DEPARTMENT OF CIVIL ENGINEERING

(ACADEMIC YEAR: 2021-2022)

CE8601 –DESIGN OF STEEL STRUCTURAL ELEMENTS

(Regulation 2017)

Semester-VI

NAME-

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STRUCTURAL STEEL

Steel can be divided into two principal groups:

(i) Steel structures - made of largely of plates or sheets

Ex: Tanks, bins, Chimney and steel roof, etc.

(ii) Framed structures - assemblies of tension, compression and flexural members.

Ex: Truss-frame, rigid frames, girder and columns, etc

ADVANTAGES OF STRUCTURAL STEEL:

1. High strength per unit weight
2. Ductile material. No sudden failure, but shows impending failure by large deflections.
3. Tough - they have both strength and ductility.
4. Easy handling and transportation. Mostly used for prefabrication.
5. Long life if properly maintained.
6. Properties donot change with time.
7. Additions and alterations can be easily made.
8. Can be erected at a faster rate

9. High scrap value amongst all building materials. (Reused).

10. Recyclable material

DISADVANTAGES:

1. Steel structures, exposed to environment are subjected to corrosion.
2. Steel structures need fire proof treatment, which increases cost.
3. Fatigue in steel, results in reduction of strength when subjected to cyclic loads/ reversal loads.
4. At places of stress concentration in member, under certain conditions, steel may lose its ductility.

APPLICATIONS:

1. High rise building skeletons.
2. Industrial buildings.
3. Transmission towers.
4. Railway bridges.
5. Overhead tanks.
6. Chimneys (stacks).
7. Bunkers and silos.

(x) MECHANICAL PROPERTIES OF STEEL:

Mechanical properties of steel depends upon the following factors:

(a) Chemical composition;

(b) Rolling methods;

(c) Rolling thickness;

(d) Heat treatment; and

(e) Stress history.

Important mechanical properties are:

1. Ultimate Strength
2. Yield stress (or proof stress).
3. Ductility.
4. Weldability.
5. Toughness.
6. Corrosion resistance, and
7. Machinability.

1. ULTIMATE STRENGTH:

Steel is designated in India as Fe310, Fe410WA, Fe 540B, Fe590, etc, where Fe stands for the steel and the number after Fe is the characteri ultimate tensile stress (in MPa). The letters A, B or C indicates the grade of steel.

W → steel is weldable.

Ce

Interded use of diff. grades of steel (IS 2062):

(i) Grade A – structures subject to normal conditions. and for non- critical loading.

(ii) Grade B – critical loading applications.

(iii) Grade C – Risk of brittle fracture requires consideration due to their design, size and/or service conditions.

2. YIELD STRESS:

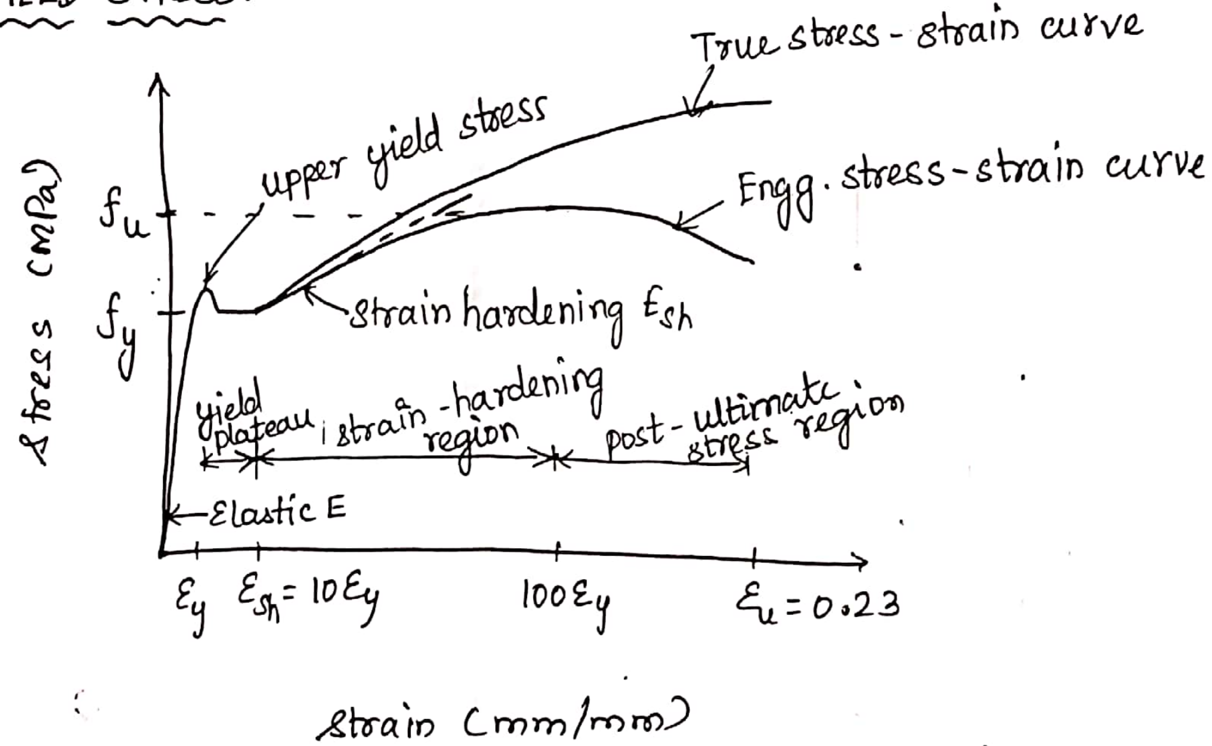


Fig 1: Stress - strain curve of mild steel specimen

* Modulus of Elasticity (E) = $\frac{\text{stress within the proportional limit (f)}}{\text{strain } (\varepsilon)}$

* E value vary in range of 200,000 to 210,000 MPa
* Steel obeys hooks law in this linear range (elastic range).

* The limit of elastic behaviour is associated with the yield stress f_y and the corresponding yield strain $\varepsilon_y = f_y / E$.
* Beyond yield limit, the steel ~~fa~~ flows plastically without increase in stress until the 'strain hardening' strain ε_{sh} is reached.

  This plastic range is usually considerable, and accounts for the ductility of steel.
* The stress increases above the yield stress f_y, when the strain hardening strain ε_{sh} is exceeded, until the ultimate tensile stress ~~f_{u}~~ f_u is reached

  Large reduction of c/s occurs at this stage
* The initial slope of strain-hardening part of the curve is termed the strain-hardening modulus, E_{sh}'
* Yield stress (f_y) = $E \times \varepsilon_y$

  ε_y for mild steel is of the order of 0.00125 or 0.125%.

3. DUCTILITY:

Ability of a material to change its shape without fracture.

* Stress-strain curve of a material also indicates ductility $\Rightarrow$ it is the amount of permanent strain, measured by determining the percentage elongation

Gauge length $= 5.65\sqrt{A_0}$

A_0 - initial cross sectional area.

$$\text{Percentage elongation} = \frac{(\text{elongated length b/w gauge point} - \text{gauge length}) \times 100}{\text{gauge length}}$$

* Minimum required percentage elongations of steel produced in India is given in Table 1 of IS : 800 : 2007.

* Eurocode 3, (i) Elongation at failure > 15 %

(ii) $\frac{f_u}{f_y} \geq 1.10$

(iii) $\varepsilon_u \geq 15\,\varepsilon_y$

} ductility requirements

4. WELDABILITY:

Ability to weld material without failure caused by Lamellar tearing.

Mostly occurs in large weld placed on thick plate.

5. TOUGHNESS:

Ability of steel to resist fracture under impact loading (ie) the capacity to absorb large amounts of energy.

* Toughness is an important design criterion to be considered ~~under~~ for earthquake loads.
* The area under stress-strain curve is a measure of toughness.
* At room temperature, structural steel is very tough & fails in ductile manner.

At temperature below $0^{\circ}C$, the yield strength of steel is only marginally affected, while there is substantial reduction in ductility & toughness [@ low temperature, steel fails suddenly without warning]

6. CORROSION RESISTANCE:

* Exposure to sea water, acid or alkaline vapours hasten the process of corrosion.
* 0.075 mm thickness reduced due to sulphur dioxide in atmosphere.
* Protection method - Galvanizing, use of paint or metallic coating, or a plastic coat in case of metallic sheeting.
* Steel with copper content of 0.2 to 0.5% have improved resistance to atmospheric corrosion but needs protection.

7. HARDNESS:

It is the measure of resistance of the material to indentations and scratching.

* Brinell Hardness Number – 150 to 190.
* Vickers Hardness Number – 157 to 190
* Rockwell hardness – 80 to 105.

8. FATIGUE RESISTANCE:

Fatigue is the term used in connection with the initiation and propogation of microscopic cracks into microcracks by repeated application of alternating stresses.

The damage ~~of~~ and failure of materials under cyclic loads is called fatigue damage.

Following members should be designed for fatigue:

(i) Members supporting lifting or moving loads.
(ii) Members subjected to wind-induced oscillations or large number of cycles.
(iii) Members subjected to repeated stress cycles from vibrating machinery, and,
(iv) Members subjected to crowd-induced oscillations.

STRUCTURAL STEEL PRODUCTS

Structural steel products can be divided into the following categories:

1) Flat hot-rolled products
2) Hot-rolled sections
3) Bolts
4) Welding electrodes, and.
5) Cold-rolled shapes.

1) FLAT HOT-ROLLED PRODUCTS:

- Plates, flat bars, sheets and strips

⟹ Plate

- Designated as:- ~~Plate~~
  - Plate – ISPL – length × width × thickness
  - Sheets – IISH – length × width × thickness
  - Strips – ISST – width × thickness
  - Flats – ISF – thickness
- Produced in steel plants from red hot steel billets by passing them through a series of rollers.

2) HOT-ROLLED SECTIONS:

- Rectangular hollow sections and tubes are manufactured by the process of extrusion
- In this process, heated metal billets are forced to pass through a die plate which has an opening of required size and shape.

Hot-rolled sections consists of the following:

(i) Rolled beams: (IS : 808 : 1989)

- Indian Standard Junior Beams (ISJB)
- Indian Standard Light weight beams (ISLB)
- Indian Standard Medium weight beams (ISMB)
- Indian Standard Wide flange beams (ISWB)

[abb. as JB, LB, MB & WB respectively].

depth in mm → JB 150

(ii) Columns / Heavy weight beams: (IS : 808 : 1989)

- Indian Standard Column Sections (ISSC)
- Indian Standard Heavy weight Beam (ISHB).

[abbreviated as SC, HB respectively]

depth in mm → HB 200

(iii) Parallel Flange Beam and Column Sections (IS 12778 : 2004)

- Indian Standard Narrow Parallel Flange Beams (ISNPB)
- Indian Standard Wide Parallel flange Beams (ISWPB)

[abbreviated as NPB & WPB respectively]

depth, width: NPB 200 × 100 × 18.42 ← mass in kg/m

(iv) Channels: (IS : 808 : 1989)

- Indian Standard Junior Channels (ISJC)
- Indian Standard light weight Channels (ISLC)
- Indian Standard Medium weight Channels (ISMC)
- Indian Standard Medium Weight Parallel flange Channels (ISMCP)

[abb. as JC, LC, MC & MCP respectively]

depth in mm → JC 150

- Indian standard equal / unequal angles (ISEA or ISA)

[abbreviated as ∠ ; Ex: ∠200×100×10 → 200 & 100 = length of legs, 10 = thickness (in mm)]

(vi) T- sections: (IS 1173: 1978)

depth in mm
HT100

- Indian Standard rolled Normal Tee Bars (ISNT)
- Indian standard rolled Deep legged Tee bars (ISDT)
- Indian standard slit light weight Tee bars (ISLT)
- Indian Standard slit Medium weight Tee bars (ISMT)
- Indian Standard slit Tee bars for H sections (ISHT),

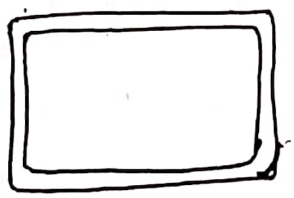
[abb. as NT, DT, LT, MT & HT]

(vii) Tubular Sections →

- As per IS 1161 : 1998

(viii) Rectangular / Square Hollow Sections:

- As per IS 4923: 1997



→ Rectangular Hollow section

3) Cold FORMED LIGHT-GAUGE SECTIONS: (IS 811-1987)

- Used when hot-rolled sections becomes uneconomical especially in small buildings subjected to lighter loads.

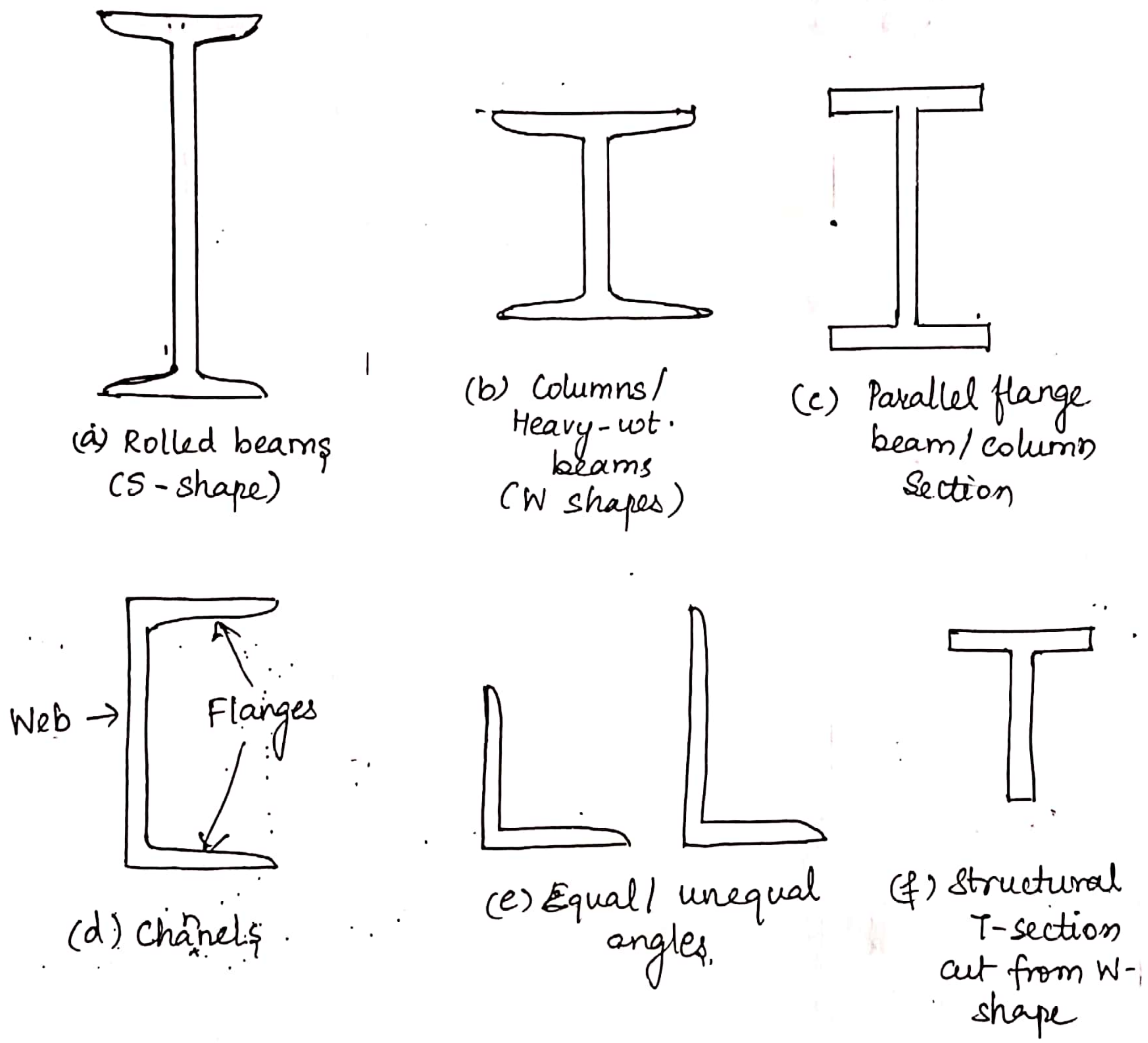


Fig 2: Types of hot rolled sections.

LIMIT STATE DESIGN CONCEPTS

To safeguard a structure against the risk of failure, safety margins are normally provided in design. Safety margins in Working stress Method - permissible stresses; in Limit State Design method - load factors.

* Uncertainities in design may be caused by variables such as loads, material strength and member dimension. Hence the structure must be designed to provide for the possibilities of overload.

* Factors affecting strength & serviceability are :-
  - Construction method
  - Workmanship
  - Quality control
  - intended Service life of the structure
  - Human errors
  - Change of use in future
  - Frequency of loading, etc.

(X) * LIMIT STATE is a state of impending failure beyond which a structure ceases to perform its intended function satisfactorily.

Limit State Considered

Ultimate limit state
- Strength
- Stability against overturning & sway
- Failure due to excessive deformation or rupture
- Fracture due to fatigue
- Brittle fracture

Serviceability limit state
- Deformation & deflection
- Vibration
- Repairable damage due to fatigue, ~~corrosion~~
- Corrosion & durability.
- Fire

LIMIT STATE OF STRENGTH:

- Multiple safety factor design format
- Load and resistance factor design format
- Partial safety factor format

(i) Multiple Safety factor design format:

In this method, the load factor shall be choosen to result in 'target reliability', such that it will produce designs which will provide the required amount of safety and at the same time results in economic structures [safety + economy]

(ii) Load and resistance factor design format:

Design resistance $(\phi R_n) \geq$ Design load effect $(\Sigma \gamma_i Q_i)$

where,

$R_n \rightarrow$ nominal strength

$\phi \rightarrow$ strength reduction factor. (accounts for understrength)

$Q_i \rightarrow$ various loads (DL, LL, WL, etc).

$\gamma_i \rightarrow$ respective overload factor

⇒ ϕ accounts for :- (ϕ is < unity)

* deviation of material strength
* Reduction in member strength due to fabrication & tolerances.
* Variation in member sixes
* Unceritainities in theoritical assumptions, and

* uncertainities in the calculation of strength of member

⇒ γ_i accounts for:- (γ_i always > unity)

* Deviation of load from characteristic value
* inaccurate assessment of load.
* uncertainity in the assessment of effects of the load
* uncertainity in assessment of limit state being considered.

$\Sigma Q_i < R_n \left(\frac{\gamma}{\phi}\right)$, represents the working stress concept.

$\frac{\gamma}{\phi}$ → denotes 'factor of safety' applied to material strength; so-called 'load factor' applied to load in order to arrive at the ultimate load for design.

(iii) Partial safety factor format:

Multiple safety factor adopted by the code is in the so-called partial safety factor format

$$R_d \geq \Sigma \gamma_{if} Q_{id}$$

where, R_d = design strength $= \frac{R_u}{\gamma_m}$

R_u → characteristic material strength

γ_m → partial safety factor for the material

LOADS ON STRUCTURES

Most unfavourable load combination (producing max. stresses) should be considered for designing the structure.

Usual forms of loads:-

Major loads
(i) Dead Loads. (DL)
(ii) Imposed loads/ Live loads. (LL /IL)
(iii) Wind Loads. (WL)
(iv) Earthquake loads. (EL)

(v) Snow loads (varies with ~~topography~~ place)

(vi) Ice loads (varies with place)

(vii) Crane loads (In industrial buildings)

Major loads — (viii) Temperature loads

(ix) Erection & fabrication loads.

(x) Dynamic loads (due to earthquake & wind)

(xi) Impact loads (bridges & cranes)

(xii) Longitudinal loads (due to vehicle/moving loads)

(xiii) Blast or explosive loads

(xiv) Loading due to fire.

(xv) Foundation settlement (especially differential settlement)

(xvi) Hydrostatic loads (liquid retaining structures)

→ uncertainities in theoritical assumptions, and

(xvii) Wave and current loads (Marine & offshore structures)

(xviii) Earth pressure (Basements, Retaining walls, columns, footings, etc)

(xix) Creep & Shrinkage loads

(xx) Fatigue effects

(xxi) Dust loads (in deserts)

(xxii) Prestressing loads.

(xxiii) Effect due to stress concentration & eccentric connections, rigidity of joints differing from design assumptions, etc.

(xxiv) Construction loads, elastic shortening of members,

⇒ Loads on the structure may be due to the following :-

1. Mass & gravitational effects
2. Mass & acceleration effects.
3. Environmental effects.

⇒ IS 800:2007 classifies actions or loads as follows:

1. Permanent actions (Q_p)
2. Variable actions (Q_v)
3. Accidental actions (Q_a)

Design a suitable I beam for a simply supported span of 5 m. and carrying a dead load of 20 kN/m and imposed load of 40 kN/m. Take fy = 250 MPa

Design load calculations :

Factored load = $\gamma_{LD} \times 20 + \gamma_{LL} \times 40$

Using partial safety factors for D.L γ_{LD} = 1.50 and for L.L γ_{LL} = 1.5 (*Cl. 5.3.3 Table 4, Page 29)*

Total factored load = 1.50 x 20 + 1.5 x 40 = 90 kN/m

Factored Bending Moment M = 90 x 5 x 5 / 8

= 281.25 kN.m

Zp required for value of *fy* = 250 MPa and

γ_{mo} = 1.10

(*Table 5, Page 30*)

Zp = (281.25 x 1000 x 1000 x 1.1) / 250 = 1237500 mm^3

= 1237.50cm3

Using shape factor = 1.14, *Ze* = 1237.50/1.14 =1085.52 *cm3*

Options ISWB 400 @ 66.7 *kg/m* or ISLB 450 @ 65.3 *kg/m*

Try ISLB 450

Ze = 1223.8 cm^3 > 1085.52

Geometrical Properties : ISLB 450

D = 450 mm , B = 170 mm , tf = 13.4 mm , tw = 8.6 mm , h1 = 384 mm , h2 = 33 mm

Ixx = 27536.1 cm4

As *fy = 250 MPa ,*

Section Classification :

B/2tf = 85 / 13.4 = 6.34 < 9.4ε

h1 / tw = 384/8.6 = 44.65 < 83.9 ε

Section is Classified as *Plastic*

Zp = 1.14 x 1223.8 = 1395.132 cm3

Design Bending Strength: Md

$$M_d = \frac{\beta_b Z_p fy}{\gamma_{mo}} = \frac{1.0x1395.132x1000x250}{1.10} = 317.075 kN.m$$

> 281.25 kN.m

βb = 1.0 for plastic section (Cl. 8.2.1.2, Page 53)

Check for Serviceability – Deflection

Load factor = γLD and γLL = 1.00 both , (Cl. 5.6.1, Page 31)

Design load = 20 + 40 = 60 kN/m

$$\delta = \frac{5x60x(5000)^4}{384\,x2x10^5\,x27536.1x10^4} = 8.866\,mm$$

Limiting deflection = Span/360 *(Table. 5.3, Page 52)*

= 5000/360 = 13.889 mm….*OK*

Hence Use ISLB 450

Working Stress Method
IS : 800 - 1984

Max Bending Moment = 60 x 5 x 5/8 = 187.5 kN.m

Max Shear Force = 60 x 5/2 = 150 kN

$$Zreq = \frac{187.5x10^6}{165} = 1136.3\,cm^3$$

Select ISLB 450 Zxx = 1223.8 Moment Capacity

= 201.927 kN.m

Check for Shear

$$q_{av} = \frac{150x1000}{450x8.6} = 38.76MPa < 100\ MPa$$

Check for Deflection

$$\delta = \frac{5x60x(5000)^4}{384\,x\,2x10^5\,x\,27536.1x10^4} = 8.866\,mm$$

Limiting deflection = Span/325 = 5000/325

= 15.38 mm…*OK*

Comparison of ISLB 450 Section

	Working Stress Method	Limit State Method
Moment Capacity	201.927 kN.m > 187.5 KNm	317.075 KNm > 281.25 KNm
Shear Capacity	387 KN > 150 KN	507.497KN > 225 KN
Section Designed	ISLB 450@ 65.3 Kg/m	ISLB 450 @ 65.3 kg/m

The Section designed as per LSM is having more reserve capacity for both BM and SF as compared to WSM

CE8601 – DESIGN OF STEEL STRUCTURES

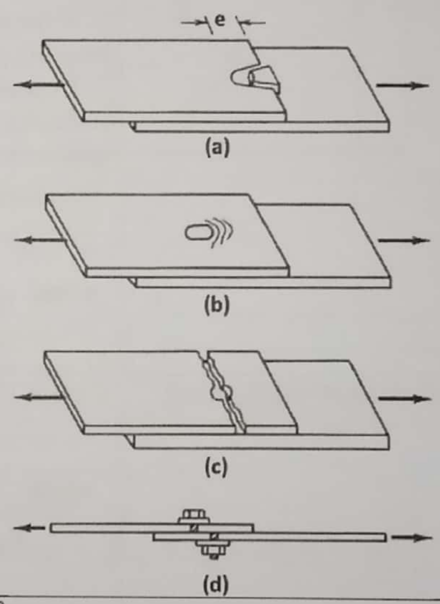
UNIT – 2

CONNECTIONS IN STEEL STRUCTURES

TOPICS TO LEARN:

1. Bolted connections – Lap joint, single cover butt joint, double cover butt joint.
   a. Analysis – Computation of efficiency, strength of joint.
   b. Design of connection.
2. Welded connections – Fillet weld, groove weld, plug weld connections
   a. Analysis – Computation of efficiency, strength of joint.
   b. Design of connection.
3. Rivet connections – Lap joint, butt joint – single cover and double cover
   a. Analysis – Computation of efficiency, strength of joint.
   b. Design of connection.

BOLTED CONNECTIONS

Failure modes of bolted connections: a) tear-out failure of sheet, b) bearing failure of sheet, c) tension failure of net section, d) bolt shear failure	
A_{nb}	$0.78x(\frac{\pi}{4}d^2)$
n_n	1
n_s	0 (since, in at least one plane of the connection threads are present)
A_{sb}	$\frac{\pi}{4}d^2$
D	Diameter of the bolt
d_o or d_h	Diameter of hole = d+2 (mm)
f_{ub}	Ultimate tensile strength of bolts – table 1 IS 800 – page 13 &14
f_u	Ultimate tensile strength of plate

t_{pkg}	Thickness of packing plate = difference between plates to be connected
T	Design thickness of plate = least of the following: i. Main plates to be connected [NOTE: Theoretically it is assumed that cover plate and packing plates do not carry any load]
Thickness of cover plate	Greater than 5/8 times of 't' (here, t is the lowest thickness of the main plates)
Design Shear strength of bolt (V_{dsb})	Clause 10.3.3 – page 75 IS 800 $$V_{dsb} = V_{nsb} / \gamma_{mb}$$ where V_{nsb} = nominal shear capacity of a bolt, calculated as follows: $$V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$ [Note: 1. In case of double cover butt joint, the bolt will be subjected to double shear, therefore shear strength of bolt should be multiplied with a factor 2, which denotes the number of shear planes 2. If the plates of unequal thickness are to be joined, packing plate should be provided. 3. Depending on the thickness of packing plate, reduction factor as per clause 10.3.3.3 should be applied to design shear strength of bolt – i.e. $V_{dsb} = V_{dsb,\text{calculated from equation}} \times \beta_{pkg}$ $$\beta_{pk} = (1 - 0.0125\, t_{pk})$$ where t_{pk} = thickness of the thicker packing, in mm.]
Design bearing strength of bolt (V_{dpb})	Clause 10.3.4 page 75 $$V_{dpb} = V_{npb} / \gamma_{mb}$$ where V_{npb} = nominal bearing strength of a bolt $= 2.5\, k_b\, d\, t\, f_u$ where k_b is smaller of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0;$
Design tension capacity of plate (T_{db})	Clause 10.3.5

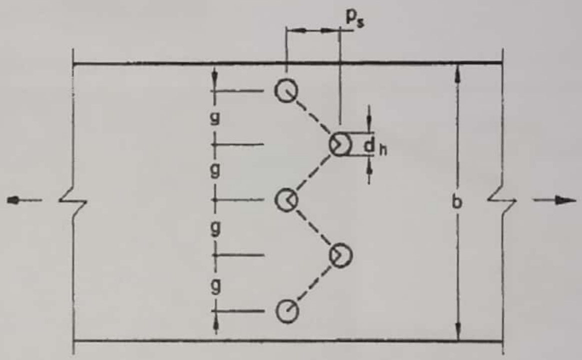
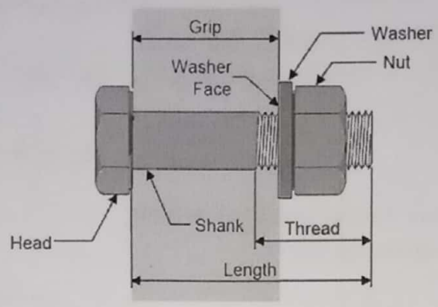
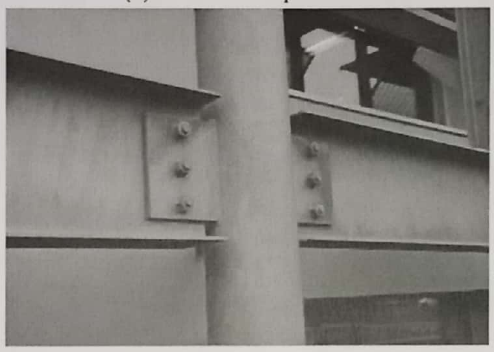
	$$T_b \le T_{db}$$ where $T_{db} = T_{nb} / \gamma_{mb}$ T_{nb} = nominal tensile capacity of the bolt, calculated as: $0.90 f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$ where, Net area Chain bolting, $A_n = [p - d_h]t$ $$A_n = \left[b - nd_h + \sum_i \frac{p_{si}^2}{4g_i} \right] t$$ Staggered bolding, where, 
Strength of plate (T_{dn})	Clause 10.3.5 $T_{dn} = 0.9 A_n f_u / \gamma_{m1}$ where, Net area $A_n = pt$
P	Pitch distance = 2.5d
E	Edge distance = $1.5d_o$
Joint efficiency	(Strength of bolt/strength of plate) x 100
Eccentrically loaded connections	F_1 = direct force = P/n F_2 = radial force = $Mr/\sum r^2$ Resultant force < V_{sb} Resultant force = $\sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$ where, n = number of bolts = $\sqrt{\frac{6M}{V_{sb}pn'}}$ n' = number of rows in which bolts are arranged p – pitch V_{sb} – design strength of bolt (lowest of shearing and bearing strength of bolt) M – bending moment = Pe e = distance between the eccentric load and the C.G. of gusset plate. P = eccentric load r = radius of extreme bolt from the CGC of the gusset

	plate. $\sum r^2$ = summation of radius from the CGC to all the bolts in the connection $\cos\theta$ = angle between F_1 and F_2 = p/r

Figures for reference



(a) Bolt – components



(b) Bolted connection in construction

(c) Bolted connection with gusset plate

WELDED CONNECTIONS

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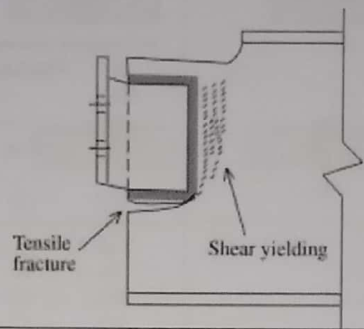
<td>Failure modes of welded connections: a) tensile fracture b) Shear yielding</td>
<td>
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<tr>
<td>Types of welded connections</td>
<td>
i. Fillet weld
ii. Grove weld (or butt weld)
iii. Plug weld
iv. Slot weld
v. Spot weld
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<tr>
<td>Types of fillet weld</td>
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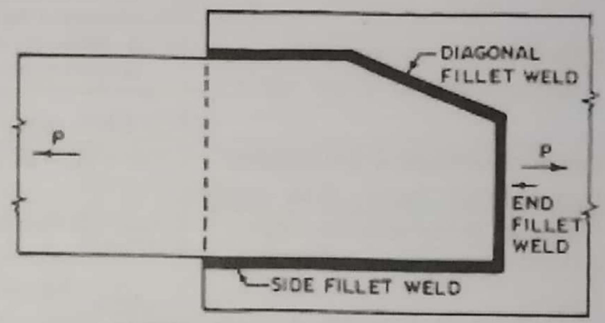

TYPES OF FILLET WELD
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<td>Fillet weld – fails mostly by shear</td>
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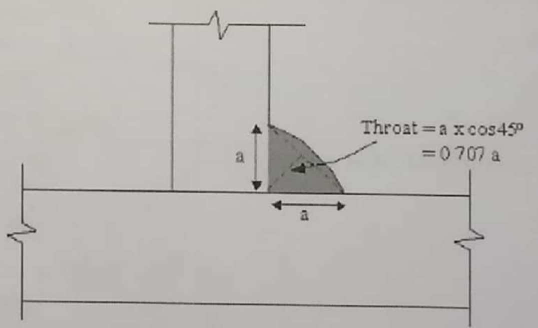

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<td>Failure plane in fillet weld</td>
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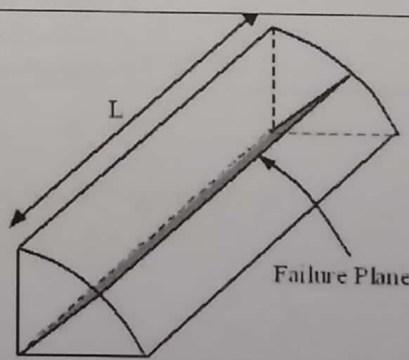

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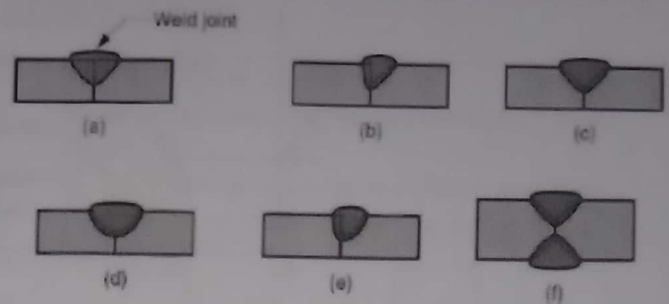
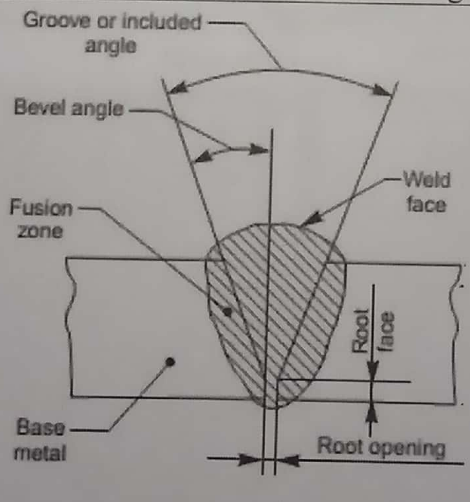
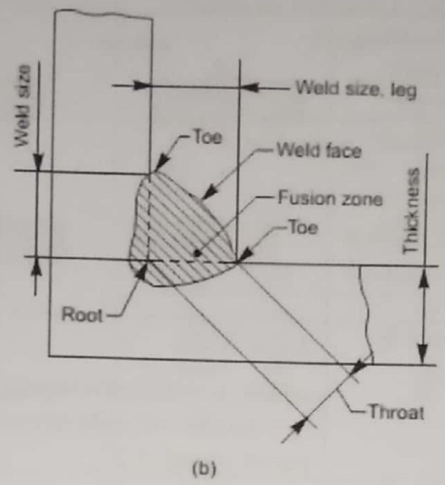
Minimum size of fillet weld	**Table 21 Minimum Size of First Run or of a Single Run Fillet Weld** (*Clause* 10.5.2.3) SI No. (1) | Thickness of Thicker Part, mm: Over (2) | Up to and Including (3) | Minimum Size, mm (4) i) | – | 10 | 3 ii) | 10 | 20 | 5 iii) | 20 | 32 | 6 iv) | 32 | 50 | 8 of first run 10 for minimum size of weld NOTES 1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld. 2 Where the thicker part is more than 50 mm thick, special precautions like pre-heating should be taken. IS 800 – page 78
Value of constant K for different angle of fusion faces – fillet weld	Table 22 – IS 800 page 78
Design stress of fillet weld	Page 79 – IS 800 $f_{wd} = f_{wn} / \gamma_{mw}$ where $f_{wn} = f_u / \sqrt{3}$, f_u = smaller of the ultimate stress of the weld or of the parent metal, and γ_{mw} = partial safety factor (*see* Table 5).
Design strength of fillet weld (P)	$f_a \text{ or } q = \dfrac{P}{t_t\, l_w}$ (page 80, IS 800) $P_{wd} = l_w t_t f_{wd}$ where, l_w = length of weld t_t = throat thickness = ks k=table 22 of code s=size of weld – table 21 of code

Types of groove weld joint – (a) Square groove weld, one side (b) single bevel groove weld (c) single V-groove weld (d) single U-groove weld (e) single J-groove weld (f) double V-groove weld for thicker sections (dashed line shows original part of edge)	
Design strength of butt weld in tension or compression	$T_{dw} = \frac{f_y l_w t_e}{\gamma_{mw}}$ where, t_e = effective throat thickness f_y = smaller of yield stress of the weld (f_{yw}) and the parent metal (f_y)
Design strength of butt weld in shear (governed by yielding)	$V_{dw} = \frac{f_{yw1} l_w t_e}{\gamma_{mw}}$ where, f_{yw1} = smaller of shear stress of the weld ($f_{yw}/\sqrt{3}$) and the parent metal ($f_y/\sqrt{3}$)
t_e	i. Single V-groove joint : greater than 5/8 times of 't' (here, t is the lowest thickness of the main plates) ii. Double V-groove joint : thickness of the thinner plate
Plug weld connection	It is provided when the design length of weld cannot be accommodated for the plates to be connected (i.e.) when $l_w > b$ b – width of plate
Joint efficiency	(Strength of weld/strength of plate) x 100

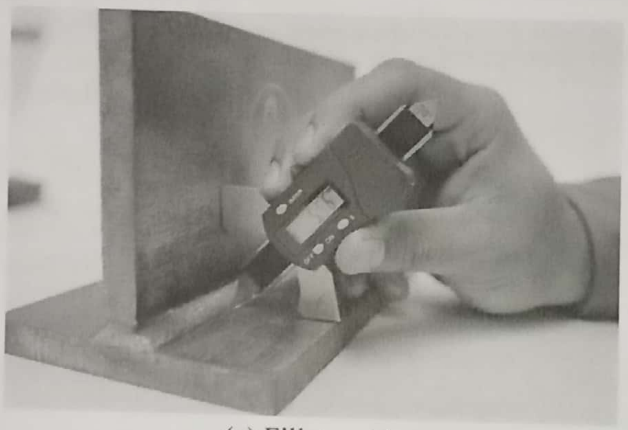


(a)

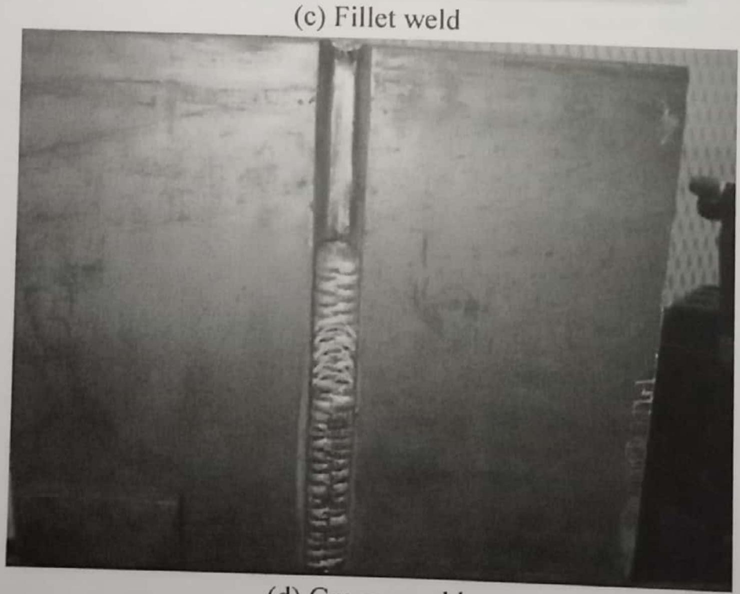
(a) Components in Single V-groove weld



(b) Components in Fillet weld

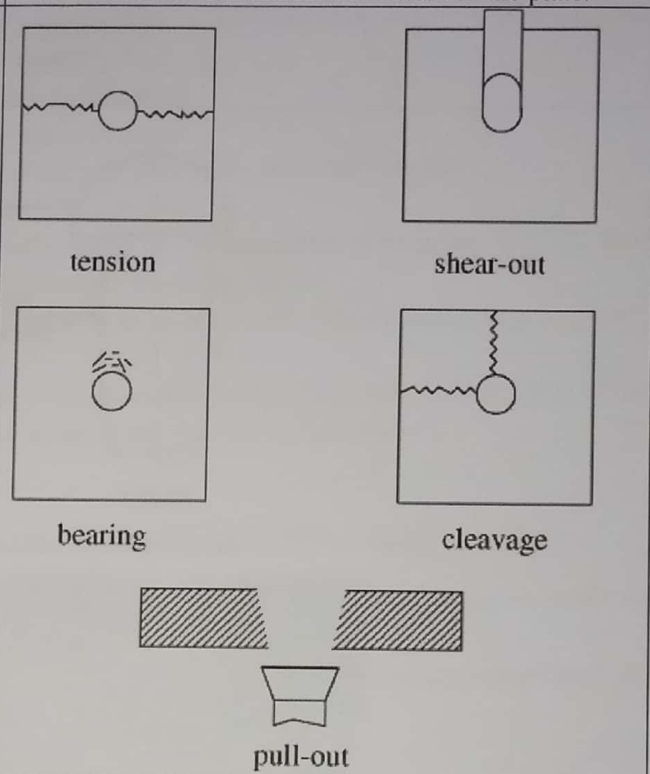


(c) Fillet weld

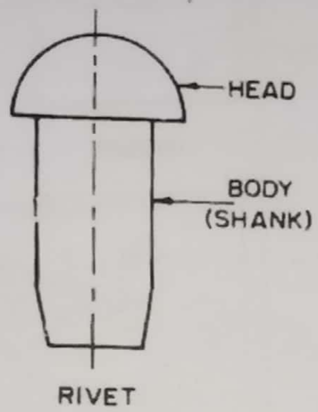


(d) Groove weld

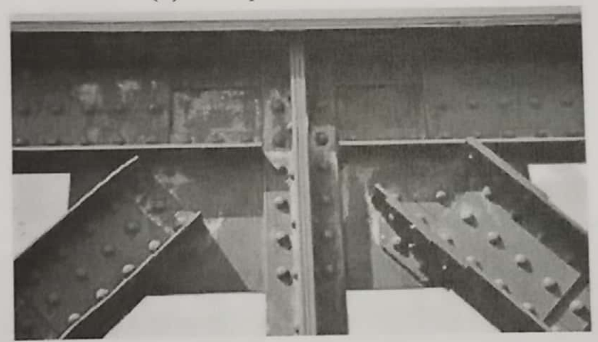
RIVET CONNECTIONS

Failure modes of riveted connections:	i. Tension failure of plates ii. Shearing failure of rivets across one or more planes. iii. Bearing failure between the plates and rivet iv. Plate shear or shear out failure in the plate.
Failure patterns in rivet connections	
Tension capacity of the plates / Tensile strength	$P_t = f_t$ x resisting section $= f_t\,(p - n_1 d)t$ where, f_t = working stress in axial tension $= 0.6f_y$ f_y = yield stress in steel p = pitch distance d = gross diameter of the rivet = nominal diameter of rivet +1.5 (in mm) n_1 = number of rows in which rivets are arranged t = thickness of the plate (least of the plates connected)
Shear capacity of the rivet/Shear strength	$P_s = f_s \times \frac{\pi}{4}d^2$ (for single shear) $P_s = f_s \times 2\frac{\pi}{4}d^2$ (for double shear) where, f_s = stress in shear
Bearing capacity of rivets/Bearing strength	$P_b = ndtf_b$ where, n = number of bolts in one row f_b = working stress in bearing
Strength of solid un-riveted plate	$= ptf_t$
P	same as in the case of bolted connections

<table>
<tr><td>Nominal diameter of the rivet (when thickness of plates to be connected is provided)</td><td>$= 6.04\sqrt{t}$</td></tr>
<tr><td>Joint efficiency</td><td>(Strength of rivet/ Strength of solid un-riveted plate) x 100</td></tr>
<tr><td>Permissible stresses in rivet</td><td>
<table>
<tr><td></td><td>f_t</td><td>f_s</td><td>f_b</td></tr>
<tr><td>Power driven</td><td></td><td></td><td></td></tr>
<tr><td>i. Shop rivet</td><td>100</td><td>100</td><td>300</td></tr>
<tr><td>ii. Field rivet</td><td>90</td><td>90</td><td>270</td></tr>
<tr><td>Hand driven</td><td>80</td><td>80</td><td>250</td></tr>
</table>
</td></tr>
</table>



(a) Components of rivet



(b) Rivet connection

STEPS INVOLVED IN SOLVING PROBLEMS

BOLTED CONNECTION – ANALYSIS:

1. Calculate the shear strength of bolt
2. Calculate the bearing strength of bolt
3. Calculate the tensile strength of plate when bolts are provided
4. Strength of bolt = least of [1], [2] and [3]
5. Calculate the strength of plate.
6. Determine efficiency.

BOLTED CONNECTION – DESIGN:

1. Determine cover plate thickness, if not provided for single cover or double cover butt joint.
2. Calculate the shear strength of bolt. [Apply correction factor for packing plate, if packing plates are provided]
3. Determine pitch and edge distance.
4. Calculate the bearing strength of bolt.
5. Calculate the tensile strength of plate when bolts are provided
6. Strength of bolt = least of [2], [4] and [5]
7. Calculate the tensile force acting on the connection if not given. [NOTE: Sometimes T = capacity of plate = strength of unbolted plate]
8. Determine the number of bolts to be provided = Tensile load acting on plate/Strength of bolt.
9. Arrange the bolts within the given dimension of the plate. [if restriction of the plate dimension is given, i.e width of the plate given in question]
10. Detail with diagram.

BOLTED CONNECTION – DESIGN – ECCENTRIC LOAD:

1. Determine cover plate thickness, if not provided for single cover or double cover butt joint.
2. Calculate the shear strength of bolt. [Apply correction factor for packing plate, if packing plates are provided]
3. Determine pitch and edge distance.
4. Calculate the bearing strength of bolt.
5. Calculate the tensile strength of plate when bolts are provided
6. Strength of bolt = least of [2], [4] and [5]
7. Calculate the moment induced by eccentric load (M)
8. Calculate the number of bolts
9. Determine direct force and radial force
10. Calculate the resultant force, resultant force $<V_{sb}$
11. Detail with diagram.

BOLTED CONNECTION – ANALYSIS – ECCENTRIC LOAD:

1. Determine cover plate thickness, if not provided for single cover or double cover butt joint.
2. Calculate the shear strength of bolt. [Apply correction factor for packing plate, if packing plates are provided]
3. Determine pitch and edge distance.
4. Calculate the bearing strength of bolt.
5. Strength of bolt = least of [2] and [4]
6. Calculate the moment induced by eccentric load (M), keep the eccentric load ‘P’ as unknown.
7. Determine direct force and radial force, keep ‘P’ as unknown.
8. Calculate the resultant force, keep ‘P’ as unknown.

9. Equate resultant force = V_{sb}. Calculate 'P'.
10. Calculate safe load to be carried by the eccentric connection = P/Load factor.
11. Load factor = 1.5

WELDED CONNECTION – ANALYSIS:

1. Calculate the thickness of weld/size of weld.
2. Calculate the length of weld.
3. Calculate the strength of weld [Lowest of tensile strength and shear strength for groove weld, shear strength for fillet weld].
4. Calculate the strength of plate.
5. Determine efficiency.

WELDED CONNECTION – DESIGN:

1. Calculate the thickness of weld/size of weld.
2. Calculate the strength of weld, keeping length of weld as 1mm. [Lowest of tensile strength and shear strength for groove weld, shear strength for fillet weld]
3. Calculate the load acting on plate, if not given in question.
4. Determine the length of weld = Load acting on plate/Strength of weld.
5. Arrange the weld in straight line of as plug welded connection [if l_w>b]
6. Detail with diagram.

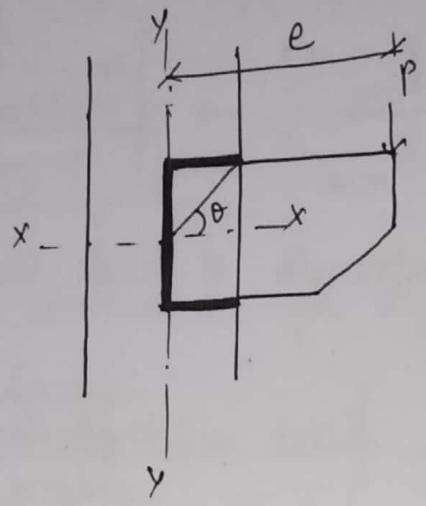
RIVETED CONNECTION – ANALYSIS:

1. Calculate the shear strength of rivet.
2. Calculate the bearing strength of rivet
3. Calculate the tearing strength of plate.
4. Strength of rivet = least of [1], [2] and [3]
5. Calculate the strength of un-riveted plate.
6. Determine efficiency.
7. Detail with diagram.

RIVETED CONNECTION – DESIGN:

1. Determine the nominal diameter and gross diameter of rivet.
2. Calculate the shear strength of rivet.
3. Calculate the bearing strength of rivet
4. Strength of rivet = least of [1] and [2]
5. Determine the number of rivets = Factored load/Strength of rivet
6. Detail with diagram.

ECCENTRIC CONNECTIONS



Direct shear stress in the weld is

$$q_1 = \frac{P}{(2b+d)\,t}$$

where P is the load (eccentrically acting).

d is the depth of the weld

b is the width of the weld

t is the effective throat thickness

The stress in the weld due to twisting moment (caused by eccentric loading) is the maximum in the weld at the extreme distance from the CG of the group of weld and acts perpendicular to the radius vector.

∴ Maximum stress due to moment;

$$q_2 = \frac{P \times e \times r_{max}}{I_{zz}}$$

where, $I_{zz} = I_{xx} + I_{yy}$ → polar moment of inertia (of welded section eccentrically located)

r_{max} = distance of the extreme weld from the CG of the group

∴ Vector sum of the stress is,

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

- For safe design, q should be less than the resistance per unit area.

PROBLEMS

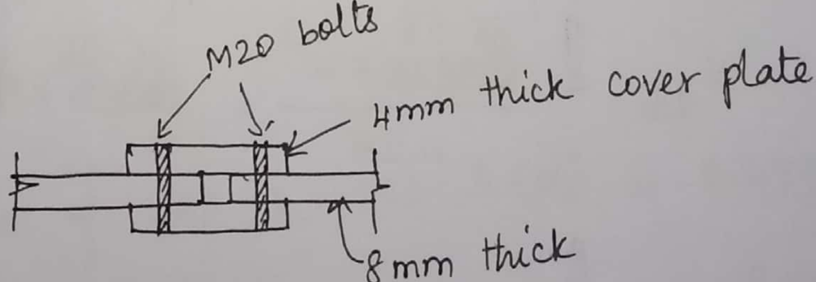
BOLT & RIVET CONNECTIONS

APR/MAY 2017

BOLT-BUTT JOINT

1.1 A single bolted double cover butt joint is used to connect two plates of 8mm thickness. Assume 20mm bolts at 50mm pitch. Calculate the efficiency of the joint. The thickness of cover plate is 4mm.

Soln:-



(M20 ↑ 20mm φ bolts)

Assume grade of bolt as 4.6, - $f_y = 410\ N/mm^2$

Referring table 1 (S.No. iv) of IS 800-2007,

$$f_{ub} = 400\ MPa$$

Step 1:

Area of bolt at throat / Net tensile stress area of bolt $\Big\} = A_{nb} = 0.78\, A_{sb}$ (or refer to table)

$$= 0.78 \times \frac{\pi}{4} \times 20^2$$

$$= 245 \text{ mm}^2$$

Step 2:

Shear strength of one bolt in double shear

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

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

$$= \frac{400}{\sqrt{3}} \left[(1 \times 245) + \left(0 \times \frac{\pi}{4} \times 20^2\right) \right]$$

$$= 56.58 \times 10^3 \text{ N} = 56.58 \text{ kN}$$

$$V_{dsb} = \frac{56.58 \times 10^3}{1.25} = 45.26 \text{ kN} \times 2 = 90.52 \text{ kN}$$

Step 3:

Design strength of one bolt in bearing

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

k_b:-

Minimum edge distance, $e = 1.5 d_0$ [clause 10.2.4.2 & 10.2.4.3 - IS: 800: 2007]

$= 1.5(20) = 30 mm$

d_0 = dia. of bolt + 2mm = $20 + 2 = 22mm$

(a) $\frac{e}{3d_0} = \frac{30}{3(22)} = 0.455$

(b) $\frac{p}{3d_0} - 0.25$

[Min. pitch distance = $2.5 d$ (clause 10.2.2 IS 800: 2007)

$= 2.5(20) = 50 mm$ = given value]

$\therefore \frac{p}{3d_0} - 0.25 = \frac{\overset{\text{given}}{50}}{3(22)} - 0.25$

$= 0.507$

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

(d) 1.0

Least value of (a), (b), (c) & (d) is 0.455

$\therefore k_b = 0.455$

$d = 20mm$

t = th. of main plate or sum of thickness of two cover plates whichever is less

$= 8mm$ or $2(4) = 8mm$

$\therefore t = 8mm$

$$V_{npb} = 2.5 \times 0.455 \times 20 \times 8 \times 410 = 74620 \text{ N}$$

$$= 74.62 \text{ kN}$$

Design strength in bearing

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{74.62}{1.25} = 59.70 \text{ kN}$$

~~Bolt strength = 59.70 kN (least of step 2 & 3)~~

Step 4:

Strength of main plate in tearing / strength of net section per pitch length

$$= 0.9 A_n \frac{f_u}{\gamma_{ml}}$$

$$= 0.9 (p-d) t \frac{f_u}{\gamma_{ml}} = 0.9 (50-20) \times 8 \times \frac{410}{1.25}$$

$$= 70848 \text{ N} \approx 70.85 \text{ kN}$$

Strength of joint per pitch length

$= 59.70 kN$ (least of step 2, 3 & 4)

<u>Step 5:</u>

Strength of solid plate per pitch length

$$= \frac{0.9\, f_u\, A}{\gamma_{ml}} = \frac{0.9 \times 410 \times 50 \times 8}{1.25}$$

$A = (p \times t)$ of plate

$= 118080 N = 118.08 kN$

<u>Step 6:</u>

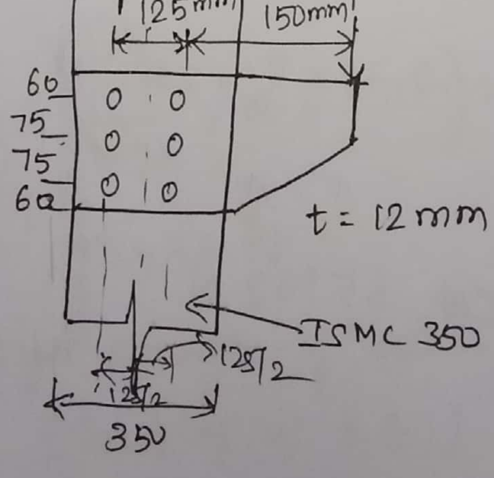
$$\text{Joint efficiency, } \eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$$

$$= \frac{59.70}{118.08} \times 100 = 50.55\%$$

NOV/DEC 2016

ECCENTRIC CONNECTIONS - BOLT

1.2 A bracket connection is shown in figure below with 24mm diameter bolts of grade 4.6 and plate of grade Fe 410 steel, determine the safe load (P) that could be transferred through the connection



Soln: Step 1: ~~G.~~ Material properties:

For ~~Bolt~~ M24 Bolts of grade 4.6,

$d = 24mm, \quad d_0 = 24+2 = 26\,mm$

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

For rolled steel sections, $f_u = 410\ N/mm^2$

Assume web section as, ISMC 350,

Thickness of web = 8.1 mm (refer steel table)

Step 2: Design strength:

∵ this is a lap joint between bracket plate and web of ISMC 300, the bolts are in single shear,

(a) ∴ Design strength of bolts in shear

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{f_{ub}}{\sqrt{3}}\left(\underbrace{n_n}_{=1} A_{nb} + n_s \underbrace{A_{sb}}_{0.78\,A_{sb}}\right)}{\gamma_{mb}}$$

$$= \left[\frac{400}{\sqrt{3}}\left(0.78 \times \frac{\pi}{4} \times 24^2\right)\right] / 1.25$$

$= 65192\ N$

(b) Strength in bearing against 8.1 mm web of ISMC 350.

k_b is least of

(i) $$\frac{e}{3d_0} = \frac{\left(\frac{350}{2} - \frac{125}{2}\right)}{3 \times 26} = 1.442$$

(ii) $$\frac{p}{3d_0} - 0.25 \; \cancel{\frac{75}{3\times 26}} = \frac{75}{3\times 26} - 0.25 = 0.712$$

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

(iv) 1.0

$\therefore \; k_b = 0.712$

$\therefore$ Design strength of bolt in bearing

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

$$= \frac{1}{1.25} \times 2.5 \times 0.712 \times 24 \times 8.1 \times 410$$

$$= 113498 \text{ N}$$

$\therefore$ Design strength of bolt = 65192 N

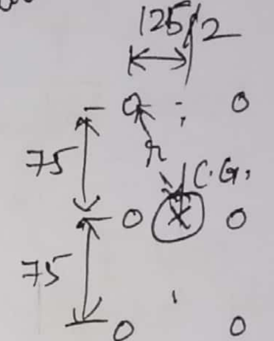
[least of (a) & (b)]

1 **Step 3: Shear force on bolt:**

Direct shear force $F_1 = \dfrac{\text{Total vertical load}}{\text{Number of bolts}}$

$$F_1 = \frac{P}{6} = 0.167\,P$$

Force due to moment (F_2). F_2 acts at rt. angles to the radial line



C.G. of the bolted connection is at centre

The 'r' value of corner 4 bolts

$$= \sqrt{75^2 + \left(\frac{125}{2}\right)^2} = 97.6\text{ mm}$$

The 'r' value of centre 2 bolts

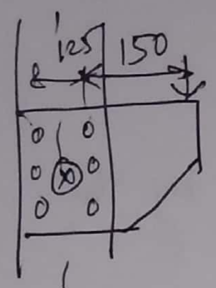
$$= \frac{125}{2} = 62.5\text{ mm}$$

$$\Sigma r^2 = \left(4 \times 97.6^2\right) + \left(2 \times 62.5^2\right) = 45915.54\text{ mm}^2$$

no. of bolts   resp. 'r' value

$\Rightarrow$ approx $\Sigma r^2 = 45915\text{ mm}^2$

Force due to moment in extreme bolt



$e = 150 + \frac{125}{2}$

$= 212.5$ mm

load eccentricity from centre of gravit

$$= \frac{Mr}{\Sigma r^2} = \frac{(Pe)\, r}{\Sigma r^2}$$

(extreme bolt alone)

[rotation possible only due to corner bolts]

$$= \frac{P \times 212.5 \times 97.6}{45915}$$

$$= 0.452\, P.$$

Angle between the two forces F_1 and F_2 is given by

$$\tan\theta = \frac{75}{\underset{(125/2)}{62.5}} = 1.2$$

$$\theta = 50.19^\circ$$

Total shear force on the extreme bolt is given by

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

$$= \sqrt{(0.167\,P)^2 + (0.452P)^2 + 2(0.167P)(0.452P)\cos 50.19}$$

$$= P\,(0.573)$$

Step 4: Safe load (P)

Equating total shear force & strength of bolt

$\Rightarrow \quad 0.573\,P = 65192$

$\Rightarrow P = 113773 \text{ N} = 113.773 \text{ kN}$

Assume ~~factor~~ load factor of 1.5.

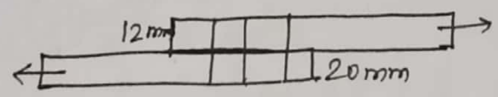
$\therefore$ Safe load that could be carried by the

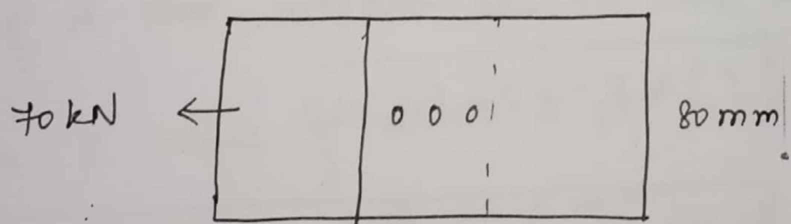
bracket $= \dfrac{113.773}{1.5} = 75.849$ kN.

BASIC PROBLEMS

1.3 Design a lap joint between two plates as shown in figure so as to transmit a factored load of 70 kN using M16 bolts of grade 4.6 and grade 410 plates

Bolt





Soln: Step 1:

Nominal diameter of the bolt (d) = 16 mm

diameter of hole = d_0 = 16 + 2 = 18 mm

Step 2:

Bolts are in single shear and hence shear capacity of the bolt;

$$V_{dsb} = \frac{(f_u/\sqrt{3})(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}}$$

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

$$= 28974\ N = 28.97\ kN \approx 29\ kN$$

Bearing capacity of the thinner plates

$$= 2.5\ k_b\ d t\ f_u/\gamma_{mb}$$

($e = 1.5 \times 16 = 24$ say 30 mm)

k_b is smaller of

$$\frac{e}{3d_0}, \quad \frac{p}{3d_0} - 0.25, \quad \frac{f_{ub}}{f_u}, \quad 1.$$

$e \leftarrow 1.5d$, $p \leftarrow 2.5d$

$$= \frac{30}{3 \times 18}, \quad \frac{2.5 \times 16}{3 \times 18} - 0.25, \quad \frac{400}{410}, \quad 1$$

$$= 0.555, \ 0.491, \ 0.975, \ 1$$

$\therefore\ k_b = 0.491$

least thickness of plate

$$\text{Bearing capacity} = V_{dpb} = \frac{2.5 \times 0.491 \times 16 \times 12 \times 410}{1.25}$$

$$= 77303\ N = 77.3\ kN$$

$\therefore$ Bolt value = 29 kN
(Bolt strength)

Step 3:

Required number of bolts $= \dfrac{\text{Factored load}}{\text{Bolt strength}}$

$$= \frac{70}{29} = 2.41 \approx 3 \text{ bolts}$$

Step 4:

Minimum pitch $= 2.5\,d = 2.5 \times 16 = 40\,mm$

Minimum edge distance

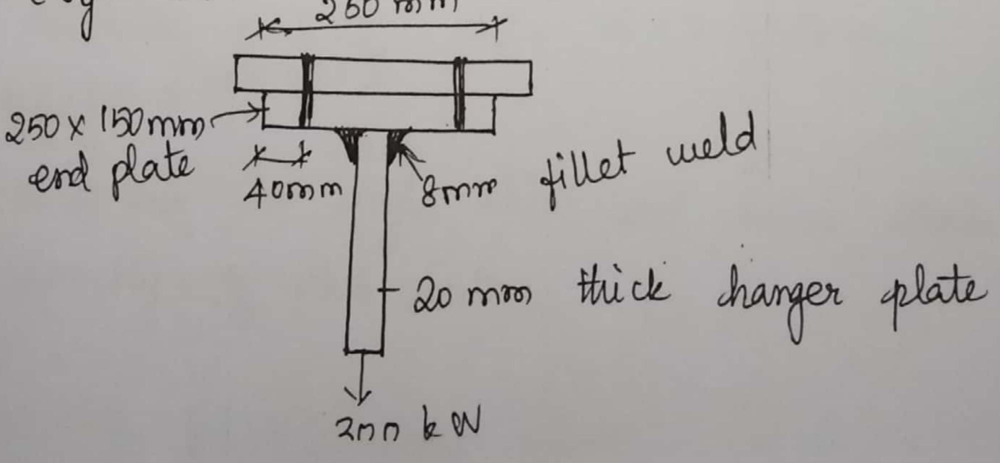
~~largest thickness~~

$= 1.5 \times$ ~~thickness of plate~~ d_0

$= 1.5 \times \cancel{20}\ 18 = 27\,mm \approx 30\,mm$ ~~30 mm~~

Bolt + Weld

Q. 1.4 Design a hanger joint as shown in figure to carry a factored load of 300 kN. Use an end plate of size 250 mm × 150 mm and appropriate thickness M24 HSFG bolts (2nos) and Fe 410 steel for end plate (f_y = 250 MPa).



Soln: Step 1:

Distance from the centre line of the bolt to the toe of the weld,

$$l_v = \left(\frac{250}{2} - \frac{\overset{\text{hanger the}}{20}}{2} - 8 - 40\right) = 67\,mm$$

Step 2:

For minimum thickness design,

$$M = \frac{\overset{\text{factored load}/2}{T}\,l_v}{2} = \left(\frac{300}{2}\right) \times \frac{67}{2} = 5025\,kN\text{-}m$$

$$t_{min} = \sqrt{\frac{4.4\,M_p \rightarrow \text{in N-m}}{f_y\,b_e}}$$

b_e ← mm

f_y ← N/mm²

$$= \sqrt{\frac{4.4 \times 5025 \times 10^3}{250 \times 150}} = 24.56\,mm \text{ say } 25\,mm$$

Step 3:

Check for prying forces,

Distance l_e from the centre line of the bolt to the point where prying force acts is the minimum of edge distance (40 mm) ← given

or $$L_e = 1.1\,t\,\sqrt{\frac{\beta f_o}{f_y}}$$ f_o ← proof stress

$$= 1.1 \times 25\,\sqrt{\frac{2 \times 197}{250}} = 34.5 \approx 35\,mm$$

$[f_0$ = proof stress = 197 N/mm² for M24 bolt$]$

$$\text{Prying force} = \frac{M}{l_e} = \frac{5025}{35} = 143.6 \text{ kN}$$

factored load

$$\text{Bolt load} = \frac{300}{2} + 143.6 = 293.6 \simeq 293 \text{ kN}$$

Tension capacity of M24 bolt

$$T_{nd} = \frac{0.9 f_u A_{nb}}{1.25} = \frac{\left(0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 24^2\right)}{1.25}$$

$$= 203.249 \simeq 203.3 \text{ kN} < 293 \text{ kN}$$

Bolt load

~~Assuming friction co-efficient of 0.55 (as per~~

{Refer table 1} ~~Assuming~~ f_{ub} = 800 MPa, f_{yb} = 640 MPa for HSFG bolts (assuming grade HSFG 8.8 bolts)}

∴ Joint is unsafe.

Step 4:

To reduce prying force and hence bolt load, use a thicker plate. Assuming a 40 mm thick plate

$$l_e = 1.1 \times 40 \sqrt{\frac{2 \times 197}{250}} = 55.2 \text{ mm}$$

Use $l_e = 40$ mm (edge distance)

Allowing prying force

$$\boxed{Q = T_{nd} - \left(\frac{\text{factored load}}{2}\right)}$$

factored tension

$= 203.3 - \left(\frac{300}{2}\right) = 53.3$ kN

$$\boxed{\text{Moment at the toe of the weld} = T l_v - Q l_e}$$

$= \left(\frac{300}{2}\right) \times 67 - (53.3 \times 40)$

$= 7918$ N-m

Moment capacity $= \frac{f_y}{1.10} \frac{b_e t^2}{4}$

$= \left(\frac{250}{1.10} \times \frac{150 \times 40^2}{4}\right) \times 10^{-3}$ mm to m

$= 13636$ N-m > 7.918 N-m

Hence the joint is safe.

Step 5:

Proof stress $= 0.7 \times 800 = 560$ MPa $= 0.56$ kN/mm^2

Prying force $\boxed{Q = \frac{l_v}{2 l_e}\left[T_e - \frac{\beta \eta f_o b_e t^4}{27 l_e l_v^2}\right]}$

clause 10.4.7 - IS 800 : 2007

pg 77

end plate width = 150 mm given

[NOTE: b_e = effective width of flange per pair of bolts

β = 2 for non pre-tensioned bolt and

1 for pre-tensioned bolt

η = 1.5

b_e = effective width of flange per pair of bolts

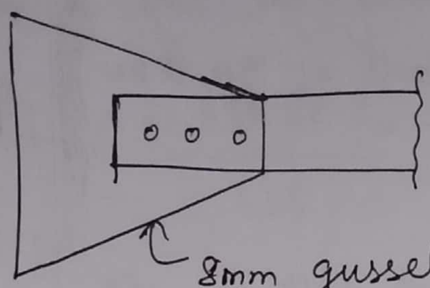
T_e = effective tension load = T.]

$$\therefore Q = \frac{67}{2\times 40}\left[150 - \frac{2\times 1.5\times 0.560\times 150\times 40^4}{27\times 40\times 67^2}\right]$$

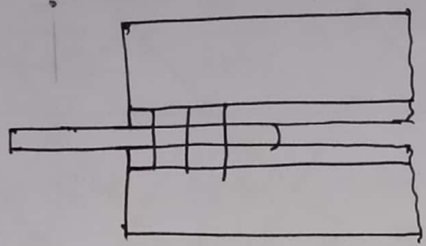
$= 14.24$ kN < 53.3 kN

$\therefore$ The joint is safe.

1.5 A member of a truss consists of two angles ISA 75×75×6 placed back to back. It carries an ultimate tensile load of 150 kN and is connected to a gusset plate 8mm thick placed inbetween the two connected legs. Determine the number of 16mm ϕ 4.6 grade ordinary bolts required for the joint. Assume f_u of plate as 410 MPa.



8mm gusset plate



refer probl.3 for cal

Soln: **Step 1:**

The bolts are in double shear, $V_{dsb} = 29\ kN$

(a) strength in double shear for 16 mm ϕ 4.6 grade bolts $= 2 \times 29 = 58\ kN$

(b) Strength in bearing on 8 mm plate

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

[$k_b \rightarrow$ from problem 1.3]

$\hookrightarrow k_b = 0.491 \approx 0.49$ (approx)

$$= \frac{2.5 \times 0.49 \times 16 \times 8 \times 400}{1.25}$$

$$= 50176 \approx 50.2\ kN$$

(c) Strength ~~of bolt~~ in bearing on two 6mm thick angles

$$= \frac{2.5 \times \cancel{400}\ 0.49 \times 16 \times (2 \times 6) \times 400}{1.25 \times 10^3} = 75.2\ kN$$

N to kN

$\therefore$ Strength of bolt = 50.2 kN

Step 2:

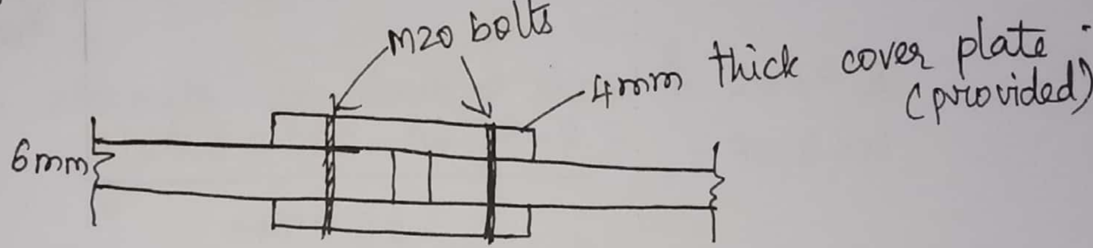
Required number of bolts = $\frac{150}{50.2}$ = $2.98 \approx 3$ bolts

---

~~Step 3~~:

NOTE: $\text{Efficiency} = \dfrac{\text{strength of joint per pitch length}}{\text{Strength of solid plate per pitch length}} \times 100$

1.7 A single-bolted double-cover butt joint is used to connect two plates 6-mm thick. Assuming the bolts of 20mm dia. at 60mm pitch, Calculate the efficiency of the joint. Use 410 MPa plates and 4.6 grade bolts.



Soln: Step 1:

Strength of a 20mm dia. bolt in double shear

$$= 45.3 \times 2 = 90.6 \text{ kN}$$

Thickness of cover plate $> \frac{5t}{8} = \frac{5 \times 6}{8} = 3.75\text{mm}$

th. of plate

∴ Provide 4mm thick cover plate

The thickness to be considered in the calculations are the thickness of the main plate or the sum of the cover plates whichever is less.

$\therefore$ Strength of bolt in bearing

$$= 2.5 k_b \, dt \, f_u/\gamma_m \qquad [\text{Take } k_b = 1.00]$$

$$= \frac{(2.5 \times 1 \times 20 \times 6 \times 400)}{(1.25 \times 1000)}$$

$$= 96 \text{ kN}$$

Strength of the plate in tearing $= \dfrac{0.9 A f_u}{\gamma_{ml}}$

$$= \frac{0.9 (60 - 22) \times 6 \times 410}{1.25 \times 1000}$$

$$= 67.3 \text{ kN}$$

$\therefore$ Strength of the joint $= 67.3$ kN (minimum of the three).

Step 2:

Strength of the solid plate per pitch length

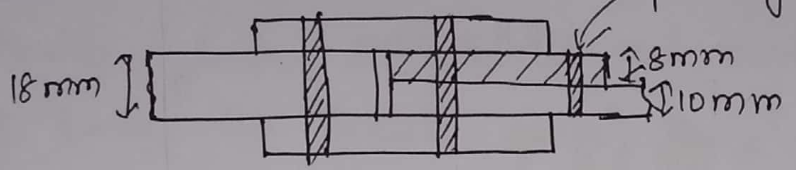
$$= \frac{0.9 \times 60 \times 6 \times 410}{(1.25 \times 1000)} = 106.3 \text{ kN}$$

Step 3:

Joint efficiency $= \dfrac{67.3}{106.3} \times 100 = 63.3\%$

1.8 Two plates 10mm and 18mm thick are to be jointed by a double-cover butt joint. Assuming cover pieces of 8mm thickness, design the joint to transmit a factored load of 500 kN. Assume Fe 410 plate and grade 4.6 bolt.

Soln.



Assume 20mm dia. bolt.

Step 1:

Bolt strength in double shear $= 2 \times 45.3$

$= 90.6$ kN

Required number of bolts $= \frac{500}{90.6} = 5.5 \approx 6$ bolts

Provide 6 bolts, three in a row

Since the joint has a packing plate greater than 6mm, the bolt strength has to be reduced by

$$\beta_{pkg} = (1 - 0.0125\, t_{pkg})$$

$$= (1 - 0.0125 \times 8) = 0.9$$

$\therefore$ Bolt strength in shear $= 0.9 \times 90.6$

$= 81.54$ kN

Strength of bolt in bearing

$$= \frac{2.5 \times \overset{k_b}{1} \times \overset{d}{20} \times \overset{t}{10} \times 400}{1.25 \times 1000}$$

$$= 160 \text{ kN}$$

$\therefore$ Required number of bolts $= \frac{500}{81.54} = 6.13$

Provide 8 bolts in two rows (four in a row)

Step 2:

Strength of bolt / pitch length $= 2 \times 81.54$

$= 163.08 \text{ kN}$

Strength of bolt / pitch length of the plate in tearing

$$\Rightarrow 163.08 = \frac{0.9\,(p - \overset{d_0 = d + 2\,mm}{22}) \times 10 \times 410}{1.25 \times 1000}$$

$\Rightarrow$ $p = 77.24\,mm > 2.5d = 50\,mm$

$\therefore$ provide pitch $= p = 75\,mm$.

Provide additional two bolts of 20mm dia on the packing plate.

Design a butt joint to connect two plates 175 x 10mm (Fe 410 grade) using M20 bolts. Arrange the bolts to give maximum efficiency.

<u>Soln.</u> Thickness of the cover plate = $\frac{5}{8} \times 10 = 6.25$ mm

Provide double cover plates each having a thickness of 8mm. The tensile force the main plate can carry (assuming two bolts in a line)

$$= \frac{0.9\left[\overset{\text{width}}{175} - \overset{2d_0}{2(22)}\right] \times \overset{t}{10} \times 410}{1.25 \times 1000}$$

$= 386.7$ kN

Shear strength of the bolt $= 2.5\,d\,t\,f_u/\gamma_m$

$[k_b = 1.00]$

$$= \frac{2.5 \times 20 \times 10 \times 400}{1.25 \times 1000}$$

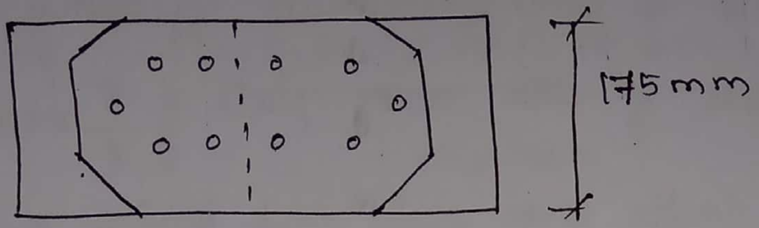
$= 160$ kN

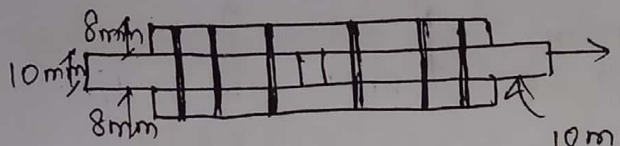
Strength of bolt in shear $= 2 \times V_{dsb} = 2 \times 45.3 = 90.6$ kN

$\therefore$ strength of bolt = 90.6 kN (least of three)

Required number of bolts $= \frac{386.7}{90.6} = 4.3$

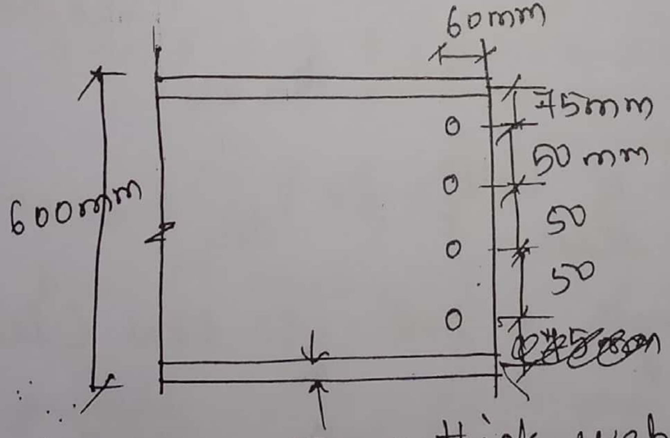
$\therefore$ Provide 5 bolts of 20mm diameter





1.10 An ISMB 600 is connected to a column by web cleats with a single row of bolts. If the reaction is 350 kN and there are four 20mm dia. bolts through the web. check if the section is adequate for block shear failure.

Soln.



~~Step1~~:

NOTE: Block Shear Capacity

$$T_{db_1} = \frac{A_{vg} f_v}{\sqrt{3} \times 1.1} + \frac{0.9 f_u A_{tn}}{1.251}$$

$$T_{db_2} = \frac{0.9 f_u A_{vn}}{\sqrt{3} \times 1.25} + \frac{f_y A_{tg}}{1.1}$$

Step 1:

Design strength of web = 350 N/mm^2

Plate or web tk = 12 mm

Net length of shear face = $[(\underbrace{3 \times 50}_{3p}) + 75] - (\underbrace{3.5 \times 22}_{3.5 d_o})$

= 148mm

Net length of the tension face = $[60 - 22/2]$

[distance from edge $- \frac{d_o}{2}$]

= 49mm

(a) A_{vg} = t (3x no. c/c dist. b/w bolts + dist. of 1st bolt from edge)

$= 12 \times [3 \times 50 + 75] = 2700 \text{ mm}^2$

(b) A_{vn} = t x net shear length

$= 12 \times 148 = 1776 \text{ mm}^2$

(c) A_{tg} = t x dist. of bolt from edge

$= 12 \times 60 = 720 \text{ mm}^2$

(d) A_{tn} = t × net length of tension face

$= 12 \times 49 = 588 \text{ mm}^2$

Step 2:

$$T_{db1} = \left[\frac{2700 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410 \times 588}{1.25}\right] \times 10^{-3}$$

$= 527.86$ kN

$$T_{db2} = \left[\frac{0.9 \times 410 \times 1776}{\sqrt{3} \times 1.25} + \frac{250 \times 720}{1.1}\right] \times 10^{-3}$$

$= 466.32$ kN

$\therefore T_{db} = 466.32$ kN (lowest of two)

The value of T_{db} is much higher than the applied reaction of 350 kN and hence there will not be any block shear failure in this case.

STIFFENED SEATED CONNECTION:

1.11) Design a seat angle connection between a beam MB 300 and column SC 200 for a reaction of a beam 100 kN. Using M20 bolts of property class 4.6. Take Fe 410 grade steel ($f_y = 250$ MPa).

Soln:

Step 1: Thickness of column flange ISSC 200

$= 15\,mm$

Strength in single shear (from ~~table~~ prev. prob)

$= 45.3\,kN$

$$\text{Strength in bearing} = \frac{2.5 \times \overset{k_b}{1} \times \overset{d}{20} \times \overset{t}{15} \times \overset{f_u}{400}}{1.25 \times 1000}$$

$= 240\,kN$

$$\text{Required number of bolts} = \frac{100}{45.3} = 2.21$$

Provide 4 bolts

Step 2:

Width of seat angle is 140 mm

Hence length of angle = 140 mm

Length of bearing required at root line of beam

$$\frac{R}{t_w\left(\frac{f_{yw}}{\gamma_{m0}}\right)}$$

R ← reaction / force

$$= \frac{100 \times 10^3}{7.7\left(\frac{250}{1.1}\right)} = 57.15\,mm$$

10^3 ← kN to N

tk. of ISMB300

Assume end clearance of beam from face of column as 5mm and tolerance of 5mm (= 10mm total)

Required length of outstanding leg

= length of bearing + 10mm

= 57.15 + 10 = 67.15mm < 75mm

length of leg adopted = 150 x 75 x 12

Length of bearing on cleat = b_1 = 57.15 − (T + r)

= 57.15 − (13.1 + 14)

t_f ISMB 300 adopted | r of ISMB 300

= 30.05 mm

For 150 x 75 x 12 angles, distance from the end of bearing on cleat to root angle

$b_2 = b_1 + 10 - (t + r_a)$ of angle

10: clearance + tolerance

= 30.05 + 10 − (12 + 10)

= 18.05mm.

Step 3: Assume UDL over bearing length b_1,

Moment at root of angle (point B) due to load to right of B

$$= \frac{\overset{R}{100} \times \overset{b_2}{18.05}}{\underset{b_1}{30.05}} \times \frac{\overset{b_2}{18.05}}{2}$$

$= 542$ N-m

Moment capacity $= 1.2\, Z \dfrac{f_y}{\gamma_{mo}}$

$$= \left(1.2 \times \underbrace{\frac{\overset{\text{(beam width)}}{140} \times 12^2}{6}}_{\frac{bd^2}{6}} \times \frac{250}{1.1}\right) \times 10^{-3}$$

$= 916$ N-m > 542 N-m

∴ Connection is safe

∴ Provide $150 \times 75 \times 12$ mm seating angle.

Step 4:

Shear capacity of the outstanding leg of cleat

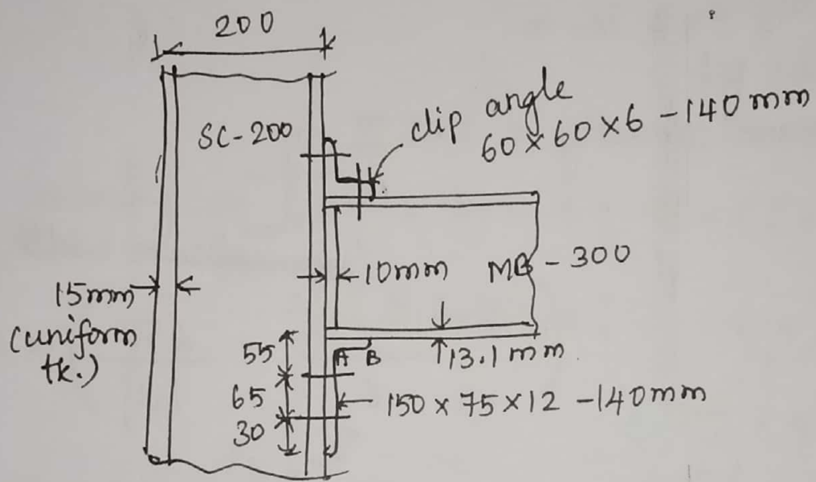
$$= \frac{w t f_y}{\sqrt{3} \times 1.1} = \frac{140 \times 12 \times 250}{\sqrt{3} \times 1.1 \times 1000} = 220 \text{ kN} > \underset{\text{force}}{100 \text{ kN}}$$

Shear strength of beam

$$V_d = \frac{A_v f_{yw}}{\sqrt{3} \times 1.10} = \frac{(300 \times 7.5) \times 250}{(\sqrt{3} \times 1.10 \times 1000)}$$

$$= 303 \text{ kN} > 100 \text{ kN}$$

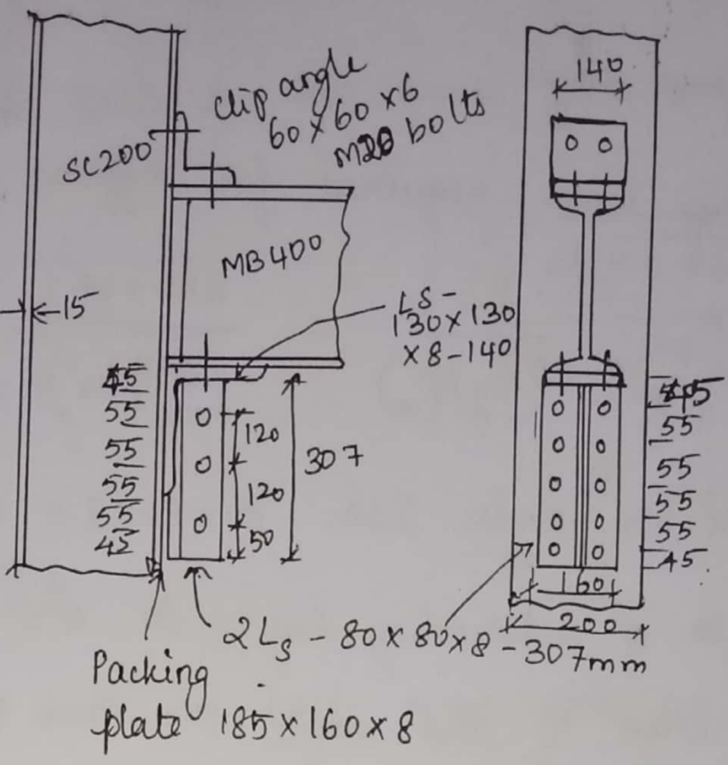
Hence web does not need any stiffener at support.



Stiffened seated connection

1.12 Design a stiffened seat angle for a reaction of 250 kN from a beam ISMB400 using M20 bolts of grade 4.6. The beam has to be connected to ISSC 200 column. Assume Fe 410 grade steel (f_y = 250 MPa)

Soln ..



Step 1:

Length of bearing required at root line of beam

$$= \frac{R}{\left(\frac{t_w f_{yw}}{\gamma_{mo}}\right)} = \frac{250 \times 1000}{\left(8.9 \times \frac{250}{1.1}\right)} = 106 \text{ mm}$$

Assuming clearance including tolerance of 10mm

Required length of the outstanding leg

$= 106 + 10 = 116$ mm

Width of beam = 140mm

Provide a seat angle of 130 x 130 x 8mm of length 140mm connected to the beam by two M20 bolts

Step 2:

Stiffener angle,

Bearing area required for stiffener angle,

$$A_{br} = \frac{R}{(f_y/\gamma_m)} = \frac{250 \times 10^3}{(250/1.1)} \approx 1100\ mm^2$$

select two angles ISA $80 \times 80 \times 8 = 1280\ mm^2$

Length of outstanding leg $= 80 - \underset{t}{8} = 72\ mm$

Thickness of the angle $t = 8mm$

$$\frac{B}{t} = \frac{72}{8} = 9 < 14$$

Hence the thickness is fine

(Clause 8.7.12 of IS code)

Distance of end reaction from column flange

$$e_x = \frac{\overset{\text{seat angle width}}{130}}{2} = 65mm$$

Provide 20mm dia. grade 4.6 bolts at a pitch of 55mm.

Step 3 (a) Strength of bolt in single shear = 45.3 kN (prob 1.6)

(b) Strength of bolt in bearing

$$= \frac{2.5 \times 1 \times 20 \times 8 \times 400}{(1.25 \times 1000)} = 128\ kN$$

$\therefore$ Strength of bolt $= 45.3\,kN = V_{df}$

Let us provide bolts in two rows with a pitch of 55 mm.

$$n = \sqrt{\frac{6M}{p\,n'\,V_{db}}} = \sqrt{\frac{(6 \times \overbrace{250 \times 65}^{R \times e_x})}{55 \times (\underbrace{2}_{n'} \times \underbrace{45.3}_{V_{db}})}}$$

$= 4.42$

Provide five bolts in each row with an edge distance of 45 mm

Step 4: Depth of stiffener angle $= (4 \times 55) + (2 \times 45)$

$= 310\,mm$

$h = 310 - 45 = 265\,mm$

The neutral axis lies at $\frac{h}{7} = \frac{265}{7} = 37.86\,mm$

Shear force in each bolt $= \frac{250}{10} = 25\,kN = V_{sf}$

The critical bolt will be at the top of the connection;

$$\Sigma y = 2\Big[(45 - \underbrace{37.86}_{h/7}) + (\underbrace{100}_{45+55} - 37.86) + (\underbrace{155}_{45+55+55} - 37.86) + (\underbrace{210}_{45+55+55+55} - 37.86) + (\underbrace{265}_{45+55+55+55+55} - 37.86)\Big]$$

$= 2\,[7.14 + 62.14 + 117.14 + 172.14 + 227.14]$

$= 1171.11\,mm$

$$\Sigma y^2 = 2\left[7.14^2 + 62.14^2 + 117.14^2 + 172.14^2 + 227.14^2\right]$$

$$= 197717.8 \text{ mm}^2$$

$$M' = \frac{M}{1+\left(\frac{2h}{21}\right)\left(\frac{\Sigma y}{\Sigma y^2}\right)} = \frac{\overset{R}{250 \times 10^3} \times \overset{e_x}{65}}{1+\left(\frac{2\times 265}{21}\right)\left(\frac{1171.4}{197717.8}\right)}$$

$$= 14136 \times 10^3 \text{ N-mm}$$

Tensile force $= \dfrac{M' y_n}{\Sigma y^2}$ ← last bolt 'y'

$$= \frac{14136 \times 10^3 \times 227.14}{197717.8} = 16240 \text{ N}$$

$$= \underline{\underline{16.24 \text{ kN} = T_f}}$$

Tensile strength of 20 mm bolt

$$= \frac{0.90 f_{ub} A_n}{\gamma_{mb}} < \frac{f_{yb} A_{sb}}{\gamma_{mo}}$$

$$= \left[0.90 \times 400 \times \left(0.78 \times \frac{\pi}{4} \times 20^2\right)\right] / (1.25 \times 10^3)$$

$$< \left[240 \times \frac{\pi}{4} \times 20^2\right] / (1.1 \times 10^3)$$

$$= 70.572 \text{ kN} < \underline{\underline{68.54 \text{ kN}}}$$

$\therefore$ Tensile strength of bolt = $\underline{\underline{68.54\ kN}} = T_{df}$

As per clause 10.4.6 IS:800-2007

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

$$\Rightarrow \left(\frac{25}{45.3}\right)^2 + \left(\frac{16.24}{68.54}\right)^2 = 0.36 < 1.0$$

Hence the connection is safe.

Provide a top ~~clip~~ clip angle of $60 \times 60 \times 6\ mm$ ~~width~~ with 4 - M16 bolts and a packing plate of size $188 \times 160 \times 8\ mm$.

UNIT - I
2 Marks

1 Mention the advantages and disadvantages of Steel Structures

Advantages

→ Due to high strength of steel per unit quantity, small sized steel structural elements are sufficient.

→ Speedy Construction is possible.

→ It can be strengthened at any later period, if necessary.

→ By using bolted connection, the steel structures are easily dismantled and transported to other sites quickly.

→ Material used for steel structures are reusable.

Disadvantages

→ The Components are easily susceptible to corrosion.

→ Steel members are costly compared to RCC structures.

→ Maintenance cost is high, since it need painting to prevent corrosion.

→ At high temperature steel loses most of its strength, leading to deformation or failure.

2 Define the limit states [AU May 2010]

The limit state design (LSM) aims for a comprehensive and rational solution to the design problem by considering safety at ultimate loads and serviceability at working loads. It uses multiple safety factor formats to provide safety. The limit states relevant for structural steel design can be grouped into two types.

Ultimate (Safety) Limit States
Serviceability Limit States

3. What are the load combinations for the design purposes?

→ Dead load + Imposed Load

→ Dead Load + Imposed Load + wind Load or earthquake load

→ Dead Load + wind Load or Earthquake load

4. Which type of Steel is most Commonly used in general Construction? Why?

Mild Steel is most Commonly used in general Construction because of its durability and malleability.

5. How the rolled Steel Sections are classified? (or) Classify the Structures based on Shape and geometry. [AU MAY 2013]

-> Indian Standard Rolled Steel Joist (or) I Section
-> Indian Standard Rolled Steel Channel Section
-> Indian Standard Rolled Steel Angle Section
-> Indian Standard Rolled Steel T Section
-> Flat Plates
-> Strips
-> Round Bars
-> Hollow Tubes

6. Define characteristic Actions

The characteristic action or characteristic load may be defined as the load which will not exceeded by a Certain accepted or pre assigned probaility (usually 5%) during the life of the Structure. Using the normal distribution the characteristic load can be expressed as

$$Q_c = Q_m + 1.65\sigma$$

7 Define Partial Safty Factor

The factor used to calculate the design strength or resistance from the characteristic strength is known as partial Safety factor. This factor is divded into two (i) for materials and for loads

8. What is working stress method (WSM)?

The stresses caused by the characteristic loads are checked against the permissible (allowable) stresses, which is a fraction of yield stress. The permissible stresses are defined by means of a factor of safety, which takes care of overload or other unknown factors.

$$\text{Permissible stress} = \frac{\text{Yield stress}}{\text{FOS}}$$

Hence in working stress method

Working stress $\leq$ Permissible stress

9. Methods of Structural Analysis

→ Elastic Analysis
→ Plastic Analysis
→ Advanced Analysis
→ Dynamic Analysis

10 Mention the Physical properties of structural steel

→ Unit mass of steel, $\rho = 7850\ kg/m^3$
→ Modulus of Elasticity, $E = 2\times10^5\ N/mm^2$
→ Poisson Ratio, $\mu = 0.3$
→ Modulus of Rigidity, $G = 0.769\times10^5\ N/mm^2$
→ Coefficient of thermal expansion, $\alpha = 12\times10^{-6}/^\circ C$.

UNIT-2

BOLTED CONNECTION

1. List out the uses of Bolted Connection (AU May 2011)

→ Bolts can be used for making end connections in tension and compression members.

→ Bolts can also be used to hold down column bases in position.

→ They can be used as separators for purlins and beams in foundations, etc

2. Define Pitch [AU May 2012]

Pitch is the distance between the centre of two consecutive rivets measured along a row of rivets

3 Define Efficiency of the joint [AU May 2014, NOV 2013]

It is the ratio of the strength of the joint to the strength of the main member expressed as a percentage. The effectiveness of a particular riveted joint is measured by the efficiency.

4 What do you mean by staggered pitch? [AU May 2011]

This is also known as alternate or reeled pitch. It is the distance measured along one bolt line, from the centre of a bolt on it to the centre of the adjoining bolt on a lower and parallel bolt line.

5. Define High tension Bolts. How it is different from common black bolt? [AU NOV 2012, May 2011]

If the bolts are made from bars of medium Carbon Steel then it is referred as Hight strength Bolts. The high strength is achieved by quenching and tempering process or by alloying steel. This makes the bolts less ductile than bolts. For these types, proof load is used instead of yield stress. The increased tensile strength redues the number of bolts. Also the vibration and impact resistance of the joints are improved considerably. These bolts are tightened to a proof load using calibrated wrenches. Hence they grip the members tightly

6 Name the different types of Connections

- → Riveted Connections
- → Welded Connections
- → Bolted Connections
- → Pinned Connections

7 What is meant by bolt Value?

The least of the strengths in shearing and bearing of one bolts is equal to the bolt value.

8 Mention the types of failure of a bolted joint. [AU May 2014]

- → Tearing failure of the plate
- → Shear failure of the Plate
- → Shear failure of the bolt
- → Bearing failure of the bolt
- → Splitting failure of the plate

9 List out the different types of bolts (AU NOV 2015, May 2012]

- → Black Bolts or unfinished Bolts or C grade Bolts
- → Turned Bolts or Close Tolerance Bolts
- → Precision Bolts or A grade Bolts
  1. Semi Precision Bolts or B grade Bolts
  2. Ribbed Bolts
- → High strength bolts.

9 What do you understand by Prying action (AU NOV 2013)

In moment resisting beam-to-column Connection, the bolts will transfer the load by direct tension which is generally located at the end of Connection. The additional force induced in the bolt is mainly due to the flexibility of Connected plates is Known as Prying forces

10 What is block Shear failure? (AU NOV 2013)

If failure occurs in shear at a row of bolt holes Parallel to the applied loads, accompanied by tensile rupture along a perpendicular face then it is referred as block Shear Failure. This type of failure results in a block of material being torn out by the applied shear force.

DEPARTMENT OF CIVIL ENGINEERING

2 MARKS

UNIT-1 & 2

1. Define Rivet line.
2. What is slip factor?
3. What are the advantages of riveted connections?
4. Define the efficiency of a joint.
5. List any two limit states.
6. Name the various types of bolted connections.
7. Write the equation for calculating the effective throat thickness of a weld.
8. A single riveted double cover butt joint in plates 12mm thick is made with 20mm diameter power driven rivets at a pitch of 90 mm. Find the safe load per pitch length of the joint.
9. A circular plate of 150 mm diameter is welded to another plate by means of 6 mm fillet weld. Calculate the greatest twisting moment that can be resisted by the weld if the permissible shearing stress in the weld is 110 N/mm^2.
10. List out some major types of connections.
11. In what circumstances the joints are most suitable.
12. What is the difference between pitch and staggered pitch?
13. The minimum pitch allowed in the code is ------------.
14. The maximum longitudinal pitch allowed in a bolted compression member is -----------.
15. The maximum longitudinal pitch allowed in a bolted tension member is -----------.
16. The minimum edge distance in a member with rolled edge is approximately ----------.
17. Write short notes on pinned connection.
18. List out some simple connections used in steel structures.
19. Write short notes on :i) Lap joint ii) Butt joint.
20. What is the difference between unstiffened and stiffened seat connection?
21. List some important advantages of welding over bolting.
22. List some important disadvantages of welding ? What are they?
23. List out four types of welds.
24. Minimum thickness of fillet weld should be -------. Minimum length of fillet weld should be ----------.
25. What is the main drawback of a butt joint?

16 MARKS

UNIT-1 & 2

1. Design riveted connections for a bracket as shown in fig, carrying an eccentric load of 250 kN at a distance of 300 mm from the centre line. Adopt limit state design concepts.

2. (i) Explain the advantages and disadvantages of bolted connections.
(ii) Two plates of 15 mm thickness have been connected in a lap joint using high strength friction grip bolts. Design the joint so as to transmit a pull equal to full strength of the plate.

3. Explain the possible failures of a riveted joint.

4. Fig. shows an arranged to support a bracket plate. The load applied to the bracket plate is 100kN. Find the greatest resistance offered by the weld per mm length. If 6 mm fillet welds are used, find the greatest stress intensity in the weld.

5. A plate girder has flange plate of 440 x 25mm and it was found from the beam analysis that this place where a joint has to be provided is subjected to a tension of 1150 kN.Design a butt weld.

6. A 16m span truss has a rafter consisting of 21SA 40,40,6 and which is subjected to 56.70 kN compression, design using rivets.

7. A 6 mm thick angle section is jointed to a 10 mm thick gusset plate. The angle is supporting a load of 60 kN. Find out the number of 16 mm diameter rivets.

8. A tie member 75 mm X 8 mm is to transmit a load of 90 kN. Design the fillet weld and calculate the necessary overlap.

9. Calculate the maximum load carried by any rivet shown in fig. Rivet and B are 200 mm^2 cross sectional area and rivet C of 400 mm^2 area.

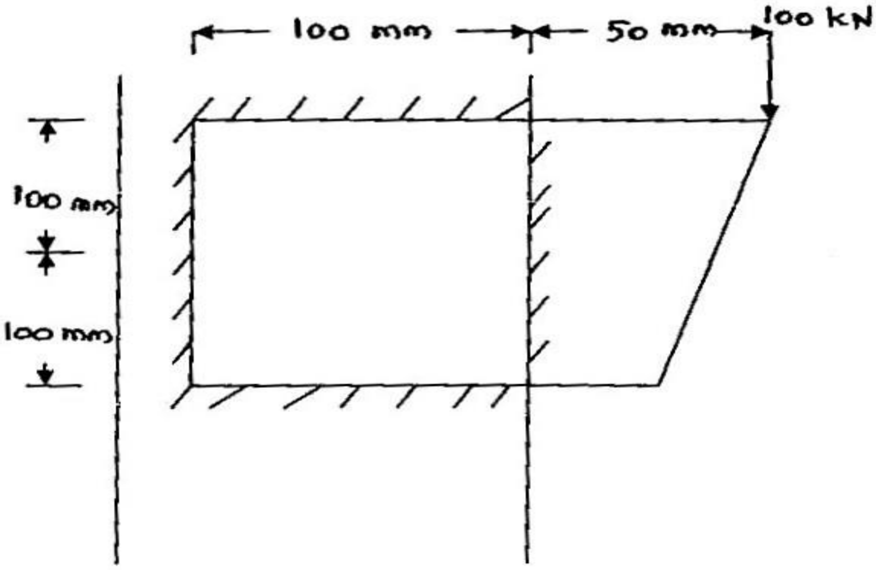
10. Design the welded joint of a truss shown in fig. take the permissible shear stress in the weld is 110 N/mm^2. Need not design the connection of the member.

11. Two plates 12 mm and 10 mm thick are joined by triple riveted lap joint, in which the pitch of the central row of rivets is 0.6 times the pitch of rivets in the outer rows. Design the joint and find its efficiency. Take σ_{at} = 150 N/mm^2, τ_{vf} = 80 N/mm^2 and σ_{vf} = 250 N /mm^2.

12. A bracket carrying a load of 100 kN is connected to column by means of two horizontal fillet welds, of 130 mm effective length and 10 mm thick. the load acts at 70 mm from the face of the column as shown in fig. Find the throat stress.

13. Explain the possible failures of a riveted joint.

14. Fig. shows an arranged to support a bracket plate. The load applied to the bracket plate is 100kN. Find the greatest resistance offered by the weld per mm length. If 6 mm fillet welds are used, find the greatest stress intensity in the weld.



DEPARTMENT OF CIVIL ENGINEERING

DESIGN OF STEEL STRUCTURAL ELEMENTS

Course material

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CHENNAI

UNIT III – DESIGN OF TENSION MEMBER

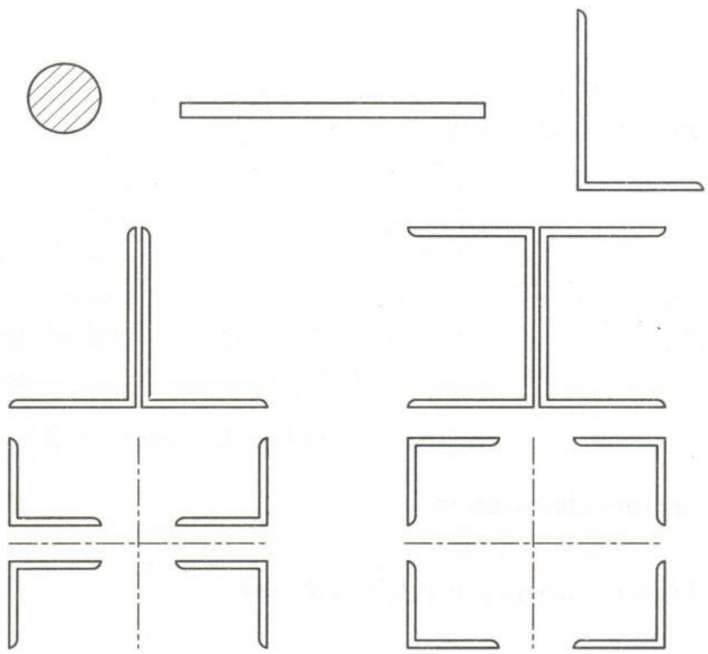
UNIT-III
DESIGN OF TENSION

Tension Members:-

Tension members are design to satisfy the design strength of the member against

(i) Yielding of gross section
(ii) Rupture of critical section
(iii) Block shear @ end of connection

Generally tension members are known as tie member.



The various shapes of tension members are solid circular sections, plates, angles, channels, I-sections, T-section & built-up section.

1. Design strength of Tension Members are due to yielding:- [cls 6.2 IS 800-2007]

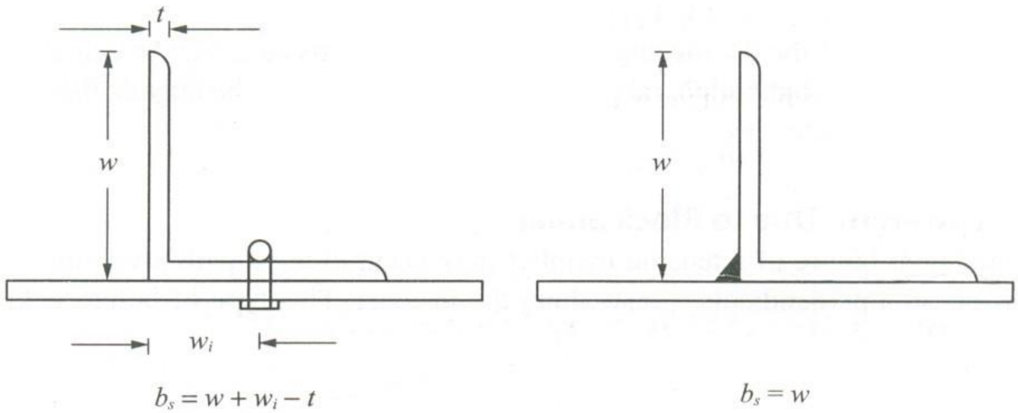
$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

2. Design strength due to rupture of critical section:- [cls 6.3 IS 800-2007] (Ultimate)

$$T_{dn} = \frac{0.9\ Anfu}{\gamma_{ml}}$$

Where,

$$An = [\, b - nd_n + \sum_{l} \frac{p_{si}}{u_{gi}}]\ t$$



a) For Angular section design strength of rupture:-

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

Where,

$$\beta = 1.4 - 0.076 \left(w/t\right)\left(f_y/f_u\right)\left(b_s/l_c\right) \leq \left(\frac{f_u \gamma_{mo}}{f_y \gamma_{ml}}\right) \geq 0.7$$

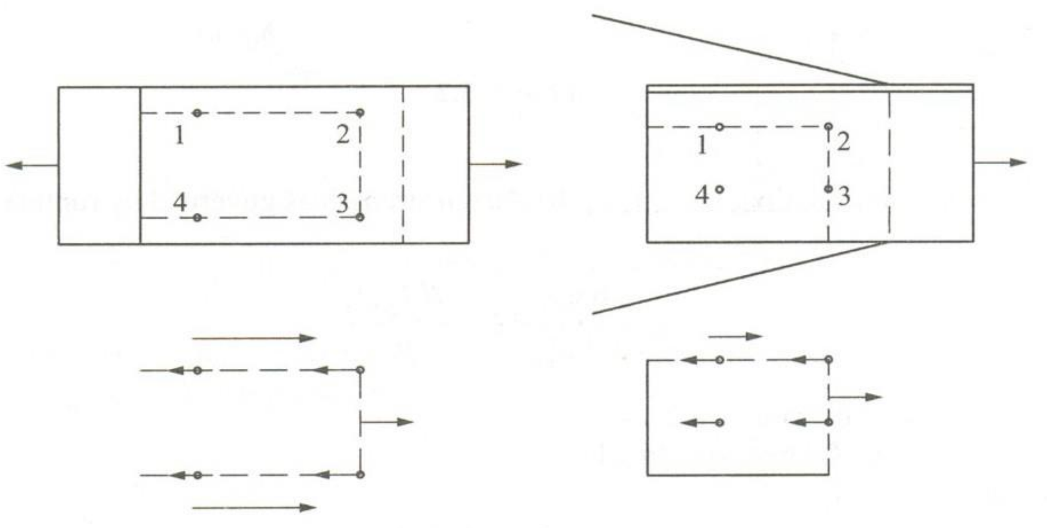
3. Design strength of member due to block shear failure @ the end connection:- [cls 6.4 IS 800-2007]

a) Bolted Connections:- [cls 6.4.1]

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

(or)

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



(a) (b)

b) Welded Connection:-

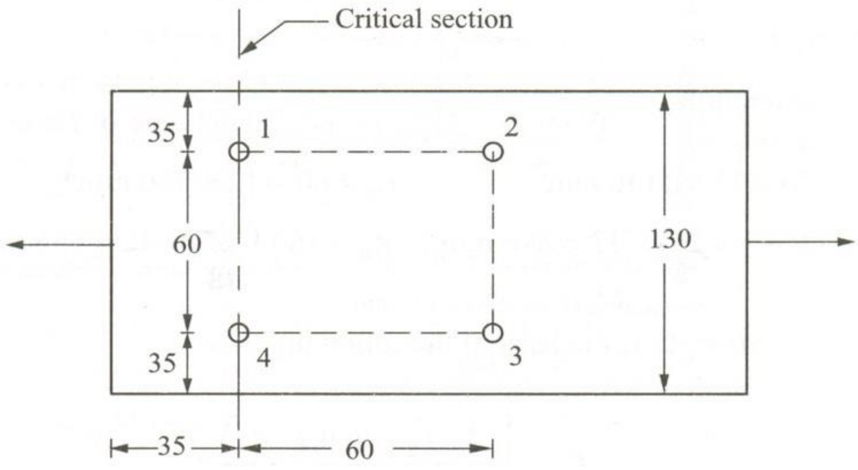
Appropriate length of member is considered around the end weld.

Preliminary section:- [cls 6.3.3 IS 800-2007]

Preliminary section is assumed from the relation is based on

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

1. Determine the design tensile strength of the plate of size 200x12mm with holes having bolts of dia 16mm (M16). The grade of steel used is Fe410.



Given:-

Size of plate = 200mm x 12mm

Dia of bolt = 16mm

Grade Fe410 => fu = 410 N/mm^2

fy = 250 N/mm^2

Sln:-

1. Design strength due to yielding:- [cls 6.2 IS 800-2007]

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

Ag = 130x12 = 1560mm^2

$$= \frac{1560 \times 250}{1.1 \Rightarrow (table\,5)}$$

T_{dg}= 354.5 KN

2. Design strength of plate @ rupture: [Along critical section] [cls 6.3 IS 800-2007]

The critical section is along the line having 2 bolts

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

$$A_n = [b - n d_n] t$$

n = 2

b = 130mm

= [130-2x18]12

A_n =1128mm^2

$$= \frac{0.9 \times 1128 \times 410}{1.25 \Rightarrow [Table-5]}$$

T_{dn} = 332.9 KN

3. Design strength due to block shear:- [cls 6.4 IS 800-2007]

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

(or)

$$T_{dn} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{mn}}\right] + \left[\frac{A_{tg} f_y}{\gamma_{mo}}\right]$$

Where,

Section considered for A_{vg} is (e+n'p) x t

Section considered for A_{vg} is (n'g) x t

Section considered for A_{vn} & A_{tn} is the net area after detecting the bolt hole.

A_{vg} = (35+60)12= 1140mm^2

A_{tg} = 60x12 = 720mm^2

A_{vn} = [35+60-18] x 12 = 924mm^2

A_{tn} = [60-18] 12 = 504mm^2

$$T_{db_1} = \left[\frac{1140 \times 250}{\sqrt{3} \times 1.1}\right] + \left[\frac{0.9 \times 504 \times 410}{1.25}\right]$$

T_{db_1} = 298.36 KN

(or)

$$T_{db} = \left[\frac{0.9 \times 924 \times 410}{\sqrt{3} \times 1.25}\right] + \left[\frac{720 \times 250}{1.1}\right]$$

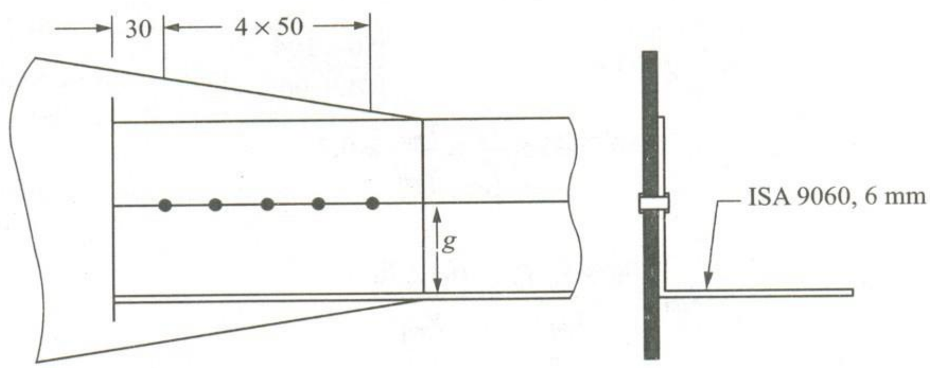
$$T_{db_2} = 321.12\,KN$$

$\therefore$ The least of the above 4 strength value is the design strength of the plate.

$\therefore$ Design strength of the plate = 298.36 KN

1. A single unequal angle ISA 90x60x6mm is connected to a 10mm tk gusset plate at the ends with 5 Nos of 16mm dia bolts to transfer tension. Determine the design tensile strength of the angle if the gusset is connected to the 90mm leg.

Given:-



Unequal angle = ISA 90x60x6

Tks of gusset plate = 10mm

φ of bolt = 16mm

Nos of bolt = 5 Nos.

Sln:-

1. Design strength of angle in yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left(90 - \frac{6}{2}\right) + \left(60 - \frac{6}{2}\right)\right] \times 6$$

$$A_g = 864\text{mm}^2$$

$$= \frac{864 \times 250}{1.1}$$

$$T_{dg} = 196.36\ KN$$

2. Design strength of angle against rupture [cls 6.3.3 IS 800-2007]

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

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

Where,

A_n=> Net area of the connected leg = (90-6/2-18) 6

A_n=414mm^2

A_g=> Gross area of the outstanding leg = (60-6/2) x 6

A_g = 342mm^2

w => outstanding leg width = 60mm

t => tks of angle = 6mm

b_s=> shear lag width = w+w_1-t

Assume, w_1=90/2=45mm ~ 50mm

Provide w_1 = 50mm

b_s=60+50-6 b_s = 104mm

L_c=> length of the end connection = 90mm

$$= \left[1.4-0.076\times\frac{60}{6}\times\frac{250}{410}\times\frac{104}{90}\right]$$

$$¿\frac{410\times1.1}{250\times1.25}\geq0.7$$

$$\beta=0.864\leq1.44\geq0.7$$

Which is true [0.7 ¿ 0.864 ¿ 1.44]

$$\therefore\beta=0.864$$

$$T_{dn} = \frac{0.9\times414\times410}{1.25}+\frac{0.864\times342\times250}{1.1}$$

$\boxed{T_{dn} = 189.369\ \text{KN}}$

3. Design strength of plate against block shear of end connection: [cls 6.4 IS 800-2007]

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

(or)

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

Where,

A_{vg}= (30+4x50)6=1380mm^2

A_{vn}= [230-(4.5x18)]6=894mm^2

Block shear failure takes place along line 1 to 3

'A_{tg}' is found along line 1-2

A_{vg} = (30+4x50)6=1380mm^2

A_{tg} is taken along line 2-3

$$A_{tn}=\left[40-\frac{1}{2}\times18\right]6$$

$$A_{tn} = 186\text{mm}^2$$

$$T_{db} = \frac{1380\times250}{\sqrt{3}\times1.1}+\frac{0.9\times186\times410}{1.25}$$

$$T_{db}=235.98\,KN$$

(or)

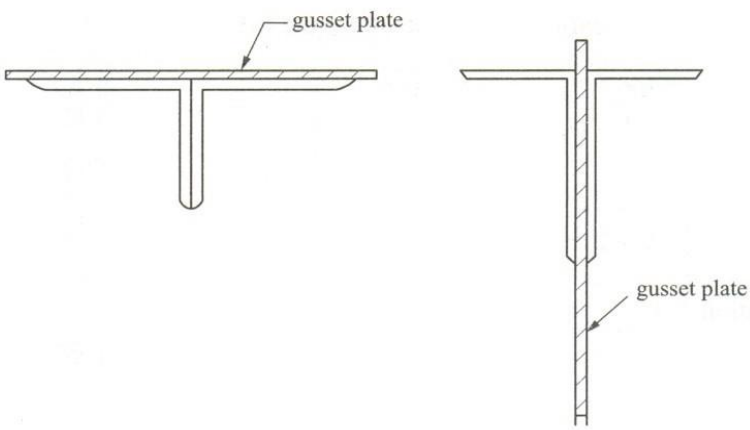
$$T_{db} = \frac{0.9\times894\times410}{\sqrt{3}\times1.25}+\frac{240\times250}{1.1}$$

T_{db} = 206.9 KN

$\therefore$ The least of strength of section in yielding, rupture and block shear is the design strength of the section.

$\therefore$ Design strength of the section = 189.369 KN

3. Find the design strength if the 60mm side is connected to the gusset plate as in the above problem.



(a) (b)

Sln:-

Here the 60mm side is connected to gusset plate.

$\therefore$ Assume the line of bolts to be placed at a distance 60/2 = 30mm

1. Design strength of angle in yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left(90 - \frac{6}{2}\right) + \left(60 - \frac{6}{2}\right)\right] \times 6$$

$$A_g = 864\text{mm}^2$$

$$= \frac{864 \times 250}{1.1}$$

$$\boxed{T_{dg} = 196.36 \text{ KN}}$$

2. Design strength of angle against rupture [cls 6.3.3 IS 800-2007]

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

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

$\Downarrow$

Where, [cls 6.3.3 IS 800-2007]

$$A_{nc} = \left(60 - \frac{6}{2} - 18\right)6$$

$= 234\text{mm}^2$

$$A_g = \left(90 - \frac{6}{2}\right) \times 6$$

$= 522\text{mm}^2$

w => 90mm

t => 6mm

b_s = w+w_1-t

= 90+30-6=114mm

b_s = 114mm

L_c = 60mm

$$\beta = 1.4 - 0.076\left(\frac{90}{6}\right)\left(\frac{250}{410}\right)\left(\frac{114}{60}\right) \le \left(\frac{410 \times 1.1}{250 \times 1.25}\right) \ge 0.7$$

$$\beta = 0.079 \le 1.44 \ge 0.7$$

Max. limit for β is $\dfrac{f_u \gamma_{mo}}{f_y \gamma_{ml}}$

$\therefore$ Provide β = 0.7

$$\therefore \quad T_{dn} = \frac{0.9 \times 234 \times 410}{1.25} + \frac{0.7 \times 522 \times 250}{1.1}$$

$$\boxed{T_{dn} = 152.12 \text{ KN}}$$

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]

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

(or)

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

Where,

$$A_{vg} = [30 + (4 \times 50)]6$$

$$A_g = 1380 mm^2$$

$$A_{tg} = 30 \times 6 = 180\ mm^2$$

$$A_{tn} = \left[30 - \frac{18}{2}\right] \times 6$$

$$A_{tn} = 126\text{mm}^2$$

$$A_{vn} = [230 - (4.5 \times 18)]6$$

$$A_{vn} = 894\ mm^2$$

$$T_{db} = \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}$$

$$\boxed{T_{db} = 218.27 \text{ KN}}$$

$$T_{db} = \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$$

$$\boxed{T_{db} = 193.27 \text{ KN}}$$

$\therefore$ Design strength
of the section =
152.12 KN

DESIGN OF TENSION MEMBER:-

Design Procedure:-

1. Find the reqd gross area to carry the factored load considering the strength at yielding.

$$\boxed{A_a = \frac{1.1T_u}{f_y}}$$

2. Select suitable section depending upon the type of structure & location of member such that the gross area is 25 to 40% [generally 30%] more than 'A_g' calculated.

3. Determine the no. of bolts are length of weld reqd and arrange them appropriately. [design of connection]

4. Find the strength of the assumed section considering

   (i) Strength of section in yielding of gross area

   (ii) Strength of section in rupture of critical section.

   (iii) Strength of section against block shear at the end o connection.

5. The strength of section obtained [Design strength of section] should be more than a factored tensile force ting on the section. If not, the section has to be revised and redesign the section.

6. The slenderness ratio has to be check for the tension member, as per table-3, IS 800-2007 [Pg.No:20]

Slenderness ratio, $\lambda = \dfrac{l_{eff}}{\gamma_{\min}}$

Where,

$\gamma_{\min}$ => The least of γ_{xx} & γ_{yy} of the section. [from steel table]

1. Design a single angle section for tension member of a roof truss to carry a factored load of 225KN. The member is subjected to possible reversal of stress due to the action of wind. The length of the member is 3m. Use 20mm shop bolts of grade 4.6 for the connection.

Given:-

T_u= 225 KN

d = 20mm

$f_y = 400 \, {}^{N}\!/\!{}_{mm^2}$

Grade 4.6 => $f_y = 250 \, {}^{N}\!/\!{}_{mm^2}$

$d_o = 22\,mm$

Sln:-

$n = \dfrac{T_u}{v}$

Required area, $A_g = \frac{1.1T_u}{f_y}$

$= \frac{1.1 \times 225 \times 10^3}{250}$

$A_g = 990\, mm^2$

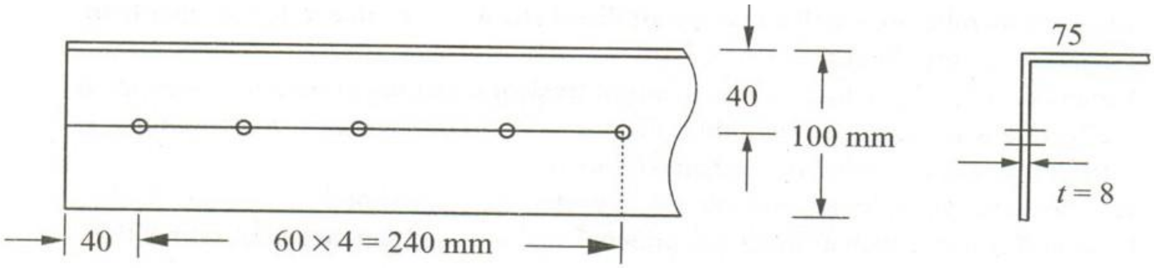
To select ISA 100x75x8mm

$A_g = 1336\, mm^2$ [from steel table]

γ_{xx}=31.4mm γ_{yy} = 21.8mm

γ_{min} = 21.8mm

$\lambda = \overline{21.8}$ is connected to the gusset plate (assumed tks 10mm) by lap jt along the 100mm side.



BOLT VALUE:-[M20]

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

$$V_{dsp} = \frac{V_{nsp}}{\gamma_{mb}}$$

$$V_{nsp} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

$$V_{dsp} = \frac{f_u}{\sqrt{3}} \frac{(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}}$$ ∙ n_s=0 f ∙ ss

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

1.25Table5 – IS 800-2007

$$\boxed{V_{dsp} = 45.27\ KN}$$

(ii) Strength of the bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dsp} = \frac{V_{nbp}}{\gamma_{mb}} = \frac{2.5 k_b d_t f_u}{\gamma_{mb}}$$

$$k_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1$$

Assume, e=1.5 d_o=1.5 x 22 = 33mm ¿ 40mm

p = 2.5d = 2.5 x 20 = 50mm ¿ 60mm

$$k_b \Rightarrow \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1$$

= 0.606, 0.659, 0.975, 1

$\therefore$ k_b = 0.606 (least value)

$$V_{dbp} = \frac{2.5 \times 0.606 \times 20 \times 8 \times 410}{1.25}$$

$$\boxed{V_{dbp} = 79.5 \text{ KN}}$$

$\therefore$ Design strength of bolt value = 45.27 KN

$\therefore$ No. of bolts, n= $\frac{T_u}{v}$

$= \frac{225}{45.27}$

= 4.97 ¿ 5 Nos.

$\therefore$ Provide 5 Nos of 20mm dia bolts pitch 60mm and the edge distance 40mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left(100 - \frac{8}{2}\right) + \left(75 - \frac{8}{2}\right)\right] \times 8$$

$$A_g = 1336\, mm^2$$

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

$$\boxed{T_{dg} = 303.636\, KN}$$

2. Design strength of the section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \geq 0.7$$

$$A_{nc} = \left[100 - 22 - \frac{8}{2}\right] 8$$

$$A_{nc} = 592\ mm^2$$

$$A_{go} = \left(75 - \frac{8}{2}\right) 8$$

$$A_{go} = 568\, mm^2$$

w = 75mm

t = 50mm

b_s = w+w_1-t

= 75+50-8

b_s = 117mm

$L_c = 100\text{mm}$

$$\beta = 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{117}{100}\right) \le \left(\frac{410\times 1.1}{250\times 1.25}\right) \ge 0.7$$

$$= 0.89 \le 1.44 \ge 0.7$$

$$\beta = 0.07 \le 0.89 \le 1.44$$

$$\therefore \beta = 0.89$$

$$T_{dn} = \frac{0.9\times 592\times 410}{1.25} + \frac{0.89\times 568\times 250}{1.1}$$

$$\boxed{T_{dn} = 289.6\text{ KN}}$$

> 225 KN

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]

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

(or)

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

Where,

$$A_{vg} = [40 + (4\times 60)]\times 8$$

$$A_{vg} = 2240\ mm^2$$

$$A_{tg} = 50\times 8$$

$$A_{tg} = 400 mm^2$$

$$A_{vn} = [280 - (4.5\times 22)]8$$

$$A_{vn} = 1448\ mm^2$$

$$A_{tn} = \left[50 - \frac{22}{2}\right]\times 8$$

$$A_{tn} = 312\ mm^2$$

$$T_{db_1} = \left[\frac{2240\times 250}{\sqrt{3}\times 1.1}\right] + \left[\frac{0.9\times 312\times 410}{1.25}\right]$$

$$\boxed{T_{db_1} = 386.026\text{ KN}}$$

$$T_{db} = \left[\frac{0.9\times 1448\times 410}{\sqrt{3}\times 1.25}\right] + \left[\frac{400\times 250}{1.1}\right]$$

$$\boxed{T_{db_2} = 337.697\ KN}$$

The above 2 values of strength against block shear 337.697 KN > 225KN

The strength of the section against yielding, rupture & block shear are greater than the external load of 225KN.

$\therefore$ The assume section ISA 100x75x8mm is safe.

2. Solve the above problem using angle section on opposite sides of gusset plate

Given:-

T_u = 225KN

d = 20mm

d_o= 22mm

Grade 4.6 => f_u= 400 N/mm^2

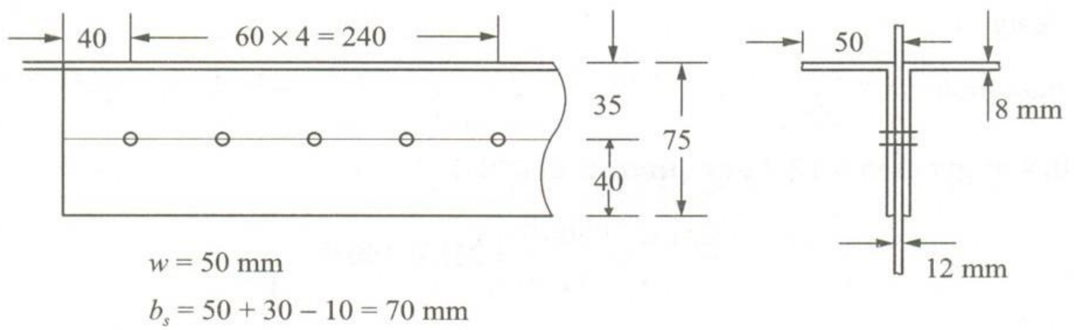
f_y= 250 N/mm^2

$$\lambda act = \frac{l_{ef}}{\gamma_{\min}}$$

$$\lambda \max = 350\, table\, 3$$

$$\lambda act < \lambda \max$$

Sln:-



To find A_g:-

$$A_g = \frac{1.1T_u}{f_y}$$

$$= \frac{1.1 \times 225 \times 10^3}{250}$$

$$A_g = 990\, mm^2$$

Area each angle reqd = 990/2 = 495mm^2

$\therefore$ Select the section from steel table having area 30% more than 495mm^2

Try ISA 70x70x5mm

A_g=667mm^2 [from steel table]

$$\gamma_{xx} = 21.5\, mm$$

$$\gamma_{yy} = 21.5\, mm$$

Bolt Value:- [M20]

(i) Strength of bolt in double shear:-

Assuming gusset plate of tks = 10mm

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

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

$$n_n = n_s = 1$$

$$A_{nb} = \frac{0.78 \times \pi \times 20^2}{4}, A_{sb} = \frac{\pi \times 20^2}{4} = 314.16\, mm^2$$

$$V_{dsb} = \frac{\frac{400}{\sqrt{3}}[1 \times 245 + 1 \times 314.16]}{1.25}$$

$$\boxed{V_{dsb} = 103.3\,KN}$$

(ii) Strength of bolt in bearing:-

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$= \frac{2.5 \mathrm{k}_b d_t f_u}{\gamma_{mb}}$$

Assume e=1.5 d_o= 33mm ¿ 40mm

p =2.5d = 50mm ¿ 60mm

$$k_b = \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1$$

= 0.606, 0.659, 0.975, 1

∴ Take k_b = 0.606 [least value]

$$V_{dbp} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25}$$

$$\boxed{V_{dbp} = 99.38 \mathrm{KN}}$$

∴ Design strength of bolt value = 99.38KN

∴ No. of bolts = 225/99.38

= 2.26 ¿ 3 Nos.

∴ Provide 3 nos. of 20mm bolts for pitch 60mm & tks edge distance 40mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left[\left(70 - \frac{5}{2}\right) + \left(70 - \frac{5}{2}\right)\right] \times 5\right] \times 2$$

$$A_g = 1334\,mm^2$$

$$= \frac{1334 \times 250}{1.1}$$

$\boxed{T_{dg} = 303.18\,KN}$ > 225KN

2. Strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta=1.4-0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)\leq\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\geq 0.7$$

$$A_{nc}=\left[70-22-\frac{5}{2}\right]\times 5\times 2$$

$$A_{nc}=456\ mm^2$$

$$A_{go}=\left(70-\frac{5}{2}\right)\times 5\times 2$$

$$A_{go}=675\ mm^2$$

w = 70mm

w_1= 35mm ¿ 40mm

b_s= w+w_1-t

= 70+40-10

=100

$$\beta=1.4-0.076\left(\frac{70}{100}\right)\left(\frac{250}{410}\right)\left(\frac{100}{160}\right)\leq\left(\frac{410\times 1.1}{250\times 1.25}\right)\geq 0.7$$

$$=1.89\leq 1.44\geq 0.7$$

$$\beta=0.7\leq 1.19\leq 1.44$$

$$\therefore\beta=1.19$$

$$T_{dn}=\frac{0.9\times 456\times 410}{1.25}+\frac{1.19\times 675\times 250}{1.1}$$

$$\boxed{T_{dn}=316.87\text{ KN}}$$

> 225 KN

3. Design strength of plate against block shear: [cls 6.4 IS 800-2007]

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

(or)

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

A_{vg} & A_{vn} are found along section 1-2 and

A_{tg} & A_{tn} are found along section 2-3

$$A_{vg}=[[40+2(60)]5]2$$

= 1600mm^2

$$A_{vn}=[[40+2(60)-2.5(22)]5]2$$

= 1050mm^2

$$A_{tg}=30\times 5\times 2=300\ mm^2$$

$$A_{tn}=[[30-0.5(22)]5]2$$

= 190mm^2

$$T_{db_1}=\frac{1600\times 250}{\sqrt{3}\times 1.1}+\frac{0.9\times 190\times 410}{1.25}$$

$$\boxed{T_{db_1} = 266.03 \text{ KN}}$$

For 2 angles

$$T_{db_2} = \frac{0.9 \times 1050 \times 410}{\sqrt{3} \times 1.25} + \frac{300 \times 250}{1.1}$$

$$\boxed{T_{db_2} = 247.137 \text{ KN}}$$

For 2 angles

T_{db}= 247.137KN [least value of these two]

Hence 2 nos. of ISA 70x70x5mm is safe against yielding, rupture & block shear conditions.

TENSION SPLICE:-

- When a single piece of reqd length is not available, for a tension member, splice plates are used to transverse the reqd tension force from 1 piece to another.
- The strength of the splice plates & the bolts connecting them should have strength atleast equal to a design load.

1. Design a splice to connect a plate of size 300x20mm width a plate of size 300x10mm. The design load is 500KN. Use 20mm block bolts fabricated in the shop. Provide a double cover butt joint with tks of cover as 10mm.

Given:-

1. Plate of size = 300x20mm
2. Plate of size = 300x10mm

   Tks of cover plate = 6mm

   d = 20mm

   d_o =22mm

   Design load = 500KN

Sln:-

Since plates have varying tks need to be provided packing plate is reqd to provide the two cover plates.

The bolts are under double shear.

1. Strength of bolt in double shear:- [cls 10.3.3 IS 800-2007]

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

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

$$n_n = n_s = 1$$

$$A_{nb} = \frac{0.78 \times \pi \times 20^2}{4} = 245 \ mm^2$$

$$A_{sb} = \frac{\pi \times 20^2}{4} = 314.16 \ mm^2$$

$$\beta_{pk} = [1 - 0.0125 \, tpk]$$

$= [1-(0.0125\times10)]$

β_{pk} = 0.875

$$V_{nsb} = \frac{400}{\sqrt{3}}[1\times245+1\times314.16]\times0.875$$

= 112.99 KN

$$V_{dsb} = \frac{112.99}{1.25}$$

$\boxed{V_{dsb} = 90.392 \text{ KN}}$

1. Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$= 2.5k_b d_t f_u$

Assume e=1.5 d_o= 33mm ¿ 40mm

p =2.5d = 50mm ¿ 60mm

$$k_b = \frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1$$

= 0.606, 0.659, 0.975, 1

∴ Take k_b = 0.606 [least value]

$V_{dbp} = 2.5\times0.606\times20\times10\times410$

= 124.23 KN

$\boxed{V_{dbp} = 99.38\text{KN}}$

∴ Design strength of bolt value = 90.39 KN

∴ No. of bolts = $\frac{T_u}{v}$

$= \frac{500}{90.39}$

$n = 5.5 \simeq 6 \, Nos.$

∴ Provide 6 nos. of 20mm bolts on each side

Providing the 6 bolts on each side of the connecting plate, it can be arrange along 2 vertical rows with 3 bolts on each vertical row as shown in fig.

Check for strength of section:-

1. Strength of the plate against yielding:- [cls 6.2 IS 800-2007]

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

$A_g = 300\times10 = 3000\, mm^2$ [Tks of thinner plate]

$$= \frac{3000\times250}{1.1}$$

T_{dg} = 681.81 KN > 500KN

2. Strength of the plate against rupture:- [cls 6.3.1 IS 800-2007]

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

A_n= The critical section where carrying of plate is occurs along the vertical line passing through the 3 bolts.

= [300-3x(22)]x10

A_n = 2340mm^2

$$= \frac{0.9 \times 2340 \times 410}{1.25}$$

$$T_{dn} = 690.77\ KN > 500\ KN$$

3. Strength of the plate against block shear:- [cls 6.4 IS 800-2007]

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

(or)

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

The block shear failure takes place along the lines 1,2,3,4 as shown in fig. [The path of block shear failure is given in fig:7 IS 800-2007]

A_{vg} & A_{vn} are found along section 1-2 and

A_{tg} & A_{tn} are found along section 2-3

$$A_{vg} = [40+60] \times 10 = 1000\,mm^2$$

$$A_{vn} = [(40+60) - 1.5(22)]\,10$$

= 670mm^2

$$A_{tg} = [2 \times 110]\,10 = 2200\,mm^2$$

$$A_{tn} = [2(110) - 2(22)]\,10$$

= 1760mm^2

$$T_{db_1} = \frac{1000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1760 \times 410}{1.25}$$

$$\boxed{T_{db_1} = 650.76\ \text{KN}}$$

$$T_{db_2} = \frac{0.9 \times 670 \times 410}{\sqrt{3} \times 1.25} + \frac{2200 \times 250}{1.1}$$

$$T_{db_2} = 614.190\ KN > 500\ KN$$

Hence the connection is safe.

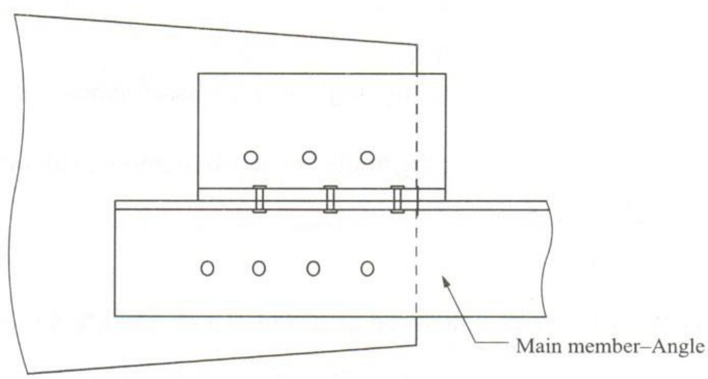
LUG ANGLES:-

- The length of end connections of heavily loaded tension members may be reduced by using lug angles as shown in fig.
- There is savings in gusset plate but additional cost is incurred from the material of lug angles & the connections for the lug angles.

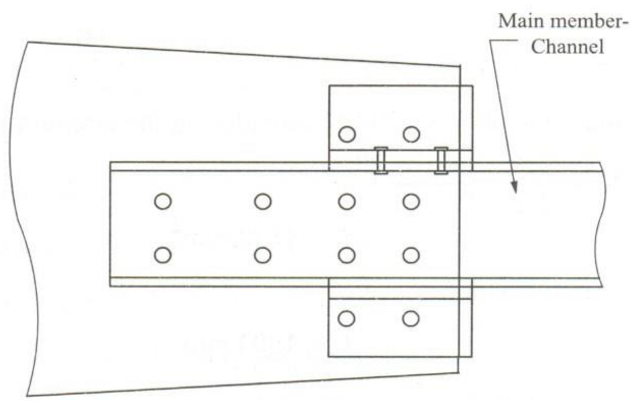
❖ The design of tension member with the use of lug angles needs to be check for the load which is share equally by the connected leg and the outstanding leg.

The following guidelines need to be satisfied.

1. The eff. Connection of the lug angle shall as for as possible.
2. It is preferable to start the lug angle in advance of a member connected.
3. A mini of 2 bolts or rivets, are provided.
4. In case of angles, the whole area can be taken rather than the net eff. Area.
5. In case of channels, the lug angles should be placed simitrical and the strength of fasterness connecting lug angle to the gusset be 10% more than the outstanding leg.
   [When main member is a channel]
6. In case of angle [Main member] the above values are 20% & 40% respectively.



(a)



(b)

1. Design a tension member of a roof truss which carries a factored axial tension of 430KN.

Design the connection when

(i) No lug angle is provided

(ii) Lug angle is provided

Hints:-

1. Without lug angle, the connections are designed for ‘T_u’ and member is check for design strength for ‘T_u’.
2. When lug angle is provided, connection in main member is design for ‘$T_u/2$’ and the connection in lug angle is design for ‘$T_u/2$’, where the connection plate & lug angle is increased by 20% and connection b/w lug angle & main plate is increased by 40%

Given:-

$T_u = 430KN$

Sln:-

(i) No lug angle is provided:-

Assume, d = 20mm

d_o=22mm

Tks of gusset plate = 12mm

BOLT VALUE:- [M20]

$$A_g = \frac{1.1T_u}{f_y}$$

$$= \frac{1.1 \times 440 \times 10^3}{250}$$

$$A_g = 1892 mm^2$$

Select a section from steel table having area 30% more than the reqd area.

Select ISA 110x110x12mm

$$A_g = 2502 mm^2$$

$$\gamma_{xx} = \gamma_{yy} = 33.4\ mm$$

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

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

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

$$n_n = 1, n_s = 0$$

$$A_{nb} = 245\ mm^2$$

$$= \frac{400}{\sqrt{3}}[1 \times 245]$$

$$V_{nsb} = 56.58\ KN$$

$$V_{dsb} = \frac{56.58}{1.25}$$

V_{dsb} = 45.264 KN

(ii) Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$V_{nbp} = 2.5 k_b d_t f_u$

Assume e = 40mm

P = 60mm

$$K_b = \frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1$$

K_b =0.606, 0.66, 0.959,1

∴ Take K_b = 0.606

$$V_{dbp} = \frac{2.5\times0.606\times20\times12\times410}{1.25}$$

$$\boxed{V_{dbp} = 119.26 \text{ KN}}$$

∴ Design strength of bolt value = 45.264 KN

∴ No. of bolts = $\frac{T_u}{v}$

$= \frac{430}{45.264}$

= 9.49 ¿ 10 Nos.

∴ Provide 10 nos of 20mm dia to bolts edge distance 40mm & pitch of 60mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left(110 - \frac{12}{2}\right) + \left(110 - \frac{12}{2}\right)\right]\times12$$

A_g = 2496mm^2

$= \frac{2496\times250}{1.1}$

T_{dg}= 567.27 KN > 430 KN

2. Strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

$$A_{go} = \left[110 - \frac{12}{2}\right]\times12$$

$= 1248\, mm^2$

$$A = \left[110 - 22 - \frac{12}{2}\right]\times12$$

= 984mm^2

w = 110 mm

w_1 = 60mm

$b_s = w + w_1 - t$

$= 110+60-12$

$= 158$mm

L_c = 580 mm

$$\beta = 1.4 - 0.076\left(\frac{110}{12}\right)\left(\frac{250}{410}\right)\left(\frac{158}{580}\right) \le \left(\frac{410\times 1.1}{250\times 1.25}\right) \ge 0.7$$

$$= 1.28 \le 1.44 \ge 0.7$$

$$\therefore \beta = 1.28$$

$$T_{dn} = \frac{0.9\times 984\times 410}{1.25} + \frac{1.28\times 1248\times 250}{1.1}$$

$$\boxed{T_{dn} = 653.53 \text{ KN}}$$

> 430 KN

3. Strength of the section against block shear:- [cls 6.4.1 IS 800-2007]

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

(or)

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

Where,

$$A_{vg} = [40 + (9\times 60)]12$$
$$= 6960 \text{mm}^2$$

$$A_{tg} = [50\times 12]$$
$$= 600 \text{mm}^2$$

$$A_{vn} = [580 - (9.5\times 22)]12$$
$$= 4452 \text{mm}^2$$

$$A_{tn} = \left[50 - \frac{22}{2}\right]12$$
$$= 468 \text{mm}^2$$

$$T_{db_1} = \frac{6960\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 468\times 410}{1.25}$$

$$T_{db_1} = 997.5 \text{ KN} > 430 \text{KN}$$

$$T_{db_2} = \frac{0.9\times 4452\times 410}{\sqrt{3}\times 1.25} + \frac{600\times 250}{1.1}$$

$$T_{db_2} = 895.13 \text{ KN} > 430 \text{KN}$$

(ii) Lug Angle is Provided:-

- When lug angle is provided the member of bolts reqd for establishing the connection reduces thereby reducing the overall length of overlap.

- The connection b/w main member gusset plate is designed for $T_u/2$
- The lug angle is designed for a force of $T_u/2$ [Increased by 30%]

◆ The connection b/w the main member lug angle is designed for 40% of $T_u/2$ and connection b/w angle & gusset plate designed for 20% of $T_u/2$

Connection for Main Member:-

$$n = \frac{T_u/2}{v} = \frac{215}{45.26} \qquad [\because v = 45.26\,KN]$$

n = 4.75 ¿ 5 Nos

∴ Provide 5 nos of 20mm bolts.

Lug Angles:-

$$A_g = \frac{1.1 \times T_u/2 \times 1.3}{f_y} = \frac{1.1 \times 215 \times 1.3 \times 10^3}{250} = 1230\text{mm}^2$$

Try ISA 80x80x12mm

$A_g = 1781\text{mm}^2$

$\gamma_{xx} = \gamma_{yy} = 23.9\,mm$

Connection b/w gusset plate & lug angle:-

$$\text{No. of bolts} = \frac{1.2 \times T_u/2}{v}$$

$$= \frac{1.2 \times 215}{45.26}$$

n = 5.7 ¿ 6 Nos

∴ Provide 6 nos of bolts b/w gusset plate and lug angle.

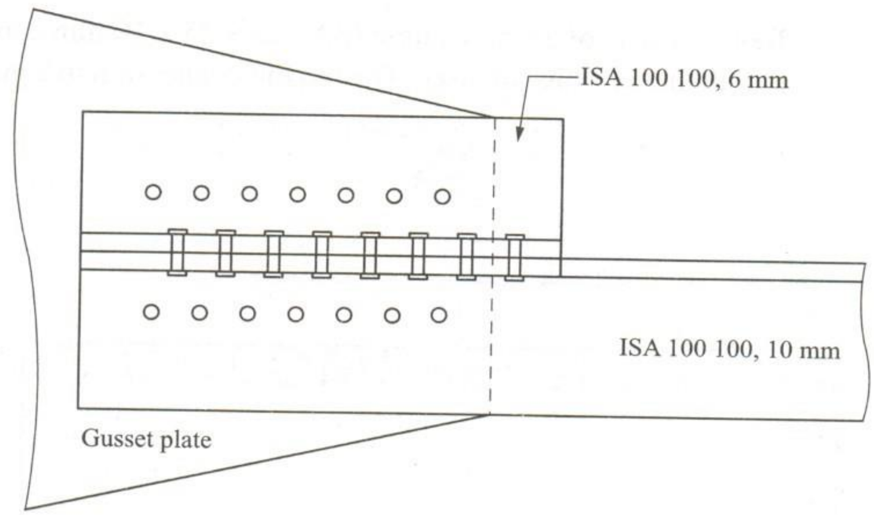
Connection b/w lug angle & main membe:-

$$n = \frac{1.4 \times T_u/2}{v}$$

$$= \frac{1.4 \times 215}{45.26}$$

n = 6.65 ¿ 7 Nos

∴ Provide 7 nos of bolts b/w lug angle and main member.


ISA 100 100, 6 mm
ISA 100 100, 10 mm
Gusset plate

UNIT IV - DESIGN OF COMPRESSION MEMBER

Classes of sections:-

1. Column -> Stanchion
2. Truss -> Strut
3. Beam -> Girder

a) Class 1 [Plastic]:-

Cross sections, which can develop plastic hinges and have the rotation capacity reqd for failure of the structures by formation of plastic mechanism. The width to tks ratio of plate elements shall be less than that specified under class 1 (plastic) in table 21.

b) Class 2 [Compact]:-

Cross-sections which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling. The width to tks ratio of plate elements shall be less than that specified under class-2 (compact), but greater than that specified under class-1 (Plastic) in table 21.

c) Class 3 [Semi-Compact]:-

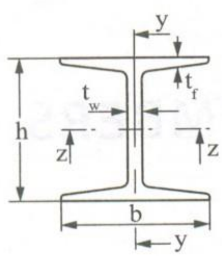
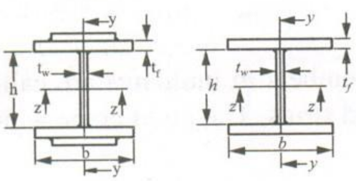
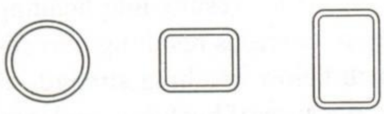
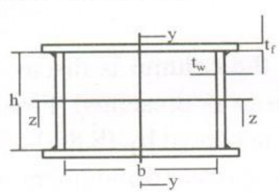
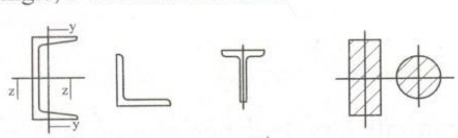
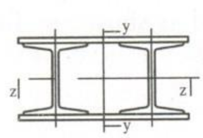
C/S in which the extreme fiber in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to tks of plate element shall be less than that specified under class-3 (Semi-Compact) but greater than that specified under class-2 in table-21.

d) Class 4 [Slender]:-

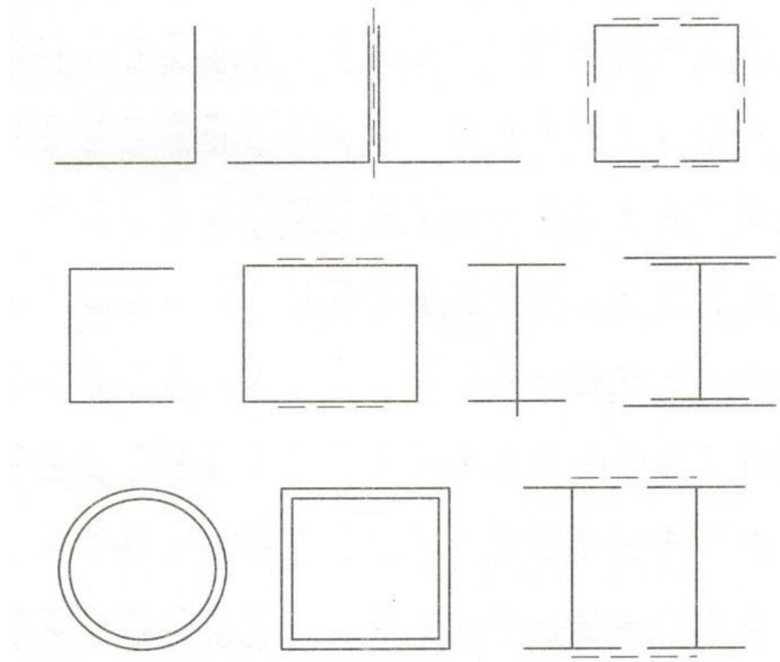
C/S in which the elements buckle locally even before reaching yield stress. The width to tks ratio of plate elements shall be greater than that specified under class-3 in table 21. In such cases, the eff. Sections for design shall be calculated either by following the provisions of IS 801 to account for the Post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

- ❖ Generally steel sections carrying axial compression fail by flexural buckling.
- ❖ The buckling strength of the compression members are affected by residual stresses, accidental eccentricities & slenderness ratio.
- ❖ To account for these factors the strength of members is subjected to axial compression defined by the above buckling classes 1,2,3&4 [Plastic, Compact, Semi-Compact & slender] given in table 10 IS 800-2007.

Table 6.1 Buckling class of cross-sections
[Refer Table 10 in IS 800]

Cross-Section **(1)**	**Limits** **(2)**	**Buckling About Axis** **(3)**	**Buckling Class** **(4)**
Rolled I-Sections 	$h/b_t > 1.2$: $t_r \le 40$ mm	z-z y-y	*a* *b*
	40 mm $< t_r \le 100$ mm	z-z y-y	*b* *c*
	$h/b_t > 1.2$: $t_t \le 100$ mm	z-z y-y	*b* *c*
	$t_f > 100$ mm	z-z y-y	*d* *d*
Welded I-Section 	$t_r \le 40$ mm	z-z y-y	*b* *c*
	$t_r > 40$ mm	z-z y-y	*c* *d*
Hollow Section 	Hot rolled	Any	*a*
	Cold formed	Any	*b*
Welded Box Section 	Generally (except as below)	Any	*b*
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z y-y	*c* *c*
Channel, Angle, *T* and Solid Sections 		Any	*c*
Built-up Member 		Any	*c*

DESIGN COMPRESSIVE STRENGTH:- [cls 7.1 IS 800-2007]



$$P_d > P_u$$

Where,

P_d = Design compressive strength of column.

P_u = External compression (or) design load.

P_d = Aexfcd

A_e = Eff. Area

f_{ed} = design compressive stress

$$f_{ed} = \frac{f_y / \gamma_{mo}}{\varphi + [\varphi^2 - \lambda^2]^{0.5}} = \frac{x f_y}{\gamma_{mo}} \leq \frac{f_y}{\gamma_{mo}}$$

Where,

$$\varphi = 0.5[1 + \alpha\,[\lambda - 0.2] + \lambda^2]$$

λ = non-dimensional eff. Slenderness ratio.

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

f_{cc} = Ruler buckling stress

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

Where,

KL/r = eff. Slender ratio (or) eff. length, KL to appropriate radius of gyration.

α = Imperfection factor given in table 7

X = Stress reduction factor [see table-8]

$$= \frac{1}{\left[\varphi + \left(\varphi^2 - \lambda^2\right)^{0.5}\right]}$$

$\lambda_{mo} =$ Partial safety factor for material strength.

KL = Depends on support condition given in table – 11

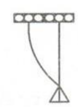
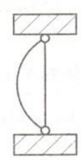
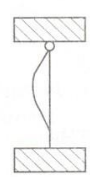
The only variable in finding the permissible comp. stress (fcd) is slenderness ratio (L/r) for the given section coming under any of the buckling class a,b,c&d.

$\therefore$ Based on the slenderness ratio, design compressive stress can be taken from table 9, 9a, 9b, 9c (or) 9d IS 800-2007.

❖ The buckling class for various section are given in Table-10 IS 800-2007 and slenderness ratio is based on eff. length given in table-11; IS 800-2007.

Table 6.2 Effective length of prismatic compression members

[Refer Table 11 in IS 800]

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

1. Determine the design axial load capacity of the column ISHB 300@ 577 N/m if the length of the column is 3m and both ends are pined.

Given:-

Section => ISHB 300@577 N/m.

L => 3m.

End condition => Both ends are pinned.

Sln:-

To find slenderness ration:-

$$\lambda = \frac{KL}{r}$$

Where,

K = 1.0 [from table-11 IS 800-2007]

$$\gamma_{xx} = 129.5\, mm$$

$$\gamma_{yy} = 54.1\, mm$$

[from steel table]

$$\therefore r_{\min} = 54.1\, mm$$

$$= \frac{1 \times 3000}{54.1}$$

$$\lambda = 55.45$$

To find design comp. stress:- [cls 7.1.2.1 IS 800-2007]

$$P_d = A_e f_{ed}$$

Where,

$$f_{ed} = \frac{f_y / \gamma_{mo}}{\varphi + [\varphi^2 - \lambda^2]^{0.5}} = \frac{x f_y}{\gamma_{mo}} \leq \frac{f_y}{\gamma_{mo}}$$

$$\varphi = 0.5 [1 + \alpha [\lambda - 0.2] + \lambda^2]$$

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

Buckling Class:- [Table-10 IS 800-2007]

Rolled steel I-section

$$\frac{h}{bf} = \frac{300}{250} = 1.2$$

tf = 10.6 < 100

About z-z axis –b

About y-y axis –c

$\therefore$ The section need to be check for buckling

Class-C

$\alpha = 0.49$ [from table-7 IS 800-2007]

$$f_{cc} = \frac{\pi^2 E}{\left(KL/r\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{(55.44)^2}$$

f_{cc} = 641.98 N/mm^2

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

λ = 0.624

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

φ = 0.79

$$\therefore \text{ Design compressive stress } f_{cd} = \frac{250/1.1}{0.79 + \left[(0.79)^2 - (0.624)^2\right]^{0.5}}$$

f_{cd} = 178.33 N/mm^2

$$\therefore P_d = 7485 \times 178.33$$

P_d = 1334.7 KN

50	183
60	168

Also referring table 9c IS 800-2007 [for buckling class-c] and λ = 55.45

f_{cd} = 174.8 N/mm^2

P_d = 7485x174.8

P_d =1308.5 KN

DESIGN OF COMPRESSION MEMBERS:-

Step:1 => Assume the design comp. stress of the member [Generally for rolled steel sections assume f_{cd} = 135 N/mm^2, for angle section assume f_{cd} = 90 N/mm^2 for builtup sections carrying large loads assume f_{cd} = 200N/mm^2

Step:2 => Reqd eff. Sectional area, $A = \frac{P_d}{f_{cd}}$

Step:3 => Select the section for the eff. Area and calculate. r_{min} [least of $\gamma_{xx} \wedge \gamma_{yy}$]

Step:4 => From the end co-ordinations, [decide the type of connection] determine the eff. Length.

Step:5 => Find the slenderness ratio and hence the design comp. stress f_{cd}

Step:6 => Find the actual load carrying capacity of the compression member.

Step:7 => If the calculated value of differs considering from the design load [P], revise the section.

1. Design a single angle strut connected to a gusset plate to carry a factored load of 180KN. Length of the strut is b/w c/c of intersection is 3m and the support condition is one end fixed & other end hinge with K=0.85

Given:-

Factored load, P = 180KN

L = 3m

K = 0.85

Sln:-

To find f_{cd}:-

Assume a design comp. stress f_{cd} = 90 N/mm^2

To find A reqd:-

Reqd Area $A = \dfrac{P_d}{f_{cd}}$

$= \dfrac{180 \times 10^3}{90}$

A = 2000mm^2

Try ISA 90x90x12mm

Properties of ISA 90x90x12mm:-

$A = 2019 mm^2$

$\gamma_{xx} = \gamma_{yy} = 27.1\, mm$

$\gamma_{uu} = 34.1\, mm, \gamma_{vv} = 17.4\, mm$

Buckling Class:-

Angle come under buckling class-c

$\dfrac{KL}{r} = \dfrac{0.85 \times 3000}{17.4}$

$\dfrac{KL}{r} = 146.55$

Refer Table 9c, IS 800-2007

140	66.2
150	59.2

f_{cd}= 61.615 N/mm^2

∴ Strength of strut = 2019 x 61.615

P_d = 232.7N/mm^2

P_d = 124.4 KN < 180KN

Revise the section:- Try ISA 130x130x8mm

$A = 2022 mm^2$

$I_{xx} = I_{yy} = 40.3$

$I_{uu} = 51.0\, mm, I_{vv} = 25.5\, mm$

$\dfrac{KL}{r} = \dfrac{0.85 \times 3000}{25.5}$

$\dfrac{KL}{r} = 100$

f_{cd} = 107 N/mm^2 [from table 9c IS 800-2007]

∴ Strength of strut = 2022x107

= 216.35 KN > 180KN

2. Design the above member when both ends are hinged.

Given:-

P = 180KN

L = 3m

Sln:-

To find A_{cd}:-

Assume a design comp. stress f_{cd} = 90N/mm^2

To find Areqd:-

$$A = \frac{P_d}{f_{cd}}$$

$$= \frac{180 \times 10^3}{90}$$

A = 2000mm^2

Try ISA 130x130x8mm

A = 2022mm^2

r_{min}= 25.5mm

$$\frac{KL}{r} = \frac{1 \times 3000}{25.5}$$

$$\frac{KL}{r} = 117.65$$

110	94.6
120	83.7

[from table 9© IS 800-2007]

f_{cd} = 86.26 N/mm^2

∴ Strength of section = 2022 x 86.26

= 174.4KN < 180KN

Hence unsafe

∴ Revise the section with r_{min} more than 25.5mm

Try ISA 150x150x10mm

A = 2903mm^2

r_{min}= 29.3 mm

$$\frac{KL}{r} = \frac{1 \times 3000}{29.3}$$

$$\frac{KL}{r} = 102.39$$

100	107
110	94.6

From table 9© f_{cd} = 104 N/mm^2

∴ Strength of section = 2903 x 104

= 301.9 KN > 180KN

Hence safe

Effective length based on connection:-

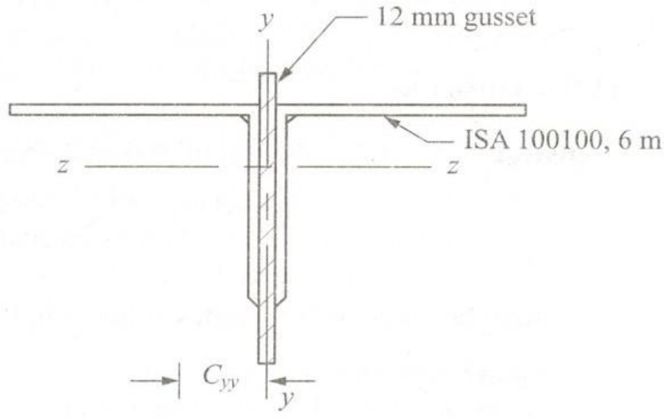
- Generally eff. Length is computed based on table-11 IS 800-2007.
- Based on connectivity, welded joints are considered to be rigid.
- For welded joints case equal to K=0.65 to 0.7

For Bolted Connection:-

a) When single bolts are provided on both sides.
b) When double bolts are provided.

K = 0.85

1. In a truss a strut which is IM long consists of 2 angles ISA 100x100x6mm. Find the design strength of the member if the angles are connected on both sides of a 12mm gusset plate using.
(i) One bolt (ii) Two bolts (iii) A rigid jt by welding



Given:-

L = 3m

2 ISA 100x100x6mm

Tks of gusset plate = 12mm

Sln:-

Section Properties of ISA 100x100x6mm:-

$A=1167mm^2$ $\qquad$ $I_{yy}= 111.3 \times 10^4 mm^4$

$r_{yy} = r_{zz} = 30.9mm$ $\qquad$ $I_{zz}= 111.3 \times 10^4 mm^4$

$C_{xx} = C_{yy} = 26.7$ mm

The local axis along the C/S is y-y & z-z as shown in fig.

r_{min} is the least of r_{yy} & r_{zz} of the composite section including 2 angles and a portion of gusset plate of size 100x12mm.

r_{zz} of the composite section is the same as r_{zz} of a single angle section.

Since the z-z axis is same for both the composite section & single angle section.

$$\therefore r_{zz} \text{ of composite section } = \sqrt{\frac{I_{zz}}{A}}$$

$$= \sqrt{\frac{111.3\times10^4}{1167}}$$

$$r_{zz} = 30.9mm$$

$$r_{yy} \text{ of composite section } = \sqrt{\frac{I_{yy}}{A}}$$

Where,

I_{yy} = M.O.I of composite section

I_{yy} = 2 [I_{yy} of one angle section +A(t/2 +cy)

$$= 2[111.3\times10^4+1167\left(\frac{12}{2}+26.7\right)^2$$

$I_{yy} = 4.72 \times 10^6 mm^4$

$$\therefore r_{yy} = \sqrt{\frac{4.72 \times 10^6}{2(1167)}}$$

$$r_{yy} = 44.97\text{mm}$$

$$\therefore r_{min} = 30.9 \text{ mm}$$

(i) One Bolt :- [Bolt ends hinged]

$$\therefore K = 1$$

$$\frac{KL}{r} = \frac{1 \times 3000}{30.9}$$

$$\frac{KL}{r} = 97.09$$

90	121
100	107

[from table 9c IS800-2007]

The member belongs to buckling class-c since it is a angle section.

$\therefore$ Refer table-9 (C)

$f_{cd} = 111.07 \text{ N/mm}^2$

Design strength of section = f_{cd} x A

= 111.07 x 2 x 1167

= 259.24 KN

(ii) Two Bolts:-

$$\therefore K = 0.85$$

$$\frac{KL}{r} = \frac{0.85 \times 3000}{30.9}$$

$$\frac{KL}{r} = 82.52$$

80	136
90	121

[from table 9c IS 800-2007]

$f_{cd} = 132.22 \text{ N/mm}^2$

$\therefore$ Design strength of section = f_{cd} x A

= 132.22 x 2 x 1167

= 308.6 KN

(iii) A rigid joint by welding:-

$$\therefore K = 0.7$$

$$\frac{KL}{r} = \frac{0.7 \times 3000}{30.9}$$

$$\frac{KL}{r} = 67.96$$

60	168
70	152

[from table 9c IS 800-2007]

$f_{cd} = 155.26 \text{ N/mm}^2$

$\therefore$ Design strength of section = f_{cd} x A

= 155.26 x 2 x 1167

= 362.38 KN

2. Determine the load carrying capacity of a column section as shown in fig. The actual length of the column is 4.5m. One end of the column is assumed as fixed and the other end hinged. The grade of steel [E250]

Given:-

L = 4.5m

Support condition = One end fixed & other end hing

$\therefore$ K = 0.8

Sln:-

The design stress 'f_{cd}' of the composite section depends on $\dfrac{KL}{r_{\min}}$ ratio and the buckling class.

Properties of ISMB 400:-

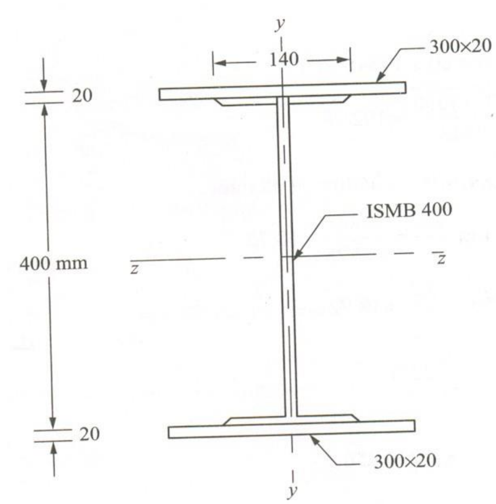
h = 400mm, bf = 140mm, tf = 16mm

tw = 8.9mm, r_{yy} = 28.2mm, r_{xx} = 161.5mm

I_{zz} = 20458.4 x $10^4 mm^4$, I_{yy} = 622.1 x $10^4 mm^4$

r_{min} is least of r_{zz} (or) r_{yy}

where, $r = \sqrt{\dfrac{I}{A}}$



I_{zz} of Composite section:-

$$I_{zz} = 20458.4 \times 10^4 + \frac{300 \times 20^3}{12} + \left[300 \times 20 \times (430 - 220)^2\right] + \frac{300 \times 20^3}{12} + \left[300 \times 20 \times (220 - 10)^2\right]$$

$$I_{zz} = 734.18 \times 10^6 \, mm^4$$

I_{yy} of Composite section:-

$$I_{yy} = 622.1 \times 10^4 + \frac{20 \times 300^3}{12} + \left[20 \times 300 \times (150 - 150)^2\right] + \frac{20 \times 300^3}{12} + \left[20 \times 300 \times (150 - 150)^2\right]$$

$$I_{yy} = 96.\ 221 \times 10^6 \ mm^4$$

$$\therefore r_{zz} = \sqrt{\frac{I_{zz}}{A}}$$

A = 7846 + (2x300x20)

A = 19846mm^2

734.18×10^6

94.152×10^6

19846

$$\therefore r_{zz} = \sqrt{¿¿¿}$$

$r_{zz} = 192.34mm$

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

$$= \sqrt{\frac{96.221 \times 10^6}{19846}}$$

$r_{yy} = 69.63mm$

$\therefore r_{min} = 69.63mm$

To find slenderness ratio:-

$$\frac{KL}{r_{min}} = \frac{0.8 \times 4500}{69.63}$$

$$\frac{KL}{r_{min}} = 51.7$$

The buckling class of the built up section based on table-10 IS 800-2007. Tks of flange is 16+20 = 36mm < 40mm

$\therefore$ Along buckling about z-z axis is buckling class 'B' and buckling about y-y axis, therefore I_{yy} is less than I_{zz}

50	183
60	168

From table 9 (C) IS 800-2007

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

$\therefore$ Load carrying capacity of the section = 180.45 x 19846

= 3581.2 KN

$$\text{Safe working load} = \frac{3581.2}{1.5}$$

= 2387.5 KN

3. Design a column 4m long to carrying a factor load of 6000KN column is effectively held at both ends and restrain in direction at one end. Design the column using beam section ISHB 450 @ 907 N/m

Given:-

L = 4m

Factor Load = 6000KN

One end fixed and other end hinged

$\therefore$ K = 0.8

Sln:-

The given section ISHB is checked for the axial load carrying capacity

$\therefore P_d = A \times f_{cd}$

Properties of ISHB450 @ 907 N/m:-

$A = 11789mm^2$

$I_{xx} = 40349.9 \times 10^4 mm^4$, $I_{yy} = 3045 \times 10^4 mm^4$

Assuming $f_{cd} = 200N/mm^2$

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

$= 30000mm^2$

$\therefore$ Area deficit = 30000-11789

= 18211mm^2

Selecting 20mm tk plate @ top 2 bottom flange portion.

2(20xb) = 18211

b = 455.275mm ¿ 500mm

$\therefore$ Assume the size of plate @ as 500x20mm @ top and bottom.

I_{zz} of composite section:-

$$I_{zz}=40349.9\times 10^4+\frac{500\times 20^3}{12}+\left[500\times 20\times(480-245)\right]^2+\frac{500\times 20^3}{12}+\left[500\times 20(245-10)\right]^2$$

$$I_{zz}=1508.66\times 10^6\ mm^4$$

I_{yy} of composite section:-

$$I_{yy}=3045\times 10^4+\frac{20\times 500^3}{12}+\left[20\times 500\times(250-250)\right]^2+\frac{20\times 500^3}{12}+\left[20\times 500\times(250-250)\right]^2$$

$$I_{yy}=447.12\times 10^6\ mm^4$$

Check for over hang:-

The over hang length is limited to 'lbt' over hang length = 500-250

= 250mm < 16(20) = 320mm

$$\therefore r_{zz}=\sqrt{\frac{I_{zz}}{A}}$$

A = 11789+(2x500x20)

A = 31789mm^2

$$r_{zz}=\sqrt{\frac{1508.66\times 10^6}{31789}}$$

r_{zz} = 217.85 mm

$$r_{yy}=\sqrt{\frac{I_{yy}}{A}}=\sqrt{\frac{447.12\times 10^6}{31789}}$$

r_{yy} = 118.60mm

$$\therefore r_{min}=118.60\ mm$$

To find slenderness ratio:-

$$\frac{KL}{r_{min}}=\frac{0.8\times 4000}{118.60}$$

$$\frac{KL}{r_{min}}=26.98$$

20	224
30	211

From table 9 © IS800-2007

f_{cd} = 214.926 N/mm^2

∴ Design load carrying capacity of the section = $f_{cd} \times A$

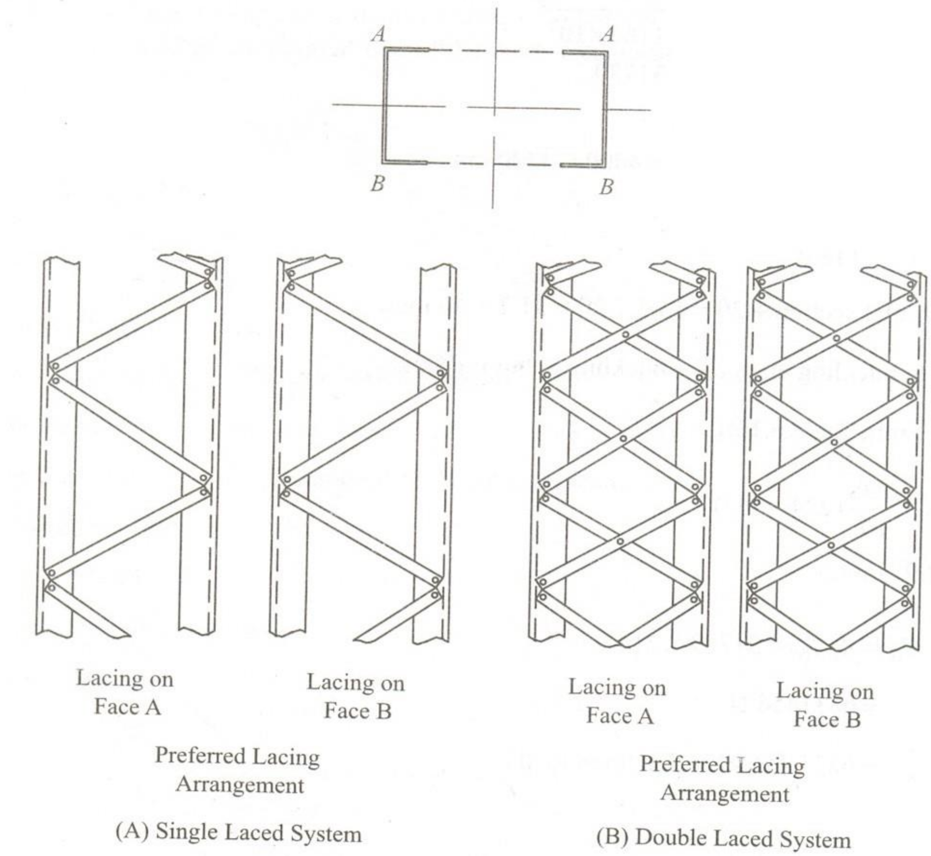
$= 214.926 \times 31789$

$P_d = 6832.3$ KN > 6000KN

Hence the assume section is safe.

Laced & Battened Columns:-

[cls 7.6 IS 800-2007] [cls 7.7 IS 800-2007]



- ❖ Lacings and battens are provided to establish a built up section. [generally using channels and angles]
- ❖ They do not increase the area of the section, but increase the mini. Radius of gyration [achieve by placing the members away from principle axis]
- ❖ The commonly used lateral systems are lacings or latticings battering.

Design of Laced Columns:-°

The general guide lines reqd are

1. The latticing system shall be uniform throughout.
2. In single lacing system, the direction of lattices on the opposite face should be the shadow of the other and not mutually opposite.
3. In bolted construction, the mini width of lacing bars shall be 3 times the nominal dia of bolts.
4. Tks of flat lacing bars shall not be less than 1/140 th of its eff. Length for single lacing & 1/16th of eff. Length for double lacings.
5. Lacing bars shall be inclined at 40° to 70° to the axis of the built up members.

6. The distance b/w the two main member should be kept, such that

   $r_{yy} > r_{zz}$ where,

   r_{yy} = Radius of gyration about the weaker axis.

   r_{zz} = Radius of gyration of stronger axis [major axis] of the individual members.
7. Maxi. Spacing of lacing bars shall be such that, the maxi. Slenderness ratio of the main member b/w consecutive lacing connections is not greater than 50 (or) 0.7 times of the unfavourable slenderness ratio of the member as a hole.
8. The lacing shall be design to resist a transverse shear, 'V_t = 2.5% P' [Axial load of column] If there are two transverse parallel systems then each system has to resist a shear force of '$V_t/2$'
9. If the column is subjected to bending also the shear due to bending moment has to be added with 'V_t'
10. The eff. Length of a single laced system is equal to the length b/w the inner faster ness. For welded joints and double lacing system, Effectively connected at the intersection, eff. Length is taken as 0.7 times the actual length.
11. The slenderness ratio KL/r for lacing shall not exceed 145. [$\because \lambda_{max} = 145$]
12. The eff. Slenderness ratio of laced columns shall be taken as 1.05 times the actual maxi. Slenderness ratio in order to account for shear deformation effects.

Design of Batten column:-

1. Similar to lacings, battens are design for transverse force V_t = 2.5% P
2. The batten plates should be symmetrical & spaced uniformly throughout. The eff. Slenderness ratio is 1.1 times the maxi. Actual slenderness ratio of the column to account for shear deformation.
3. Spacing shall be such that slenderness ratio of the column in any part is not greater than 50 and not greater than 0.7 times the slenderness ratio of the member as a hole about z-z axis.
4. The design shear and moment for the batten plates is given by the following relations.

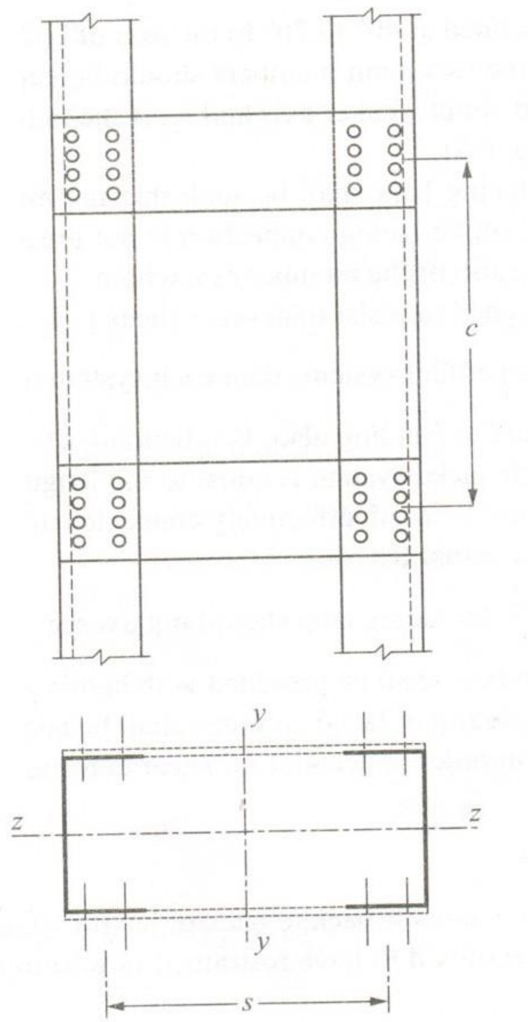
$$V_b = \frac{V_c C}{N_s}$$

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

Where,

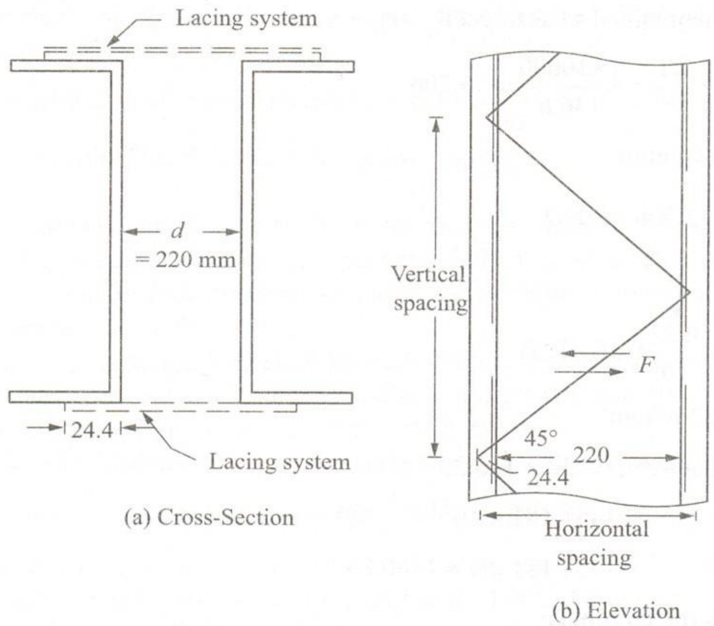
C = c/c distance along longitudinal direction.

N = No. of batten plates.



1. Design a laced column with 2 channels back to back of length 10m to carry an axial

factored load of 1400KN. The column may be assume to have restrain in position but not in direction at both ends. [Hinged ends]



(a) Cross-Section

(b) Elevation

Given:-

P = 1400KN, L = 10m, K = 1

Condition Both ends are hinged.

Sln:-

Assume f_{cd} as 135N/mm^2

To find A_{reqd}:-

$$A_{reqd} = \frac{P}{f_{cd}}$$

$$= \frac{1400 \times 10^3}{135}$$

$A_{reqd} = 10370.37mm^2$

$\therefore$ Area of each channel reqd $= \frac{10370.37}{2}$

$= 5185.2mm^2$

Try 2-ISMC 350 @ 421 N/m

$A = 5366mm^2$

$W = 421N/m$; $I_{zz} = 10008.0x10^4mm^4$; $I_{yy} = 430.6x10^4mm^4$

$r_{zz} = 136.6mm$; $r_{yy} = 28.3mm$; $c_{yy} = 24.4mm$

The lacing system is provided such that $r_{yy} > r_{zz}$. This is achieve by providing sufficient spacing b/w the two channels.

$$\therefore r_{\min} = r_{zz}$$

r_{zz} of combined section = r_{zz} of individual channel section.

$\therefore$ r_{zz} of combined section = 136.6mm

Slenderness ratio:-

$$\frac{KL}{r_{\min}} = \frac{1 \times 10000}{136.6}$$

$$\frac{KL}{r_{\min}} = 73.206$$

For laced columns the maxi. Slenderness ratio can be increased by 5%

$$\therefore \quad \frac{KL}{r_{\min}} = 73.206 \text{ x } 1.05$$

$= 76.86$

70	152
80	136

From table 9 © IS800-2007

From table 10 the builtup section comes under the buckling class 'C'

$f_{cd} = 141.024N/mm^2$

$\therefore$ Load carrying capacity of column, $P_d = f_{cd}$ x A

$= 141.024 \text{ x } 5366 \text{ x } 2$

$P_d = 1513.46$ KN > 1400KN

$\therefore$ Assumed section 2ISMC 350 is sufficient.

Design of Lateral system:- [Lacing System]

The clear distance b/w the two channels is arrived based on the condition r_{yy} ¿ r_{zz}

$I_{yy} = I_{zz}$

I_{zz} of composite section is twice the I_{zz} of an individual channel section.

$$I_{zz}(comp) = 2I_{zz}(individual)$$

$= 2 \text{ x } 10008 \text{ x } 10^4$

$I_{zz} = 2.0016 \text{ x } 10^8mm^4$

I_{yy}:-

I_{yy} of composite section is found for the 2 channels from the centroidal Axis

$$I_{yy(comp)} = 2\left[I_{yy_{(self)}} + Ah^2\right]$$

(one channel)

$$= 2\left[430.6\times10^4 + 5366\times\left(\frac{d}{2}+24.4\right)^2\right]$$

$I_{yy} = I_{zz}$

$$2\left[430.6\times10^4+5366\left(\frac{d}{2}+24.4\right)^2\right]=2.0016\times10^8$$

$$2\left[430.6\times10^4+5366\left(\frac{d^2}{4}+595.36+24.4d\right)\right]=2.0016\times10^8$$

$$2\left[430.6\times10^4+1341.5d^2+3194701.76+130930.4d\right]=2.0016\times10^8$$

$$2\left[1341.5d^2+130.930\times10^3 d+750.07\times10^4\right]=2.0016\times10^8$$

$$1341.5d^2+130.93\times10^3\ d+750.07\times10^4=100.08\times10^6$$

$$1341.5d^2+130.93\times10^3\ d=92.579\times10^6$$

d = 218

$\therefore$ d = 220mm

Assume the lacings to be provided at 45º to the horizontal.

Horizontal Spacing = d+24.4+24.4

= 220+48.8

= 268.8mm

Hori spacing = 268.8mm

Vertical spacing = 2[horizontal spacing]

= 2x268.8

= 537.6mm

The limit for slenderness ratio for each channel b/w the lacings vertically is 50

$\therefore$ Slenderness ratio for vertical spacing

[Each Channel] $= \frac{KL}{r}$

$= \frac{1\times537.6}{28.3}$

= 18.99 < 50

Transverse shear to be resisted by each lacing system is 2.5% of axial load. [Clause 7.6.6.1 IS 800-2007]

Load = $\frac{2.5\times1400}{100}$

= 35KN

$\therefore$ Transverse shear to be resisted by each lacing bar is 17.5KN

$$L=\frac{268.8}{\cos 45^\circ}$$

L = 380.14mm

Mini tks of lacing bar = $\frac{L}{40}$

$= \frac{380.14}{40}$

$= 9.5mm$

$\therefore$ Provide 10mm tk flat plates for lacing bar. Assume dia of bolt as 20mm,
width of lacing bar = 3xdia

= 3x20

b = 60mm

$\therefore$ The assumed lacing bar is 60x10mm

Connection for lacing Bar:- [20mm dia]

1. Strength of bolt is single shear:- [cls 10.3.3 IS 800-2007]

$$V_{dsp} = \frac{V_{nsp}}{\gamma_{mb}}$$

$$V_{nsp} = \frac{fu}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

$$n_n = 1, \; n_s = 0$$

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

V_{nsp} = 56.59 KN

V_{dsp} = 45.272 KN

2. Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}} = \frac{2.5 k_b d_t f_u}{\gamma_{mb}}$$

$$k_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1$$

$$e = 1.5d_o = 33 \; mm \simeq 40 \; mm$$

$$p = 2.5 \, d = 50 \; mm \simeq 60 \; mm$$

k_b = 0.606, 0.659, 0.975, 1

$\therefore$ k_b = 0.606

$$= \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25}$$

V_{dbp} = 99.384KN

$\therefore$ The strength of bolt value = 45.272KN

$\therefore$ No. of bolts = $\frac{17.5}{45.27} = 0.39 \, N/N_o$

$\therefore$ Provide one 20mm φ bolt on each side of connection.

Strength of lacing bar:- [60x10mm]

Slenderness ratio of lacing bar = $\frac{KL}{r}$

$$= \frac{1 \times 380.14}{\gamma_{min}}$$

$$r_{min} = \sqrt{\frac{I_{zz}}{A}} \text{ (or) } \sqrt{\frac{I_{yy}}{A}}$$

$$I_{zz} = \frac{60 \times 10^3}{12} = 5000\, mm^4$$

$$I_{yy} = \frac{60^3 \times 10}{12} = 180 \times 10^3\, mm^4$$

$$= \sqrt{\frac{5000}{600}}$$

$$r_{min} = 2.88$$

$$= \frac{1 \times 380.14}{2.88}$$

Slenderness ratio = 131.99 < 145 [cls 7.6.6.3 IS 800-2007]

130	74.3
140	66.2

From table 9© IS 800-2007

f_{cd} = 72.68 N/mm^2

Load Carrying capacity of section = 72.68 x 60 x 10

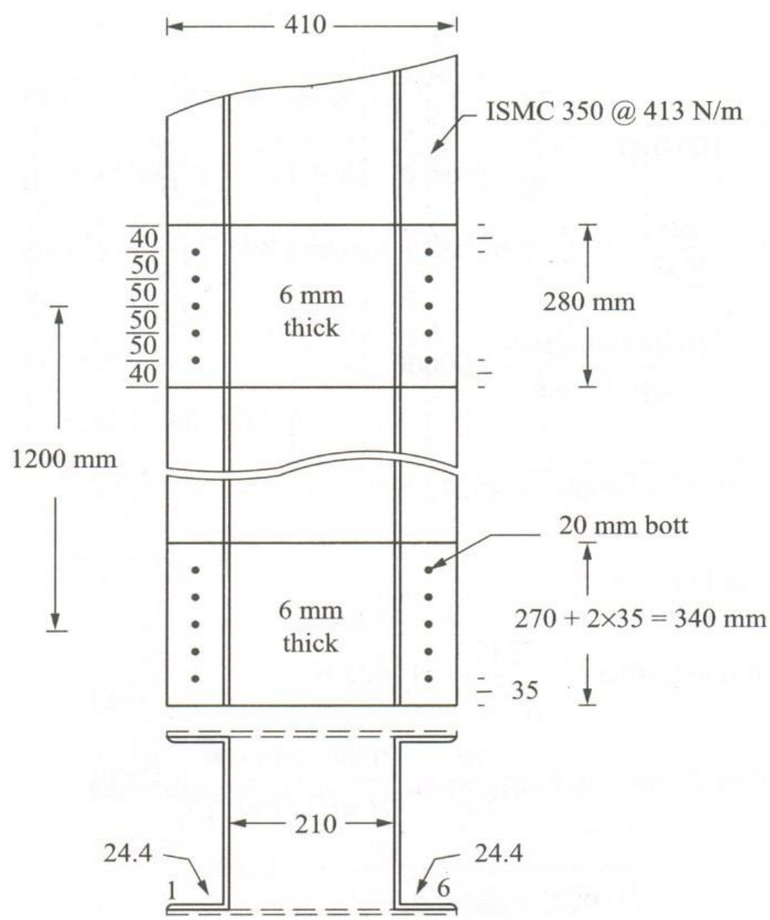
p_d = 43.61KN > 17.5KN

Hence the lacing system is safe.

2. Design the above built up column using battens as lateral system. The sections selected are 2ISMC350@413N/m with clear spacing of 220mm.

[$\therefore$ The section is design as per the previous problem]

Sln:-



d = 220mm

Slenderness ratio = $\frac{KL}{r} = 73.21$

Slenderness ratio of battens is 1.1 times $\frac{KL}{r}$

= 73.21 x 1.1

= 80.53

C/C horizontal distance b/w the batten plate S = d+24.4+24.4

S = 268.8mm

If 'C' is the spacing of the battens. The value of 'C' is found based the relation C/rmin<50 [The slenderness ratio of each channel b/w 2 battens plates is limited to 50]

C = 28.3 x 50

C = 141.5mm

∴ Assume C = 1200mm

Transverse shear $V_t = \frac{2.5 \times 1400}{100}$

V_t = 35KN

As per clause 7.7.2.1 IS 800-2007, shear to be resisted by Batten plate

$$V_b = \frac{V_t C}{NS}$$

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

Where,

N = 2

$$\therefore V_b = \frac{35 \times 1200}{2 \times 268.8}$$

V_b = 78.125KN

$$\therefore M = \frac{35 \times 1200}{2 \times 2}$$

= 105.00KN mm

M = 10.5KN.m

Width & tks of Batten Plate:-

The end batten plate should have width (depth) greater than S (268.8mm) $\therefore$ Provide width of batten plate as 270mm width of intermediate batten plate should be greater than ¾ width of end batten plate.

$\therefore$ Provide width of intermediate batten plate = $\frac{3}{4} \times 270$

= 202.5mm

$\therefore$ Width as 210mm

Tks of batten plate should be greater than S/50

$$t = \frac{268.8}{50}$$

t = 5.376mm

$\therefore$ Provide the tks of 6mm

Check for stresses in Batten Plate:-

Shear stress $= \frac{V_b}{A}$

$$= \frac{78.125 \times 10^3}{210 \times 6}$$

= 62 N/mm^2

Permissible shear stress $= \frac{f_y}{1.1\sqrt{3}}$

= 131.21 N/mm^2

Shear Act stress < Permissible shear stress

Actual bending stress $\sigma_b = \frac{M}{Z}$

$$= \frac{10.5 \times 10^6 \times 6}{td^2}$$

$$= \frac{10.5 \times 10^6 \times 6}{6 \times 210^2}$$

$= 38.09\ N/mm^2$

Permissible bending stress = $\frac{f_y}{1.1}$

$= 227.27\ N/mm^2$

Act bending stress < Permissible bending stress

Hence the breadth of the section has to be increased.

Providing an edge of 35mm on both sides over all depth of section is 210+35+35 = 280mm

To find Actual bending stress:-

Actual bending stress = $\frac{10.5\times10^6\times6}{(280)^2\times6}$

$= 133.9\ N/mm^2$

Shear stress $= 46.5\ N/mm^2$

$\therefore$ Provide intermediate plate of size 280 x 6mm and end batten plate of size [270+70=340mm] 340 x 6mm

Connections for intermediate batten plate:-

- Bolts are placed along a vertical line on the batten plate.
- Force in the extreme bolt should be less than the bolt value for the connection to be safe.

Assume 20mm dia bolt

$\therefore$ Bolt value = 45.27 KN

The transverse shear acting on a connection = 78.125KN

$\therefore$ No. of bolts = $\frac{78.125}{45.27}$

= 1.72 ¿ 3 Nos.

Since moment also acts on the connection provide 3 Nos of bolts.

The force due to moment on extreme.

Bolt $F_m = \frac{M\gamma}{\sum \gamma^2}$

$= \frac{10.5\times10^6\times10^5}{(105^2+105^2)}$

$F_m = 50KN$

$F_s = \frac{F}{n} = \frac{78.125}{3}$

$= 26.04\ KN$

Resultant force = $\sqrt{F_m^2 + F_s^2}$

$= \sqrt{50^2 + 26.04^2}$

= 56.57 KN > 45.27 KN

$\therefore$ 3 bolts are not sufficient we have to increase the no. of bolts.

$\therefore$ Assume 5 Nos of bolt along the vertical line.

Force due to moment $F_m = \frac{M\gamma}{\sum \gamma^2}$

$$= \frac{10.5\times10^6\times10^5}{2\left(105^2\right)+2\left(105^2\right)}$$

$= 25KN$

$$F_s = \frac{78.125}{5}$$

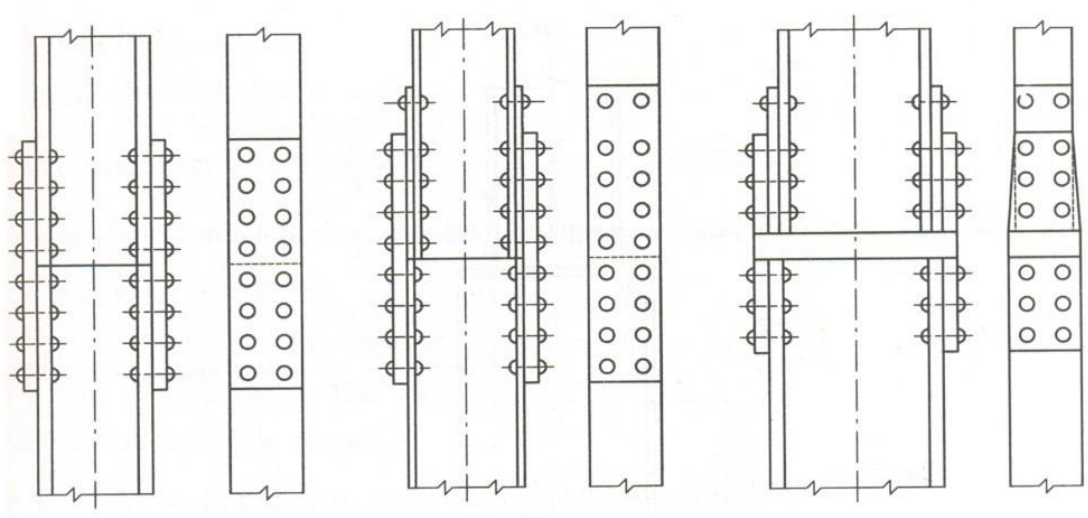
$= 15.62$ KN

$$\text{Resultant Force} = \sqrt{25^2+15.62^2}$$

$= 29.48$ KN < 45.27 KN

Hence 5 Nos of 20mm dia bolts are provided in both sides.

COLUMN SPLICE:-

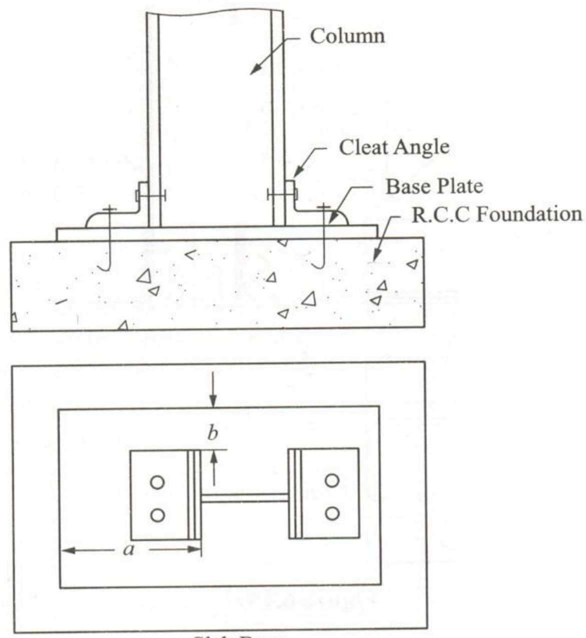


(a) (b) (c)

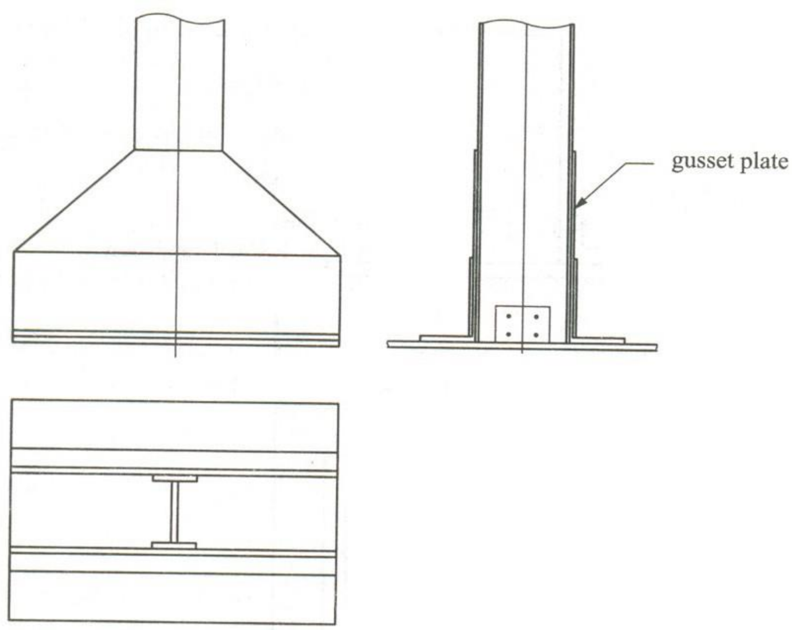
- When two pieces of the section are connected together to get the reqd length of column, is called a column splice.
- In a building the section of column may be change from storey to storey (for economy) and in cases when the length reqd exceeds standard size of the section available.

COLUMN BASES:-

(i) Slab Base, (ii) Gussetted Base, (iii) Grillage Foundation



Slab Base



(i) Slab Base:-

- ➢ It is used in columns carrying small loads. [Approximately upto 1000KN]
- ➢ The load is transferred to the base plate through bearing, with the help of cleat angles.

(ii) Gussetted Base:-

- ➢ Gussetted Base when the column carries heavy load [App. 1000-2000KN]
- ➢ The column is connected to the base plate using gusset plates and cleat angles.
- ➢ The load is transferred to the base party to bearing & party to gusset.

Design of slab base (or) simple base:-

1. The bearing strength of concrete is $0.45f_{ck}$
2. Area of base plate reqd is $\dfrac{P_u}{0.4f_{ck}}$

   Assume the size of plate such that the projections of base plate from the column on both sides (a&b) are kept more or less same.
3. Find the base intensity pressure

$$w = \frac{P_u}{Areaofbaseplate}$$

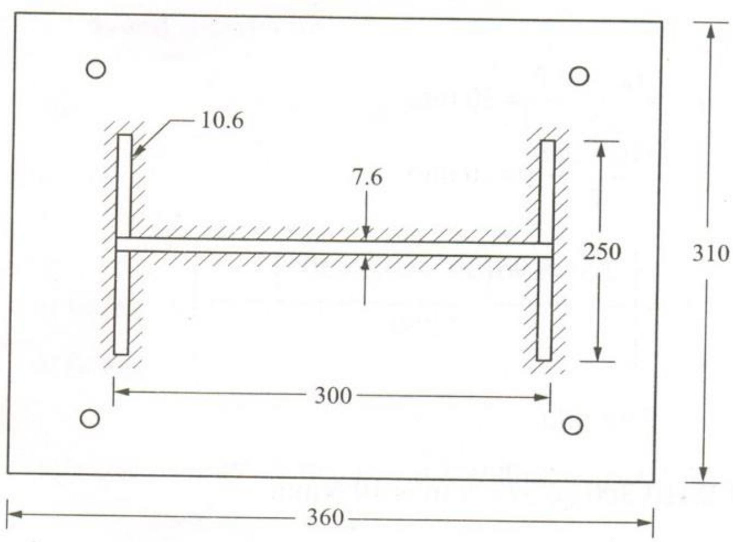
4. Min thickness of base plate reqd. is

$$t_s = \left[\frac{2.5w\left(a^2 - 0.3b^2\right)\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$

5. Connection: If bolted connection is provided 2 cleat angles of size ISA 65x65x6mm are used which are connected with 20mm dia of bolts.

   If welded connection is used the size of weld is arrived based on the length of weld available alround the column.
6. The Base Plate is connected to the foundation concrete using 4 Nos of 20mm dia and 300mm long Anchor bolts.

0. Design a slab base for a column ISHB 300@577 N/m which is subjected to factored axial load of 1000KN use M20 concrete for the concrete pedestal.



Given:-

ISHB 300@577 N/m

$P_u = 1000KN$

$f_{ck} = 20N/mm^2$

Sln:-

1. Bearing stress in concrete $\sigma_{bc} = 0.45 f_{ck}$

$= 0.45 \times 20$

σ_{bc} = 9 N/mm^2

2. Area of base plate reqd, $A = \dfrac{P_u}{0.45 f_{ck}}$

$= \dfrac{1000 \times 10^3}{9}$

A = 111.11 x 10^{3}mm^2

Assuming equal projection on both sides with a = b =30mm, size of base plate assumed is 310x360mm.

∴ Area Provided = 111.6x10^{3}mm^2

3. Pressure intensity @ base $w = \dfrac{P_u}{Area of base plate}$

$= \dfrac{1000 \times 10^3}{111.6 \times 10^3}$

w = 8.96 N/mm^2 < 9N/mm^2

4. Mini. Tks of base plate $= \left[\dfrac{2.5w\left(a^2 - 0.3b^2\right)\gamma m_o}{f_y}\right]^{0.5}$

$= \left[\dfrac{2.5 \times 8.96\left(30^2 - 0.3 \times 30^2\right) \times 1.1}{250}\right]^{0.5}$

$(t_s) 7.87\, mm < 10.6\, mm (t_f)$

∴ Provide tks of base plate as 12mm

5. Bolted Connection:-

Provide 2 cleat angle ISA 65x65x6mm connected using 20mm dia ‘J’ anchor bolts for a length of 300mm.

2) Design the above problem using welded connection

Sln:-

1. Bearing stress in concrete $\sigma_{bc} = 0.45 f_{ck}$

= 0.45 x 20

σ_{bc} = 9 N/mm^2

2. Area of base plate reqd, $A = \dfrac{P_u}{0.45 f_{ck}}$

$= \dfrac{1000 \times 10^3}{9}$

A = 111.11 x 10^{3}mm^2

Assuming equal projection on both sides (for economy) size of plate adopted is 310x360mm.

∴ Area Provided = 111.6 x 10^{3}mm^2

3. Pressure intensity @ base $w = \dfrac{P_u}{Actual area}$

$= \dfrac{1000 \times 10^3}{111.6 \times 10^3}$

$w = 8.96\ N/mm^2 < 9N/mm^2$

4. Mini. Tks of base plate $= \left[\dfrac{2.5w\left(a^2-0.3b^2\right)\gamma m_o}{f_y}\right]^{0.5}$

$$= \left[\frac{2.5\times 8.96\left(30^2-0.3\times 30^2\right)\times 1.1}{250}\right]^{0.5}$$

Tks of flange = 10.6mm

$\therefore$ Provide tks of base plate as 12mm.

5. Welded Connection:-

Providing fillet weld alround the I-section, length available is

Length available = 4(250)-2(7.6)+2(300)-2(10)

1w = 1563.6mm

Design strength of weld:-

Providing a weld of grade 410 $N/mm^2 (f_u)$

Stress x Area

↓ ↓

$$\text{Design strength of weld} = \frac{f_u/\sqrt{3}}{\gamma m_w}\times(lw\times t)$$

$$= \frac{410/\sqrt{3}}{1.25}\times(1563.6\times 0.7)$$

For the available length, the size of weld reqd is found.

$1000 \times 10^3 = 189.37 \times 0.73 \times 1563.6$

$$S = \frac{1000\times 10^3}{189.37\times 0.7\times 1563.6}$$

S = 4.82mm

Provide 6mm fillet weld alround the column provide 20mm dia ‘J’ anchor bolts at the 4 corners of the base plate with length 300mm.

Gusseted Base:-

- When the load on the column is higher gusset plates are provided along the flanges of the C/S.
- The load is transferred by bearing through the base plate and also partly through the gusset plate.

Design Procedure:-

1. Area of base plate, $A = \dfrac{P_u}{0.45 f_{ck}}$
2. Assume various members of gusset base
   (i) Tks of gusset plate assumed as 16mm
   (ii) Size of gusset angle is assume such that the vertical leg can accomadate 2 bolts in one vertical line. The other leg is assume such that 1 bolt can be provided.

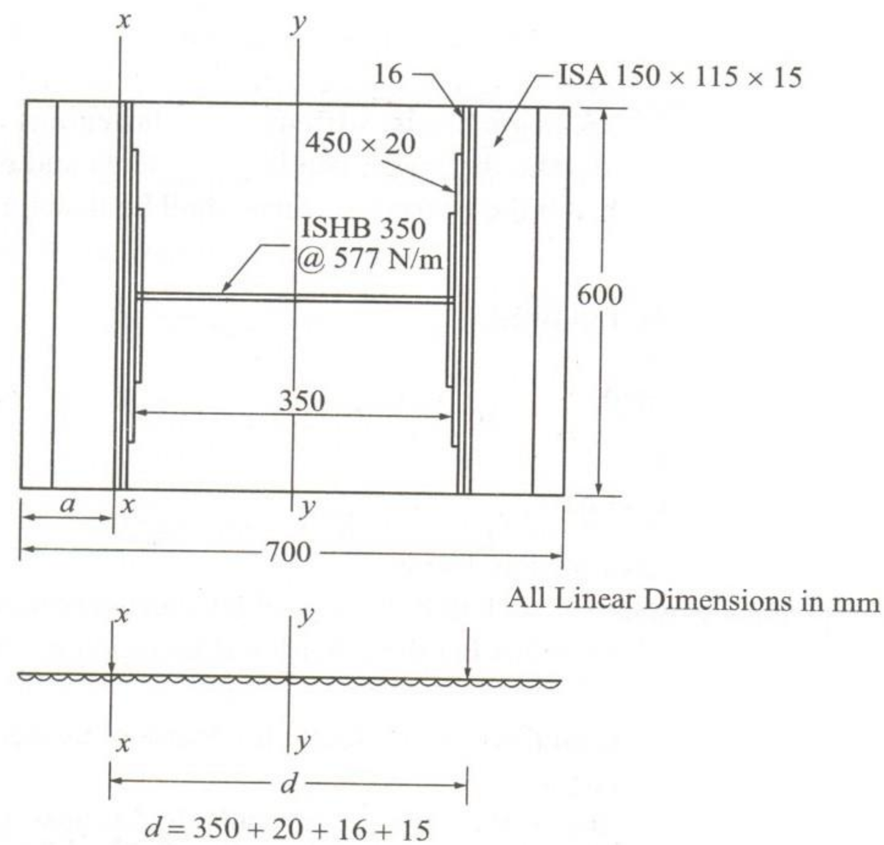
(iii) The tks of angle is kept approximately equal to the tks of gusset plate.

3. Width of gusseted base is kept suc that it will just project the outside the gusset angle and hence

$$\text{Length} = \frac{\underline{Areaofplate}}{width}$$

4. The load is assumed to be transferred 50% by bearing and 50% by fasteners.
5. Tks of base plate is computed by flexural strength at the critical sections.

1. Design a gusseted base for a column ISHB 350@710N/m with 2 plates 450 x 20mm carrying a factored load of 3600KN. The column is to be supported on concrete pedestals to be built with M20 concrete.



ASSIGNMENT-I

1. Draw the various possible forces in bolted connections:-

a) Shear Plane on thread:-

b) Two planes subject to shear:-

c) Bolts in Direct Tension:-
d) Bolts resisting pure moment:-
e) Bolts subject to shear and tension:-

2. Beam ISLB 500 at 750 N/m carries total factored ude of 300KN. It is supported on columns ISHB 300 at 630N/m at each end. The connection is made using M20 bolts of grade 4.6 and steel Fe410. Design the connection.

Given:-

ISLB 500

D = 500mm, bf=180mm, tf=14.1mm, tw=9.2mm

ISHB 300

D = 300mm, bf=250mm, tf=10.6mm

Sln:-

Try angle 100x100x8mm one on each side of beam.

a) Angle connecting beam web:-

The connecting bolts will be in double shear

Strength of bolts in double shear = $2\times\left[0.462 f_u (n_n A_{n6})\right]$

= $2\times0.462\times400\times245 / 1000$

= 90.55KN

Strength of bolts in bearing = $2d_{tpfu}$

= 2x20x9.2x410

= 147KN

Least bolt value = 90.55KN

No. of bolts = $\frac{Reaction @ each end}{bolt value}$

= $\frac{300/2}{90.55}$

Nos. of bolt = 1.7 says 2 Nos.

Provide 2 bolts @ 50mm pitch with edge distance of 40mm.

Mini. Length of angle reqd = 2x40+50

= 130mm

b) Angle connecting column flange:-

Connecting bolts will be in single shear and bearing on 8mm tks of angle

Strength of bolts in single shear = $0.462 f_u (n_n A_{nb})$

= 0.462x400x245

= 45.3KN

Strength of bolts in bearing = $2 d t_p f_u$

= 2x20x8x410

= 31KN

Least bolt value = 45.3KN

No. of bolts = $\frac{300/2}{45.3}$

= 3.31 says 4Nos.

Provide 2 bolts on each side of flange check the tks of the angle:-

Factored shear resistance = $\frac{A_v \cdot f_{yw}}{\sqrt{3}\gamma_{mo}}$

= 0.525 $A_v f_{yw}$

= 0.525x2x100x8x250

= 209.9KN > 150KN

Hence safe

ASSIGNMENT-II

1. Design a tension member to carry a factored load of 340KN use 20mm dia of black bolt and gusset plate of 8mm thickness.

Given:-

Factored load = 340KN

d = 20mm, d_o = 22mm

Thickness of gusset plate = 8m

Sln:-

No. of bolts = Tu/V

To find A_g:-

$$A_g = \frac{1.1 \times T_u}{f_y}$$

$$= \frac{1.1 \times 340 \times 10^3}{250}$$

A_g = 1496mm^2

Try ISA 100x100x8mm

A_g =1539mm^2

$\gamma_{xx} = \gamma_{yy} = 30.7\,mm$

BOLT VALUE:-

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

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

$$= \frac{f}{\sqrt{3}} \left[\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}} \right]$$

n_n = 1, n_s =0

$$A_{nb} = \frac{0.78 \times \pi \times 20^2}{4}$$

= 245mm^2

$$= \frac{400}{\sqrt{3}} \left[\frac{1 \times 245.04}{1.25} \right]$$

V_{dsb} = 45.27KN

(ii) Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$= \frac{2.5 k_b d_t f_u}{\gamma_{mb}}$$

Assume,

e = 1.5d_o = 33mm ¿ 40mm

p = 2.5d = 50mm ¿ 60mm

$$k_b = \frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1$$

= 0.606, 0.66, 0.959, 1

Take k_b = 0.606

$$= \frac{2.5\times0.606\times20\times8\times410}{1.25}$$

V_{dbp} = 79.5KN

∴ Design strength of bolt value = 45.27KN

$$\therefore \text{No. of bolts} = \frac{340}{45.27}$$

= 7.51 ¿ 8Nos

∴ Provide 20mm φ bolt of 8 Nos.

Check for strength of section:-

(i) Design strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \frac{\left(100 - \frac{8}{2}\right)\times250}{1.1}$$

T_{dg} = 349.04KN

(ii) Design strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{l_c}\right) \le \frac{f_y \gamma_{mo}}{f_u \gamma_{ml}} \ge 0.7$$

w = 100

w_1 =50

b_s = 100+50-8

b_s = 142mm

L_s = 460mm

$$\beta = 1.4 - 0.076\left(\frac{100}{8}\right)\left(\frac{250}{410}\right)\left(\frac{142}{460}\right) \le \frac{250\times1.1}{410\times1.25} \ge 0.7$$

= 1.22 ¿ 1.44 ¿ 0.7

∴ β = 1.22

$$A_{nc} = \left(100 - 22 - \frac{8}{2}\right)8 = 592\, mm^2$$

$$A_{go} = \left(100 - \frac{8}{2}\right)8 = 768\ mm^2$$

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.22 \times 768 \times 250}{1.1}$$

T_{dn} =387.7KN > 340KN

3) Design strength of plate against block shear:- [cls 6.3.4 IS800-2007]

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

(or)

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

Where,

$$A_{vg} = [40 + 7(60)]^8 \quad = 3680 mm^2$$

$$A_{vn} = [460 - 7.5(22)]^8$$

$= 2360 mm^2$

$$A_{tg} = 50 \times 8 = 400\ mm^2$$

$$A_{tn} = \left(50 - \frac{22}{2}\right)^8$$

$= 312 mm^2$

$$T_{db1} = \frac{3680 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 312 \times 410}{1.25}$$

= 574.97KN

$$T_{db2} = \frac{0.9 \times 2360 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.1}$$

= 493.13KN > 340KN

Design strength of section is against yielding rupture & block shear as greater than the external load of 340KN

∴ The assumed section ISx100x100x8mm is safe.

2. Explain different modes of failure of tension member.

1. Cross section yielding:-

Generally a tension member without bolt holes, can resist loads upto the ultimate load without failure. But such a member will deform in the longitudinal direction considerably nears 10% to 15% of its original length before fracture. At such a large deformation a structure become in serviceable.

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

2. Net section Rupture:-

A tension member is after connected to the main of other members by bolts or welds, when connected using bolts tension members have holes & hence reduced cross section being referred to the net area.

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

(iii) Block shear failure:-

Originally observed is bolted shear connection at sloped beams ends. Block shear is now reqd as potential failure of made the ends of axially load tension member also.

In this failure made the failure of member occurs along a parts including fussion on one plates & shear on ¿ lr plane along the fasterners as shown in fig.

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 f_u A_{vn}}{\gamma_{ml}}$$

(or)

$$T_{db2} = \frac{0.9 f_y A_{vn}}{\sqrt{3}\,\gamma_{ml}} + \frac{f_y A_{tg}}{\gamma_{mo}}$$

Where,

A_{vg}, A_{vn} = min gross area & net area in shear along section (1-2) & (4-3)

A_{tg}, A_{tn} = min gross area & net area from hole to toe of the angle section (2-3)

Working stress method of steel design:-

Permissible stresses:-

1. Axial tension, $\sigma_{at} = 0.6 f_y$
2. Axial compression, $\sigma_{ac} \leq 0.6 f_y$ [depends upon L/R ratio]
3. Bending compression, $\sigma_{bc} = 0.66 f_y$
4. Per shear stress, $\tau_c = 0.45 f_y$ [generally taken as $0.4 f_y$]

ASSIGNMENT-III

1. Procedure for finding permissible and compressive stress of steel sections:-

PROCEDURE:-

- Assume design stress of the member (generally rolled steel section assumed f_{cd} = 135N/mm^2) for angle section f_{cd} = 90N/mm^2, for builtup section f_{cd} = 200N/mm^2
- Required eff. Sectional area is $A = \frac{P_d}{f_{cd}}$
- Select a section for the eff. Area calculate $\gamma_{\min}$ (least of $\gamma_{xx} \wedge \gamma_{yy}$)
- From the end condition (decide the type of connection) determine eff. Length
- Find slenderness ratio and hence design stress f_{cd}
- Find actual load carrying capacity of compression member.

$p_d = f_{cd} \times A_e$

- If the calculating value of p_d difference consider by from design load P, revise the section.

2. Design a double angle discontinues strut to carry factored axial load 170KN. The length of the strut b/w c/c of intersection is 3.85m, f_y =250N/mm^2

Given:-

L=3.85m
$f_y = 250 N/mm^2$
P = 170KN

Sln:-

Assume $f_{cd} = 90 \ N/mm^2$

$$A_g = \frac{170\times10^3}{2\times90}$$

$A_g = 944.4mm^2$

For safe design increase 30%

$$A_g = 944.4\times1.3$$

$$A_g = 1227.2 \ mm^2$$

∴ Try 2ISA 90x90x8mm

$$I_{xx} = I_{yy} = 104.2\times10^4 \ mm^4$$

$$\gamma_{xx} \ \gamma_{yy} = 25.1 \ mm$$

$$\gamma_{xx} = \gamma_{yy} = 27.5 \ mm$$

$$r_{uu} = 34.7 \ mm$$

$$r_{vv} = 17.5 \ mm$$

$A^* = 2x1379 = 2758mm^2$

$I_y {}^* = 2[104.2x10^4+1379(25.1+10/2)^2$

$$r_y {}^* ¿ \sqrt{\frac{4.583\times10^6}{2758}}$$

$r_y {}^* ¿$ = 40.76mm

¿

K = 1

$$\therefore \frac{KL}{\gamma_{\min}} = \frac{1\times3850}{27.5}$$

$$\frac{KL}{\gamma_{\min}} = 140$$

$\therefore \ f_{cd} = 66.2 \ N/mm^2$ [from table 9c]

$\therefore P_d = f_{cd} \times A$

= 66.2x2758

= 182.58KN > 170KN

∴ Hence the assumed section is safe.

$$\partial_{act} < \partial_{allow}$$

∴ Hence the section is safe in deflection

4. Check for web buckling:-

$$F_{cdw} = (b_1 + n_1) t_w f_c$$

$$n_1 = \frac{h}{2} = \frac{600}{2} = 300 \ mm$$

b_1 =100mm

$$\lambda w = \frac{2.5d}{tw}$$

$$= \frac{2.5 \times 523.4}{11.2}$$

λw = 116.83

110	94.6
120	83.7

[from table 9 © IS800-2007]

$\therefore$ $f_c = 87.155$ N/mm^2

$\therefore \hbar dw = (100+300) \times 11.2 \times 87.155$

$\hbar dw$ = 390.454KN > 225KN

Hence the section is safe against web buckling.

5. Check for web crippling:-

$$F_w = \frac{(b_1 + n_2) t_w f_{yw}}{\gamma_{mo}}$$

$n_2 = 2.5 (t_f + t_1)$

= 2.5 (21.3+17) = 82.5mm

$$F_w = \frac{(100 + 82.5) 11.2 \times 250}{1.1}$$

F_w = 464.545KN > 225KN

Hence the section is safe against web crippling.

3. An ISMB section of depth 500mm is used as a beam over as a span of 6m with s.s ends. Determine the maxi. Factored udl that the beam can carry if the ends are restrained against torsion, but compression flange is laterally unsupported.

Given:-

Span = 6m

Depth = 500mm

Section => ISMB 500

Sln:-

Section Properties of ISMB500:-

A = 11074mm^2, b_f = 180mm, t_f = 17.2mm, t_w = 10.2mm, I_{zz} = 45218.3x10^{3}mm^4, I_{yy} = 1369.8x10^{3}mm^4, r_1 = 17mm, z_{pz} = 2074.67x10^{3}mm^3, z_{ez} = 1808.7x10^{3}mm^3, r_{yy} = 35.2mm, r_{zz} = 202.1mm

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

= 500-2(17.2+17)

d = 431.6mm

To find maxi.Moment & S.F:-

The maxi. Moment of the beam M = $\frac{wl^2}{8}$

The design moment capacity of the section $M_d = \beta_b Z_p f_{bd}$

Where,

f_{bd} => Taken from table-13 for $f_{cr, b}$ given in table-14 [cls 8.2.2 IS800-2007]

$f_{cr, b}$:- [Table-14 IS 800-2007]

The critical stress f_{cr}, b is found based on slenderness ratio $\frac{KL}{r}$ and $\frac{h}{t_f}$ ratio

Here, K=1

$\therefore \frac{KL}{\gamma_{min}} => \frac{1\times 6000}{35.2} = 170.45$

$\frac{h}{t_f} => \frac{500}{17.2} = 29.07$

$\frac{KL}{r}$	h / t_f		
	25	29.07	30
170	136.7	124.16	121.3
170.45	136.26	123.75	120.89
180	127.1	114.97	112.2

From table-14 [IS 800-2007]

$\therefore$ Critical stress $f_{cr}, b = 123.75\, N/mm^2$

f_{bd} :-

Refer [table-13 IS800-2007] α_{LT} =0.21 (for R.S section)

$\therefore$ Refer table – 13(a) IS 800-2007

150	106.8
100	77.3

From table-13(a)

$\therefore$ f_{bd} = 91.31 N/mm^2 (for f_{cr}, b = 123.75N/mm^2

$\therefore$ The design bending strength of ISMB $M_d = \beta_b Z_p f_{bd}$

Buckling Class:- [Table-2 IS 800-2007]

$\frac{b}{t_f} = \frac{0.5b_f}{t_f} = \frac{90}{17.2} = 5.2 \sum 9.4 \sum ¿$

¿

$\frac{d}{t_w} = \frac{431.6}{10.2} = 42.31 \sum 84 \sum ¿$

¿

$\therefore$ The section comes under plastic

$\therefore$ $\beta_b = 1$ [cls 8.2.2 IS 800-2007]

$M_d = 1\times 2074.67\times 10^3 \times 91.31$

M_d =189.44KN.m

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

M_d = 189.44KN.m

The safe udl the section can carry is found by equating M & M_d

M = M_d

$$189.44 \times 10^6 = \frac{w \times 6000^2}{8}$$

$$w = 42.09 KN/m$$

UNIT-V

ROOF TRUSSES & INDUSTRIAL STRUCTURES

The Industrial buildings are low rise steel structures, characterised by their low height, lack of interior floors, walls and partitions. The roof in system for such buildings are,

1. Flat roofing consisting of R.C.C construction
2. Slope roofing.
   i) truss roofing
   ii) shell roofing of R.C.C. or steel.

* Steel roof trusses are ~~commonly~~ used one of the best cheapest & most convinient roof in system for various types of buildings, both for small as well as large span.

* The steel roof trusses are commonly used for industrial buildings, workshop buildings, storage godowns, warehouse & residential building, school building and office. where the construction work is to be completed in a short duration of site.

* One of the greatest advantages of the roof truss that its midspan depth is greatest - specially to resist the max. BM.

* The slope is forces of roof truss facilitate to easy drainage of rain water.

* A roof truss is basically a framed structure formed by connecting various members at their ends to form a system of triangles, arranged in predisided pattern depending upon the span, type of loading & functional requirements.

* The members can be jointed through rivets, bolt or weld using gusset plate.

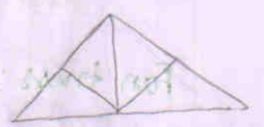
* The rivet or bolted joint permit some rotation of the members and hence the joints are considered as pin joint, where BM is zero.

* The external loads are generally applied at joints only, so that the member of the truss carry the direct forces either tension or compression.

* The members carrying compressive force are called strut & carrying tensile forces are called ties.

Types of Roof truss:

King post truss.



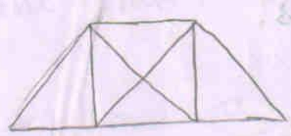
(Span < 6 m)

It's basically timber.

Truss also be made of steel & timber.

Queen post truss:

It's made up of combination of steel & timber.



(Span 6-10 m)

$$\text{Slope} = \frac{\text{rise}}{\text{half of span}} = \frac{1.6}{4} = 0.4$$

$\tan\theta = 0.4$

$\therefore \theta = \tan^{-1}(0.4)$

$\theta = 21.8^{\circ}$

weight of the roof materials $= 250 \times 1.5 = 375$ N/m

Assume dead load of purlin $= 65$ N/m

$\therefore$ Total dead load $=$ 440 N/m

The component of dead load normal to the roof $= 440 \cos 21.8^{\circ} = 408.53$ N/m.

wind load $= 1200 \times 1.5 \times 1 = 1800$ N/m.

Total load normal to the roof $=$ 2208.53 N/m

Step 2: BM calculation

$$M = \frac{wl^2}{10} = \frac{2208.53 \times (8)^2}{10} = 14.135 \text{ kNm}.$$

$$Z_p \text{ required} = \frac{M}{1.33 f_y} = \frac{14.135 \times 10^6}{1.33 \times 250} = 42.51 \times 10^3 \text{ mm}^3$$

Step 3: selection of angle section:

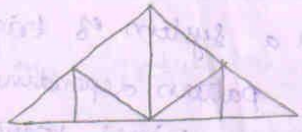
Depth $= \frac{1}{45} \times 8000 = 178$ mm

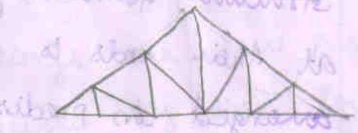
width $= \frac{1}{60} \times 8000 = 134$ mm

$\therefore$ from steel table

let us try $200 \times 150 \times 15$ mm

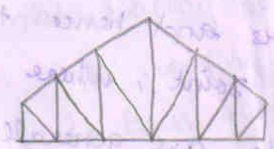
Howe triangular truss :



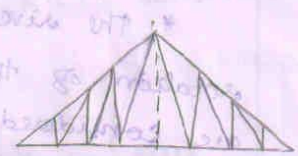


(Span upto 6-24 m)

PRATT TRUSS

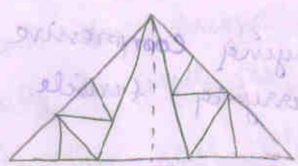


Flat



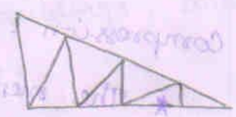
triangular (Span 6-30 m)

Fink & french truss :



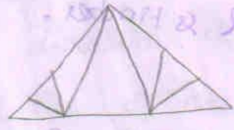
(Span 10-20 m)

North light truss



(Span 8-10 m)

Fan truss :

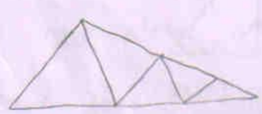


(Span < 12 m)

Compound fan truss :

(Span 20-30 m)

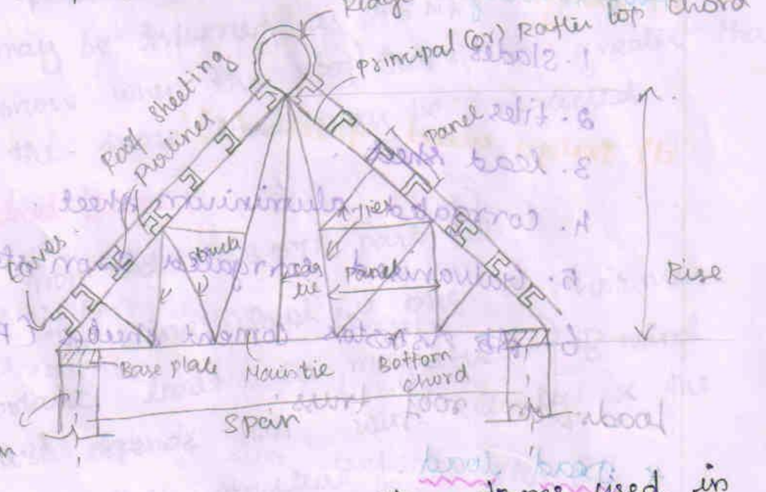
SAW TOOTH TRUSS ;



(Span 5 to 8 m)

Components of Roof truss:

1. Principal rafter (or) top chord
2. Bottom chord (or) main tie
3. Ties
4. Struts
5. Sag tie
6. Purlin's
7. Ridge
8. Roof covering
9. Eaves
10. Base plate (or) Anchor plate
11. panel.



The following some other terms used in truss design.

Bay:- It is defined as the distance between the two adjacent truss.

Rise:- It is defined as the distance from the heighest point to the line joining supports.

Span:

It is the distance b/w the Centre of the End supports.

(X) pitch:

for a symmetrical truss, it is define as the ratio of rise to the span.

(X) Slope:

for a Symmetrical truss, it is defined as the ratio of its rise to half of the Span.

Roof coverage:

1. slades
2. tiles
3. lead sheet.
4. corrgated aluminium sheet.
5. Galvanised corrgated Iron steel (GCIS)
6. ~~Ab~~ Asbestos coment sheets (AC)

Loads on roof truss:

* Dead load:

a) weight of roof covering, weight

1. Trafford Asbestos sheets 159 N/m^2
2. 20 gauge CGI sheets 112.7 N/m^2

b) weight of purlins

1. Glazzed roof 70 – 120 N/m^2
2. G.I sheeting 60 – 90 N/m^2
3. A.C sheeting 90 – 150 N/m^2

c) weight of bracing :

May be assumed as 12 to 15 N/m^2

d) weight of trusses (w) :

$$w = \left(\frac{span}{3} + 5\right) \times 10$$

* Imposed load :

This load taken from IS code 875 part - II .

* Snow load :

The load due to snow depends upon the pitch of the roof and the roofing materials. It may be assumed as 2.5 N/m^2 per mm depth of snow when the roof slope is greater than 50° the snow load may be neglected.

* wind load :

from code IS 875 part - III

The wind load is one of the most important loads, the magnitude of wind pressure depends on wind velocity & the shape of the structure.

a) Basic wind speed (V_b) :

The wind speed measured in a 50 years return period. The basic wind speed map of our country as applicable to 10m height above the ground level for different zones. As per code IS875 part - III 1987. The 6 wind zone have wind speed of 55, 50, 47, 44, 39, 33 m/s respectively.

b) Design wind speed (V_z):

$$V_z = k_1 k_2 k_3 V_b$$

where, V_z - Design wind speed at any height in

k_1 - Risk coefficient factor

k_2 - Terrain, height & structure size factor

k_3 - Topography factor.

V_b - basic wind speed.

In the above $k_1 k_2 k_3$ factors have been described in IS 875 part-III 1987.

c) Design wind pressure:

$$P_z = 0.6 (V_z)^2 \ \mathrm{N/m^2}$$

d) Design wind force (F):

$$F = (C_{pe} - C_{pi}) A P_z$$

where,

C_{pe} - External pressure coefficient.

C_{pi} - internal pressure coefficient.

A - surface area of the element

P_z - design wind pressure.

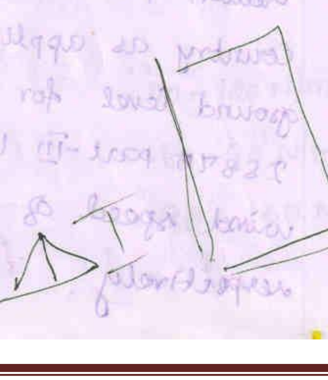
Topography factor:

$$k_3 = (1 + C \times S)$$

for, $3^\circ < \varphi \le 17^\circ$

$C = 1.2 \, Z/L$

$L_e = L$



for $\varphi > 17°$

$C = 0.36$; $L = \frac{z}{0.3}$

where, φ - upward slope of the ground in wind direction.

L - Actual length of the upward wind slope in the direction of wind.

z - Effective height of the crest.

Le - Effective horizontal length.

S - factor = 1.0

1\. A roof truss shed is to be built in Jodhpur city area for an industrial use. the size of the shed is 18m x 30m. The height of the height of the truss is 20m at the eaves. Determine the basic wind pressure.

Soln:

Step 1: Basic wind speed:

from code IS 875 part - III 1987

The basic wind speed in a Jodhpur city is 47 m/s

Step 2: Design wind speed:

Risk coefficient factor k_1:

The design life to be assumed as 50 years from code IS 875 part - III.

table - 1:

The risk coefficient for $V_b = 47$ m/s

Terrain, height, structure and size factor.

The size of the shed is $\boxed{18 \times 30}$ m.

Category - I height < 1.5 m

II height 1.5 to 10 m

III height > 10 m.

Building :

class A - dimension < 20m

B - 20 to 50 m

C - > 50 m

The max. horizontal dimension if the building is 20 - 50 m and height of the truss in 20m

$\therefore$ from code let choose the category type 3 and class 'B'.

$k_2 = 0.98$

Topography factor

$\theta = 0^\circ$; $c = 0$

$k_3 = 1 + (0) = 1$

$V_z = 1 \times 1 \times 0.98 \times 47$

$= 46.06$

Step 2: Basic wind pressure (P_z):

$P_z = 0.6 (V_z)^2$

$= 0.6 (46.06)^2$

$P_z = 1272.91$ N/m^2

18/4/15 Q2. A power plant structure having max. dimensions more than 60m is proposed to built on down hill near pune. The height of the hill is 400m with a slope of 1 in 3. If the location is 250m from the crest of the hill on downward slope and its eave board is at a height of 9m. Determine the design wind pressure.

Soln:

Step-1: Basic wind speed (V_b):

from code IS 875 part - 3. The basic wind speed in pune = 39 m/s

Step-2: Design wind speed (V_z):

i) Risk co-efficient (k_1):

The design life of power plant structure assumed as 100 year's

∴ from code IS 875 Table-1. The Risk coefficient for V_b = 39 m/s.

$k_1 = 1.06$

ii) Terrain height of structure & size factor (k_2):

The size of structure more than 60m (> 50m)

∴ It's belong's to class 'C'

The height at eave board = 9m — (1.5m to 10m)

∴ It's belong's to category - 2 structure

$k_2 = 0.98$.

iii) Topography factor (k_3) :

The effective height of the hill (z) = 400m

Slope is 1 in 3.

$\therefore \theta = \tan^{-1}(1/3) = 18.43^\circ$

$c = 0.36$ $[\because \theta > 17^\circ)$

$S = 1.0$

$k_3 = 1 + (0.36 \times 1)$.

$= 1.36$

$\therefore V_z = k_1 k_2 k_3 V_b$

$= 1.06 \times 0.93 \times 1.36 \times 39$

$V_z = 52.28$ m/s

Step 3: Basic wind pressure (P_z)

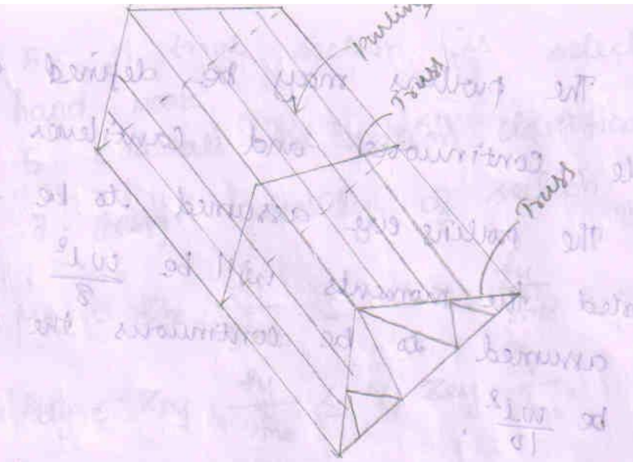
$P_z = 0.6 (V_z)^2$

$= 0.6 \times (52.28)^2$

$= 1640$ N/m^2.

Design of purlins:

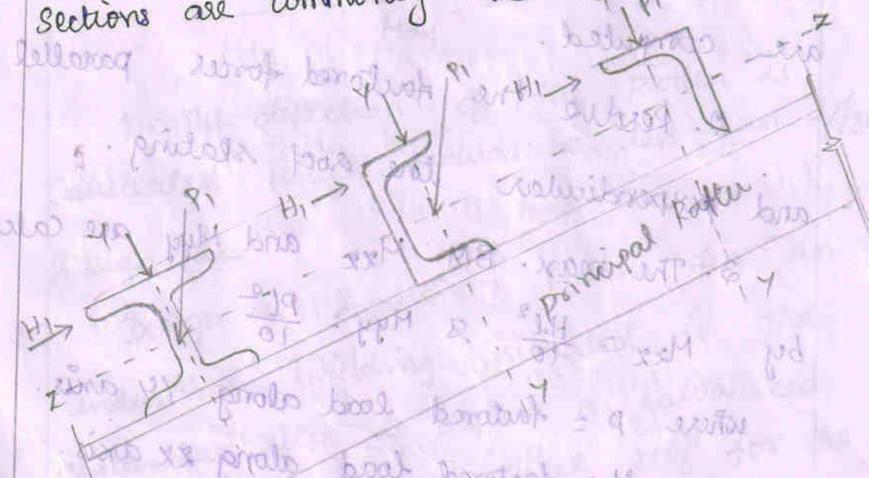
It is defined as the beam, provided over truss to support the roofing materials b/w the adjacent trusses.



Spacing of the truss = Span of the purlin

These are placed in a tilted position over the principle rafter.

I-section, channel section and angle sections are commonly used as a purlin.



The wind force is assumed to act normal to the roof truss and the gravity loads pass through the COG of purlin section.

18/11/15

The purlins may be defined as simple, continuous and cantilever beams.

The purlins are assumed to be simply supported the moments will be $\frac{wl^2}{8}$, if they are assumed to be continuous the moments will be $\frac{wl^2}{10}$.

IS code 800 : 2007 reccomends the purlins to be designed as continuous beam

Design procedure for channel & I-section purlin

1. The gravity load P_1, due to roof seating, load and the load h_1 due to wind force are computed.

2. Resolve the factored forces parallel to and perpendicular the roof seating.

3. The max. BM M_{zz} and M_{yy} are calcula[ted] by $M_{zz} = \frac{Hl^2}{10}$ & $M_{yy} = \frac{Pl^2}{10}$

where P = factored load along YY axis.

H = factored load along zz axis.

l = span of the purlin (c/c distance [of] the adjacent t[russes])

4. purlin subjected to biaxial bending, th[e] required value of section modulus

$$Z_p)_{required} = M_{zz}\frac{\gamma_{mo}}{f_y} + M_{yy}\frac{\gamma_{mo}}{f_y}$$

5. A trial section is selected from IS hand book.

6. Calculate the section classification.

7. Design capacity of section.

$$M_{dz} = Z_{pz} \frac{f_y}{\gamma_{mo}} \leq 1.2 Z_{ex} \frac{f_y}{\gamma_{mo}}$$

$$M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}} \leq \gamma_f Z_{ey} \frac{f_y}{\gamma_{mo}}$$

$$Z_{py} = \frac{b^2 t_f}{2}$$

8. The local capacity of the section is checked using the following intraction equation.

$$\frac{M_{xx}}{M_{dx}} + \frac{M_{yy}}{M_{dy}} \leq 1$$

9. The deflection of the purlin is calculated which should be less than $l/180$.

~~Design~~ an

1. Design an I-section purlin for an industrial building situated in the allahabad city to support a galvanized corrugated (GCI) Iron sheet roof for the ~~fell~~ following data i) spacing of the truss c/c = 6m. ii) span of the truss = 12m iii) spacing of purlin c/c = 1.5m. iv) Intensity of wind pressure = 2kN/m² v) weight of Galvanized sheet = 180 N/m². The grade of steel Fe... then $\theta = 30°$.

80th.

Step 1: load calculation:

weight of G.I sheet = $130 \times 1.5 = 195$ N/m

Assumed dead load of purlin = 100 N/m

∴ Total dead load = 295 N/m

The dead load acts vertically downward.

The dead load ⊥r to roof = $295 \sin 30^\circ$

$= 147.5$ N/m.

The component of dead load normal to the roof = $295 \cos 30^\circ$

$= 255.47$ N/m

∴ The Since the wind load is act as normal to the roof = $2 \times 10^3 \times 1.5$

$= 3000$ N/m.

The load on purlin normal to the roof

$H = 3000 + 255.47$

$= 3255.47$ N/m

∴ factored load normal to the roof

$H = 1.5 \times 3255.47$

$= 4883.2$ N/m.

factored load $\perp^r$ to the roof,

$$P = 1.5 \times 147.5 = 221.25 \text{ N/m}$$

Step 2: Max. BM @ zz & yy axis

$$M_{zz} = \frac{HL^2}{10} = \frac{4883.2 \times 6^2}{10} = 17.58 \text{ kNm},$$

$$M_{yy} = \frac{pl^2}{10} = \frac{221.25 \times (6)^2}{10} = 0.796 \text{ kNm}.$$

$$\therefore Z_p)_{required} = M_{xx}\frac{\gamma_{mo}}{f_y} + M_{yy}\frac{\gamma_{mo}}{f_y}.$$

$$= \frac{17.58 \times 10^6 \times 1.1}{250} + \frac{0.796 \times 10^6 \times 1.1}{250}$$

$$Z_p = 80.85 \times 10^3 \text{ mm}^3.$$

$\therefore$ Let us try ISWB 150 @ 170 N/m

Step 3: properties of that section:

$h = 150$mm, $d = h - 2(t_f + R_1) = 150 - 2(7+8) = 120$ mm.

$b_f = 100$mm, $I_{xx} = 839.1 \times 10^4 \text{ mm}^4$

$t_f = 7$mm. $Z_{px} = 126.86 \times 10^3 \text{ mm}^3$.

$t_w = 5.4$mm, $Z_{ex} = 111.9 \times 10^3 \text{ mm}^3$.

$R_1 = 8$mm, $Z_{ey} = 19 \times 10^3 \text{ mm}^3$.

Step 4: Section classification.

(from code pg: 18 table - 2)

$b/t_f = 50/7 = 7.14 < 9.4$

$d/t_w = \frac{120}{5.4} = 22.22 < 84.$

Hence the section is plastic.

Step 5: Design capacity of the section.

i) $M_{dz} = Z_{pz} \frac{f_y}{\gamma_{mo}}$

$= 126.86 \times 10^3 \times \frac{250}{1.1}$

$= 28.83$ kNm.

$\leq 1.2 Z_{ez} \frac{f_y}{\gamma_{mo}} = 1.2 \times 111.9 \times 10^3 \times \frac{250}{1.1}$

$= 30.52$ kN.m.

ii) $M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}}$

$Z_{py} = \frac{b^2 t_f}{2} = \frac{100^2 \times 7.4}{2} = 35 \times 10^3 \text{ mm}^3.$

$M_{dy} = 35 \times 10^3 \times \frac{250}{1.1} = 7.95$ kNm

$\leq \gamma_f Z_{ey} \frac{f_y}{\gamma_{mo}} = 1.5 \times 19 \times 10^3 \times \frac{250}{1.1}$

$= 6.47 \text{ kNm} > 0.796$ kNm.

Hence the section adequate.

Step 6: Local capacity of the section.

$\frac{M_{zz}}{M_{dz}} + \frac{M_{yy}}{M_{dy}} \leq 1$

$\frac{17.58}{28.83} + \frac{0.796}{6.47} \leq 1$

$0.733 < 1$

Step - 7 : check for deflection :

from code pg: 31 table - 6

deflection will be checked @ service load only.

permissible deflection $\delta = \frac{l}{180}$

$= \frac{6000}{180} = 33.33\,mm$.

Max. calculated deflection $\delta = \frac{5\,wl^3}{384\,EI}$

$= \frac{5}{384} \times \frac{3255.47 \times 10^{-3} \times (6000)^3}{2 \times 10^5 \times 839.1 \times 10^4}$

$= 0.005\,mm$.

Hence the design is safe.

18/4/15 Design procedure for Angle section purlins :

1. The gravity loads and the wind loads are determined, both the loads are assumed to be normal to the roof truss.

2. Angle section can be used as a purlin, provide the slope of the roof truss is less than 30°.

3. The max. BM computed by $\frac{wl^2}{10}$ (or) $\frac{WL}{10}$

where, W - Unfactored concentrated load at centroid.

w - unfactored uniformly distributed load.

L - Span of the purlin.

4. The modulus of section required is

calculated by

$$Z_p = \frac{M}{1.33 f_y}$$

5. A trial section of angle purlin is arrived at by assuming the depth of the angle section is 1/45 of span & width of angle section is 1/60 of the span.

6. the depth and width must not be less than the specified values.

7. A suitable angle section is selected from IS hand book.

8. Modulus of the section provided should be more than the modulus of the section calculated.

1. Design the angle for the following specifications. i) the spacing of the truss = 8m c/c ii) pitch = 1/5 of the span iii) spacing of purlin 1.5m c/c iv) load from roofing materials = 25 ... v) wind load = 1200 N/m².

Soln:

Step 1: Load calculation:

$$\text{pitch} = \frac{\text{rise}}{\text{span}}$$

$$\frac{1}{5} = \frac{\text{rise}}{8} \Rightarrow \frac{1}{5} \times 8 = 1.6m$$

$\therefore$ rise = 1.6m.

$d = 200mm$ $\quad Z = 145.4 \times 10^3 mm^3$

$b = 150mm$ $\quad I_{zz} = 2005.6 \times 10^4 mm^4$.

$t = 15mm$

$\therefore Z)$ required $<$ $Z)$ provided

$42.51 \times 10^3 < 145.4 \times 10^3 mm^3$

Hence the section is safe.

Step 4: Section classification :

$b/t = \frac{150}{15} = 10 < 10.5$

$d/t = \frac{200}{15} = 13.33 < 15.7$

The section is compact.

Step 5: check for deflection.

permissible deflection $= \frac{l}{180}$

$= \frac{8000}{180} = 44.44mm$.

Max. calculated deflection $(\delta) = \frac{5}{384} \frac{wl^3}{EI}$

$$\delta = \frac{5 \times 2208.53 \times 10^{-3} \times (8000)^3}{384 \times 2 \times 10^5 \times 2005.6 \times 10^4}$$

$= 0.0036$ mm.

Hence the design is safe.

2. Design a purlin for the following data. i) spacing of roof truss = 6m

ii) Spacing of purlin = 1.4m

iii) pitch = 1/4

iv) Wt. of GI sheet = 133. N/m^2.

v) wind load intensity Normal to the roof = 1500 N/m^2.

Use channel section.

~~Soln~~:

3. Design a angle Iron purlin for a trussed roof from the following data.

i) span of roof truss = 12m.

ii) spacing of roof truss = 5m.

iii) spacing of purlin along the slope of roof truss = 1.2m.

iv) The slope of the roof truss = $\frac{1}{2} \frac{\text{Vertical}}{\text{Horizontal}}$

v) wind load on roof truss normal to the roof = 1.04 kN/m^2

vi) vertical load from roof sheeting = 0.2 kN/m^2.