution slab base.

1) Design a slab base for a column ISHB 300 @ 577 N/mm Carrying a factored load of 1000 kN. M20 Concrete is used for foundation. Provide welding Connection b/w Column & base plate.

Sol:



ISHB 300 @ 577 N/m.

M20

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

$P_u = 1000 kN$

$b_f = 250 mm$

$h = 300 mm$

$t_f = 10.6 mm$

Step 1: Size of base plate.

(1) Bearing strength of Concrete $\Rightarrow 0.45 f_{ck}$

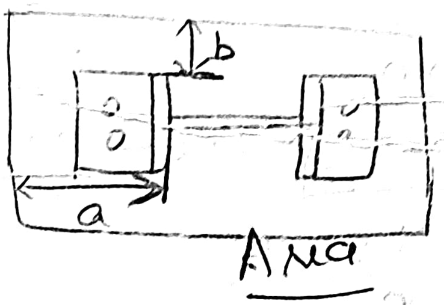
$= 0.45 \times 20$

$= 9 N/mm^2$

(2) Area $= \dfrac{P_u}{0.45 f_{ck}} = \dfrac{1000 \times 10^3}{9} = 111111.11 mm^2$

(3) Choose a section such that $a = b$

larger projection $a = \dfrac{x - 300}{2}$, $b = \dfrac{y - 250}{2}$ smaller projection



$\dfrac{x - 300}{2} = \dfrac{y - 250}{2}$

$x - y = 50 \rightarrow (1)$

$xy = 111111.11 \rightarrow (2)$

from (1) & (2)

we get $x = 359.20 \simeq 360 mm$

$y = 309.20 \simeq 310 mm$

$a = 30 mm$, $b = 30 mm$

Provide $\Rightarrow$ **360 × 310 mm**

Step - III : Thickness of base plate.

(i) Pressure intensity (w) = $\frac{P_u}{Area}$

$= \frac{1000 \times 10^3 N}{360 \times 310 mm^2}$

$= 8.96 N/mm^2$ (pg. no. 47)

(ii) $t_s = \sqrt{\frac{2.5 w (a^2 - 0.3 b^2) \gamma_{mo}}{f_y}} > t_f = 10.6 mm$

$= \sqrt{\frac{2.5 \times 8.96 (30^2 - 0.3 \times 30^2) \times 1.1}{250}}$

$= 7.8 < t_f.$

Let $t_s = 12 mm$

Connection:

Connect slab 360 x 310 x 12 mm with 4 bolts of 20mm dia φ, 300 long to anchor the plate

Welded Connection: $t_f = 10.6 mm$

min size of weld = 5mm

max size of weld = $t - t/4 = 10.6 - \frac{10.6}{4}$

$= 4.95 mm$

$\approx 8 mm.$

$S = 8 mm$

throat thickness = k . s

$= 0.7 \times 8$

$= 5.6 mm.$

Design strength of weld = $\frac{A_g f_u}{\sqrt{3}\gamma_{mw}}$ ($L_w \times t$)

$= \frac{L_w \times t \times 410}{\sqrt{3} \times 1.25}$

Equal to design load,

$\frac{L_w \times t \times 410}{\sqrt{3} \times 1.25} = 1000 \times 10^3 N$

$L_w = \frac{1000 \times 10^3 \times \sqrt{3} \times 1.25}{5.6 \times 410}$

$L_w = 942.94 mm$

Check: for length:

$t_w = 7.6$ $t_f = 10.6$

total available length of weld = 2[250 + 250 + 7.6 + 300 - 2(10.6)]

= 1542.11 mm.

$b = L + 2S \times$ No of turns

$L = b - 2S \times$ No of turns.

$= 1542.11 - 2 \times 8 \times 12$

$= 1398.12 mm$

$\frac{A_g f_u}{\sqrt{3}\gamma_{mw}}$

∴ Design strength = $\frac{1398.12 \times 5.6 \times 410}{\sqrt{3} \times 1.25}$

$= 1112005.57$

$= 1112.05 kN > 1000 kN$

∴ Hence safe.

① Design a simply supported beam of effective Span 1.5m carrying a factored concentrated load of 360kN at mid span.

Sol:

$W_u = 360kN$ $\quad l = 1.5m$

$l = 1500mm$

Choose section

1) Maximum moment occur at mid span,

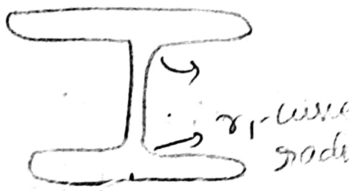
$$M = \frac{WL}{4} = \frac{360\times1.5}{4} = 135\ kN\text{-}m$$
$$= 135\times10^6\ N\text{-}mm.$$

Zp (plastic section modulus) $= \frac{M\gamma_{mo}}{f_y}$ (Pg:No:30 table:5)

$$= \frac{135\times10^6\times1.1}{250}$$
$$= 594.0\times10^3\ mm^3.$$

Select trial section as ISMB 300 which has Zp value $651.781\times10^3\ mm^3$.

2) Section properties:



$h = 300mm$

$t_f = 12.4mm$

$b = 140mm$

depth of web. $d = h - 2(t_f + r_1) = 300 - 2(12.4 + 14)$
$= 247.2mm.$

h_f = centre to centre distance of flange.

$$= 300 - \frac{12.4}{2} = 293.8mm$$

web thickness $t_w = 7.5mm$.

$I_{zz} = 8603\times10^4\ mm^4$ (Pg:No:138)

$Z_e = 573.6\times10^3\ mm^4.$

$Z_p = 651.73\times10^3\ mm^4.$

Self wt of beam = 0.4336 kN/m → (from steel table)

Factored wt = 1.5 × 0.4336 = 0.649

Additional factored moment due to self wt } = $1.5 \times \frac{w l^2}{8}$

$= 1.5 \times 0.4336 \times \frac{1.5^2}{8} = 0.183$ kN·m

Total factored moment M = 135 + 0.183 = 135.183 kN-m

factored shear force due to self wt = $1.5 \times \frac{w l}{2}$

$= 1.5 \times 0.4336 \times \frac{1.5}{2}$

= 0.488 kN.

∴ Total factored shear force = $\frac{360}{2}$ (at centre) + 0.488 = 180.48 kN.

Section classification: $\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$ (pg: No 59)

Overhang $b = \frac{140}{2} = 70$

(Table: 2 Pg: No: 18) IS-800

$\frac{b}{t_f} < 9.4\varepsilon$, $\frac{d}{t_w} < 84\varepsilon$

$\frac{70}{12.4} = 5.64 < 9.4$, $\frac{247.2}{7.5} = 32.96 < 84$.

It is classified as a plastic [class - I section

3). Shear capacity of section: (Pg: No: 59 cl: 8.4.1)

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5$

$= 295.235 \times 10^3$ N

= 295.235 kN > shear force (180.48)

∴ Hence ok

(c) Shear capacity $0.6V_d = 0.6 \times 295.25 = 177.14$ KN

$$V > 0.6V_d$$

4). Moment capacity of Section: (Pg. No: 70 Cl. 9.2.2)

Since $V > 0.6V_d$ & section belongs to plastic Category.

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$$

$$M_d = Z_p f_y / \gamma_{mo} \leq 1.2 Z_e f_y \times 1/\gamma_{mo}$$

$$Z_p f_y \, 1/\gamma_{mo} = 651.7 \times 10^3 \times 250 \times 1/1.1 = 148.114 \times 10^6 \text{ N-mm}$$

$$1.2 Z_e f_y \, 1/\gamma_{mo} = 1.2 \times 573.57 \times 10^3 \times 250 \times 1/1.1$$

$$\Rightarrow 156.428 \times 10^6 \text{ N-mm}.$$

$$M_d = 148.114 \times 10^6 \qquad \beta = \left(\frac{2V}{V_d} - 1\right)^2$$

$$\beta = \left(\frac{2 \times 180.48}{295.235} - 1\right)^2 = 0.05$$

$$\beta = 0.05$$

Since it is double symmetric section furb obtained from (table-14) of code.

$$\frac{KL}{r} = \frac{1500}{28.4 \text{ (from steel table)}} = 52.8 \quad \& \quad \frac{h_f}{t_f} = \frac{293.8}{12.4} = 23.69.$$

Refer to (table 14) by double interpolation.

h/t_f 20 23.69 25

$\Rightarrow$ Interpolation KL/r 50 — 995.3 & 951.7

52.8 — ?

60 — 728.4 & 684.7

At X, $f_{crb} = 995.3 - \frac{(23.69-20)}{(25-20)} \times (995.3 - 957.7)$

$= 963.123\ N/mm^2$

At Y, $f_{crb} = 726.4 - \frac{(23.69-20)}{(25-20)} \times (726.4 - 654.6)$

$= 695.55\ N/mm^2$.

At G, $f_{crb} = 963.123 - \frac{(52.8-50)}{(60-50)} \times (963.123 - 695.55)$

$= 888.2\ N/mm^2$. (Table 13 (A))

$\boxed{f_{crb} = 888.2}$

$f_{cd} = 204.5 - \frac{88.2}{100}(206.8 - 204.5)$

$= 204.77$

$M_{fd} = f_{cd} \times A = 888.$

$= 204.77 \times 5626 = 1.15204 \times 10^6\ N\text{-}mm$.

$M_{dr} = 148.114 \times 10^6 - 0.05(148.114 \times 10^6 - 1.1852 \times 10^6)$

$M_{dr} = 140.77 \times 10^6 > 135\ kN.m$.

5) max deflection $\delta = \frac{WL^3}{48EI} = \frac{360 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 8603 \times 10^4}$.

$= 1.68\ mm < \frac{1500}{300}$. → Length to breadth reason.

∴ Hence safe. Use $\boxed{ISMB\ 300}$

NOTE:

$V < 0.6 V_d$ = Low Shear

$V > 0.6 V_d$ = High Shear

2). 8m x 12m size, thick = 100mm, I beam 3m apart
floor finish = 1.5 kN/m², load 1.5 kN/m².

Sol:

Clear span = 8m.

thick = 100mm = 0.1m.

width = 3m.

F.F = 1.5 kN/m²

L.L = 1.5 kN/m².

Sol:

1) load calculation,

l x b x t x Unit wt

self wt of slab = 1 x 3 x 0.1 x 25 = 7.5 kN/m.

F.F = 1.5 x 3 = 4.5 kN/m

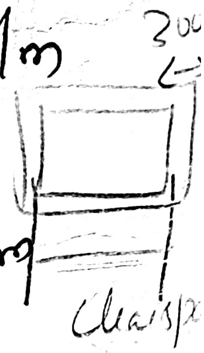
L.L = 1.5 x 3 = 4.5 kN/m

Total L = 16.5 kN/m.

2). Self wt of beam assumed as 0.8 kN/m.

t.wt = 16.5 + 0.8 = 17.3 kN/m

3). Factored wt = 1.5 x 17.3 = 25.95 kN/m



4). clear span = 8m

Assume width of support = 300mm

5). Effective span = Clear span + width of support

= 8 + 0.3

= 8.3m.

b) $B.m = \frac{wl^2}{8} = \frac{25.95 \times 8.3^2}{8} = 223.46 kN.m$

7) $S.F \Rightarrow \frac{wl}{2} = \frac{25.95 \times 8.3}{2} = 107.69 kN.$

8) $Z_p = \frac{M_u}{f_y} \times \gamma_{mo} = \frac{223.46}{250} \times 1.1 = 981.2 \times 10^3$

9) Solution. 18LB-400. (PG: NO: 138).

$Z_p = 1099.45 \times 10^3 mm^3$

$Z_e = 965.3 \times 10^3$

$I_{xx} = 19306.3 \times 10^4 mm^4$

$h = 400 mm$

$r_1 = 16 mm$

$b_f = 165 mm$

$t_f = 12.5 mm$

$A = 72.43 \times 10^2 mm^2$

$t_w = 8 mm$

$W_t = 56.9 kg/m$

$= 57.9 \times 9.81 = 558.18 kN/m < 0.8$

(i) Section classification:

$b = \frac{b_f}{2} = \frac{165}{2} = 82.5 mm$

$d = h - 2(t_f + r_1) = 400 - 2(12.5 + 16) = 343 mm$

(a) $\varepsilon = \sqrt{\frac{250}{250}} = 1$

(b) $b/t_f = \frac{82.5}{12.5} = 6.6$

(c) $d/t_w = \frac{343}{8} = 42.87$

Refer table (1.2) (2)

$\varepsilon = 1,\ 6.6 < 9.4\varepsilon$

∴ plastic section.

11). Check shear:

$V_d = \frac{f_y}{\sqrt{3}} \times 1/1.1 \times h \times t_w$

$= \frac{250}{\sqrt{3}} \times 1/1.1 \times 400 \times 8$

$= 420 \times 10^3 = 420kN > 107.69.$

Safe

12). Check moment:

$V < 0.6V_d$

$107.69 < 252$ → pg: 53 in IS800 cls: 8.2.1.2

$M_d = \beta_b Z_p f_y \times 1/\gamma_{m0}$ $\beta_b = 1$ for plastic section

$= 1 \times 1099.4 \times 250 \times 10 \times 1/1.1$

$= 249.87 \times 10^6 > M_u\ (223.46 \times 10^6)$

∴ ok.

13). Check deflection:

$Def = 5/384 \frac{wl^4}{EI}$ (for UDL)

$= 5/384 \times \frac{17.3 \times 10^{-3} \times 8.3 \times 10^3}{2 \times 10^5 \times 19306.3 \times 10^4}$

$= 26.37mm < l/300 = \frac{8300}{300} = 27.67$

∴ Hence safe.

Unsupported beam: (PS: NO: 54)

(1) Calculate the moment carrying capacity of a laterally unrestrained ISMB 400 member of length 3m.

Sol:

Step 1: Section properties SP (6).

$h = 400mm$ $\quad d = h - 2(t_f + R)$

$b = 140mm$ $\quad = 400 - 2(16+14) = 340mm.$

$t_f = 16mm$ $\quad t_w = 8.9mm.$

$I_{xx} = 20458.4 \times 10^4 mm^4$ $\quad I_y = 622.1 \times 10^4 mm^4.$

$Z_e = 1020 \times 10^3 mm^3$

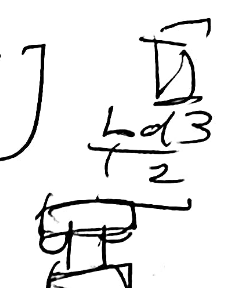
$Z_p = 1175.2 \times 10^3 mm^3.$

Step 2: If laterally unsupported, design bending strength is found by calculating buckling moment.

$$M_{cr} = \sqrt{\frac{\pi^2 E I_y}{(KL)^2}\left(G I_t + \frac{\pi^2 E I_w}{(KL)^2}\right)}$$ (available in your IS-800)

$$G = \frac{E}{2(1+\mu)} = \frac{2 \times 10^5}{2(1+0.3)} = 76.92 \times 10^3 N/mm^2.$$

$$I_t = \sum \frac{b_i t_i^3}{3} \left[\frac{2 \times 140 \times 16^3}{3} + \frac{(400-16) \times 8.9^3}{3}\right]$$

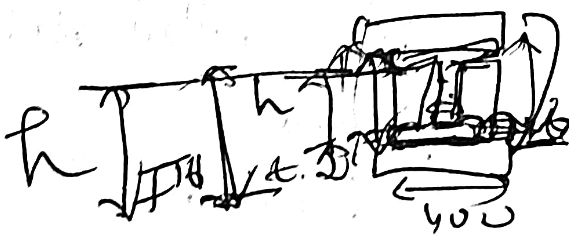


$= 4.725 \times 10^5 mm^4$

$KL = 3000mm.$

$h_f = 400 - 16 = 384mm.$

$I_w = (1 - \beta_f)\beta_f I_y h_f^2$



$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$I_w = (1-0.5) \times 0.5 \times 622.1 \times 10^4 \times 384^2$

$I_w = 2.29 \times 10^{11}\ mm^6$.

$$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 622.1 \times 10^4}{3000}\left[76.92 \times 10^3 \times 4.725 \times 10^5 + \frac{\pi^2 \times 2 \times 10^5 \times 2.29 \times 10^{11}}{3000^2}\right]}$$

$$\boxed{M_{cr} = 343.68\ kN.m}$$

Step 3: Classification. (pg: No: 44).

$$b/t_f = \frac{70}{16} = 4.38 < 9.4$$

$\therefore$ plastic

Step 4: moment

$$\boxed{M_d = \beta_b Z_p f_{bd}}$$

$$\boxed{f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}}$$

(All formula available in IS 800)

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi^2_{LT} - \lambda^2_{LT}]^{0.5}} \leq 1.0$$

~~$= \frac{1}{1.004[}$~~

For rolled steel section $\boxed{\alpha_{LT} = 0.21}$

otherwise,

Formula $\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2_{LT}]$

$= 0.5[1 + 0.21[0.925 - 0.2] + 0.925^2]$

$= 1.004$

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}}$$ $\beta_b = 1$

$$= \sqrt{\frac{1 \times 1175.2 \times 10^3 \times 250}{343.68 \times 10^6}} = 0.925$$

$$\chi_{LT} = \frac{1}{1.004 + [1.004^2 - 0.925^2]^{0.5}} = 0.71751$$

$$f_{bd} = \frac{0.717 \times 250}{1.1} = 163\ N/mm^2$$

$$M_d = \beta_b Z_p f_{bd} = 1 \times 1175.2 \times 10^3 \times 163$$

$$M_d = 191.56\ kN\text{-}m$$

Plate Girder

→ A beam built up of steel plates & shapes which may be welded / bolted together to form a deep beam larger than can be produced by a rolling mill. As such, it is capable of supporting greater loads on larger span.

→ The typical welded plate girder consists of flange plate welded to a deep web plate.

→ Stiffeners — vertical (→ flange, web) — to avoid buckling.
→ horizontal
↳ provide in web.

→ 1) depth of plate girder 1/10 to 1/12 of span length.

① Design a welded plate girder of span 30m to carry a super imposed load of 35kN/m. (Avoid) use of bearing & intermediate stiffner. Use Fe 415 & E250 steel.

Sol:

$udl = 35kN/m$.

factored load $= 1.5 \times 35 = 52.5kN.m$

1). Self wt of plate girder $w_s = \frac{w}{200}$ $= \frac{52.5}{200}$

$= 0.26 kN/m$

2) Total wt $w_t = w_s + w$ $\Rightarrow 0.26 + 52.5 = 52.76 kN/m$

3) $M_u = \frac{w_t l^2}{8}$ $= \frac{52.76 \times 30^2}{8}$ $= 5935.5 kN.m$

4). Shear $V_u = \frac{wl}{2}$ $= \frac{52.76 \times 30}{2}$ $= 791.4 kN$.

5). $d = \sqrt{\frac{M_u k}{f_y}}$ [$67 = k$ for without stiffener]

depth of web plate

$= \sqrt{\frac{5935.5 \times 10^6 \times 67}{250}}$ $= 1162.34 mm \simeq 1100 mm$.

$k = \frac{d}{t_w}$ $= \frac{1100}{t_w} = 67$

$t_w = 16.4$ $\simeq 18mm$

∴ Web plate size } 1100 × 18 mm.

6). Design of flange plate:

Area $\frac{A_f \times f_y \times d}{1.1} \geq M$.

$\frac{A_f \times 250 \times 1100}{1.1} \geq 5935.5 \times 10^6$

$250 \times 10^3 A_f = 5935.5 \times 10^6$

$A_f = 23.74 \times 10^3 \; mm^2$

Assume semi compact

$b/t_f = 13.6$, $A = 23.74 \times 10^3$.

$A = b \times t_f$.

$b = 13.6 \times t_f$

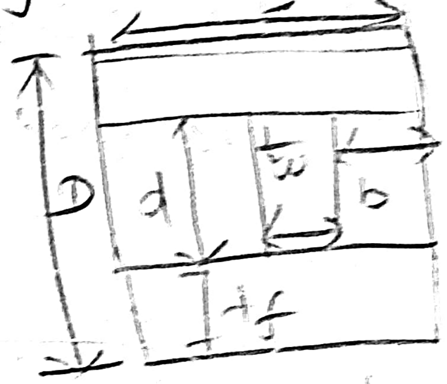
$23.74 \times 10^3 = 13.6 \times t_f$, $t_f = 41.78 \simeq 40mm$

$t_f = 40mm$

$b/t_f = 13.6$

$b = 13.6 \times 40 = 544 \simeq 550 \, mm$.

∴ Provide flange plate size } 550 × 40 mm.



7). Moment:

$M = \frac{Z_e f_y}{\gamma_{mo}}$, $Z_e = \frac{I_{xx}}{y/2}$

$I_{xx} = \frac{BD^3}{12} - \frac{2 \times b_1 \times d^3}{12}$.

B = 550 (mm)

d = 1100

tf = 40mm

D = 2×40 + 1100

= 1180

$b = \frac{B - t_w}{2}$ ← 266

$= \frac{550 - 18}{2} = \frac{532}{2}$

$$= \frac{550 \times 1180^3}{12} - \frac{2 \times 266 \times 1100^3}{12} = 7.63 \times 10^{10}\ mm^4$$

$y = D = \text{depth} = 1180.$

$$Ze = \frac{7.63 \times 10^{10}}{1180/2} = 27.61 \times 10^6\ mm^3$$

$M = 27.61 \times 10^6 \times 250/1.1 = 6.27 \times 10^9\ N\text{-}mm > M_u$ (5935.5)

Hence safe.

8). Check for shear:

(i) Shear capacity: $$V_d = \frac{d \times t_w \times f_y}{\gamma_{mo} \times \sqrt{3}}$$

$$= \frac{1100 \times 18 \times 250}{1.1 \times \sqrt{3}} = 2.59 \times 10^6 > V_u$$

No stiffener.

9) Check bearing

$$F_w = (b_1 + n_2)\, t_w \cdot \frac{f_y}{\gamma_m}.$$

Assume, b_1 & $n_2 = 100$

$$F_w = (100 + 100) \times 18 \times \frac{250}{1.1} = 818.18 \times 10^3 > V_u$$

∴ Safe.

10. Design weld connection web plate & flange plate

(a). $q = F / I_{b}\, (a\bar{y})$ — shear area; b → breadth of flange; I → Imm

$$= \frac{791.14 \times 10^3}{7.63 \times 10^{10} \times 550}\left[550 \times 40 \times \frac{1180}{2}\right]$$

(550 × 40 = area flange; 1180/2 → D/2 = $\bar{y}$)

$= 1.1165\ N/mm^2$

(b) shear force at Junction $= q \times b$

$= 1.145 \times 550 = 62.97$

Strength of weld :-

$$2 \times 0.7 \times 8 \times \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = S.F$$

↓

62.975.

$S = 2.346 \simeq 5m$

∴ As per IS 800 min weld thick 5mm.

UNIT – V

ROOF TRUSS & INDUSTRIAL STRUCTURE

Truss

→ Truss are triangular frame work, consist of essentially axially loaded member which are more efficient in resisting external loads since the cross section is nearly uniformly stress.

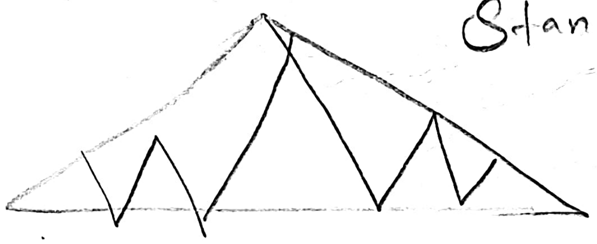
→ It is used in roof of single story industrial buildings, long span floors & roof of multistory buildings, to resist gravity loads.

→ It is also used in walls & horizontal planes of industrial buildings to resist lateral loads and give lateral stability.

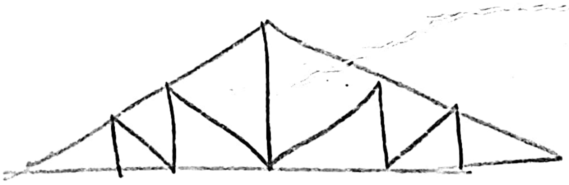
types:

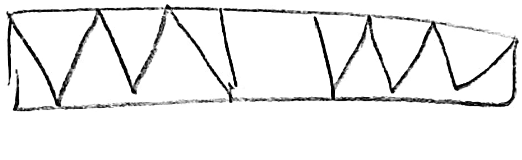
→ trusses are named according to their web configuration.

(1) King post, (2) Fan, (3) Fink | Howe truss



Standard roof truss





Parallel Chord

Top chord : - Established the upper edge.

Bottom chord : - lower edge of the truss.

Girder - supporting secondary truss.

Struts - The member which do not belong to top & bottom chord subjected to compressive force

Spacing of truss : distance b/w 2 consecutive truss

Purlins : beam provided over truss to support the roofing b/w adjacent truss.

load:
(1) Dead load
(2) Imposed load
(3) wind load
(4) other load.

Dead load:
(1) U.W of G.I. Sheet = 80N/m^2
(2) A.C Sheet = 130N/m^2
(3) In general N of covering wt including
Connection (1) 100-150N/m^2 for G.I
(2) 170-200 N/m^2 for A.C sheet.

Live [imposed:
Upto 10° slope = 0.75KN/m^2
For more than 10° slope : 0.75-0.02 (θ-10) KN/m^2
θ - slope of roofing.

Wind load (No 53, IS875, part-3).

① roof truss in Lucknow is to be built for an Industry. The size of shed is 24m × 40m. The ht of building is 12m at the eves. Determine the basic wind pressure.

Sol:

1) Basic wind pressure $P_z = 0.6 V_z^2$ (pg:no:12)

$V_z = K_1 K_2 K_3 V_b$ V_b - basic wind pressure. (pg:no:8)

(pg:no:53 : IS 875-III)

$V_b = 47$ m/s for Lucknow

2) To find k_1 (Risk coefficient).

life of structure = 50.

(pg:no: 11 part:3, 875) table-1.

Assume general building. $K_1 = 1$

3) K_2 - Terrain ht & Structure size K_2

(table:2 - IS 875 part 3 - K_2 - page 12).

pg:no:8   Category - 3 - 875 part III.

pg:no:11   Class - B   "

→ Size of building 24×40m - max dim 20 & 50m.

→ building belongs to Class B

→ building category 3 & ht 12m.

pg:no:12

Ht	B
10	0.88
12	x
15	0.94

$$x = 0.88 - \frac{(12-10)}{(15-10)} \times (0.94-0.88)$$

$K_2 = 0.904$

To find k_3:

SP: 64, PS: No: 58 (iii).

$k_3 = 1 + C_s$; $C = Z/L$

If Ground $c > 0$. $c > 0$

$V_z = 1 \times 0.904 \times 1 \times 47 = 42.488 \, M/S$

wind pressure $P_z = 0.6 \times (42.48)^2 = 1083.14 \, N/m^2$.

$P_z = 1.08 \, kN/m^2$

). Dimension 60m – Dehradun. ht = 400m location, i 250m from crest of slope 1 in 3. on downward slope & its ever board in hill of 9m. Determine the wind pressure

Sol:

1). $P_z = 0.6 V_z^2$ $V_b = 47$

2). $K_1 = 1.07 \, m/s$ Power plant life = 100

3). $K_2 = 0.93$ (category – 2) more than 60. (table – 5.3)

4). $K_3 = 1 + Cs$, $Cs = Z/L$. (PS: No: 58)

Slope 1 in 3. $\tan\theta = 1/3$.

$\theta = 18.43$

2 → height of hill 400m :

$\theta > 17^\circ$ $C = 0.36$

$C = Z/L \Rightarrow L = \frac{400}{(1/3)} = 1200.$

$H = 9m, \; x = 250$ $Le = \frac{Z}{0.3} = \frac{400}{0.3} = 1333.3$

$H/Le = \frac{9}{1333.33}$

$X/Le = \frac{250}{1333.3} = 0.187.$

$S = 1$, (Pg. No: 57) fig 4.5

$K_3 = 1 + Cs$

$= 1 + 0.36 \times 1 = 1.36$

$V_2 = 1.07 \times 0.93 \times 1.36 \times 47$

$V_2 = 63.61$ m/s.

$P_2 = 0.6 \times (63.61)^2$

$P_2 = 2.42$ kN/m2

Purlins. (No design of same simply supported beam)

1) The slope of roof should be less than 30°
2) width of angle leg ⊥ to sheeting ≥ L/45
3) width of angle leg parallel to sheeting ≥ L/60
4) Find its Dead load (m):

(a) wt of sheeting ⇒ wt of sheeting per m^2 × spacing of Purlins.

(b) self weight of purlin /m = D.L T.L/m.

5) F.D.L = 1.5 × D.L/m

6) D.L normal to sheeting D.Lu × cos θ

θ ⇒ slope of roof

7) Live load per m = live load/m^2 × spacing of purlins

FL = 1.5 × L.L

L.L normal to sheeting = L.Lu cos θ

8) W.L normal to sheeting if it is suction put (-ive sign)

9) load calculation. (a) D.Lu + L.Lu

(b) D.Lu + W.Lu. max of above value (W_u)

10) B.Mu = $w_u l^2/10$

11) Check section is plastic / S.C / C

12) Md > B.Mu

13) check deflection

Problem:

1). Design a angle pulins for the following data by Simplified method. Spaing of truss = 4m. Spaing of Purlins = 1.6m, wt of A.C Sheets including Laps & fixe 0.25 kN/m². L.L = 0.6 kN/m². W.L = 1 kN/m² suction inclination of main rafter of truss = 21°.

Sol:

1). $\theta = 21^\circ$

2). $L/45 = \frac{4000}{45} = 88.89\text{mm} \approx 100\text{mm}$

3). $L/60 = \frac{4000}{60} = 66.67\text{mm} \approx 75\text{mm}$.

∴ Section provide = 100×75×6 mm

Self wt = 8 kg/m = 8×9.81 = 78.48 N/m.

4). Dead load:

wt of sheet/m² × spaing of pulins = 0.25 × 1.6

= 0.4 kN/m.

Self wt. of purlin = 0.0785 kN/m.

Total wt/m = 0.4785 kN/m.

5). F.L = 1.5 × 0.4785 ≈ 0.609 kN/m.

6) Normal to Sheeting D.L × cos θ = 0.609 × cos 21°

= 0.5685 kN/m.

7). L.L: L.L = 0.6 × 1.6 = 0.96 kN/m

L.L = 1.5 × L.L = = 1.5 × 0.96 = 1.44 kN/m

Normal to Sheeting L.L cos θ = 1.44 × cos 21°

= 1.34 kN/m.

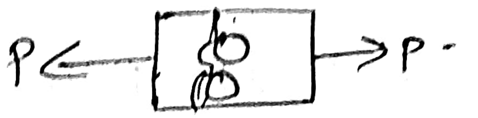
Failure

Rivet.

Procedure:

(1) Tension failure in plate.

(1) Tension failure in plate
(2) Shear failure across one or more planes of rivet
(3) Bearing failure b/w plate & rivets
(4) plate shear / shear out failure in the plate.

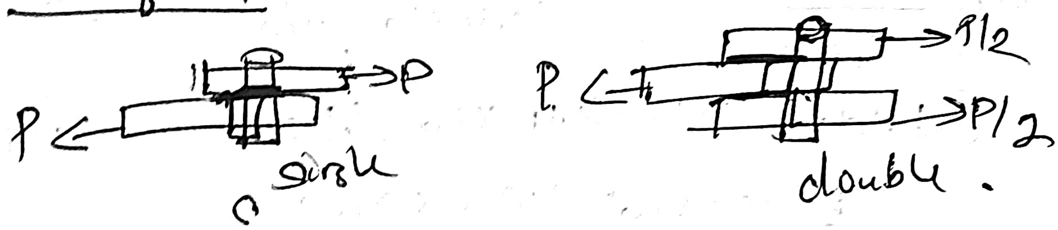


$$P_t = f_t \times \text{resistance section}$$

$$P_t = f_t\,(p-d)t$$

when,
P_t = Pull applied.
f_t = safety tensile stress of plate.
p – pitch. perpendicular to force applied.
t = thickness
d – gross dia.

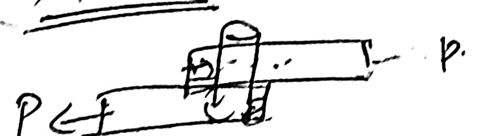
(2) shear failure



$$P_S = \pi/4\, d^2\, f_s \rightarrow \text{for single}$$

$$P_S = 2 \times \pi/4\, d^2\, f_s \rightarrow \text{double.}$$

(3) Bearing



$$P_b = f_b \cdot d \cdot t$$

If n-rows of rivets given,

$$P_t = (p-d)t \cdot f_t \cdot x$$

$$P_s = \pi/4\, d^2 \cdot f_s \times n \quad \& \quad 2 \times \pi/4\, d^2 \cdot f_s \times n$$

$$P_b = f_b \cdot d \cdot t \cdot n.$$

Efficiency $\cdot\ \eta = \dfrac{\text{Least value of } P_t, P_s, P_b}{\text{Strength of solid plate}} \times 100\%$

solid plate $\Rightarrow$ $f_t\ p \cdot t = P$

Permissible stress,

$f_s = 100\,mpa$

$f_b = 300\,mpa$

$f_t = 150\,mpa$

Formula for calculating dia of rivet:

1) Unwin's formula

$d = 6.05\sqrt{t}$

2) French $d = 1.5t + 4$ $t > 15mm$

3) Common $d = \sqrt{50t} - 2$

Dia of rivet

(1) Roof truss, transmission tower, sign board = 12 to 16 mm

(2) Large bridge towers = 25 to 28mm

(3) Bridge work = 22 to 25 mm

(4) Workshops, building = 20mm,

Rivet min - IS-800

1). $p \nless 2.5d$

2) $p \ngtr 32t$ or 300mm

3) $p \ngtr 16t$ or 200mm - tension

4) $p \ngtr 12t$ or 200mm - Compression.

5). $g \ngtr (4t + 100mm)$ or 200mm.

6).

Nominal diameter (mm)	Diameter of hole (mm)	Distance to shear or hand flame cut hole	Distance to rolled, machine flame cut
12 (or) below	13.5 (or) below	19	17
14	15.5	25	22
16	17.5	29	25
18	19.5	32	29
20	21.5	32	29
22	23.5	38	32
24	25.5	44	38
27	29.0	51	44
30	32.0	57	51
33	35.0	57	57

Permissible stress

Type of rivet	Axial Tensile stress	Shear stress	Bearing stress
Power driven rivets	100 N/mm²	100 N/mm²	300 N/mm²
Hand driven rivets	80 N/mm²	80 N/mm²	250 N/mm²

f.

$\leq 25\,mm = +1.5$

$> 25 = +2\,mm$

(1) Rivets, after they have fitted in position, completely fill the hole & hence slip b/w plate is impossible

(2) applied load is resisted equally by all rivets in joints

(3) The centroid of rivet group is on the axis of loading

(4) Deformation of plates is neglected

(5) Shear stress distribution across section of rivet shank is uniform

(6). Bending - neglected.

Problem!

(1) Determine the safe load & efficiency of double cover butt joint. The main plates are 12mm thick connected by 18mm dia rivet at pitch of 100mm. Design cover plate also. What is the percentage reduction in the efficiency of joint if the plates are lap jointed?

Design a Gantry Girder for the following data?

Cran capacity => 200kN

span of gantry girder - 7.5m

Span of crane girder - 15m

self weight of crane girder => 200kN

self weight of trolley = 40kN

min hook approach = 1.2m

Wheel base of crane = 3.5m

self wt of rail section = 300kN

Take f_y = 250MPa. Assume no lateral restrai along span.

Sol:

① Max wheel load: (W)

W = Cran capacity + wt of trolley

= 200 + 40 = 240kN

② wt of crane girder per m = $\frac{\text{Total wt of crane girder}}{\text{Length of crane.}}$

$= \frac{200}{15} = 13.33$ kN/m