GOVERNMENT POLYTECHNIC

BHUBANESWAR-23

DEPARTMENT OF CIVIL ENGINEERING

LECTURE NOTES

Year & Semester: 3rd Year, 5th Semester
Subject code/Name -Th-2, STRUCTURAL DESIGN-II
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Ch-1

Steel Structure

* It is an assemble of a group of a members (elements) expected to sustain their share of applied forces and to transfer them safely to the ground.

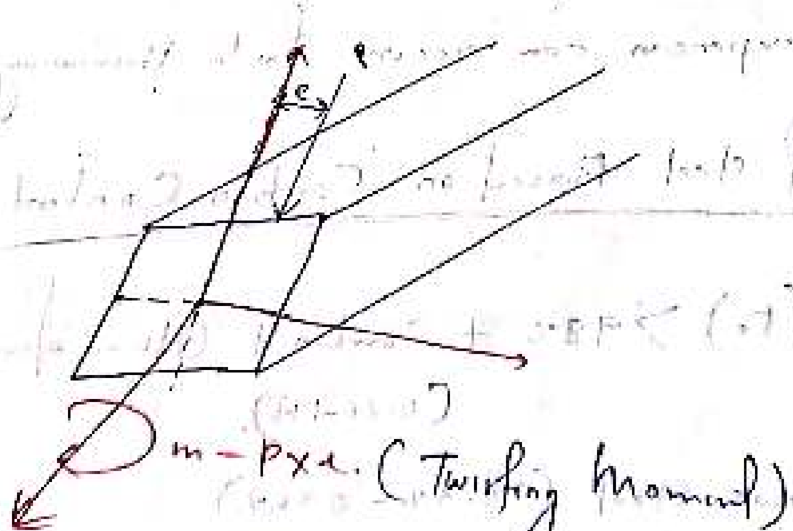
Type of structure

on the basis of supporting system.

* Line structure (cables, beam, column)
* Surface structure.
* Framed structure (space structure)

Structural Elements

1) Tension Member
2) Compression Member
3) flexural Member
4) Beam - Column.
5) Torsion - Member



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Advantages of steel

* Steel member have high strength per unit weight. (Long span bridges, high-rise buildings).
* Ductile materials. (% elongation >15%)
  (% elongation < 5%) brittle materials.
* Conveniently handle and transported.
* Modification can be easily done.
* erected at fasten rate.
* Highest scrap Value.
* Properties of steel mostly donot change with time.
* Properly maintained steel structure have a very longlife.

Disadvantages

* When placed in exposed condition, subjected to corrosion.
* Steel structure need fire proof treatment.
* Fatigue failure. (Reversal stress).
* Thermal expansion can occure due to Increasing Temp.

Classification of steel Based on Carbon Content.

Iron (Fe) >98% + Carbon + Other elements
(0.1%-1.1%).

1) Low carbon steel (0.1% - 0.25%)
2) Medium carbon steel (0.25% - 0.6%).
3) High carbon steel (0.6% - 1.1%)

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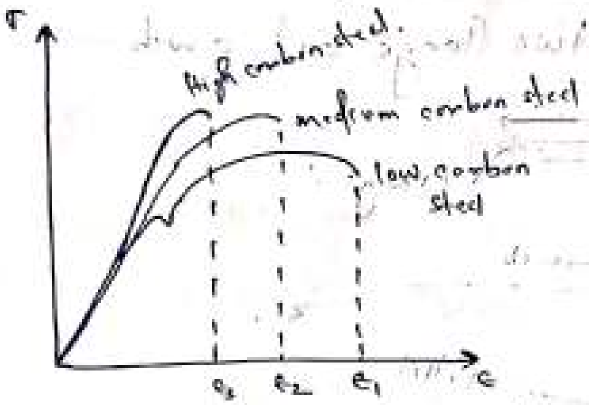
Various Iron-carbon Alloys.

* Cast Iron - >2% → Brittle, Hard, Rigid.
* Steel — Carbon content (1% - 2%)
* Wrough Iron Carbon Content <0.05% (Soft, weldable)



Rolled Steel Section

* Steel is manufactured in rolling mill.
* Hot steel is passed from series of rollers to get the desire shape.
* Generally those shape are preferred which provide higher section Modulus.

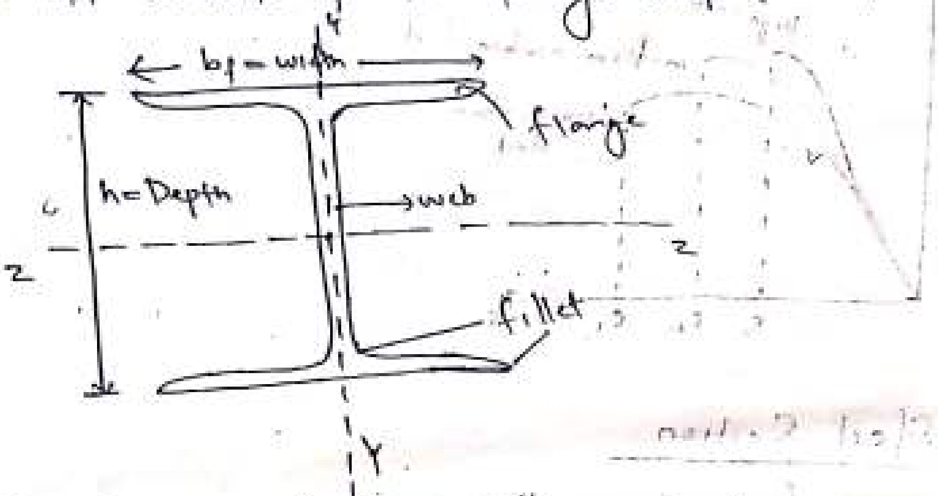
Types of Rolled steel Section

1) Rolled steel Beam Section (I).
2) Rolled steel Channel Section (C)
3) Rolled steel T- Section.
4) Rolled steel Angle Section. (1st Sectn manufactured in 1819)
5) Rolled steel Hollow Section.

6) Rolled Steel Plates.
7) Rolled steel flats.
8) Rolled Steel Bars.
9) Rolled Steel Strip
10) Rolled steel sheet.

1) Rolled steel Beam Sec (I- Section).

* It consists of two flange and a web.



Different Series of Beam Section

* ISLB → Indian Standard Light Beam.
* ISMB → Indian standard Medium weight Beam. (Beam)
* ISHB → Indian standard Heavy weight Beam. (column)
* ISWB → Indian standard wide flange Beam.
* ISJB → Indian standard junior Beam.

Beam Section are designated with depth of the Section & Mass per unit length.

Eg:- ISWB 600 @ 50.6 kg/m.

depth

mass per unit length.

2) Rolled Steel Channel Section ([).

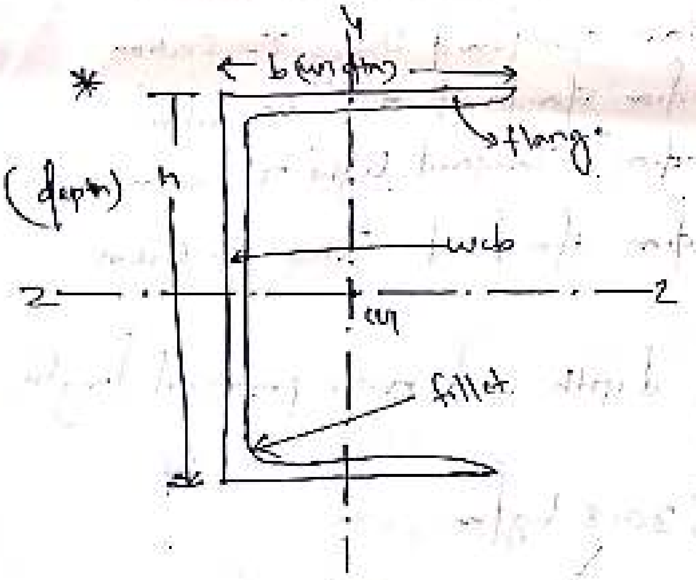
* Channel Section are used as purlins in a roof truss.

* Specially channel Section are used to make builtup Column Section.

following are the types of channel Section.

1. ISLC → Indian ~~standard~~ Light Channel
2. ISMC → Indian standard ~~Special~~ medium Channel.
3. ISSC → Indian standard Special Channel
4. ISJC → Indian standard Junior channel.

*



* They are designated with depth of Section and mass per unit length.

ISMC 300 @ 30.8 kg/m

depth (mm)

mass per unit length.

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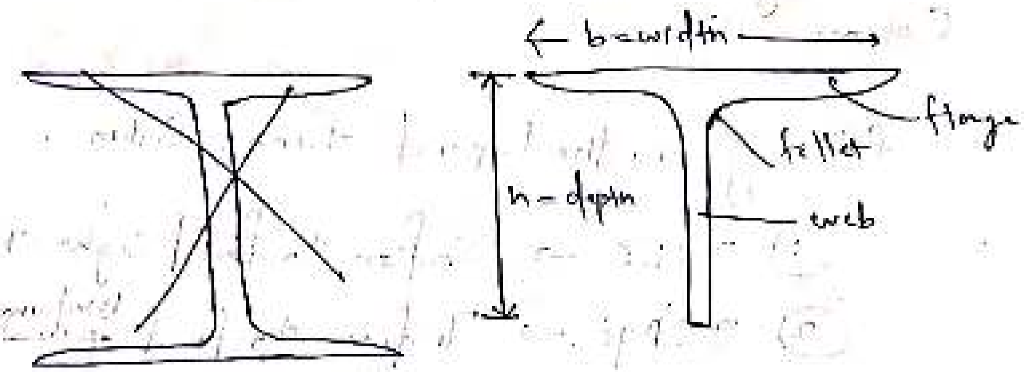
3) Rolled Steel T-Section

* It has one flange and a web.
* T-section are used in column, bracket connection and in steel water tank.



There are 5 Series of T-Section.

1. ISNT – Indian standard normal T-Section
2. ISHT – Indian standard Heavy T-Section
3. ISST – Indian standard short T-Section
4. ISLT – Indian Standard light T-Section
5. ISJT – Indian standard Junior T-Section

* It is designated by depth and mass per unit length

ISHT 200 @ 30.8 kg/m

d = 200mm, mass = 30.8 kg/m.

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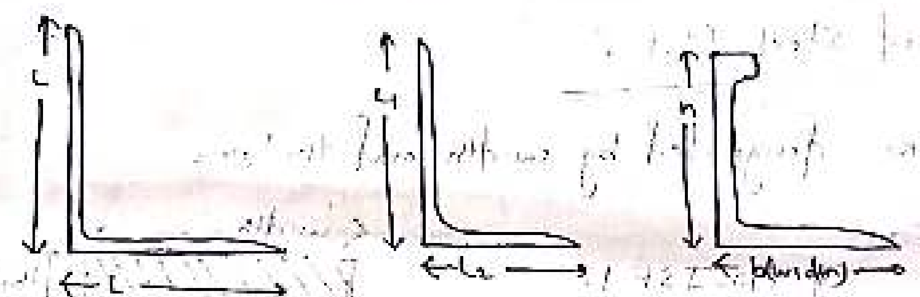
4) Rolled Steel Angle Section,

* This is the 1st Section to be manufactured.

* Angle Section are used to make builtup section, purlin's beam, stiffners & to establish connection.

Angle Section are 3 types, !

1. Equal Angle Section,
2. Unequal Angle section
3. Bulb Angle section



(equal angle sec^n) (Unequal Angle sec^n) (Bulb angle)

* Angle section are designated with both leg size & thickness

eg:- ISA 60×60×8 (equal Angle)

ISA 100×60×8 (Unequal Angle).

* Bulb angle section has consolidated area at one of the leg which provides stiffness to the angle.
* They are used in ship buildings & in Gantry cranes.

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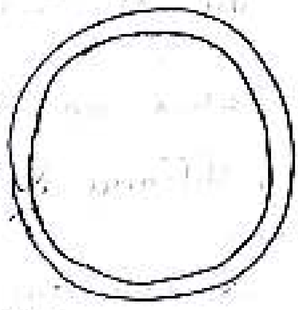
5) Rolled steel tube Section

* They are designated by outside dimension & thickness

* Hollow section are preferred in those member where torsion is dominant.



There are 3 types of tube Section,

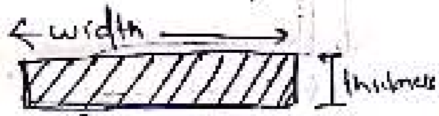
1. Square hollow section
2. Rectangle hollow section
3. circular hollow section.

6) Rolled steel flat.

* They are designated by width and thickness.

For eg:- 100 ISF 10

100mm width of flat
10mm thickness of flat.



(steel flat section)

(Generally length of flat upto ≤ 400mm)

* flat Sections are used in lacing system.
(Thickness of flat $t \geq 5$ mm)

7) Rolled steel Plate Section ($t \geq 5$mm)

* It is designated by length, width and its thickness.

For eg:- ISPL 2000 x 1000 x 8

* Plate are used in column base.

* Gusset plate to form a connection between different Members.

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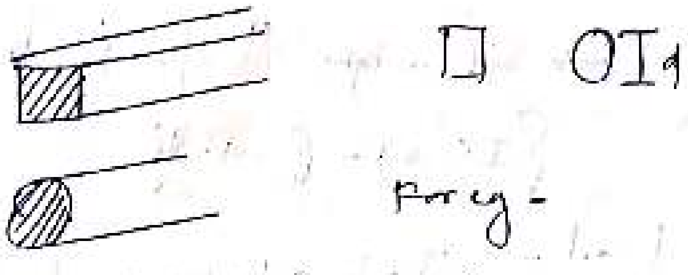
8) Rolled Steel Bars



For eg -

Steel bar are classified into 2 type

① Indian Standard Round bar (ISRO)

② Indian Standard Square Bar (ISSQ)

* These Section are designated by diameter incase of round bars and by sides incase of Square bar.

Physical Property of Steel

* Specific gravity of steel = 7.85
* Unit Mass of steel (ρ) = 7850 kg/m³
* Modulus of elasticity (E) = 2×10^5 mpa
* Poisson's Ratio (μ) = 0.30 (IS 800:2007)
* Modulus of Rigidity (G) = 0.769×10^5 mpa
* Coefficient of thermal expansion (α) = 12×10^{-6} /°C

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Designe Philosphies

① Loads

a) Dead load → Self weight of structural element
{IS 875 (part-I)}

b) Imposed load → Due to activities inside the structure
(Table fornilure etc)
{IS 875 (part-II)} (part-IV)

c) Wind load → It depends upon varies factors such as pressure intensity of the wind, topography, type of structure, shap & size and Risk co-efficient.
{IS 875 (part-III)}

d) Earthquake load & Seismic Load – {IS 1893}

Load Combination

	FOS (Load factor).
→ DL + IL	1.5
→ DL + WL/EL	1.5
→ DL + IL + WL/EL	1.2
→ DL + AL	0.9 or 1.5

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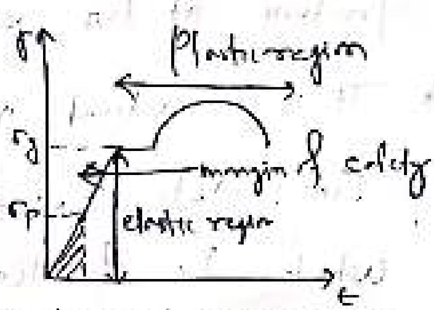
Types of Design Philosophies

1. Working stress Method (WSM)
2. Ultimate strength Method (USM)
3. Limit state Method (LSM).

1. Working stress Method

Assumption

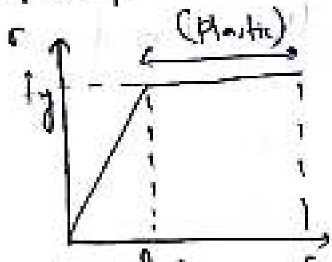


* Material behaves elastically (plastic strength of steel neglected)
* Structural member designed for worst/worst combination of loading.
* Uneconomical → bcz. In this design we reduce stress by using FOS. So, number of material required more.

$\boxed{\sigma_p = \sigma_y / FOS}$

2. Ultimate strength Method (Plastic Method of design)

* In this method Plastic region of stress strain curve of mild steel was also considered.
* Members are designed for collapse or Ultimate load.



collapse load = working load × load factor.

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* The result of this method was section size requirement was smaller, Hence economical.

* In this method serviceability condition we are not considering. (whether excessive deformation is occurs or not, Deflection and vibration).

3 Limit state of Method

* In this method, the design objective is to make structure fit for use. for some target reliability.
* It is a statical method of design.
* Limit state considers all the posibilities Because of which the structure will become unfit for use.

There are two types of limit state

Limit state of strength	Limit state of serviceability
* Strength (Yielding, buckling).	* Deflection
* stability against overturning and sway.	* Vibration
* for fracture due to fatigue	* fatigue check (Including repairable damage due to fatigue).
* Plastic collapse.	* Corrosion.
* Brittle Fracture.	* fire.

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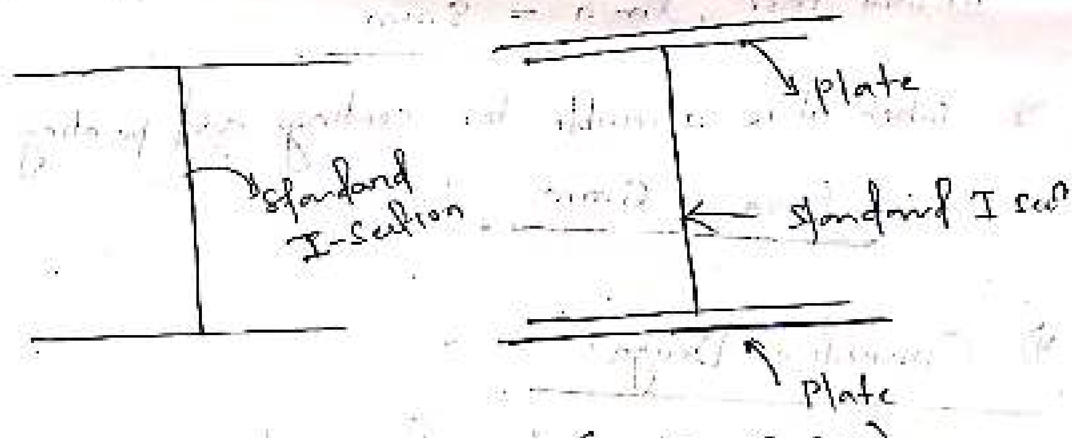
Special Consideration in steel designe

1) Size and Shape
2) buckling
3) Minimum thickness
4) Connection designe.

① Size and shape

The member of steel structure should be designed to consists of any of the avilable sections or combination of them.

ex- A beam section may be a standard I-Section or it may consists of builtup Section.



(Builtup Section)

② Buckling

The permissible load per unit area in steel is much higher as compared to permissible value in concrete. Therefore the same load the cross-sectional area of a steel member is smaller.

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* As the member in steel structure are more slender the compression member in steel structure are liable to buckling.

* To account for buckling the code specify that part of section be taken as ineffective.

Minimum Thickness

If very thin section are used & small amount of corrosion may result into large percentage reduction in effective area. hence minimum thickness to be used in structural member.

* When the member is directly expose to whether & it is not accessible to perform coating or painting in that case, $t_{min} = 8\,mm$.

* when it is accessible for coating and painting,

  $t_{min} = 6\,mm$.

4) Connection Design

There are three types of connection are commonly used.

a) Riveted Connection
b) bolted Connection.
c) Welded Connection

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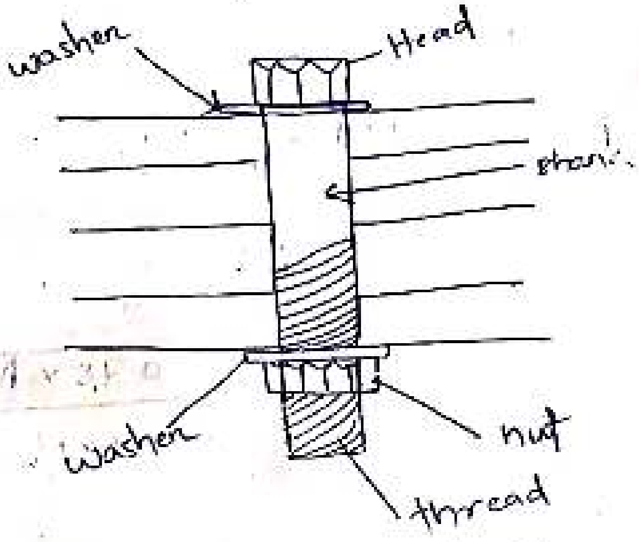
Ch-2

Bolted connection

* Bolt can be defined as the piece of metal with head form at one end and shank threaded at other end.

* Threads are required to receive a nut.

*



Types of bolts

① Unfinished bolt

② High strength friction grip bolts (HSFG bolts)

① Unfinished bolts.

* They are also called as ordinary, Rough, Black & bearing type bolts.

* Unfinished bolts are made from low carbon steel circular steel.

* They are used when loading is static.

* They are not recommended when the loading is impact, vibration, Reversal of stresses.

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* The size of ordinary bolt is in between <u>5mm - 36mm</u>.
* Designated as M_{16}, M_{20}.

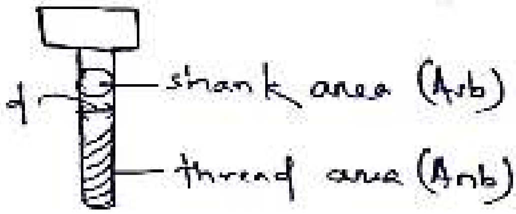
16 → Diameter of bolts.

* The cross-sectional area at the threaded portion can be assumed equal to 0.78 times the shank area.

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

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

$$= 0.78 \times \pi/4 \times d^2$$



* Generally, the grade of bolt used is property <u>class 4.6</u>.

4 → 1st no → Designate 1/100 th of ultimate strength of bolt material.

$$f_{ub} = 4 \times 100 = 400 \text{ N/mm}^2$$

0.6 → 2nd No → Designated Ratio of yield strength and Ultimate strength of bolts.

$$f_{yb} = 0.6 \times 400 = 240 \text{ N/mm}^2.$$

* <u>Bolt hole.</u> (IS 800 : 2007 - Pg No - 73 - (T-19) - (cl - 10.2.1))

$d < 12 - 14 \Rightarrow d_0 = 1\text{mm} + d$

$16 < d < 24 \text{ mm} \Rightarrow d_0 = 2\text{mm} + d$

$d > 24\text{mm} \Rightarrow d_0 = 3\text{mm} + d$

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* In this bolt clamping action is negligible, therefore frictional resistance is zero.
* Hence load is transfer by bearing mechanism.

② High strength friction grip Bolt (HSFG) bolts.

* These bolts are made from medium carbon steel.
* There strength is increased by the Quenching process or by adding alloys.
* These bolts can be tightened up to a large degree as compared to ordinary bolts.
* Due to which large clamping action is developed, hence frictional resistance is generated between connecting elements.
* Load is transfered mainly by friction - mechanism.
* These bolts are used when the loading is, Dynamic, impact & vibration.
* Generally dia of HSFG Bolts is 12mm - 36mm.
* Generally property classes 10.9 S is used.

S → strength, $f_{ub} = 10 \times 100 = 1000\ N/mm^2$

$f_{yb} = 1000 \times 0.9 = 900\ N/mm^2$

* Due to tightening of nut bolt is pretensioned which generate friction between connecting elements.

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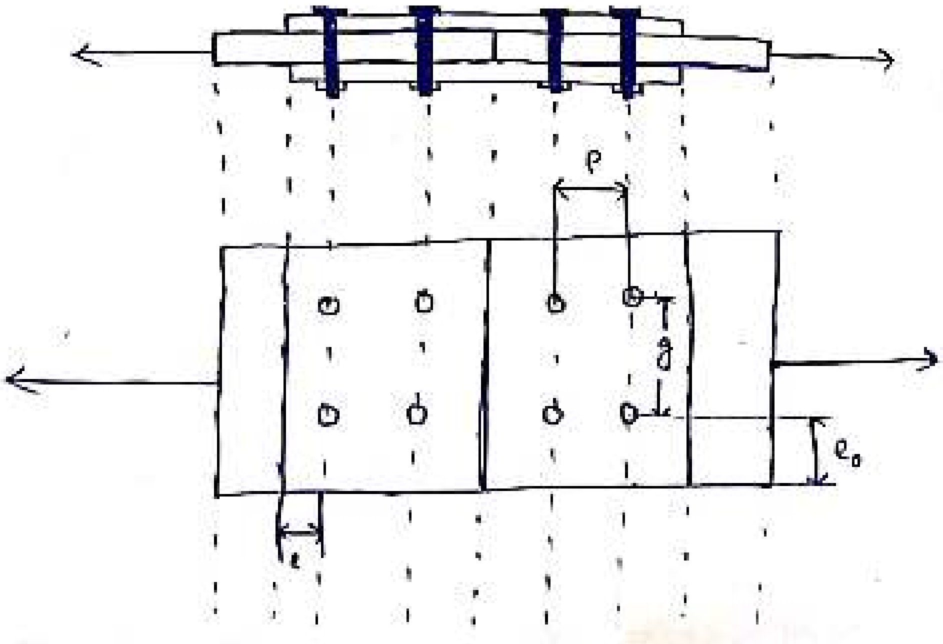
Advantages of Bolted connection

* Making joint is noise less.
* Do not need skill labour.
* Need less labour.
* Connection can be made quickly
* Structure can be put to use immidiately.
* Modification if any can be done easily.
* Working area required in the field is less.

Dis advantages of Bolted Connection

* Due to vibration nuts are likely to loosen.
* Rigidity of joints is reduced due to loose fit resulting into excessive deflection.

Different Terminology



p = Pitch
g = gauge
e_0 = edge distance
e = end distance

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1. Pitch (P)

It is the center to center distance between two consecutive bolt holes in the direction of parallel to apply force / stress.

* It is the center to center distance between two consecutive bolt holes measured in longer direction of connection.

* Minimum Pitch (P_{min}) - As per IS 800-2007 page No. - 73
CL. 10.2.2

$$\boxed{P_{min} = 2.5 \times d}$$ $\therefore d \rightarrow$ Nominal diameter of bolt.

* Maximum Pitch (P_{max}) = As per IS 800-2007 Pag No - 74
CL - 10.2.3.2

1. For Tension Member - (P_{max}) :- 16t or 200 mm, min^m.
2. For Compression Member (P_{max}) = 12t or 200 mm min^m.

$\therefore t \rightarrow$ Thickness of

* As per IS 800-2007 pag No - 74, CL- [redacted] 10.2.3.1 -
The distance between the centers of any two adjacent bolts shall not exceed. 32t or 300 mm.

$\therefore t \rightarrow$ Thickness of thinner plate.

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② Gauge length (g).

* Center to center distance between two bolts measured in the direction perpendicular to applied force/stress.

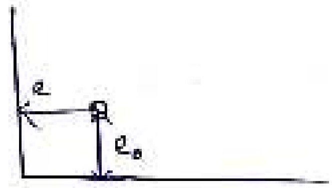
* In case of wide plate it is defined as the distance between two adjacent bolts measured in shorter direction.

End distance (e)

* It is the distance between center of bolt to the nearest edge of the member in the direction of applied force.

Edge distance. (e_0)

* It is the distance between the center of bolts to the nearest edge of the member measured in the direction perpendicular to applied force / stress.



* According to IS 800:2007 cl- 10.2.4.2 Pg No-74
Minimum value of end & edge distance.

$(e \, \& \, e_0)$ minimum $= 1.7 \times d_0$ (For hand flame cut edge.)
$= 1.5 \times d_0$ (For machined flame cut edge.)

$\therefore d_0 \rightarrow$ Diameter of bolt holes.

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* According to IS 800:2007 cl-10.2.4.3 Pg No-74.
For Maximum value of e.g.e.

$(e.d.e)_{max} \rightarrow 12te$ ∴ $t \rightarrow$ thickness of thinner member

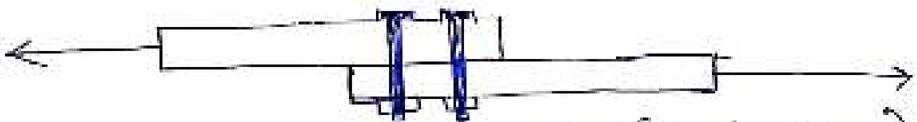
$\epsilon = \sqrt{\frac{250}{f_y}}$

Types of Bolted Connection.

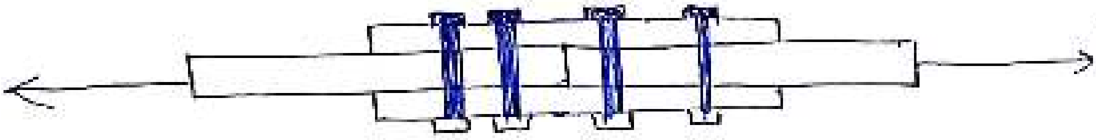
(1) On the basis of types of ~~bolt~~ force in the bolt.

a) Shear Connection :- Bolt are subjected to shear stress at a single or multiple cross-section

For eg:- Lap Connection,
Butt Connection.



Lap Connection (single Shear).



b) Hanger Connection → In this Connection bolt cross-section will be under tensile stress.

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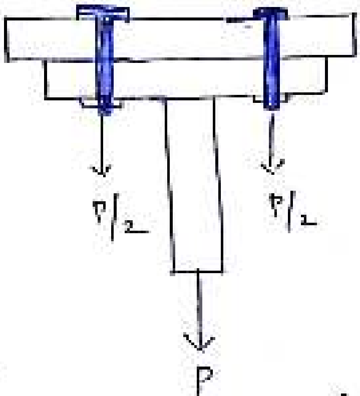
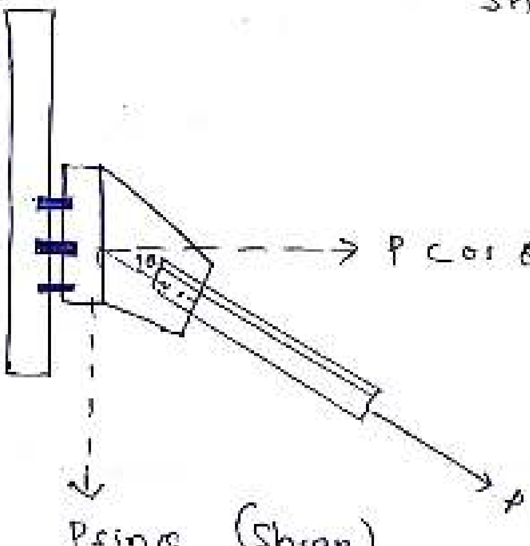


(Tension Connection)

c) Inclined bracket connection → Bolt are under combined state of stress due to shear and tension.



(Tension + Shear Connection)

Types of bolted joint

① Lap joint.

② Butt joint → Single cover butt joint
→ Double cover butt joint.

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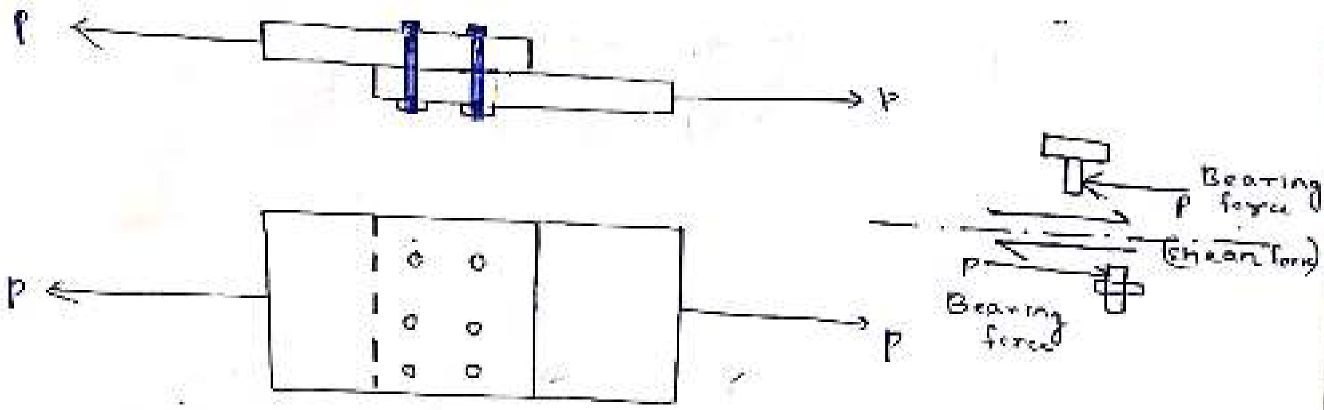
① Lap joint

* When two structural elements are overlapped with each other then the joint formed is called Bolted lap joint.



(Double bolted lap joint)

* The bolt will fail mainly due to shear

② Butt joint

* In this joint the elements to be connected are placed edge to edge and then cover plate is provided at the top or at the top and bottom of the element.

* If the cover plate is provided only at top → single cover butt joint

* If the cover plate is provided on both side → Double cover butt joint.

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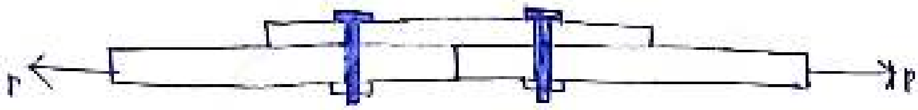
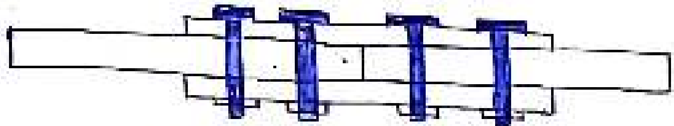
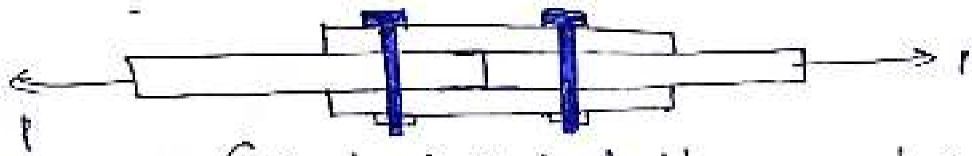


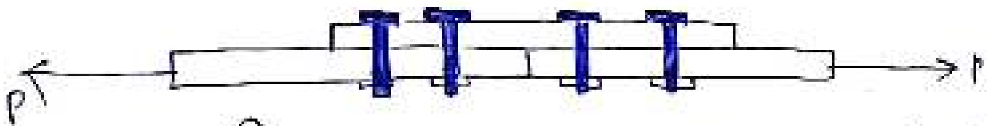
(Single bolted single cover butt joint).



(Double bolted double cover butt joint)



(Single bolted double cover butt joint)



(Double bolted single cover butt joint)

Failure of bolted joint

failure of bolted joint can occure in two ways.

1. failure of bolt
   - shear failure of bolt
   - Bearing failure of bolt
   - Tensile failure of bolt
2. failure of Member
   - Tensile failure of member
   - Bearing failure of member
   - shear tear out of member
   - Block shear failure of member

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Strength of bolted joint (Bearing - type bolts) (unfinished bolts)

Assumption

* Frictional resistance between the connected elements will be neglected.
* All the bolts will share the load equally.
* Any undesirable bending stress will be neglected.
* The stress distribution is assumed to be uniform.

① Design shear strength of bolts.

* The nominal shear strength of bolt depends upon grade of bolt, no of shear plane & Location of shear planes.

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

(IS 800-2007 Pg No - 75 cl - 10.3.3)

∴ V_{nsb} - Nominal shear capacity of a bolt

f_{ub} - Ultimate tensile strength of bolt

n_s → no. of shear plane intercepting in the shank portion.

n_n → no. of shear plane intercepting in the threaded portion

A_{sb} → Shank Area

A_{nb} → Threaded Area = $0.78 \times A_{sb}$.

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Case-1 – Lap joint or single cover butt joint [1 shear plane]

$n_s = 1, \quad n_n = 0$

$n_s = 0, \quad n_n = 1$

Case-II Double cover butt joint [2 shear plane]

$n_s = 2 \quad n_n = 0$

$n_s = 0 \quad n_n = 2$

$n_s = 1 \quad n_n = 1$

Design shear capacity of Bolt

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

$\gamma_{mb} \rightarrow$ Partial Safety Factor (IS 800-2007 Pg No-30, T-5, cl-5.4.1) ($\gamma_{mb} = 1.25$)

$$\boxed{V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \times [n_s \times A_{sb} + n_n \times A_{nb}]}$$

* Reduction factor

① Reduction factor for long joint [IS 800-2007 Pg No-75 Cl No-10.3.3.1]

* When the distance between first and last bolts in the joints measured in the direction of load transfer exceeds $15 \times d$. The nominal shear capacity 'V_{dsb}' reduced by factor (β_{lj}).

If $L_j > 15d$

$$\boxed{\beta_{lj} = 1.075 - \frac{L_j}{200d}}$$

$L_j \rightarrow$ Length of joint.

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② Reduction Factor for Large grip length [IS 800 2007 Pg 140-75 CL-10.3.3.2]

* Grip length 'L_g' equal to total thickness of connected Plates exceed $5 \times d$, then shear capacity reduced by Reduction factor (β_{lg}).

$$\boxed{\beta_{lg} = \frac{8d}{3d + L_j}}$$

d = nominal dia of bolt.

③ Reduction Factor due to Packing Plate:- [IS 800 2007 Pg 140-75, CL-10.3.3.3]

* If the thickness of packing plate '$t_{pkg} > 6\,mm$' then bending effect are generated in the bolt.

* Due to which shear capacity of the bolt is reduced by reduction factor 'β_{pkg}'

$$\boxed{\beta_{pkg} = 1 - 0.0125 \times t_{pkg}}$$

② Bearing strength of Bolt. [IS 800-2007, Pg-75, CL-10.3.4]

* Bolt failure will occur, if high grade plates are connected using low grade of bolts.

* Bearing strength is independent of the grade of bolt.

* Bearing strength is dependent upon arrangements of bolt holes in the plate. (Gap between bolt holes, e, e_1)

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* Nominal Bearing strength of bolt.

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

where,

$f_u \rightarrow$ Ultimate tensile stress of plate

$d \rightarrow$ Nominal diameter of bolt.

$t \rightarrow$ Thickness of thinner mainplate
on
Submation of cover butt jointplate
} Min^m

k_b :- Bearing factor
depend upon

i) $\frac{e}{3d_0}$

ii) $\frac{p}{3d_0} - 0.25$

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

iv) 1

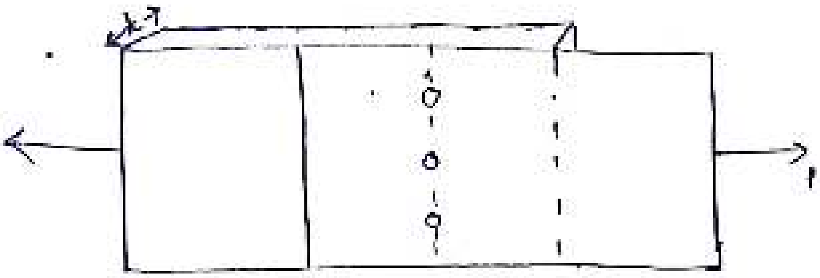
} Min^m $\Rightarrow k_b$

∴ $d_0 \rightarrow$ diameter bolt holes
$p \rightarrow$ pitch
$e \rightarrow$ end distance
$f_{ub} \rightarrow$ Ultimate ~~tensile~~ tensile stress of bolt.

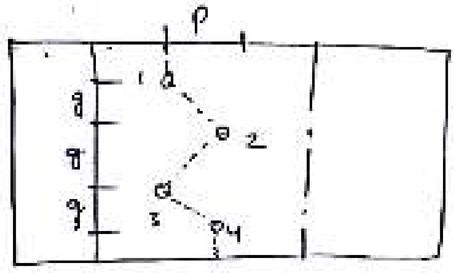
Design Bearing strength of bolt $\boxed{(V_{dpb}) = \frac{V_{npb}}{\gamma_{mb}}}$

∴ $\gamma_{mb} = 1.25$ (Pg-31, T-5).

(5) Design Tensile strength of plate. (T_{dp}). [IS 800-2007 Pg No - 32. Cl - 6.3.1]



(Chain pattern)



(Staggered pattern)

$A_n \rightarrow (B - 3 \times d_0) \times t$

$A_n = \left[B - 4 \times d_0 + \frac{p^2}{4g} \times 3\right] \times t$

In general

$$\boxed{A_n = (B - n \times d_0) \times t}$$

* Staggered pattern of bolting is better than the chain pattern. Because it provides large value of net sectional Area in tension 'A_n'.

$$\text{Staggered pattern } (A_n) = \left[B - n\,d_0 + \sum_{i=1}^{n'} \frac{P_s^2}{4g_i}\right] \times t$$

$B \rightarrow$ width of plate

$n \rightarrow$ no. of bolt holes at the section consider

$d_0 \rightarrow$ Diameter of bolt holes.

$n' \Rightarrow$ no. of staggered pitch traved by crack

$P_s \Rightarrow$ Staggered pitch.

$g \Rightarrow$ Gauge Distance.

$t \Rightarrow$ Thickness of thinner Mainmember.

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Design Tensile strength of plate.

$$\boxed{T_{dp} = \frac{0.9 \times f_u \times A_n}{\gamma_{ml}}}$$

$\therefore \gamma_{ml} \rightarrow$ Partial safety factor for member resisting.

$\gamma_{ml} - 1.25$ [IS 800-2007, Pg-30, T-5]

$f_u \rightarrow$ ultimate tensile stress of plate.

$A_n \rightarrow$ Net Sectional Area.

4) Effeciency of Joints.

* Efficiency of bolted joint is defined as the percentage of load that can be transffered by a bolted joint in terms of the strength of solid plate.

$$\text{Effeciency of joint } (\eta) = \frac{\text{strength of joint}}{\text{strength of Solid plate}} \times 100$$

Strength of joint :- $\min^m$
1) shear strength of bolt
2) Bearing strength of bolt
3) Tensile strength of Plate

(Bolt value)

$$\text{Strength of Solid plate} = (T_{dg}) = \frac{A_g \times f_y}{\gamma_{mo}} \quad \left[\begin{array}{l}\text{Pg No - 32} \\ \text{cl - 6.2}\end{array}\right]$$

$A_g \rightarrow$ gross Area of cross section.

$f_y \rightarrow$ yield stress of materials.

$\gamma_{mo} \rightarrow$ partial safety factor $= 1.1$ (Pg-30, T-5).

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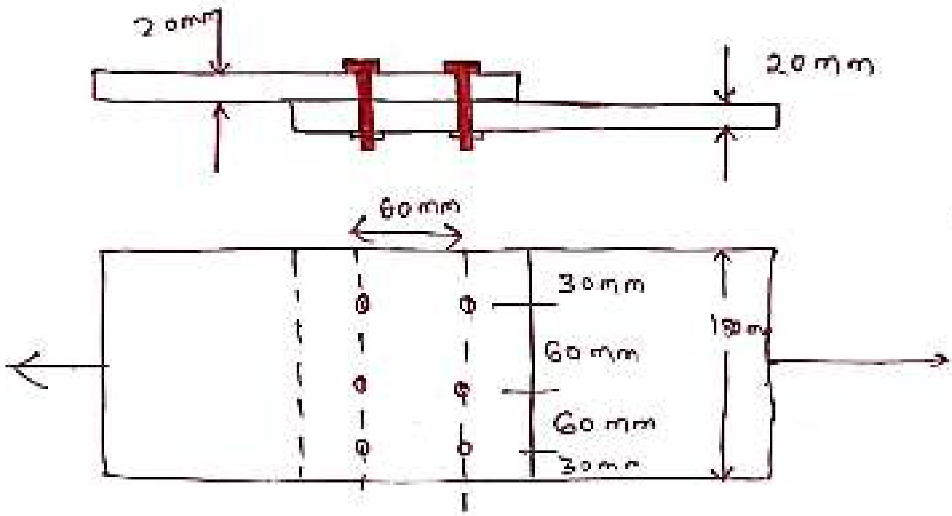
(9)

Question - 1

find the efficiency of a lap joint are shown in fig. M_{20} bolts of grade 4.6 and Fe410 (E250) plate are used.



Data given

for 4.6 grade of bolt

$f_{ub} = 4 \times 100 = 400\ N/mm^2$

$f_{yb} = 400 \times 0.6 = 240\ N/mm^2$

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

thickness of plate = 20mm

width of plate (B) = 180mm

diameter of bolt (d) = 20mm

Diameter of bolt hole (d_0) = 20+2 = 22mm.

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① Tensile strength of plate

Net sectional Area $(A_n) = (B - nd_0) \times t$

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

$= 2280 \ mm^2.$

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

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

$= 673056 \ N$ or $673.056 \ kN.$

② Strength of Bolt (Number of Bolt → 6)

a) Shear strength of bolt

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

Single shear, $n_n = 1$, $n_s = 0$ (Assume).

A_{sb} = Nominal shank area of bolt

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 20^2 = 314.159 mm^2$

(Threaded Area of bolt) $A_{nb} = 0.78 \times A_{sb}$

$= 0.78 \times 314.159 = 245.044 \ mm^2.$

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(11)

Design Shear strength of one bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times \left[n_s \times A_{sb} + n_n \times A_{nb} \right]$$

$$= \frac{400}{\sqrt{3} \times 1.25} \times \left[0 \times A_{sb} + 1 \times 245.044 \right]$$

$$= 45272.39\ N \simeq 45.272\ kN$$

Design shear strength of 6 bolts

$$V_{dsb} = 45.272 \times 6 = 271.632\ kN.$$

(b) Bearing strength of bolt.

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

k_b :- Bearing factor

$$\Rightarrow \frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.45$$

$$\Rightarrow \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.65$$

$$\Rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$$

$$\Rightarrow 1$$

which one is Minimum

$$\therefore K_b = 0.45.$$

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∴ Design Bearing strength of one bolt

$$V_{dpb} = \frac{2.5 \times 0.45 \times 20 \times 20 \times 410}{1.25}$$

$$= 147.60 \, kN$$

∴ Bearing strength of 6 no of bolts.

$$V_{dpb} = 6 \times 147.60 = 885.6 \, kN.$$

$$\text{efficiency of joint } (\eta) = \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$$

∴ Strength of joint ⇒ min^m (271.63 kN, 673.045 kN, 885.6 kN).

∴ strength of joint = 271.63 kN

$$\text{Strength of solid plate} = \frac{A_g \times f_y}{\gamma_{m0}}$$

$$= \frac{6 \; 20 \times 180 \times 250}{1.1}$$

$$= 818.81 \, kN$$

$$\text{efficiency of joint } (\eta) = \frac{271.63}{818.81} \times 100$$

$$= 33.17 \,\%$$

Ans

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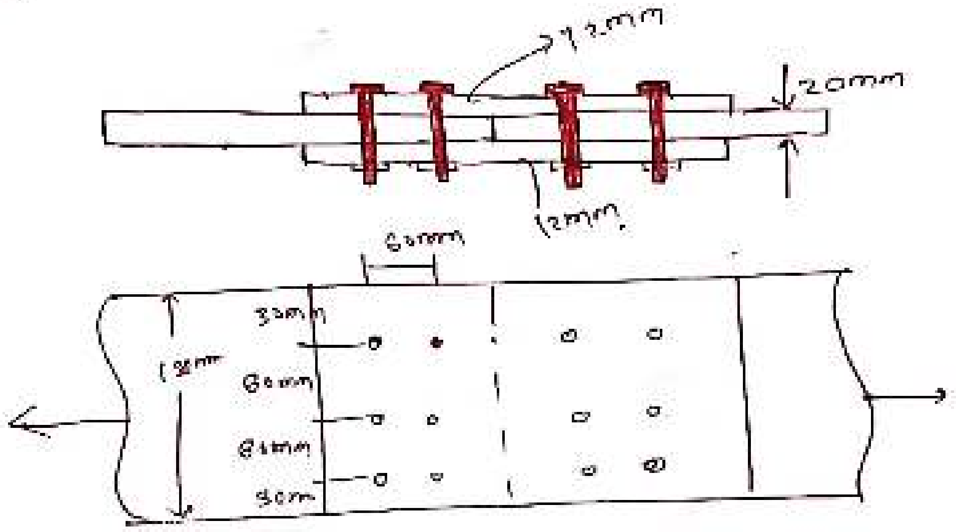
11

Question-2

find the efficiency of joint if in the above example instead of lap joint, but joint is made using two cover plate each of size 12 mm and 6 Nos. of bolt on each plate.



Data given

$f_{ub} = 400\ N/mm^2$

$f_u = 410\ N/mm^2$

$f_{yb} = 240\ N/mm^2$

$f_y = 250\ N/mm^2$

width of plate $(B) = 180\ mm$

thickness of plate $(t) = 20\ mm$

diameter of bolt $(d) = 20\ mm$

diameter of bolt hole $(d_0) = 20 + 2 = 22\ mm$

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① Desinge Tencile of plate

Net Sectional area $(A_n) = (B - nd_0) \times t$

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

$= 2280\ mm^2$

Design Tensile strength of plate

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

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

$= 673.056\ kN.$

Strength of Bolt

a) Design shear capacity of bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times \left[n_s \times A_{sb} + n_n \times A_{nb}\right]$$

for double shear plane

$n_s = 1$
$n_n = 2$ } Assume.

Shank Area $A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 20^2 = 314.159\ mm^2$

Thread Area $A_{nb} = 0.78 \times A_{sb} = 245.044\ mm^2$

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Designe shear capacity of one bolt

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 314.159 + 1 \times 245.04]$$

$$= 103.21 \text{ kN.}$$

Design shear strength of 6 bolts.

$$V_{dsb} = 6 \times 103.21 = 619.86 \text{ kN.}$$

b) Design Bearing stress of bolt.

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

k_b = Bearing factor.

$$k_b \Rightarrow \frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.45$$

$$\Rightarrow \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.65$$

$$\Rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97 *$$

$$\Rightarrow 1$$

which one is Min?

$\therefore k_b = 0.45$

Design bearing stress of one bolt

$$V_{dsb} = \frac{2.5 \times 0.45 \times 20 \times 20 \times 410}{1.25} = 147.60 \text{ kN.}$$

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Design Bearing stress of 6 Nos of bolt.

$V_{dpb} = 6 \times 147.60 = 885.6$ kN.

Effeciency of joint (η) = $\frac{\text{Strength of joint}}{\text{Strength of Solid plate}} \times 100$

∴ Strength of joint → min^m (673.056 kN, 619.86 kN & 885.6 kN)

∴ Strength of joint = 619.86 kN.

∴ Strength of Solid plate = $\frac{A_g \times f_y}{\gamma_{m0}}$

$= \frac{180 \times 20 \times 250}{1.1}$

$= 818.18$ kN

Efficiency of joint $(\eta) = \frac{619.86}{818.18} \times 100$

$= 75.76\%$.

∴ Double cover joint having more effiency than lap joint. So, DCBJ generally prefered.

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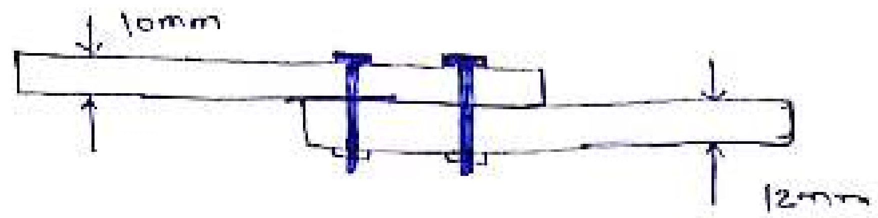
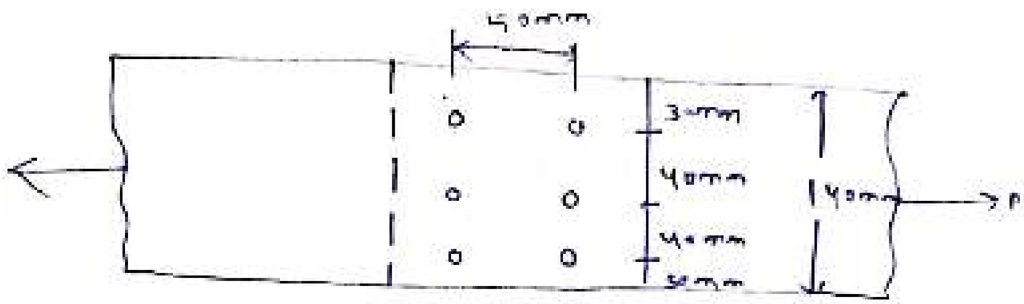
(13)

Question-3

Find the maximum force that can be transmitted through a double bolted chain lap joint consisting of '6' bolts into two rows connecting two plates of thickness 12mm and 10mm given that M16 bolts of grade 4.6 and Fe410 plate used.





Data given

$f_{ub} = 400\ N/mm^2$

$f_u = 410\ N/mm^2$

$f_{yb} = 400 \times 0.6 = 240\ N/mm^2$

$f_y = 250\ N/mm^2$

Diameter of bolt (d) = 16mm

Diameter of bolt hole (d_0) = 16+2 = 18mm

width of ~~thickness~~ plate (B) = 140mm

thickness of plate (t) = 10mm.

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① Design tensile strength of plate

Net sectional Area $(A_n) = (B - nd_0) \times t$

$= (140 - 3 \times 18) \times 10$

$= 860 \text{ mm}^2$

Design tensile strength of plate

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

$$= \frac{0.9 \times 410 \times 860}{1.25}$$

$= 253.87$ KN.

② Bolt strength.

a) Design shear capacity of Bolt.

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \times \left[n_s \times A_{sb} + n_n \times A_{nb} \right]$$

Single shear

$n_n = 1$

$n_s = 0$

Shank area $A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 16^2 = 201.06 \text{ mm}^2$

Thread Area $(A_{nb}) = 0.78 \times A_{sb} = 0.78 \times 201.06 = 156.82 \text{ mm}^2$

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Designe shear capacity of one bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times \left[n_s \times A_{sb} + n_n \times A_{nb}\right]$$

$$= \frac{400}{\sqrt{3} \times 1.25} \times \left[0 \times A_{sb} + 1 \times 156.82\right]$$

$$= 28.97 \text{ kN}$$

∴ Design shear capacity of 6 Nos of Bolts.

$$V_{bsb} = 28.97 \times 6 = 173.82 \text{ kN.}$$

b) Design Bearing stress of Bolt

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

k_b = Bearing factor

$$\left.\begin{array}{l} \Rightarrow \dfrac{e}{3d_0} = \dfrac{30}{3 \times 18} = 0.55 \\ \Rightarrow \dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.67 \\ \Rightarrow \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\ \Rightarrow 1 \end{array}\right\} \text{which one is Min}^{m}$$

$\therefore k_b = 0.55$.

Design Bearing capacity of one bolt

$$V_{dpb} = \frac{2.5 \times 0.55 \times 16 \times 10 \times 410}{1.25} = 72.16 \text{ kN}$$

Design Bearing capacity of 6 Nos of Bolt $= 6 \times 72.16$
$= 432.96$ kN

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∴ strength of joint = min^{m}(263.87 kN, 173.82 kN, 432.98 kN.)

∴ strength of joint = 173.82 kN.

∴ Maximum force can be transmitted in joint is 173.82 kN.

Design of Bolted joint

Question

Design a lap joint between the two plate each of width 120mm, If the thickness of plate are 16mm and 12mm. The joint has to transfer a design load of 160kN. The plate are of Fe 410, use bearing type bolt.

Data Given

Assuming 4.6 property classes and M16 grade of bearing type bolt

f_{ub} = 400 N/mm^2

f_{yb} = 400×0.6 = 240 N/mm^2

f_u = 410 N/mm^2

f_y = 250 N/mm^2

width of plate (B) = 120mm

Diameter of bolt (d) = 16mm, d_0 = 16+2 = 18mm

thickness of plate = 12mm

$e = 1.5 \times d_0$ (flame cut)

$= 1.5 \times 18 = 27 \simeq 30$mm

$p = 2.5 \times d = 2.5 \times 16 = 40$mm

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(15)

① Bolt Strength

a) Design shear capacity of Bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \times \left[n_s \times A_{sb} + n_n \times A_{nb} \right].$$

Lap joint → Single Shear

$n_n = 1$

$n_s = 0$

Shank Area $(A_{sb}) = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 16^2 = 201.06\ mm^2$

Thread Area $(A_{nb}) = 0.78 \times A_{sb} = 0.78 \times 201.06 = 156.82\ mm^2$

Design shear capacity of a bolt

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times \left[1 \times 156.82 \right]$$

$= 28.97$ kN.

b) Design Bearing stress of a bolt

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

k_b ⇒ Bearing factor

$$\left.\begin{aligned} &\Rightarrow \frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.55 \\ &\Rightarrow \frac{p}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49 \\ &\Rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97 \\ &\Rightarrow 1 \end{aligned}\right\} \text{which one is min}$$

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$\therefore k_b = 0.49$

Design Bearing stress of bolt.

$$V_{dpb} = \frac{2.5 \times 16 \times 12 \times 0.49 \times 410}{1.25}$$

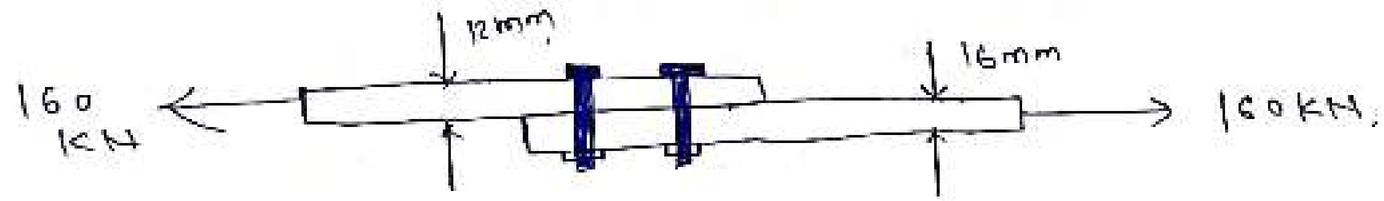
$$= 77.145 \text{ KN}.$$

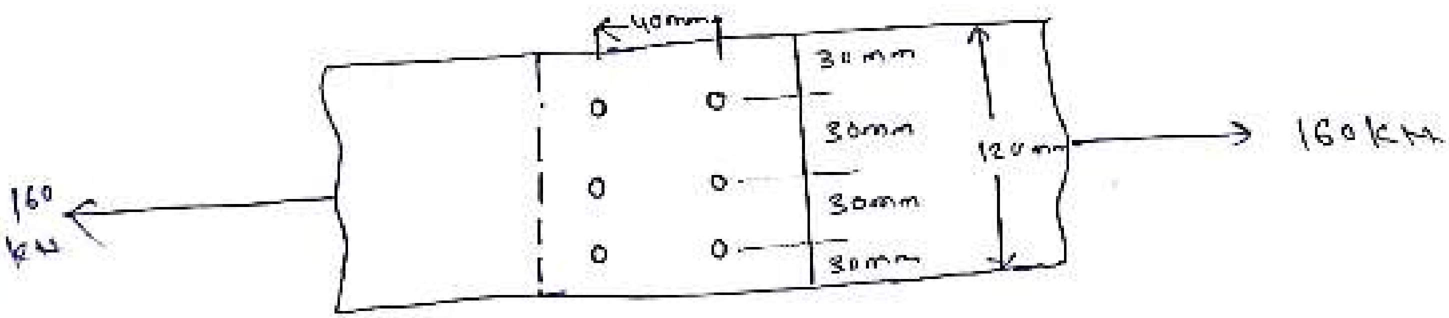
$\therefore$ Bolt Value $= \min(28.97 \text{ KN}, 77.145 \text{ KN})$.

$\therefore$ Bolt value $= 28.97$ KN.

Total factored load $= 160$ KN.

No of Bolts required $= \dfrac{160}{28.97} = 5.52 \simeq 6$ nos.





Check

Designe Tensile strength of plate.

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

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Net Sectional Area of plate

$$A_n = (B - n d_0) \times t$$

$$= (120 - 3 \times 18) \times 12$$

$$= 792 \text{ mm}^2.$$

Design Tensile strength of plate

$$T_{dp} = \frac{0.9 \times 410 \times 792}{1.25}$$

$$= 233.79 \text{ KN} > 160 \text{ KN}$$

(Design is Safe)

Question -2

Design a lap joint to connect two plates 300mm wide and 16mm thick using 20mm diameter of bolts of property class 4.6, then the applied factored load 370 kN.

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Shearing Capacity of HSFG Bolt. (IS 800-2007, Pg-75, Cl-10.4)

*! These are the bolts made of high tensile steel which are pretension and them provide with nuts here resistance to shear force is mainly by friction.

* Nominal shear capacity of HSFG Bolts. (IS 800-2007, Pg-76, Cl-10.4.3)

$$\boxed{V_{nsf} \Rightarrow \mu_F \times n_e \times k_h \times F_o}$$

Where,

F_o = Pretension force. 'kN'

$F_o = f_o \times A_{nb}$

A_{nb} = net Area of bolt

f_o = proof stress $= 0.7 \times f_{ub}$

f_{ub} = Ultimate strength of bolt

n_e = no. of shear interface offering frictional Resistance.

K_h = Hole factor

$\Rightarrow$ 1 (Standard sized holes)

$\Rightarrow$ 0.8 (Over sized holes

Short slotted holes

long slotted holes) (load is $\perp^{ar}$ hole)

$\Rightarrow$ 0.7 (long slotted hole) (load is parrallel to hole)

- μ_F = Slip factor or co-efficient of friction specified in [Table-20]. ($\mu_f \le 0.55$)

Design shear capacity of friction type Bolt

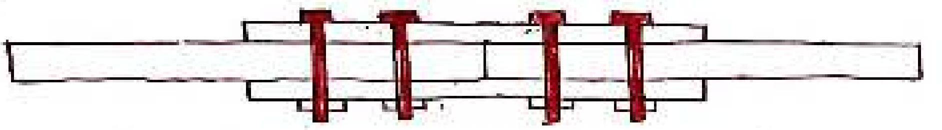
$$\boxed{V_{dsf} = V_{nsf}/\gamma_{mf}}$$

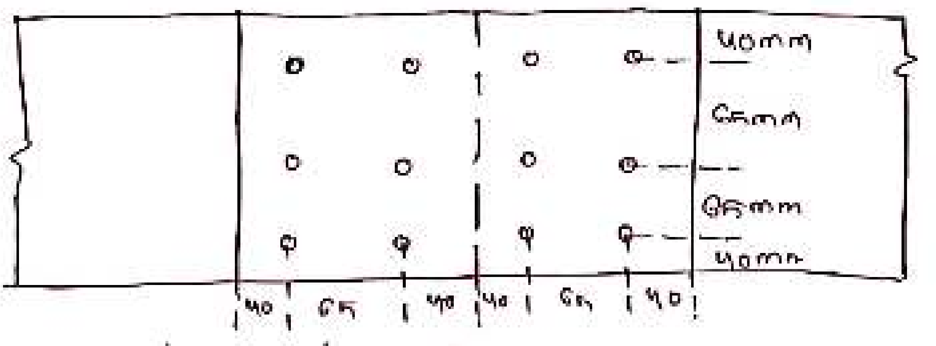
$\gamma_{mf} \Rightarrow$ 1.1 [Service load condition]

$\Rightarrow$ 1.25 [Ultimate load condition].

Question

Determine the shear capacity of HSFG Bolts used in connecting two plates are shown in fig.





Q-1) Slip resistance designed at Service load

Q-2) Slip resistance designed at Ultimate load

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⇒ HSFG Bolt of property class 8.8 are used.
⇒ fasteners are in clearance holes.
⇒ Co-efficient of friction = 0.3.

Data given

f_{ub} = 800 N/mm²
f_{yb} = 800 × 0.8 = 640 N/mm²
n_e = 2 (Double cover butt joint)
K_h = 1 (for fastener clearance holes).
μ_f = 0.3

P = 65 mm

⇒ P = 2.5 × d ⇒ d = 65/2.5 = 26 mm

∴ d = 20 mm. (diameter of bolt).

d_0 = 20 + 2 = 22 mm (Diameter of bolt holes).

f_0 = proof stress

f_0 = 0.7 × f_{ub}

= 0.7 × 800 = 560 N/mm²

Proof load (T_0) = f_0 × A_{nb}

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Net Area of bolt of threaded portion

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

$$= 0.78 \times \pi/4 \times 20^2$$

$$= 245.04\ mm^2$$

$F_o = f_o \times A_{nb} = 560 \times 245.04 = 137.22$ kN

Nominal shear capacity of bolt

$$V_{nsf} = \mu_F \times K_h \times n_e \times F_o$$

$$= 0.3 \times 1 \times 2 \times 137.72$$

$$= 82.33\ kN.$$

Designe slip resistance per bolt

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}$$

a) Design slip resistance per bolt (for service load cond^n).

$\gamma_{mf} = 1.1$

$V_{dsf} = \frac{82.33}{1.1} = 74.84$ kN.

Slip resistance for 6 nos of bolt (V_{dsf}) = 6×74.84
= 449.04 kN.

b) $V_{dsf} = \frac{82.33}{1.25}$ (for ultimate load cond^n)

= 65.864 kN.

Slip resistance for 6 nos of bolt (V_{dsf}) = $6 \times 65.86 = 395.16$ kN.

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Welded Connection

* when the steel members are connected with the help of welding then, they are called as welded joint.
* There are different ways to perform welding

  For eg:- Gas welding
  Electric Arc welding
  Electric Resistance welding
  Seam welding.

* Generally steel structural elements are joint together with the help of electric arc welding.

Advantages & Disadvantages of welding

Advantages of welding

* Welding is more adoptable than bolting
* Self weight of welded structure is less.
* welded connection are consider to be rigid.
* There is no need to drill holes inside member due to which the effective area will become = Gross Area of member.
* Welded joint are strong in case of reversal of stresses.

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* It is possible to achieve 100% of efficiency.
* Welded connection have good aesthetic appearance
* Welded connection is air tight and water tight.

Disadvantages of Welded Connection.

* Welding is effected by the field condition.
* Skilled person is required to perform welding.
* Inspection of welded joint is difficult.
* Residual stresses are present in the welded joint.
* A welded joint fails earlier than bolted joint because of brittle in nature.

Types of weld

1. Butt weld
2. Fillet weld
3. Slot weld
4. Plug weld

1. Butt weld

* Butt weld are also called as groove weld.
* Groove weld are provided when the elements to be joint lies in same plane.

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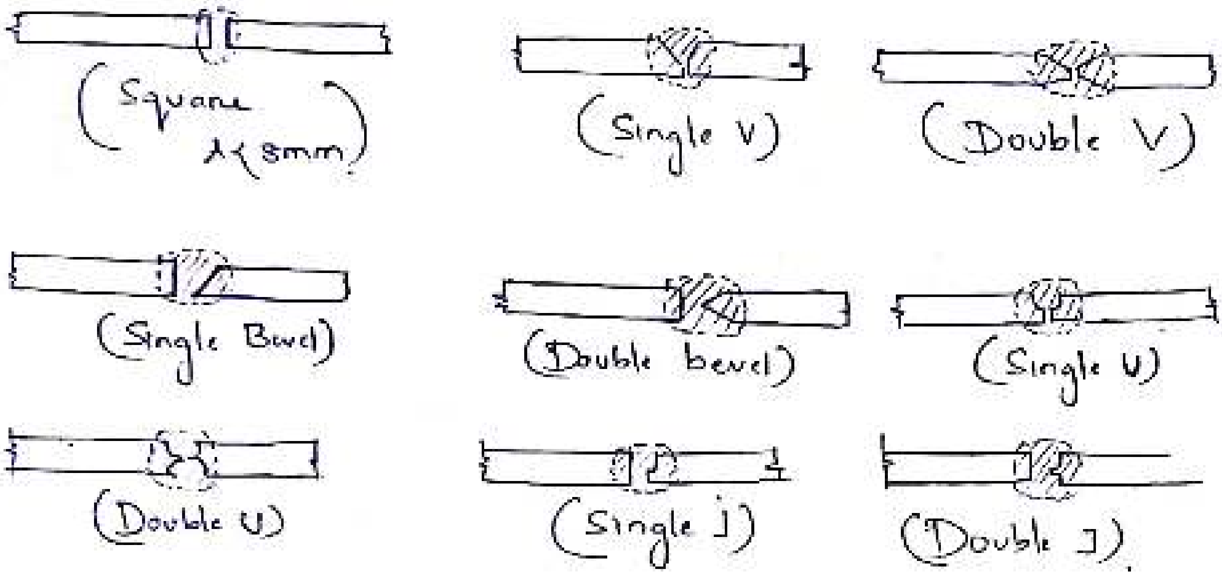
If $t > 8mm$ → edge preparation is required

$t < 8mm$ → no edge preparation is required, This weld is known as square Butt weld.



Assumption in Analysis of Welding

* Weld material is homogeneous and isotropic.
* Deformation due to loading will be neglected i.e, welded joint is rigid.
* Residual stresses are also neglected.
* Butt weld / Groove weld are designed to resists axial tensile / Compressive stress.

Note

fillet weld are design to resists shear stress.

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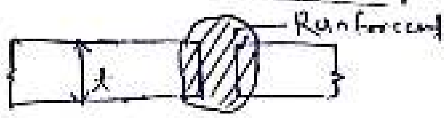
Design Consideration of Butt joint

* Reinforcement :- extra weld metal above the element surface which increase the size of weld by at least 10%.

* The maximum size of reinforcement shall not exceed 3mm.



* Size of butt weld :-

* It is defined in terms of effective throat thickness (t_e)

* t_e (throat thickness) = $\frac{5}{8} \times t$ (Partial penetration)

throat thickness (t_e) = t (for Double penetration).

Designe strength of goove weld.

* Design axial strength of goove weld.

$$\boxed{P_{dw} = \frac{f_y' \times A_e}{\gamma_{mw}}}$$

Design shear strength of butt weld (V_{dw})

$$= \frac{f_y' \, l_w \times t_t}{\sqrt{3}\, \gamma_{mw}}$$

where,

→ A_e = effective Area = $l_w \times t_t$, l_w = effective length of butt weld ;

t_e = effective throat thickness.

⇒ f_y' → f_{yw} → yield strength of weld metal

f_y → yield strength of structural element

(which one is min^m)

⇒ γ_{mw} = partial safety factor for weld material

γ_{mw} = 1.25 for shop weld

γ_{mw} = 1.5 for field weld

{ IS 800-2007 , Pg-30 T-5 }

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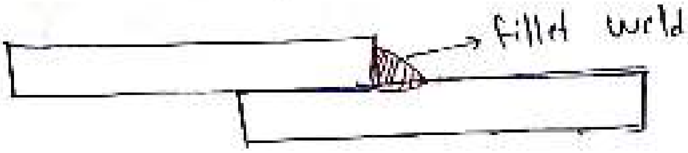
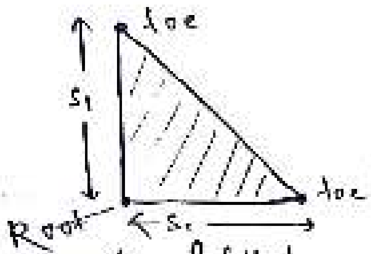
(21)

Fillet weld

* Fillet weld are designed when the two members to be joined lies in a different plane (there is some inclination between the connecting pieces).
* Fillet weld are provided when the member to be jointed are overlaped with each other.
* Fillet weld are critical in shear-stress.





c/s of fillet weld

Leg size :- It is the distance from the root to toe in the cross section of fillet weld.

Size of weld (s) → It is the minm of two leg size.

$$S \rightarrow \left.\begin{matrix} S_1 \\ S_2 \end{matrix}\right\} \text{Min}^m$$

* Minimum Size of fillet weld

* It depends upon thickness of thicker element connected. [IS 800-2007, Pg - 78, T-21
cl - 10.5.2.3]
* The size of fillet weld shall not be less than 3mm.

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Maximum Size of fillet weld

* It depends upon thickness of thinner member connected.

$$\boxed{S_{max} = t - 1.5mm} \rightarrow \text{Square edge}$$

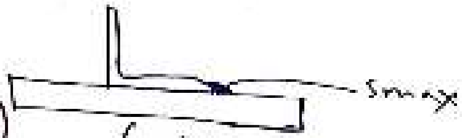
(fillet weld on square edges)

(IS 800-2007, Pg-79
Cl-10.5.8.1)

t = thickness of thinner member.

$$\boxed{S_{max} = t \times \frac{3}{4}} \rightarrow \text{Rounded toe}$$

(Pg-79, Cl-10.5.8.2)



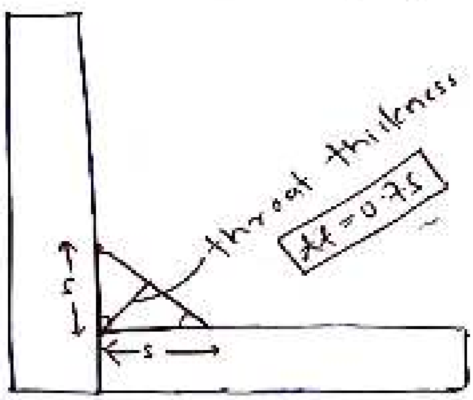
(Fillet weld on Rounded toe)

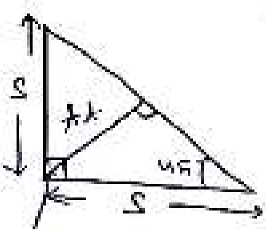
t = thickness of thinner member.

effective throat thickness

* It is defined as the minimum dimension in the crossection of fillet weld.

* Throat thickness shall not be less than 3mm.





$(\theta = 90°)$ $\sin 45 = \frac{t_t}{S}$

$t_t = S.\sin 45°$

$= S\frac{1}{\sqrt{2}} = 0.7S$.

* Throat thickness depends upon angle of inclination 'θ'

e.g. If $\theta = 90°$, $t_t = 0.7S$

In general $\boxed{t_t = KS}$ (IS 800-2007, Pg-78
T-22)

∴ K = constant which depends upon angle of inclination

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* effective length of fillet weld L_w (IS 800-2007, Pg-78 Cl- 10.5.4.1)

$$\boxed{L_w = L - 2 \times \text{Size of weld}}$$

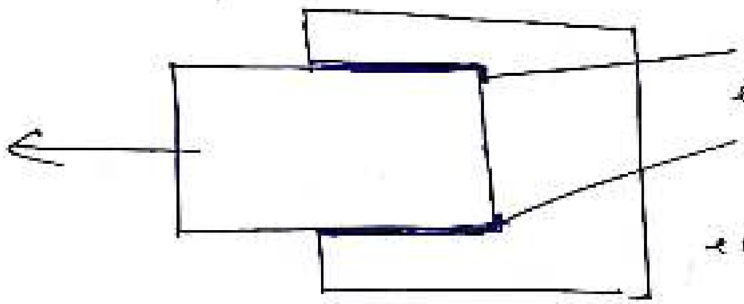
L_w = effective length of weld.

L = overal length of weld

* effective length of weld shall never be less than $4 \times S$

S = size of weld $\boxed{L_w = 4 \times S}$

End Return



end return should not be less than 2X size of weld.

Designe strength of fillet weld

Designe stress in fillet weld, $f_{wd} = \dfrac{f_{wn}}{\gamma_{mw}}$

f_{wn} = nominal stress in fillet weld.

γ_{mw} = partial safety factor.

$\gamma_{mw} = 1.25$ for shop weld (IS 800-2007, Pg-30 T-5)

$\gamma_{mw} = 1.5$ for field weld

Designe strength of fillet weld = Designe stress $\times A_e$

$$= \frac{f_{nw}}{\gamma_{mw}} \times A_e$$

$$P_{dw} = \frac{f_{nw}}{\gamma_{mw}} \times A_e$$

$$= \frac{f_u'}{\sqrt{3}\,\gamma_{mw}} \times (L_w \times t_t)$$

$$\boxed{P_{dw} = \frac{f_u'}{\sqrt{3}\,\gamma_{mw}} \times L_w \times t_t}$$

$$f_u' \rightarrow \left.\begin{array}{l} f_{uw} \\ f_u \end{array}\right\} \text{min}^m$$

f_{uw} = ultimate strength of weld metal

f_u = Ultimate strength of element.

If load is given, then for a particular size of weld, the length of weld required will be:

$$P_u = P_{wd} = \frac{f_u'}{\sqrt{3}\,\gamma_{mw}} \times (L_w \times t_t)$$

$$\boxed{L_w = \frac{P_u \times \sqrt{3} \times \gamma_{mw}}{f_u' \times K \times S}}$$

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Question-1

A 15 mm thick plate is joint to a 16mm plate by 200 mm long (effective) butt welded determine the strength of joint. If

1) A double V butt weld is used.

2) A single V butt weld is used.

Assume Fe410 grade plate and shop welded are used.

Data given

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

$l_w = 200mm$ (effective length of weld)

$\gamma_{mw} = 1.25$ (shop weld).

thickness of thinner plate $(t) = 16mm$.

double V butt weld

Throat thickness (t_t) = (t) thickness of thinner plate.

= 16mm (fully penitration)

Design strength of weld $(p_{dw}) = \dfrac{f_y' \times l_w \times t_t}{\gamma_{mw}}$

$\Rightarrow f_y' = f_y = 250\ N/mm^2$ $\qquad = \dfrac{250 \times 200 \times 16}{1.25}$

$= 640\ kN.$

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Single V butt weld

Throat thickness (t_t) = $\frac{5}{8} \times t$ (partially penetration).

$= \frac{5}{8} \times 16 = 10mm$

Design strength of butt weld

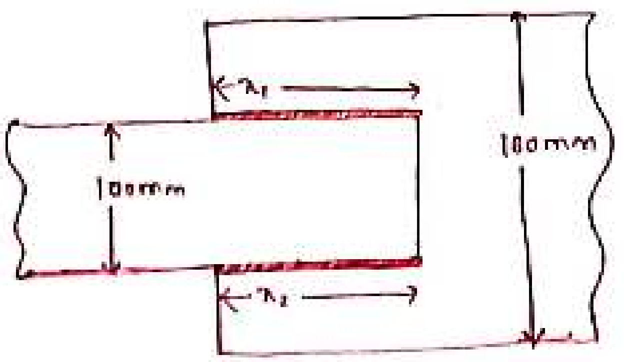
$$P_{dw} = \frac{f_y' \times l_w \times t_t}{\gamma_{mw}}$$

$$= \frac{250 \times 200 \times 10}{1.25}$$

$= 400$ kN.

Question - 3

Design a suitable longitudinal fillet weld to connect the plates as shown in fig, to transmit a pull equal to the full strength of small plate, given plates 12mm thick, grade of plate Fe410 and welding should be made in workship.



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(24).

Data given

$f_u = 410$ N/mm² | $\gamma_{mw} = 1.25$ (work shop welding).

$f_y = 250$ N/mm²

thickness of plate (t) = 12 mm

Minimum size of weld (S_{min}) = 5 mm [IS 800 - 2007, Pg - 78, T-21]

Maximum size of weld (S_{max}) = 12-1.5 = 10.5 mm (square edge)

∴ Size of weld (S) = 10 mm (Assume).

Gross Area of small plate (A_g) = B×t = 100×12 = 1200 mm²

Strength of solid plate (T_g) = $\frac{A_g \times f_y}{\gamma_{m0}}$

$= \frac{1200 \times 250}{1.1}$

= 272.72 KN.

Throat thickness of fillet weld (t_t) = KS ($\theta = 90°$).

= 0.7×10 = 7 mm

Strength of fillet weld = $\frac{f_u' \times l_w \times t_t}{\sqrt{3} \times \gamma_{mw}}$

$= \frac{410 \times l_w \times 7}{\sqrt{3} \times 1.25}$

= 1.32 l_w KN

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Design strength of weld = strength of solid plate

$\Rightarrow \quad 1.33 \times l_w = 272.72$

$\Rightarrow \quad l_w = \dfrac{272.72}{1.33} = 205.05\,mm \simeq 210\,mm$

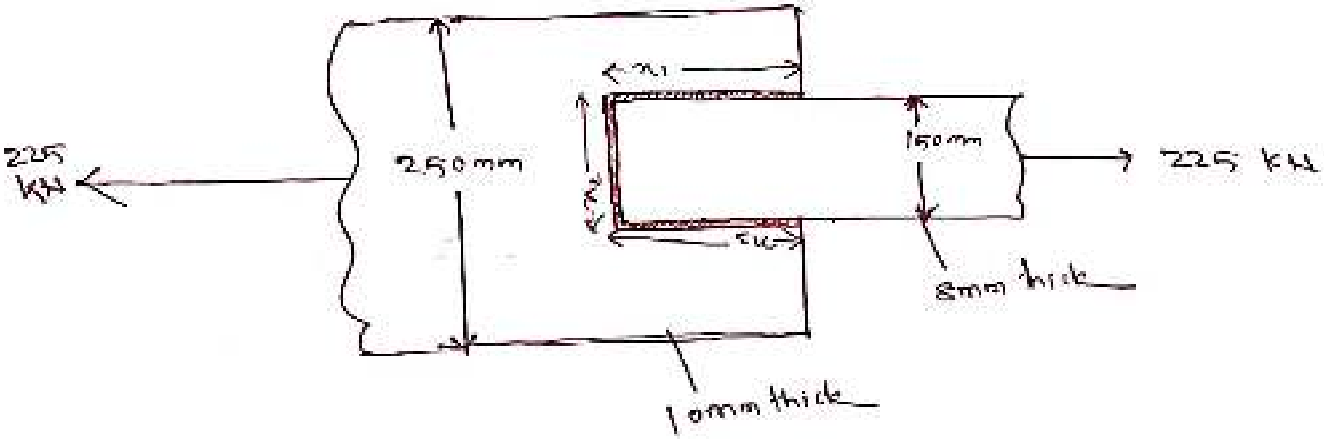
$\therefore$ effective length of weld $l_w = 210$ mm

$x_1 + x_2 = 210mm \quad (x_1 = x_2)$

$x_1 = x_2 = \dfrac{210}{2} = 105\,mm$ Ans

Assignment

Designe a suitable fillet weld to connect two plate are shown in below use Fe410 grade of plate is used.



$x_1 = ?$
$x_2 = ?$
$x_3 = ?$

Data given :

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

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

thickness of thinner plate = 8mm
thickness of thicker plate = 10mm

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(2h)

Minimum size of weld (S_{min}) = 3mm (IS 800-2007, Pg-78 T-21)

Maximum size of fillet weld (S_{max}) = 8-1.5

(Size of weld) = 6.5mm (Square edge).

∴ S = 6mm.

effective throat thickness (t_t) = K×S (θ=90°)

= 0.7×6 = 4.2 mm

∴ Tensile load = 225 kN

⇒ Designe strength of weld = $\frac{L_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$

(γ_{mw}=1.25, Assume shop weld) = $\frac{L_w \times 4.2 \times 410}{\sqrt{3} \times 1.25}$

= L_w×0.795 kN

⇒ Designe strength of weld = Tensile load (Designe purpose)

⇒ L_w×0.795 = 225

⇒ $L_w = \frac{225}{0.795}$ = 283.01mm ≃ 290mm.

∴ effective length of weld L_w = 290mm

$x_1 + x_2 + x_3$ = 290 mm

x_2 = 150mm

$x_1 = x_3 = \frac{290-150}{2}$ = 70mm.

Ans

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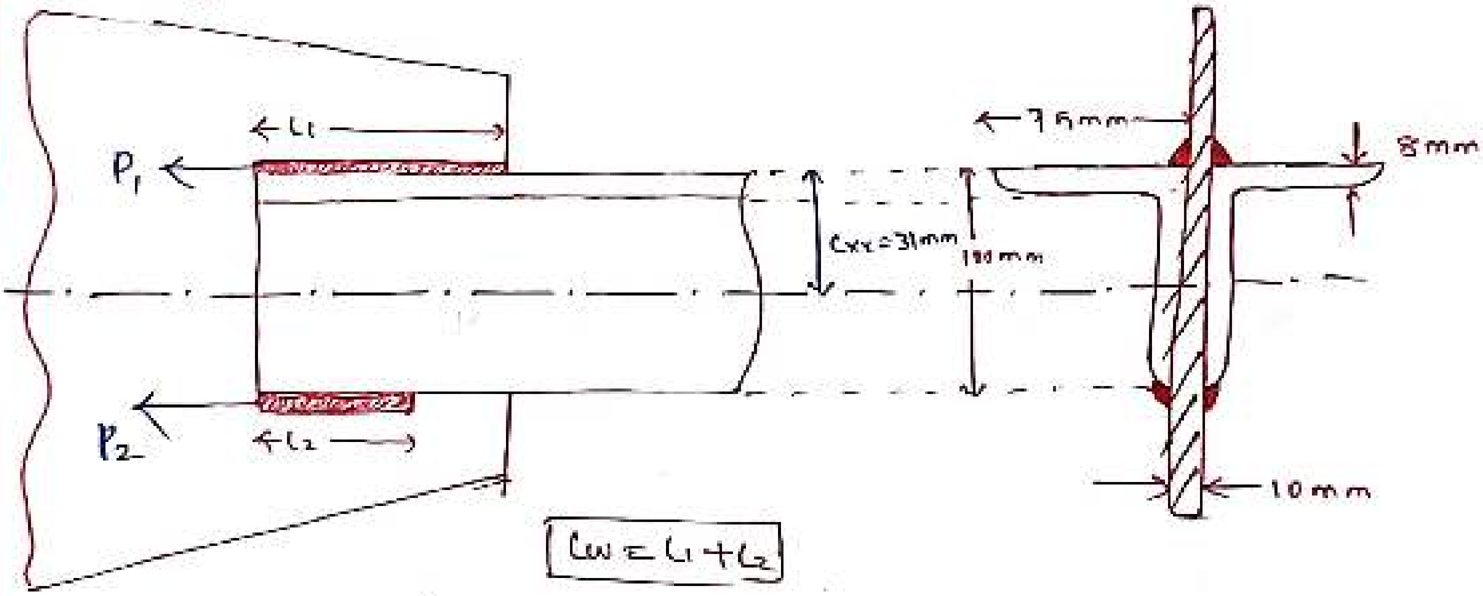
Question

A tie member of a roof truss consists of ISA 100×75×8 mm the angle are completely either sides of a 100 mm gusset plate and the member is subjected to a working pull of 300 kN. Design the welded connection assume the connection are made in workshop Fe410 grade plate are used.



Data given

$f_u = 410\ N/mm^2$

working pull = 300 kN

factored load/pull = 300×1.5 = 450 kN

factored load per one angle = 450/2 = 225 kN

$\gamma_{mw} = 1.25$ (shop weld).

$t_{min} = 8$ mm, $t_{max} = 10$ mm

Minimum size of weld (S_{min}) = 3 mm. (T-21, Pg-75)

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Maximum size of weld $(S_{max}) = \frac{3}{4} \times t$ (Round shape edge).

$= \frac{3}{4} \times 8 = 6mm$

$\therefore S = 6mm$

effective throat thickness $(t_t) = K \times S$ $(\theta = 90^\circ)$

$= 0.7 \times 6 = 4.2 mm$

Design strength of weld $(P_{dw}) = \frac{f_u \times L_w \times t_t}{\sqrt{3} \times \gamma_{mw}}$

$= \frac{410 \times L_w \times 4.2}{\sqrt{3} \times 1.25}$

$= 0.795 \times L_w$ kN.

$\therefore$ Des strength of weld = factored load

$\Rightarrow$ $0.795 \times L_w = 225$ kN

$\Rightarrow$ $L_w = \frac{225}{0.795} = 283.01 mm \simeq 290 mm.$

$\therefore L_w = 290 mm.$

cg of Angle $C_{xx} = 21mm$ (Steel table, pg- 14)
ISA

taking Moment about cg.

$P_1 \times C_{xx} = P_2 \times (100 - C_{xx})$

$\Rightarrow \frac{f_u \times L_1 \times t_t}{\sqrt{3} \times \gamma_{mw}} \times C_{xx} = \frac{f_u \times L_2 \times t_t}{\sqrt{3} \gamma_{mw}} \times (100 - C_{xx})$

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$\Rightarrow \quad l_1 \times 31 = l_2 \times (100-31)$

$\Rightarrow \quad l_1 = l_2 \times \left(\frac{100-31}{31} \right)$

$$\boxed{l_1 = 2.22 l_2}$$

$l_0 = l_1 + l_2$

$\Rightarrow \quad 290 = 2.22 l_2 + l_2$

$\Rightarrow \quad l_2 = \frac{290}{3.22} = 90\,mm$

$\Rightarrow \quad l_1 = 290 - 90 = 200\,mm.$

Ans

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Ch-3

Iliyas...

Design of Tension Member.

* A structural element which is subjected to two pulling forces at its end are called tension member.

Types of Section member

1) **Cables** :- * They are ideal tension member because, they can not resists compression and bending stress.

* They are made from multiple wires which are hellically rounded

* They are used in suspension bridge.

2) **Flat or plates** :- In plate, bolting can be done in a way that the CG of member coinside ~~to~~ with the CG of connection which no bending effect are generated.

* In case of compression ~~member~~ applied to flat or plates they will fail easily, because they have less radious of gyration.

* They can be used in lacing or battening.

3) **Angle or Builtup Section**

* For a given area angle section provides more stiffness as compared to flat sections.

* Angle section can be connected either with bolting or welding.

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* In case of heavy loading builtup section or I-section is used as a tension member.

4) Hollow Section

* In addition to tensile strength hollow section have good compressive strength therefore they are used as Bracing element.

* Hollow section are generally connected with the help of welding.

Failure of tension Member

* failure of tension member is governed by two limit state

1) Yielding failure 2) Rupture/fracture failure

* There are 3 modes in which these limit state could be reached.

① Gross Section Yielding

* Due to gross section yielding the member deforms to a very large value due to which the member becomes unserviciable, Hence it will fails in limit state of serviciability.

② Net Section Rupture

* When Ultimate stress are reached at the net section the rupture take place in the connecting element through the bolt holes.

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2

③ Block shear failure

* In this failure a segment of member get cutted out from the joint. In this failure shear occures at one plane and tension occurs at adjacent plane.

Design strength of Tension Member.

* It depends upon the following strength's criteria

① Design strength in Gross section yielding

(IS 800-2007, pg-32, CL-6.2)

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

f_y = yield strength of member.

A_g = Gross sectional Area

γ_{m0} = Partial Safety factor (1.1 - Pg-30 > T-5)

② Design strength of Tension member due to net section Rupture:-

[IS 800-2007, Pg-32 CL-6.3]

a) Flat & plate

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

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Where,

f_u = Ultimate strength of member.

γ_{m1} = partial safety factor = 1.25 (pg-30, T-5)

A_n = Net Sectional Area.

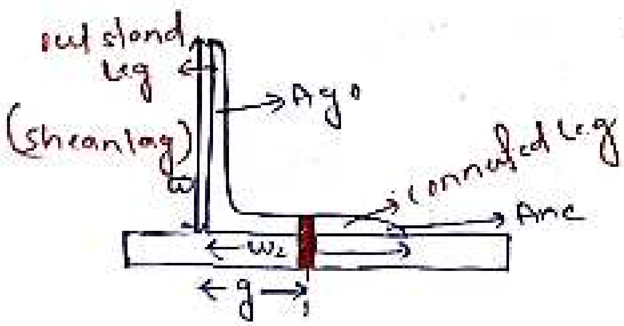
$A_n = (B - n d_0) \times t$ (for chain bolting)

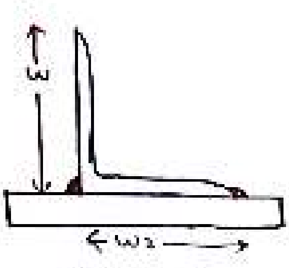
$A_n = \left[B - n d_0 + \sum_{i=1}^{m} \frac{p_s^2}{4 g_i}\right] \times t$ (for staggered bolting)

b) Angles Section

* In case of angle section connected with one leg only with gusset plate. The out standing leg will be effected by shear lag.

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





$\boxed{b_s = w + g - t}$

$\boxed{b_s = w}$

$A_{go} = (w_1 - t/2) \times t$

$A_{nc} = (w_2 - t/2 - n d_0) \times t$

$A_{go} = (w - t/2) \times t$

$A_{nc} = (w_2 - t/2) \times t$

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Where,

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

$$\ge 0.7$$

A_{nc} = Net sectional area of connected leg.

A_{go} = Gross sectional area of outstanding leg.

w = width of out standing leg.

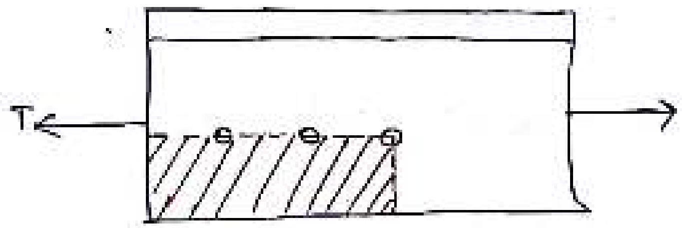
t = thickness of element

b_s = Distance from tip of out standing leg to the center of first bolt line.

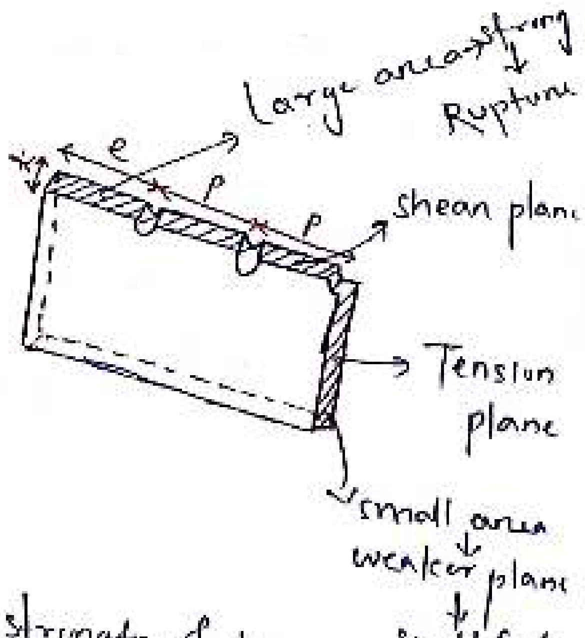
L_c = Length of connection.

③ Design strength in Block shear

* The Block shear failure occurs due to combined effect of shear ~~on~~ one plane and tension on the other plane.



Shaded area will tear out from member.



∴ Block shear strength = Rupture strength of stronger plane + Yield strength of weaker plane.

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[IS 800-2007, pg-33, cl-6.4]

Case-1

Rupture of shear plane + Yielding of tension plane

$$\boxed{T_{db1} = \frac{0.9 f_u \times A_{vn}}{\sqrt{3} \times \gamma_{m1}} + \frac{f_y \times A_{tg}}{\gamma_{m0}}}$$

Case-2

Rupture of tension plane + Yielding of shear plane

$$\boxed{T_{db2} = \frac{0.9 \times f_u \times A_{tn}}{\gamma_{m1}} + \frac{f_y \times A_{vg}}{\sqrt{3} \times \gamma_{m0}}}$$

where

$A_{vg} \rightarrow$ Gross area in shear $= (e+2p) \times t$ (from figure)

$A_{vn} \rightarrow$ Net area in shear $= (e+2p-2.5 \times d_0) \times t$

$A_{tg} \rightarrow$ Gross area in tension $= e_0 \times t$

$A_{tn} \rightarrow$ Net area in tension $= (e_0 - 0.5 \times d_0) \times t$

Design strength of tension in Block shear

$= \min^m (T_{db1}, T_{db2})$

$\therefore$ Design strength of tension member as per limit state

$$\text{Min}^m \leftarrow \begin{cases} T_{dg} \\ T_{dn} \\ T_{db} \end{cases}$$

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(4)

Check for Slenderness Ratio

Slenderness ratio for tension member is defined as the ratio of unsupported length of the member to its least radious of gyration.

Slenderness Ratio $\boxed{\lambda = \frac{l_e}{r_{min}}}$

l_e = unsupported length

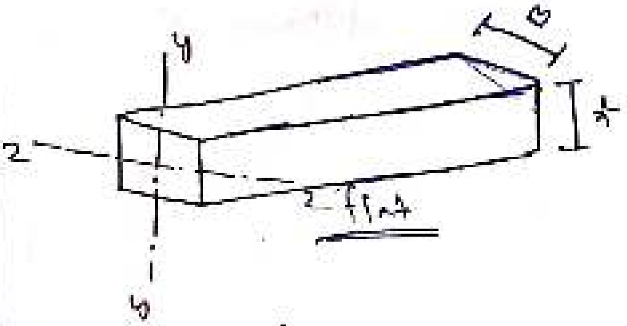
r_{min} = Least radious of gyration

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

$$\boxed{I_{min} = A\, r_{yy}^2}$$

I = moment of inertia

A = Area of section



$I_{zz} = \frac{Bt^3}{12}$

$I_{yy} = \frac{t \times B^3}{12}$

$\Big\}$ I_{zz} = min^m, I_{yy}

$$\boxed{r_{min} = \sqrt{\frac{\frac{Bt^3}{12}}{B \times t}} = \frac{t}{\sqrt{12}}}$$

IS 800-2007 has kept a restriction over the slenderness ratio of the tension member subjected to reversal of stresses

[IS 800-2007, Pg-20, T-3]

Member	Maximum effective Slenderness Ratio
(1) A tension member in which reversal of direct stress occurs due to loads other than wind & Seismic loading (DL+LL)	180
(2) A member normally acting as a tie in roof truss or a bracing system but subjected to possible of reversal stress resulting from action of wind or earthquake force	350.

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③ for any other tension member 400.
(other than prestressed member)

<u>Design procedure</u> (Limit state)

given ⇒ Applied load, 'T' ⇒ $T_u = \gamma_f \times T$ (applied factored load)

<u>Step-1</u> → Determine the required area of crosssection

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

$$\boxed{A_g = \frac{T_u \times \gamma_{mo}}{f_y}}$$

<u>Note</u>

∴ (A_g increasing by 25%)

<u>Step-2</u> → From IS Hand book no.-1 sellect a Rolled steel Section of a particular shape which provides the gross area equals to or greater than the required gross area.

<u>Step-3</u> → Design of connection.

<u>Bolting</u>

* Diameter of bolt $= 6.04\sqrt{t}$
* strength of Bolt → B_v
* no of bolt $(n) \rightarrow \frac{P_u}{B_v}$
* Arrange the bolts suitably into the members.

<u>Step-4</u> → Determine the Designe tensile streng of the member (T_d)

i) Gross section yeilding strength 'T_{dg}'
ii) Net section rupture strength 'T_{dn}'
ii) Block shear strength T_{db}

} $min^m = T_d$

$\boxed{T_d \geq T_u}$

<u>Safe</u>

<u>Step-5</u> → Check for slenderness Ratio :-

$\lambda_{member} \not> \lambda_{max}$ (Give IS code)

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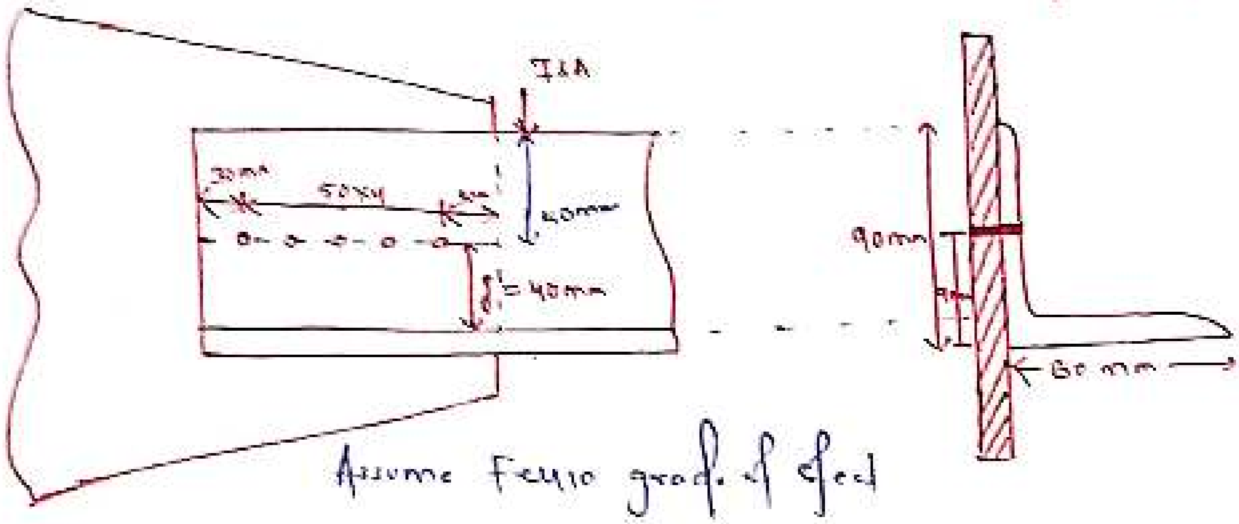
⑤

Question-1

A single unequal angle ISA 90×60×6 mm is connected to a 10 mm gusset plate at the ends with 5 nos of 16 mm bolts to transfer tension determine the design tensile strength of the angle if 90 mm leg is connected to gusset plate!



Assume Fe410 grade of steel

Data given

$f_y = 250$ mpa , $d = 16$mm

$f_u = 410$ N/mm² $d_0 = 16+2 = 18$mm.

① Gross section yielding strength

$$T_{dg} = \frac{f_y \times A_g}{\gamma_{m0}} \qquad (\gamma_{m0} = 1.1)$$

A_g = gross area of ~~plate~~ Angle section

$A_g = 8.65 \text{ cm}^2 = 865 \text{ mm}^2$ (from steel table Pg-14)

$$T_{dg} = \frac{250 \times 865}{1.1} = 196.59 \text{ kN}.$$

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② Net Section Rupture strength

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

A_{nc} = Area of connected leg

$A_{nc} = (90 - t/2 - n d_0) \times t$

$= (90 - 6/2 - 15) \times 6 =$

$= 414\ mm^2$

A_{go} = Area of out stand leg.

$A_{go} = (60 - t/2) \times t$

$= (60 - 6/2) \times 6$

$= 342\ mm^2$

$\beta = 1.4 - 0.076 \times \frac{w}{t} \times f_y/f_u \times \frac{b_s}{L_c}$

w = length of out stand leg = 60 mm

t = 6mm

$b_s = w + g - t = 60 + 40 - 6$

$= 94$ mm.

$L_c = 50 \times 4 = 200$ mm

$\Rightarrow \beta = 1.4 - 0.076 \times \frac{60}{6} \times \frac{250}{410} \times \frac{94}{200}$

$= 1.18$

Check $\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44 \geq 1.18$

$0.7 \leq 1.18$

$\therefore \beta = 1.18$

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(6)

$$T_{dn} = \frac{0.9 \times 410 \times 414}{1.25} + \frac{342 \times 250 \times 1.18}{1.1}$$

$$= 213.93\ \text{kN}$$

(3) Block Shear strength

$$A_{vn} = (e + 4 \times p - 4.5 d_0) \times t = (30 + 4 \times 50 - 4.5 \times 18) \times 6 = 894\ \text{mm}^2$$

$$A_{vg} = (e + 4 \times p) \times t = (30 + 4 \times 50) \times 6 = 1380\ \text{mm}^2$$

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

$$A_{tn} = (50 - 0.5 \times d_0) \times t = (50 - 0.5 \times 18) \times 6 = 246\ \text{mm}^2.$$

$$T_{db_1} = \frac{0.9 \times f_u \times A_{tn}}{\gamma_{m1}} + \frac{f_y \times A_{vg}}{\sqrt{3} \times \gamma_{m0}}$$

$$= \frac{0.9 \times 410 \times 246}{1.25} + \frac{250 \times 1380}{\sqrt{3} \times 1.1}$$

$$= 253.69\ \text{kN}$$

$$T_{db_2} = \frac{0.9 \times f_u \times A_{vn}}{\sqrt{3} \times \gamma_{m1}} + \frac{A_{tg} \times f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 410 \times 894}{\sqrt{3} \times 1.25} + \frac{300 \times 250}{1.1}$$

$$= 220.54\ \text{kN}$$

$$\therefore T_{db} = \min^m (T_{db_1}, T_{db_2})$$

$$T_{db} = 220.54\ \text{kN}.$$

$\therefore$ Design tensile strength of tension member $= \min^m (196.59\ \text{kN}, 213.93\ \text{kN}\ \&\ 220.54\ \text{kN})$

$$\therefore T_d = 196.59\ \text{kN}.$$

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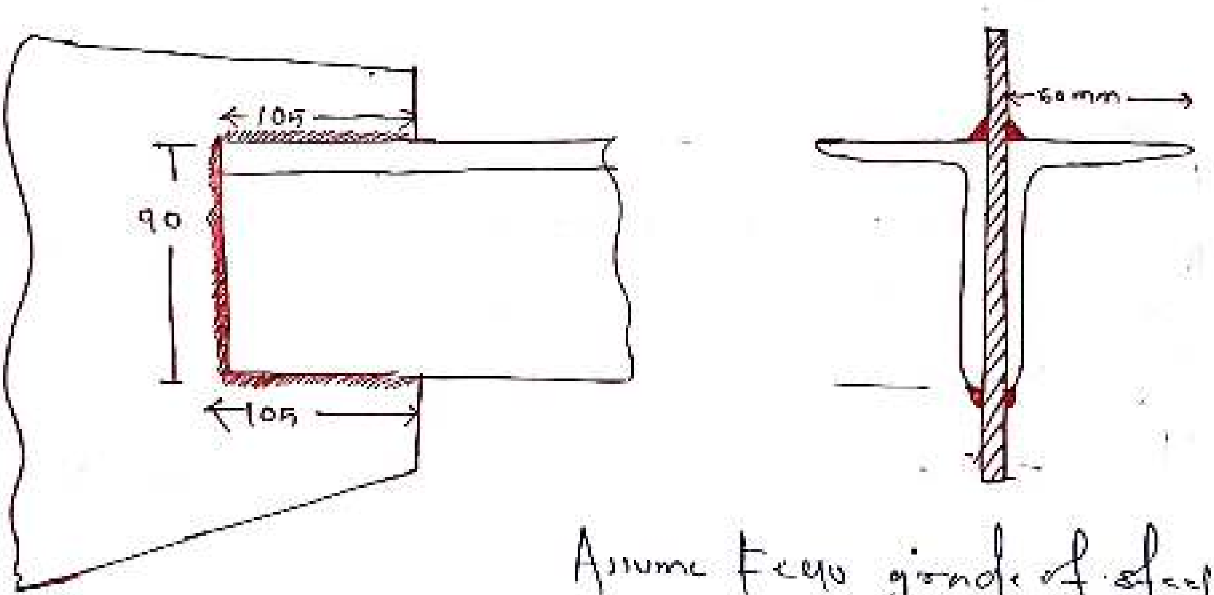
Question-2

Determine the tensile strength of a roof truss member 2 ISA 90x60x6 mm^2 connected to the gusset plate of 8mm thickness by 4mm weld as shown in fig. The effective length of the weld is 300mm.



Assume Fe410 grade of steel

Data given

ISA 90x60x6mm

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

Thickness of gusset plate = 8mm.

Size of weld = 4mm

effective length of weld = 300mm.

$$\text{Area of connected leg } (A_{nc}) = (90 - t/2) \times t \times 2 \rightarrow 2\text{ angle section}$$

$$= (90 - 6/2) \times 6 \times 2$$

$$= 1044\ mm^2.$$

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⑦

Area of out standing leg $= (60 - t/2) \times t \times 2$

$= (60 - 6/2) \times 6 \times 2$

$= 684 \text{ mm}^2.$

Gross Area $(A_g) = 8.65 \text{ cm}^2$ (from steel table)

$= 865 \text{ mm}^2 \times 2 = 1730 \text{ mm}^2$

① Gross Yielding strength

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

$$= \frac{250 \times 1730}{1.1}$$

$$= 393.18 \text{ KN.}$$

② Designe strength due to Net Rupture

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

$$= \frac{0.9 \times 410 \times 1044}{1.25} + \frac{\beta \times 250 \times 684}{1.1}$$

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

$w = 60 \text{ mm}$, $L_c = 105 \text{ mm} = \left(\frac{105+105}{2}\right).$

$t = 6 \text{mm}$, $b_s = 60 \text{ mm}$

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$$\beta = 1.4 - 0.076 \times \left(\frac{60}{6}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{60}{105}\right)$$

$$= 1.13$$

Check

$$\frac{f_u \times \gamma_{m0}}{f_y \times \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44 \geq 1.13$$

$$0.7 \leq 1.13 \quad (\text{ok})$$

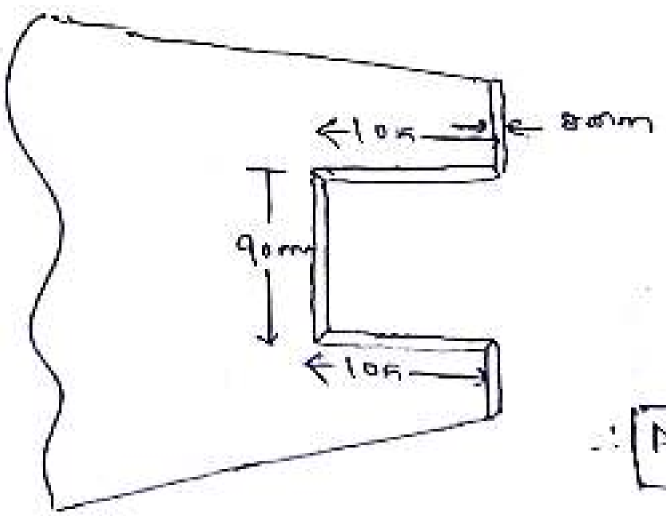
$$\therefore \beta = 1.13$$

$$T_{dn} = \frac{0.9 \times 410 \times 1044}{1.25} + \frac{1.13 \times 250 \times 684}{1.1}$$

$$= 483.85 \text{ kN}$$

(3) Block shear strength

* In the case of welding block shear failure mainly occurs in gusset plate, So, we determined the Block shear strength of gusset plate.



gp Thickness

$$A_{vn} = A_{vg} = \left(\frac{105 + 105}{2}\right) \times 8 \times 2$$

$$= 1680 \text{ mm}^2$$

$$A_{tg} = A_{tn} = 90 \times 8 = 720 \text{ mm}^2$$

$$\therefore \boxed{A_{vn} = A_{vg} = 2 \times L_c \times t \,(\text{gp})}$$

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⑧

$$T_{db1} = \frac{0.9 \times A_{tn} \times f_u}{\gamma_{m1}} + \frac{f_y \times A_{vg}}{\sqrt{3} \times \gamma_{m0}}$$

$$= \frac{0.9 \times 720 \times 410}{1.25} + \frac{250 \times 1680}{\sqrt{3} \times 1.1}$$

$$= 432.98 \text{ kN}$$

$$T_{db2} = \frac{0.9 \times f_u \times A_{vn}}{\sqrt{3} \times \gamma_{m1}} + \frac{f_y \times A_{tg}}{\gamma_{m0}}$$

$$= \frac{0.9 \times 410 \times 1680}{\sqrt{3} \times 1.25} + \frac{250 \times 720}{1.1}$$

$$= 449.96 \text{ kN}$$

$$\therefore T_{db} = \min^m (T_{db1}, T_{db2})$$

$$(T_{db} = 432.98 \text{ kN})$$

∴ Design Tensile strength of tension member

$$T_d = \min^m (T_{dg}, T_{dn}, T_{db})$$

$$= \min^m (393.18, 483.85 \text{ kN}, 432.98)$$

$$\therefore (T_d = 393.18 \text{ kN})$$

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Designe problem

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 reversal stress due to the action of ~~load~~ wind. The effective length of the member is 3m use 20 mm shop bolts of grade 4.6 for the connection

Data given

factored load $(T_u) = 225$ kN.

effective length = 3m

Diameter of bolt $(d) = 20$ mm

Diameter of bolt holes $(d_0) = 20+2$ mm.

$f_{yb} = 400 \times 0.6 = 240$ N/mm^2

$f_{ub} = 400$ N/mm^2

Assume Fe410 grade of member

$f_y = 250$ N/mm^2

$f_u = 410$ N/mm^2.

Step-1 Gross Area of tension member

$$A_g = \frac{T_u \times \gamma_{m0}}{f_y}$$

$$= \frac{225 \times 10^3 \times 1.1}{250} = 990 \text{mm}^2.$$

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(9)

* Ag increase by 25%. (generall 25 – 40%) Increase

$$A_g = 990 \times 1.25 = 1237.5\ mm^2$$

Step-2

* Try ISA 10075 × 8 mm,

$A_g = 1336\ mm^2$ (from steel table pg – 14)

Step-3 Design of Connection

* Bolt strength

$d = 20\ mm$
$d_0 = 22\ mm$

$e = 1.5 \times d_0 = 1.5 \times 22 = 33 \not\approx 40\ mm$

$P = 2.5 \times d = 2.5 \times 20 = 50\ mm \not\approx 60\ mm$

(1) Design Shear strength of a Bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left[n_s \times A_{sb} + n_n \times A_{nb}\right]$$

1 Single shear $n_s = 0$, $n_n = 1$

Area of bolt at shank $(A_{sb}) = \frac{\pi}{4} \times 20^2 = 314.159\ mm^2$

Area of thread portion $(A_{nb}) = 0.78 \times A_{sb} = 0.78 \times 314.159$
$= 245.044\ mm^2$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times \left[1 \times 245.044\right]$$

$$= 45.272\ KN$$

(2) Design Bearing strength of a Bolt

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

$t = 8\ mm$, $d = 20\ mm$, $f_u = 410\ N/mm^2$

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$\Rightarrow$ k_b = Bearing factor.

$$\Rightarrow \frac{e}{3d_0} = \frac{40}{3\times 22} = 0.6$$

$$\Rightarrow \frac{p}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.65$$

$$\Rightarrow \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$$

$\Rightarrow 1$

$\therefore k_b = 0.6$

$$V_{dpb} = \frac{2.5\times 0.6\times 20\times 8\times 410}{1.25}$$

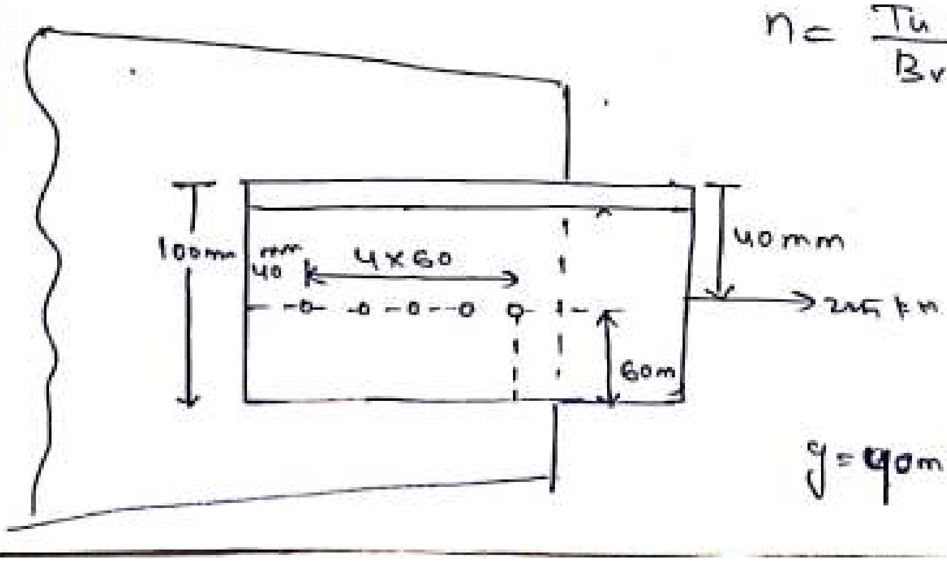
$= 78.72$ KN.

$\therefore$ Bolt strength $= \min^m(45.77\text{ KN}, 78.72\text{ KN})$

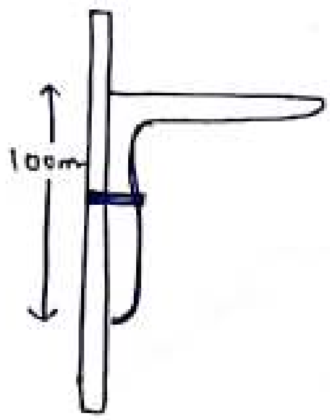
$B_v = 45.77$ KN.

* No of Bolt Required for connection

$$n = \frac{T_u}{B_v} = \frac{225}{45.77} = 4.9 \simeq 5 \text{ nos}$$



$y = 90$mm.



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(10)

Design Tensile strength of tension member (T_d)

① Design strength due to Gross yielding

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

$$A_g = 1336\ mm^2$$

$$T_{dg} = \frac{250 \times 1336}{1.1}$$

$$= 303.63\ kN.$$

② Designe Rupture strength

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

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

$w =$ [redacted] $75\ mm$

$t = 8\ mm$

$$b_s = (w + g - t)$$

$$= (75 + 40 - 8)$$

$$= 107\ mm$$

$$L_c = 4 \times 60 = 240\ mm$$

$$\beta = 1.4 - 0.076 \times \left(\frac{75}{8}\right) \times \left(\frac{240}{410}\right) \times \left(\frac{107}{240}\right)$$

$$= 1.2$$

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Check

$$\frac{\gamma_{m0} \times f_u}{\gamma_{m1} \times f_y} = \frac{1.1 \times 410}{1.25 \times 250} = 1.44 \geq 1.2$$

$$0.7 \leq 1.2$$

$$\therefore \beta = 1.2$$

$$A_{nc} = (100 - t/2 - d_0) \times t$$

$$= (100 - 8/2 - 22) \times 8 = 592\ mm^2$$

$$A_{go} = (75 - t/2) \times t = (75 - 8/2) \times 8$$

$$= 568\ mm^2$$

$$T_{dn} = \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.2 \times 568 \times 250}{1.1}$$

$$= 329.66\ kN.$$

③ Design block shear strength

$$A_{vn} = (e + 4 \times p - 4.5 \times d_0) \times t = (40 + 4 \times 60 - 4.5 \times 22) \times 8$$
$$= 1448\ mm^2$$

$$A_{vg} = (e + 4 \times p) \times t = (40 + 4 \times 60) \times 8 = 2240\ mm^2$$

$$A_{tg} = 60 \times t = 60 \times 8 = 480\ mm^2$$

$$A_{tn} = (60 - 0.5 d_0) \times t = (60 - 0.5 \times 22) \times 8 = 392\ mm^2$$

$$T_{db1} = \frac{0.9 \times A_{tn} \times f_u}{\gamma_{m1}} + \frac{f_y \times A_{vg}}{\gamma_{m0} \times \sqrt{3}}$$

$$= \frac{0.9 \times 392 \times 410}{1.25} + \frac{250 \times 2240}{1.1 \times \sqrt{3}}$$

$$= 409.64\ kN.$$

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(11)

$$T_{db1} = \frac{0.9 \times f_u \times A_{vn}}{\sqrt{3} \times \gamma_{m1}} + \frac{f_y \times A_{vg}}{\gamma_{m0}}$$

$$= \frac{0.9 \times 410 \times 1448}{\sqrt{3} \times 1.25} + \frac{250 \times 480}{1.1}$$

$$= 355.87 \, kN$$

$\therefore$ $T_{db} = \min^m(T_{db1}, T_{db2})$

$T_{db} = 355.87 \, kN$

$\therefore$ Design tensile strength of tension Member

$T_d = \min^m(T_{dg}, T_{dn}, T_{db})$

$(T_d = 303.63 \, kN)$

$T_d = 303.63 > 225 \, kN$

$(T_d > T_u)$

Design is ok

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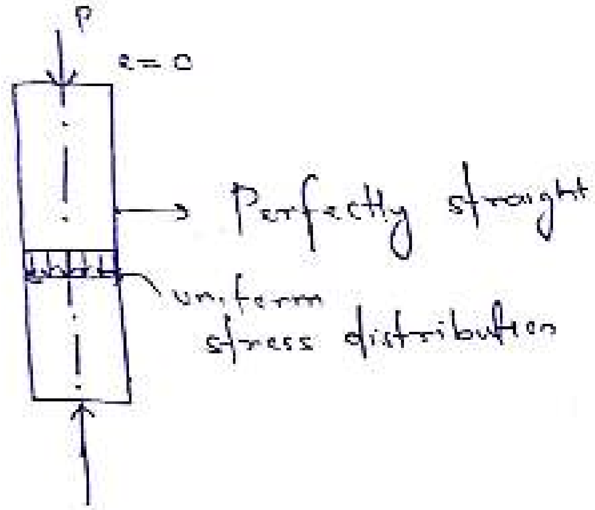
Ch-4

Design of compression Member.

* A compression member is a straight structural element which is subjected to two compressive force.



* An ideal compression member is the one which is:-
  a) Perfectly straight (no intial curvature present inside)
  b) load is none eccentric.
  c) No residual stresses.
  d) No imperfection (crookedness)

Note:- Practically above condition is not exist.

Types of compression member

① Columns, stanchions or posts

* Column is a long vertical compression member which is used to support floor on slab in framed building.

② Strut

* It is an inclined compression member in a truss or a bracing system

Strut are two types

ⓐ Continuous strut :- When a strut passes through multiple joints.

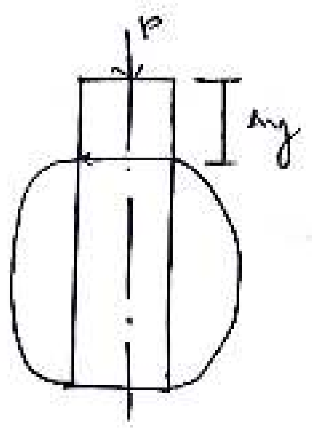
b) Discontinuous strut :- when a strut spans between two joints only.

③ Principal Rafter

* It is the top most inclined compression member in a roof truss.

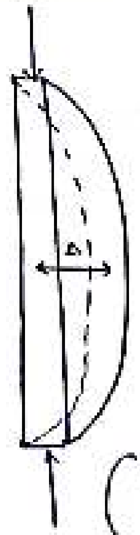
④ Boom :- It is the principal compression element in a gantry crane.

Short column



(Yielding or crushing failure)

Long Column



(Lateral dirn)

(elastic bukling)

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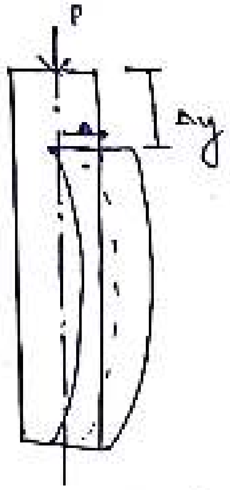
(2)

Intermediate column



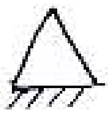
Yielding + Buckling

(Inelastic Buckling)

Effective length (l_e)

* It is the length of compression member which undergoes buckling /curvature.
* It can be defined as the length between two points of hinged end.
* Effective length is depends upon unsupported length of the member and the end support conditions.

* A every support two types of restrained may exist
  a) Rotational Restrained 'θ'.
  b) Lateral sway or translational Restrained. (Δ)



Hinged supports $\theta \rightarrow$ free $\Delta \rightarrow$ Restrained

fixed support $\theta \rightarrow$ Restrained $\Delta \rightarrow$ Restrained

Slider Guided Roller $\theta \rightarrow$ Restrained , $\Delta \rightarrow$ free.

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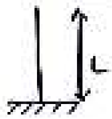
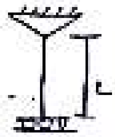
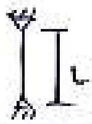
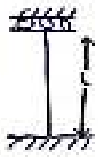
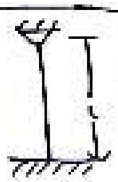
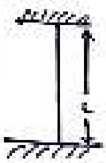
Acc. IS 800-2007, Pg-45, T-11, effective length of prismatic Compression member.

Support condⁿ				Symbol	Theoretical value (L_e)	IS code (L_e)
At one end		At other end				
θ	Δ	θ	Δ			
1) Restrained	Restrained	free	free		2L	2L
2) Restrained	free	free	Restrained		2L	2L
3) free	restrained	free	Restrained		2L	2L
4) Restrained	Restrained	Restrained	free		L	1.2L
5) Restrained	Restrained	free	Restrained		0.7L	0.8L
Restrained	Restrained	Restrained	Restrained		0.5L	0.65L

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Slenderness Ratio

* The ~~due~~ tendency of the member to fail by buckling is measured by a non-dimensional number or ratio known as slenderness Ratio. 'λ'.

* Longer the length of member for a given area greater will be tendency to fail by buckling and lesser will be its load carrying capacity.

* Slenderness ratio is measured by the ratio of effective length of the member & Appropriate Radius of gyration. 'r'.

$$\text{Slenderness Ratio } (\lambda) = \frac{L_e}{r} \text{ or } \frac{KL}{r}$$

L → unsupported length of member

K → constant for effective length.

r → Radius of gyration.

$$r = \sqrt{\frac{I}{A}}$$

I = Moment of Inertia

A = Cross sectional Area of Section

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* IS code has kept a limitation on the maximum value of slenderness ratio of the member.

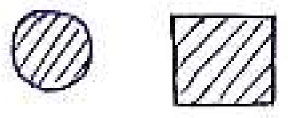
IS 800-2007, Pg-20, T-3.

Member	Slenderness Ratio
a) A member carrying compressive load from dead & imposed load →	180
b) A member subjected to compressive force resulting only from combination with wind/ earthquake action →	250
c) Compression flange of a beam restrained against torsional buckling →	300.

Types of Section

* Generally those section are preffered which provide maximum value of least radious of gyration.

① Bars or Rod



* In case of solid bars most of the area of cross section is located near by to principal axis, due to which they have less value of moment of innertia for a given area.

* They are not prefered if the length of the member is greater than 3m.

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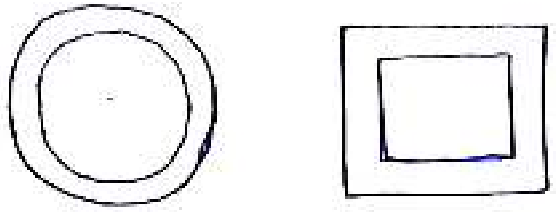
(4)

(2) Hollow tube Section



* In this section most of the area is located Away from principal axies, Hence they provide high value of moment of inertia / Radious of gyration.

* Hollow Section have good resistance towards torsion also.

* for bracing element hollow section are used.

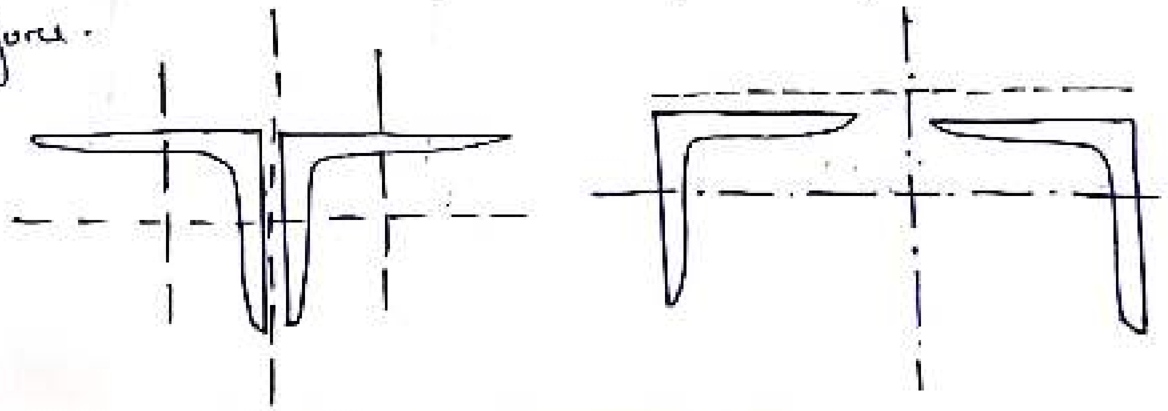
(3) Single angle Section

* It can be used as a roof truss element connected with the help of bolting / Reveting or welding

* Equal angles are prefered over unequal angles as they provide larger value of minimum radious of gyration,

(4) Double angle Section

* Strut in a truss are generally made from double angle section connected back to back or spaced apart as shown in figures.



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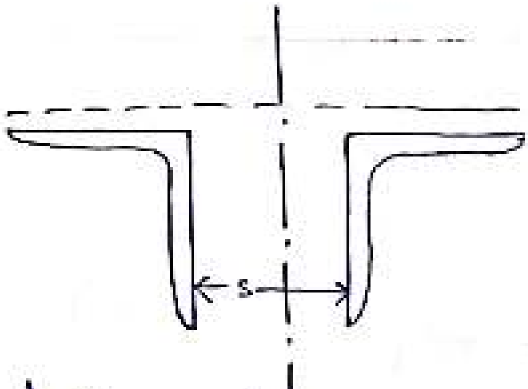
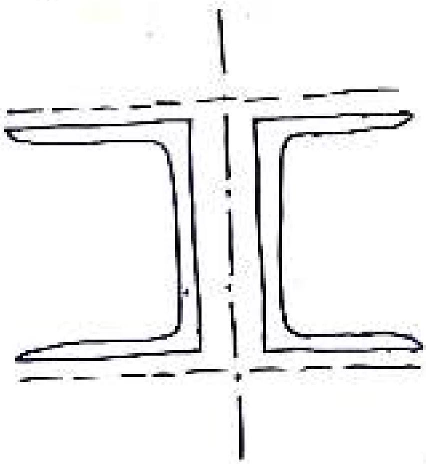


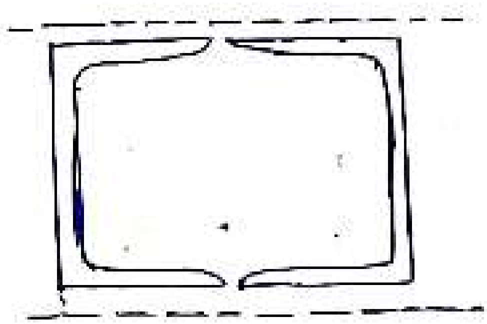
* In case of double angle section connected back to back unequal angle section are preferred. with their longer leg is connected.

(Star pattern or Cruciform)

5) Double Channel Section

* In case of moderate or heavy loaded structure angle section are not used. Double channel Section are used which can be connected back to back or toe to toe.





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⑤

Types of Buckling

① Flexural Buckling

* In this buckling the member moves ~~and~~/ translates in a lateral direction, without undergoing rotation.
* This type of failure occurs when the section has large torsional resistance as compared to bending resistance.
* Those section which have large moment of inertia about one axis as compared to other axes fails by flexural buckling.

② Torsional Buckling

* In this buckling the member rotate about its longitudinal axis.

③ Flexural torsional Buckling

* It is a combined failure of member due to flexural and torsion.

Design Strength of Compression Member.

Pery - Roberston Approach.

* In peri Roberston approach all the imperfection, Residual, shape of section buckling axis & Method of fabrication, all these factors are considers,

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* In this approach the buckling was classified into 4 categories. such as a, b, c & d.

Buckling class	Description
a	→ quasi-imperfection in the member.
b	→ Moderate level of imperfection
c	→ High level of imperfection
d	→ extreme level of imperfection

[Acc. to IS-800-2007, Pg-44, T-10] Buckling class of cross-section.

eg:- I section	$\frac{h}{bf} > 1.2$	z-z	a
	$tf \le 40mm$	y-y	b.

* Design compressive strength of member

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

where,

A_e = effective sectional Area / Gross area of a Section

f_{cd} = Designe Compressive stress.

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⑥.

The design compressive stress 'f_{cd}' of axially loaded compression member:

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

where,

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

λ = non-dimensional effective slenderness ratio.

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

f_{cc} = euler's buckling stress

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

where KL/r = effective slenderness ratio on ratio of effective length, KL to appropriate radious of gyration.

α = Imperfection factor (T-7, Pg-35)

γ_{m0}: Partial safety factor

Value of α (Imperfection factor)

Buckling classes	a	b	c	d
α	0.21	0.34	0.49	0.76

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Design steps for Compression Member

Given Data → Load (P_u), Length of the member & end support condition

Steps-1 :- Assume suitable value of slenderness ratio 'λ' (Generally for a column 3-5 m height, slenderness ratio between 40-60 gives satisfactory result).

Step-2 Corresponding to the slenderness ratio (λ) assumed in (step-1) determine 'f_{cd}' for a particular buckling classes. [IS-800-2007, Pg-40, 41, 42 & 43]

Step-3 Determine required gross sectional Area.

$$\boxed{A_g = \frac{P_u}{f_{cd}}}$$

Step-4 from IS hand book no.1 a suitable section is selected which provides the required area As per step 3.

Step-5 Determine actual slenderness ratio of the member

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

∴ Check the above value of slenderness ratio $\not> \lambda_{max}$ (Given IS code Pg-20 - T-3).

Step-6 Determine the actual value of ~~slenderness~~ 'f_{cd}' for the section selected.

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

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2], \quad \lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

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

Step-7 Determine the design compressive strength of Member (P_d)

$$P_d = f_{cd} \times A_e \geq P_u \text{ (Factored or design load)}.$$

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(7).

Question-1

Determine the design axial load capacity of the column ISHB 300 @ 577 N/m If the length of the column is 3m and its both end are pined.

Ans for rolled steel section ISHB 300@ 577 N/m

(from steel table)

$A_g = 74.85\ cm^2 = 7485\ mm^2$

$h = 300\ mm$

$b_f = 250\ mm$

$t_f = 10.6\ mm$

$f_y = 250\ N/mm^2$

$f_u = 410\ N/mm^2$

effective length of column

$L_e = L$ (both end hinged)

$E = 2\times10^5\ N/mm^2$

Minimum radius of gyration

$r_{min} = 54.1\ mm$

* Buckling classes of cross section

$\rightarrow \dfrac{h}{b_f} = \dfrac{300}{250} \leq 1.2$

$\rightarrow t_f = 10.6\ mm \leq 100\ mm$

for y-y axis

Buckling class - c (IS 800-2007 pg-44)

Designe compressive strength

$$\boxed{T_d = A_e \times f_{cd}}$$

$\alpha = 0.49$ [code book pg-35] Imperfection factor

$$f_{cc} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 2\times10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92\ N/mm^2$$

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$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624$$

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

$$= 0.5\left[1 + 0.49(0.624 - 0.2) + 0.624^2\right]$$

$$= 0.915.$$

Design compressive stress

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{0.915 + [0.915^2 - 0.624^2]^{0.5}}$$

$$= 143.46 \leq f_y/\gamma_{m0}$$

$$\leq \frac{250}{1.1} = 227.72 \text{ mpa}$$

∴ Design compressive strength of column

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

$$= 143.46 \times 7854 = 1126.73 \text{ kN}$$

Question -2

Calculate the design compressive load for an ISHB 250 @ 53.6 kN/m 4m height the column is restrained in position only of both the ends. It is to be used as uncased column in a single story building.

ISHB 250 @ 536 kN/m (from steel table)

$h = 250$ mm

$b_f = 250$ mm

$t_f = 9.7$ mm

$r_{min} = 53.7$ mm

$A_g = 6971$ mm²

$L_e = 0.65L$ (Both end fixed).

$= 0.65 \times 4000$

$= 2600$ mm.

$f_y = 250$, $f_u = 410$ N/mm²

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(5)

Buckling class

$\rightarrow \frac{h}{b_f} = \frac{250}{250} \leq 1.2$

$\rightarrow t_f = 9.7 < 100\,mm$

Buckling class

$y\text{-}y$ dir^n = c

$\therefore \alpha = 0.49$ [IS cod] (pg-55)

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2\times10^5}{\left(\frac{2600}{53.7}\right)^2} = 842.02\ N/mm^2$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{842.03}} = 0.544$$

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

$$= 0.5\times[1+0.49(0.544-0.2)+0.544^2]$$

$$= 0.732$$

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

$$= \frac{250/1.1}{0.732+[0.732^2-0.544^2]^{0.5}}$$

$$= 186.017\ N/mm^2 \leq f_y/\gamma_{m0} = 227.7\ N/mm^2$$

Design Compressive strength

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

$$= 6971\times186.017 = 1296.724\ kN$$

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Design problem

Design a single angle strut connected to the gusset plate to carry 180 kN factored load. the length of the strut is 3m. and its one end is fixed and other end is hinged.

Data given

$P_u = 180\,kN$, $l = 3m$

effective length = $0.8L = 0.8 \times 3 = 2400\,mm$ (One end fixed & Other end hinged)

Step-1 Assume slenderness ratio

$$\lambda = 40$$

Step-2 $\lambda = 40$, buckling class crosssection = c (angle section).

[IS 800-2007 Pg-42 T-9 $f_y = 250\,N/mm^2$] $f_{cd} = 198\,N/mm^2$

Step-3 Gross Area required

$$A_g = \frac{P_u}{f_{cd}} = \frac{180 \times 10^3}{198}$$

$$= 909.09\,mm^2.$$

Step-4

Select ISA 70×70×8 mm (from steel table Pg-10)

$A_g = 10.58\,cm^2 = 1058\,mm^2.$

Step-5 Min^m radius of gyration $r_{min} = 1.35\,cm$
$= 13.5\,mm$

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(4).

Slenderness Ratio $(\lambda) = \frac{KL}{r_{min}}$

$= \frac{2400}{13.5} = 177.77 < 180 \; (\lambda_{max})$ (OK)

(IS code T-3)

<u>Step-6</u> Design Compressive stress 'f_{cd}'

$\alpha = 0.49$ Buckling class 'c'

$$f_{cc} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 2\times10^5}{(177.77)^2} = 62.51 \; N/mm^2$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{62.45}} = 2.$$

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

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

$$= 2.941$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{2.941 + [2.941^2 - 2^2]^{0.5}}$$

$$= 44.587 \leq f_y/\gamma_{m0} = 227.27 \; N/mm^2$$

<u>Step-7</u> Design Compressive Strength of column

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

$$= 1058 \times 44.587 = 471.73 \; kN > P_u = 160 \; kN$$

<u>OK</u>

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