LECTURE NOTES ON

STRUCTURAL DESIGN-II (Th 2)

Diploma 5th Semester

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LECTURER STAGE-I

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Introduction to Steel Structures

Advantages

(a) High strength per unit wt. Hence little self-weight is able to resist heavy load.

(b) Light in weight so it can be transported and handled easily

(c) Long life around 50 years.

(d) Ductile hence no sudden failure. (e) Properties does not change with time.

(f) They can be erect at a faster rate. (g) High scrap value.

Disadvantages

(a) Subject to corrosion, hence require frequent painting need fire proof treatment, which increase cost.
(b) The material cost of steel is 30% of total cost and around 70% cost is fabrication and erection (to achieve economy fabrication need to be reduced)
(c) Steel need fireproof treatment which increases cost.
(d) Steel is an excellent heat conductor, hence transmit heat inside the building.
(e) Fatigue is also a major drawback.

Properties of Steel

a) Unit mass of steel, $\rho = 7\,850$ kg/m^3
b) Modulus of elasticity, $E = 2.0 \times 10^5$ N/mm^2 (MPa)
c) Poisson ratio, $\mu = 0.3$
d) Modulus of rigidity, $G = 0.769 \times 10^5$ N/mm^2 (MPa)
e) Co-efficient of thermal expansion $\alpha_t = 12 \times 10^{-6}$ /°C

- Codes like (IS2062) divides the steel in three grades, A,B,C (Grade A is used for normal condition, Grade B is used for non-critical applications and Grade C is used for low temperature and impact)
- Carbon in steel plays a important role. For Carbon steel the carbon is less than 2%, and for alloy steel carbon is more than 2%., As the carbon increase, the tensile strength and hardness increases but ductility decreases hence we use alloy steel using chromium, nickel and vanadium so that tensile strength can be increased while retaining the ductility.
- **Stainless Steel** $\Rightarrow$Low carbon steel with 10.5% chromium and 0.50% nickel is added
- High Carbon Steel $\Rightarrow$ High carbon content, used in Transmission lines and microwave towers.
- Rolled section Example ISLB500 @ 750N/m i.e depth 500mm and weight 750 N per meter. (same for channel and T section)
- Angle ISA 40x25x6mm means one leg 40mm, other 25mm and thickness 6mm
- IS means Indian standard. For I JB (junior beam), LB(light beam), MB(medium beam), WB(wide flange beam), HB(heavy beam)
- Channel JC,LT,MC
- T section JT,LT,ST(short legged),NT (Normal legged), HT

Stress Strain Curve of Mild Steel

Tension Test **Gauge length** $L_0 = 5.65\sqrt{A_0}$ (where A_0 is cross section of undeformed bar)

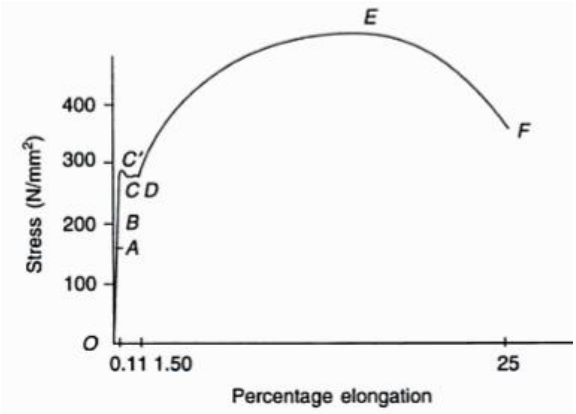


Fig. 1.1 *Stress–strain Curve of Mild Steel*

OA =obeys hooks law (stress strain) straight line.

AB=elastic limit but does not obey hooks law i.e not a straight line (mostly A & B coincide)

C'/C= upper/Lower yield, increase in strain, without increase of stress. If rate of loading is slow we observe lower yield and if rate of loading is rapid we observe upper yield.(strain is of the order of .00125)

CD plastic range,

DE strain hardening, internal structure changes to take additional load but strain increase fast.

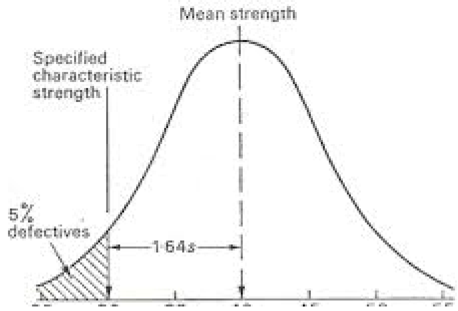
E represent the Ultimate Stress

EF, after reaching ultimate tensile stress, a localized reduction of cross section of the specimen called necking takes place, this zone is known as necking zone.

- If at C strain is ϵ_y than stain at D is $12\epsilon_y$ and at E is $200\epsilon_y$
- This test is at room temperature but as temperature increases the strength decreases but below 0^0 the strength of the steel increases but toughness and ductility decreases.

$$Ultimate\ Tensile\ Strength = \frac{Ultimate\ Tensile\ load}{Original\ Area\ of\ Cross\ Section}$$

- Bauchinger Effect $\Longrightarrow$ The Bauschinger effect refers to a property of materials where the material's stress/strain characteristics change as a result of the microscopic stress distribution of the material. An increase in tensile yield strength occurs at the expense of compressive yield strength. While more tensile cold working increases the tensile yield strength, the local initial compressive yield strength after tensile cold working is actually reduced. The greater the tensile cold working, the lower the compressive yield strength, where deflection and buckling is primary concern Bauchinger effect must be considered.
- Characteristic Strength f_k $\Longrightarrow$ The Strength below which not more than 5% of corresponding strength of specimen tested are expect to fall.
- $f_k = f_{mean} - 1.64\sigma$ where $\sigma = \sqrt{\frac{\Sigma(f_{mean}-f)^2}{N-1}}$

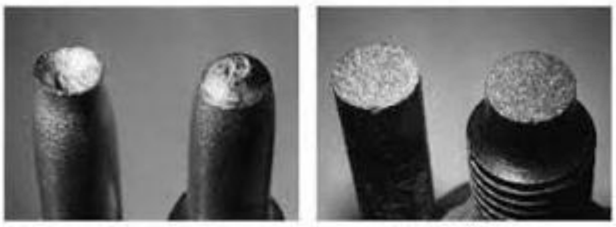


- For account of corrosion, accidental damages the design strength is characteristic strength divided by partial safety factor (usually 1.1)

- Ductility ⟹capacity of structure to undergo large inelastic deformation without significant loss of strength, the amount of permanent strain i.e the strain exceeding proportional limit up to the point of fracture is represented by ductility, its around 23%.

$$\%\,elongation = \frac{elongated\ length - gauge\ length}{gauge\ length} \times 100;$$

- After failure, if the fractured surface is cup and cone arrangement than it indicates ductile failure.



Cup-and-Cone Surfaces Ductile Materials

Flat Surfaces Brittle Materials

Fracture Surfaces

Ductility V/s Malleability⟹ Ductility is ability to deform under tensile stress i.e material ability to be stretched into wire. Malleability is ability to deform under compression i.e ability to form thin sheet by hammering or rolling.

Steel is ductile and malleable but lead is only malleable.

Toughness V/s Hardness⟹ Toughness is the ability of a material to withstand both shock and vibration i.e resistance to absorb energy. Hardness is resist scratching, abrasion indentation or penetration.

Charpy V Notch test is used to kine the Toughness. Steel should have min 20J at 23 +/- 5^{o}C

Brinell or Vicker or Rockwell Number are used to denote hardness.

Generally, rubber is tougher than steel but less hard than steel.

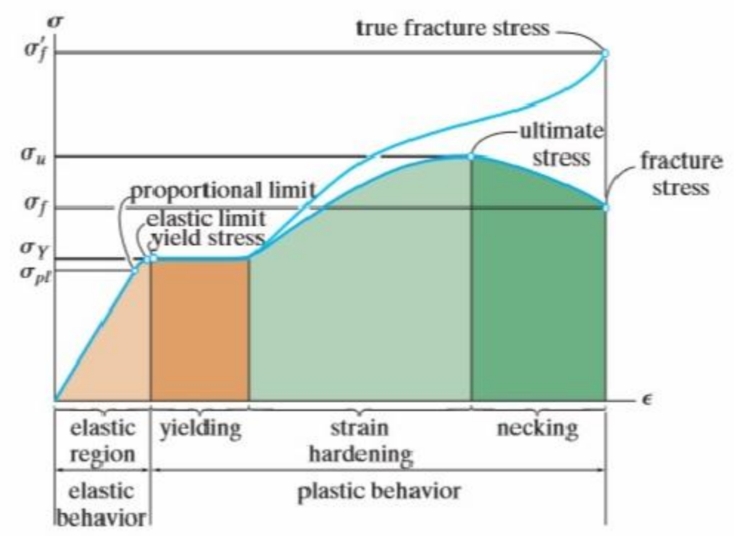
Aluminum V/s Steel Aluminum Section used in aircraft structures . aluminum has greater strength to unit weight than steel but also less stiffness and more buckling characteristic. Also thermal expansion of 'Al' is twice of Steel

Fatigue failure is defined as the tendency of a material to fracture by mean of progressive repeated alternating or cyclic stress of an intensity considerably below the normal strength. It is brittle failure.

Weldabitlity of Steel ⟹ For good weldability, steel should not show high hardness in welded parts, but should have adequate elongation and notch toughness even in the heat affected zone adjacent to weld. Lower the carbon equivalent better is the weldability.

$$C_{eq} = \frac{C + Mn}{6} + \frac{C_r + M_o + V}{5} + \frac{Ni + Cu}{15}$$

- The failure criteria for Cast Iron is Brittle and The failure criteria for Mild steel is Ductile.
- Using Von Mises theory of failure, $f_y^2 = f_b^2 + 3f_q^2$



Conventional and true stress-strain diagrams for ductile material (steel) (not to scale)

Fact and Figures of Steel Industry

The steel industry is often considered an indicator of economic progress.

Steel is one of the most recycled materials with rate of 60%.

Total steel produce in world is 1700 Million Tonnes, (China 850 MT, Japan 104MT, India 101.4 MT)

PerCapita Consumption of Steel in World is 208Kg but in India it is 65Kg only, China 438Kg and US 282Kg.

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IS800-2007

- Switching over from working stress design to Plastic Limit and then limit state design.
- Plastic Analysis or Ultimate Strength method does not guarantee serviceability criteria, instability fatigue and brittle fracture.
- The load factor for Plastic limit is 1.7 for all loads except combination DL+LL+WL/EL (1.3)
- The axis along the member length is x-x, the major axis is z-z, and minor axis is y-y (as same computer programs)

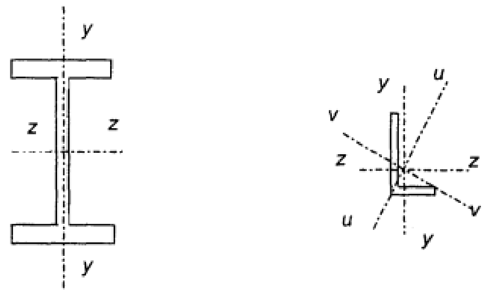


FIG. 1 AXES OF MEMBERS

- No limit of minimum thickness is provided for steel section is IS800-2007, but a minimum of 6mm is recommended.
- The partial safety factor accounts material, workmanship, error in construction and fabrication (i.e uncertainty in load, material etc)

Table 1 (*Concluded*)

Sl No.	Indian Standard	Grade/Classification	Properties: Yield Stress MPa, *Min*			Ultimate Tensile Stress MPa, *Min*	Elongation, Percent, *Min*
(1)	(2)	(3)	(4)			(5)	(6)
			d or *t*				
			< 20	20-40	> 40		
viii)	IS 2062	E 165 (Fe 290)	165	165	165	290	23
		E 250 (Fe 410 W) A	250	240	230	410	23
		E 250 (Fe 410 W) B	250	240	230	410	23
		E 250 (Fe 410 W) C	250	240	230	410	23
		E 300 (Fe 440)	300	290	280	440	22
		E 350 (Fe 490)	350	330	320	490	22
		E 410 (Fe 540)	410	390	380	540	20
		E 450 (Fe 570) D	450	430	420	570	20
		E 450 (Fe 590) E	450	430	420	590	20

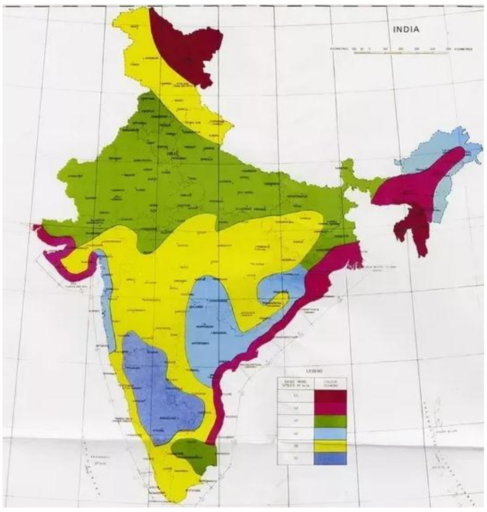
Loads (Action)

- Dead load (IS: 875,Part-1) are gravity loads and are constant with time.
- Live load (IS:875, Part-2,4) are loads which changes with time (position or magnitude) Service load, Earth pressure, water current, impact load, snow load and thermal load but excluding temperature loads.
- Wind Load (IS: 875,Part-3)
- Earth Quake load (IS:1893)

Wind Forces

- Return Period (R)⟹ Wind load varies from year to year based on the wind speed and the maximum that can be expected to occur at a given location only once in so many year, This period is known as Return Period, thus return period is number of years , the reciprocal of which gives the probability of extreme wind exceeding a given wind speed in any one year. Ex. The Return period of 200Km/hr at a certain locality is 50year, the probability of that there will be a wind speed greater than 200Km/hr in any one year is 1/R=1/50=0.02
- But in design we are not interested in the probability that wind will exceed in any one year but rather the probability that it will be exceeded during the life of the structure, Hence if 1/R is the probability that wind speed will be exceeded in any year, (1-1/R) is the probability that it will not be exceeded, If we consider N (50year) as life of the structure in years, then the probability that wind will exceed at least once during the life of the structure will be $\mathrm{P_n} = 1 - \left(1 - \frac{1}{\mathrm{R}}\right)^{\mathrm{N}} = 1 - \left(1 - \frac{1}{50}\right)^{50} = 0.635$
- There are 63.5% chance that the structure will be exposed to a wind of speed 200Km/hr

- Design wind speed $\boldsymbol{V_z = V_b * K_1 * K_2 * K_3 * K_4}$



$\boldsymbol{V_b}$ is basic wind speed for any site obtained from wind map of the india. Code defines the basic wind speed as the peak gust wind speed averaged over a period of 3 seconds.
The wind speeds are assessed with the aid of anemometers or anemographs, which are installed at meteorological observatories at heights generally varying from 10m meters above ground.

- $\boldsymbol{K_1}$ = Risk Coefficient, based on 50 year life of structure, mostly the value is taken as 0.63 but it depend on importance of structures, if structure is important risk coefficient will be high.

$\boldsymbol{K_2}$ = it depend on height with different terrain, terrain has been grouped under 4 categories based obstruction constituting the ground surface roughness, category-1 is less than 1.5m and categrory-4 is height more than 25m.

$\boldsymbol{K_3}$ = The basic wind speed V_b takes account of the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges, which can significantly affect the wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliffs, steep escarpments, or ridges (For level ground K_3 is 1)

$\boldsymbol{K_4}$ = Importance Factor for Cyclonic Region (Hospital, schools =1.30, Industrial = 1.15, Others = 1.0)

CP1 – Photograph Indicative of Terrain Category 1 Features

CP2 – Photograph Indicative of Terrain Category 2 Features

CP3 – Photograph Indicative of Terrain Category 3 Features

CP4 – Photograph Indicative of Terrain Category 4 Features

Design Wind Pressure $= \mathrm{P_z} = 0.6\mathrm{V_z^2}\ \frac{\mathrm{N}}{\mathrm{m^2}}$ **Design Wind Force** $= \mathrm{F} = \mathrm{C_f A_e P_z}$

Where, $\mathrm{C_f}$ is force co-efficent, $\mathrm{A_e} =$ effective forntal area and $\mathrm{P_z}$ is design wind pressure.

Normal Force $\mathrm{F_n} = \mathrm{C_{fn}} * \mathrm{P_z} * \mathrm{k} * \mathrm{l} * \mathrm{b}$ **Transverse Force** $\mathrm{F_t} = \mathrm{C_{ft}} * \mathrm{P_z} * \mathrm{k} * \mathrm{l} * \mathrm{b}$

l is the length and b is the width of the member in the direction of wind, k is a coefficient depend on l and b.

Wind force on Roof $\mathrm{F} = (\mathrm{C_{pe}} - \mathrm{C_{pi}}) * \mathrm{AP_z}$, $C_{pi}\ and\ C_{pe}$ are internal and external pressure coefficient.

Seismic Loads

Wind load are external loads applied to exposed surface of the structure whereas the earthquake load are inertial force and function of mass of the structure.

Zone factor The seismic zone is revised with only four zones instead of five zones, zone **I** is merged with zone **II**, hence zone **I** does not appear, in the new zoning only zone **II**, **III**, IV, V .

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	High
Z	0.1	0.16	0.24	0.36

Importance Factor Depending on importance of the building (it ranges from 2 to 1, hospital, railway station, fire station =1.5)

Response Reduction Factor ⟹ a structure is allowed to be damaged in case of severe shaking, therefore structure is designed for seismic forces much less than those expected under strong shaking. Response reduction factor is the factor by which the actual base shear force what would be generated if the structure were to remain elastic during its response to the earthquake, should be reduced to obtain the design lateral force but it can't be used in zone IV and V.

- More is the ductility or rotational capacity of the structure more is the response reduction capacity.

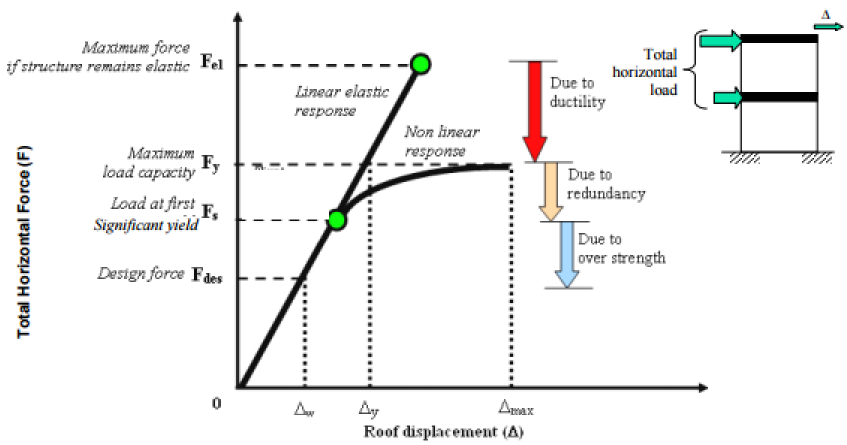


Figure C 8- Concept of Response Reduction Factor

Table 23 Response Reduction Factor (*R*) for Building System

Sl No. (1)	Lateral Load Resisting System (2)	*R* (3)
i)	*Braced Frame Systems*:	
	a) Ordinary Concentrically Braced Frames (OCBF)	4
	b) Special Concentrically Braced Frame (SCBF)	4.5
	c) Eccentrically Braced Frame (EBF)	5
ii)	*Moment Frame System*:	
	a) Ordinary Moment Frame (OMF)	4
	b) Special Moment Frame (SMF)	5

Fundamental Natural Period (T_a)

RC Frame Building $T_a = 0.075h^{0.75}$ where, h is the height of the building in 'meter' above ground level.

Steel Frame Building $T_a = 0.085h^{0.75}$

For Brick infill panel $T_a = \frac{0.09h}{\sqrt{d}}$

Seismic Coefficient Method $\Rightarrow$ also known as static method in which we don't require dynamic analysis for building height not more than 40m in all zone and in zone II and III for building 90m.

Design Seismic Base Shear $V_b = A_h W$ where, $A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$ and W is the seismic weight of the building.

$\frac{S_a}{g}$ is the response acceleration coefficient depend on T_a

Seismic weight of the building is dead load plus some percentage of imposed load (25% up to 3KN/m^2 and 50% above 3.0 KN/m^2)

Distribution of Design Force $Q_i = V_b \frac{W_i h_i^2}{\sum_1^n W_i h_i^2}$

Snow Load

- The load acts vertically and may be assumed as 2.5 N/m^2 per mm depth of snow.

Load Combination

Table 4 Partial Safety Factors for Loads, γ_f, for Limit States
(*Clauses* 3.5.1 *and* 5.3.3)

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1]		WL/EL	AL	DL	LL[1]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)[2]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.
[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

Abbreviations:
DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

Table 6 Deflection Limits

Type of Building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane+ wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter storey drift	—	Storey height/300

- Brittle cladding are asbestos sheeting, fiber concrete sheeting, un reinforced masonry etc, which crack easily under large deflection
- All Metal cladding is elastic cladding.

Example1 Estimate the design wind pressure for a 100m high mobile tower for 100 years and basic wind speed is 47m/s. Given $K_1 = 1.07, K_2 = 1.17, K_3 = 1.0$

Solution $\boldsymbol{V_z = V_b * K_1 * K_2 * K_3}$ $\quad V_z = 47 * 1.07 * 1.17 * 1 = 58.84\, m/s$

$$\mathrm{P_z} = 0.6\, \mathrm{V_z^2} \implies 0.6 * 58.84^2 = 2077.29 \frac{\mathrm{N}}{\mathrm{m}^2}$$

<u>Example 2</u> A steel chimney 3.0m in diameter is situated where the wind pressure is 1200N/m^2. Find the shear force due to wind load at a level of 15m.Shape factor 0.7.

<u>Solution</u> Design wind load $= \mathrm{KP_1A} = 0.7 * 1200 * 45 = 37800\mathrm{N}$ A=projected area=3.0*15=45m^2

<u>Example 3</u> A Building of seismic weight 2000KN of height 10.5m and 8m base dimension. Given seismic zone factor Z=0.16, Importance factor 1.5 and response reduction factor R=5.for $\mathrm{T_a} = 0.334$ $\quad \frac{\mathrm{S_a}}{\mathrm{g}} = 2.5$

<u>Solution</u> W=2000, h=10.5m, d=8m

the fundamental natural frequency is $\mathrm{T_a} = \frac{0.09\mathrm{h}}{\sqrt{\mathrm{d}}} = .09 * \frac{10.5}{\sqrt{8}} = 0.334\ \mathrm{sec}$

Hence $\frac{S_a}{g} = 2.5$,

$$A_h = \frac{ZI\left(\frac{S_a}{g}\right)}{2R} = \frac{0.15*1.5*2.5}{2*5} = 0.06$$

Design base shear = $V_b = A_h W = 0.06 * 2000 = 120KN$

Example4 For a steel building the live load statistical data in KN/m^2 is as follow, 13@3.2,15@3.8,35@4,10@4.2,10@4.4 (Ex 13 slabs, each having a load of 3.2 KN/m2)

Determine the characteristic load as per IS800.

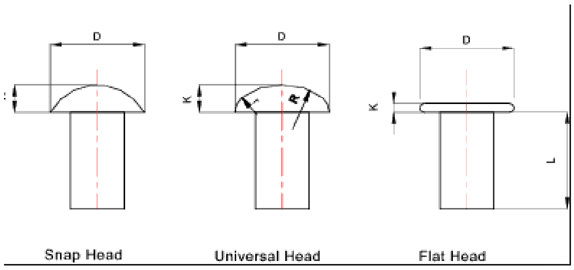
Solution Total number of samples, N=13+15+35+10+10=83

Average load $Q_m = \frac{13*3.2+15*3.8+35*4+10*4.2+10*4.4}{83} = 3.91\ KN/m^2$

Standard deviation $\sigma = \sqrt{\frac{(13(3.2-3.91)^2+15*(3.8-3.91)^2+35*(4-3.91)^2+10*(4.2-3.91)^2+10(4.4-3.91)^2)}{82}}$

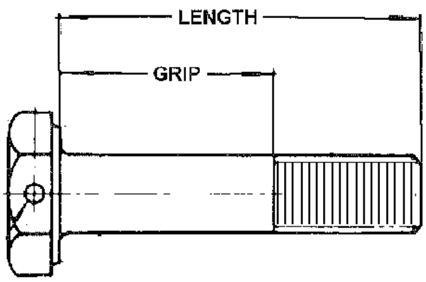
Characteristic Load $Q = Q_m + 1.65\sigma = 3.91 + 1.65 * 0.353 = 4.49\ \frac{KN}{m^2}$

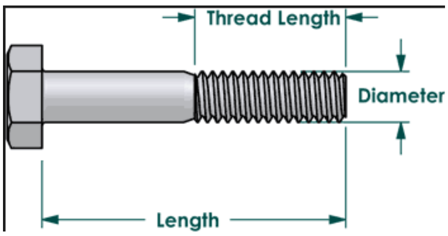
- Rivet is made up of shank and one head, the grip length of the rivet is distance between the underside of the two heads, if grip length is long the rivet will be subjected to bending in addition to shear and bearing stresses.
- The grip length should not be more than 4 times the diameter of rivet.
- The diameter of the shank is called nominal diameter of the rivet. The diameter of the hole in which rivet is inserted is called as gross diameter.
- Inserting the rivet in hole and making a second head is known as riveting, this can be done in hot stage or cold stage, cold riveting need high pressure at room temperature but strength of cold driven riveting is more than hot driven riveting.



Bolted Connection and its Advantage and Disadvantages over Rivets

- The speed of erection is faster than rivets, less skilled labor is required and overall cost of the bolt is cheaper than riveting (but cost of material is more) with less number of bolt required than riveting.
- The tensile strength of the bolt is less than rivets and bolts are misfit when structure is subject to vibration.
- The holes for bolting can be done by either (a) punching (b) drilling, punching is easy but it leads to brittle failure because it reduces ductility and toughness.
- The deflection of beams with bolted connection is more than riveted connection since slip in bolts is more than rivet connection
- Bolt is made up of head at one end and shank threaded at the other end to receive a nut.
- As per IS 800, punching of holes is allowed in only those material having yield strength greater than 360Mpa and where thickness of member does not exceed $\frac{5600}{f_y}$ and washer of minimum thickness 4mm shall be used for uniform distribution of load.





Unfinished Bolts ⟹ Also known as ordinary, common, black bolts. They are not recommended for structures subjected to vibration. They are available in market in size 5 to 36mm, designated as M16, which means 16mm diameter, and class as 4.6 bolt which means ultimate tensile strength is 400MPa and yield stress is 60% of 400 i.e 240MPa

- Table 19 of IS800 gives clearance for Bolt holes.

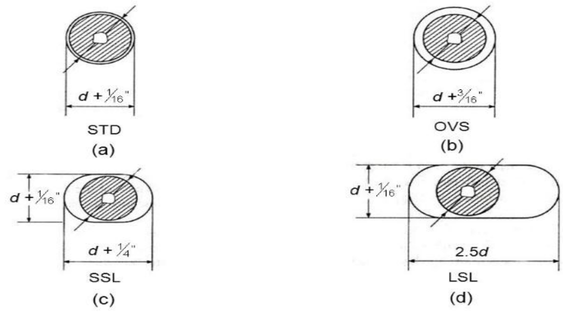


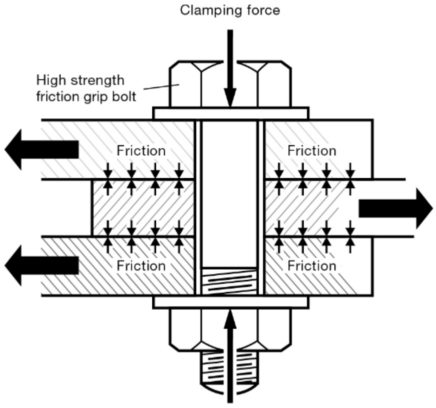
Table 19 Clearances for Fastener Holes

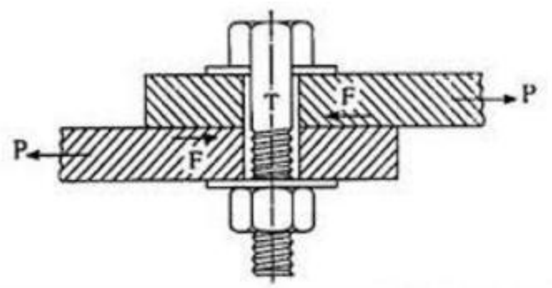
(*Clause* 10.2.1)

Sl No.	Nominal Size of Fastener, *d* mm	Size of the Hole = Nominal Diameter of the Fastener + Clearances mm			
		Standard Clearance in Diameter and Width of Slot	Over Size Clearance in Diameter	Clearance in the Length of the Slot	
				Short Slot	Long Slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 – 14	1.0	3.0	4.0	2.5 *d*
ii)	16 – 22	2.0	4.0	6.0	2.5 *d*
iii)	24	2.0	6.0	8.0	2.5 *d*
iv)	Larger than 24	3.0	8.0	10.0	2.5 *d*

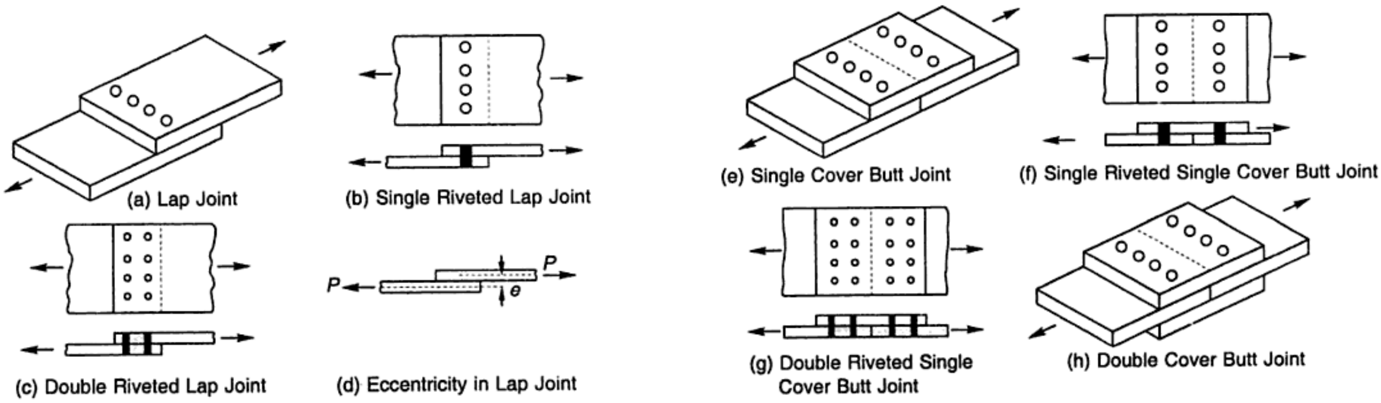
High Strength Bolts

- The high strength is achieved quenching, tempering of steel.
- These bolts are tightened until high tensile stresses are generated.
- They transfer load primarily by friction not by shear, hence these connection are called non slip connection or friction type connection.
- Due to their high tensile strength as compared to ordinary black bolt, a fewer number of bolt is required.
- Joints formed by these bolts are known as pre tensioned joints.

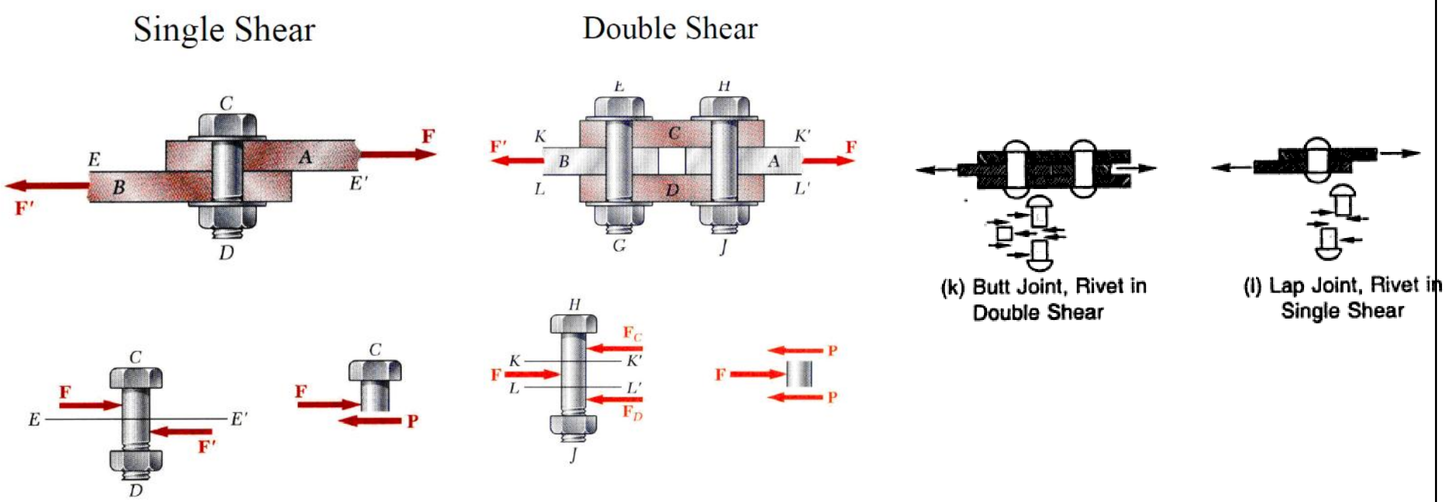




Types of joints



- Lap joint produces eccentricity thus a couple is formed. To minimize the effect of bending at least two rivets/bolts in a line should be placed.
- Butt joint is always better than lap joint.



Failure of Joints

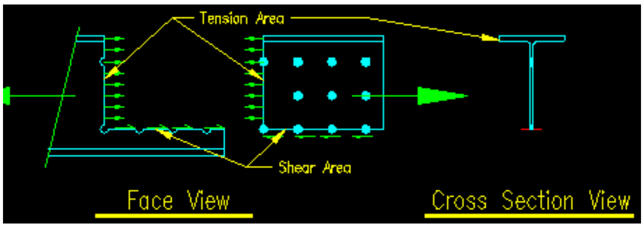
(a) Shear failure of Bolts ⟹ Shear failure happens when plates slips due to applied forces, this type of failure occur at shear plane (either single shear or double shear)

(b) Bearing failure of Bolts ⟹The bolt is crushed around its circumference by heaviest stressed plate. The plate may be strong in bearing and bolt is of weak steel than this type of failure occur.

(c) Bearing failure of Plates ⟹ when a slip of plates occur and plates comes in contact with bolt and in this if plate is weaker than bolt than failure of plate by bearing. The main reason of weakness in plate is due to less edge distance or spacing of the bolt.

(d) Tension failure of plates⟹ tensile stress at the net cross section > working tensile stress i.e rivet is strong than plate

(e) Block Shear Failure ⟹ When bolts are place at a lesser edge distance it leads to the plates to shear, this is more in high strength bolts when fewer bolts are used for making connection length small.



PITCH

Pitch it is the center to center distance between two consecutive bolts measured along the stresses line or line of action of the member.

The minimum distance between the two bolts should not be less than 2.5 times the nominal diameter of bolt. (to make installation easier)

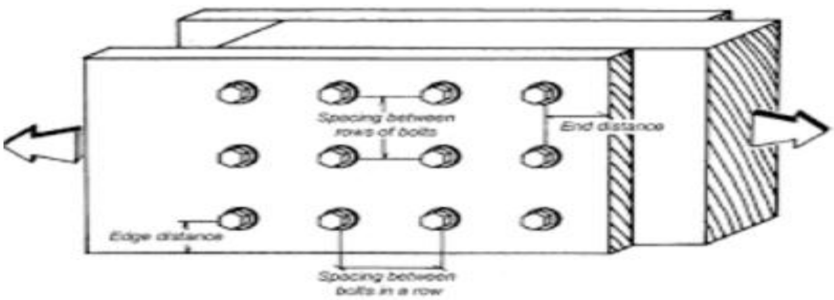
The maximum distance between the bolts is to reduced the length of connection/gusset plate and to have uniform stress in the bolts.

(a) 32t or 200mm for any fasteners

(b)16t or 200 mm for tension member 12t or 200mm for compression member (t is the thickness of thinner member)

Gauge Line or Blot Line ⟹ the distance between center to center of the rivet perpendicular to line of action of force.

(a) The maximum gauge distance is 100mm+4t or 200mm whichever is less.

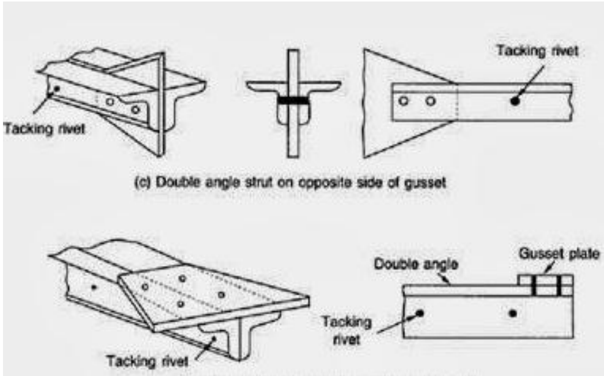


- When the bolts are staggered at equal interval and guage does not exceed 75mm, the maximum distance specified in (b) and (c) can be increased by 50%.

Edge Distance and End distance ⟹ The minimum edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand-flame cut edges; and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edges.

Maximum edge distance is $12t\sqrt{\frac{250}{f_y}}$ (t is thickness of thinner outer plate) but the members subjected to corrosive influence, the maximum edge distance shall not exceed 40mm+4t.

Tacking Bolts ⟹ if distance between the bolts as per clause of 16t or 200 mm for tension member 12t or 200mm for compression member (t is the thickness of thinner member) exceeds additional bolts are provided known as tacking bolts or stitch bolts. The maximum spacing of tacking bolts should not exceed 600mm for compression and 1000mm for tension.



Combination of bolt, welding ⟹ normally one type of fasteners either bolt or rivet or welding be designed to carry the total load but as per IS800 Clause 10.1.5 fully tensioned friction grip bolts can be designed to share the load with welding.

- When load is greater than frictional resistance of bolts than plates slip a little over each other making the bolts in shear and bearing, this is called bearing connection. in bearing connection we can have two condition one in which threads excluded from shear planes or other case in which threads are included. Threads excluded is most economical bolted connection as we need fewer bolts than threads included.

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

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\beta_{ij}\beta_{lg}\beta_{pkg} \quad \text{for long connection}$$

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

where,

V_{dsb} is design strength of bolt γ_{mb} is partial safety factor =1.25

V_{nb} is nominal shear strength, n_n is number of shear planes with threads intercepting the shear plane

n_s is number of shear planes without threads intercepting the shear planes, A_{sb} nominal shank area , A_{nb} net tensile stress area in each shear plane.

$\beta_{ij}(0.75<\beta_{lj} \leq 1.0)$ is reduction factor for the overloading of the end blots in long connection long connection means length of joint exceed 15d (d is the nominal diameter of bolt)

. $\beta_{ij} = 1.075 - \frac{l_j}{200d}$

β_{lg} is reduction factor for large grip length $\beta_{lg} = \frac{8d}{3d+l_g}$ (l_g is grip length and $l_g$should not be greater than 8d)

$$\beta_{lg} \text{ should not be greater than } \beta_{ij}$$

β_{pkg} is reduction factor for packing plate in excess of 6mm $\beta_{pkg} = 1 - 0.0125t_{pkg}$

t_{pkg} is the thickness of the thicker packing plate in mm.

Bearing Strength of Bolt ⟹ Bearing failure is only possible when grade of bolts is very low and strength of plate is very high, the possibility of which is very low. The tests on the bolt joints reveal that the bearing strength of bolts is independent of grade of bolts and plate in contact with them. Bolts never fails in bearing but bearing stress affects the efficiency of the connection, hence given bearing strength is to protect the connection not the bolts.

10.3.4 *Bearing Capacity of the Bolt*

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

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

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;

e, p = end and pitch distances of the fastener along bearing direction;

d_0 = diameter of the hole;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively;

d = nominal diameter of the bolt; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb}, by the factors given below:

a) Over size and short slotted holes – 0.7, and
b) Long slotted holes – 0.5.

γ_{mb} is partial safety factor =1.25

High Strength Friction Grip Bolt (HSFG Bolts)

In friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear, after full friction is utilized, shear and bearing will then come in the picture.

Nominal Shear Strength of HSFG Bolts $\Longrightarrow V_{nsf} = \mu_f n_e K_h F_0$

Design Shear Strength $\Rightarrow V_{dsf} = \frac{V_{ns}}{\gamma_{mf}}$

γ_{mf} is partial safety factor under serviceability condition its value is 1.1 and 1.25 at ultimate load.

μ is slip factor Table given in IS code. n_e is number of interfaces offering frictional resistance to slip

k_h =1.0 for clearance holes, 0.85 for oversized and short slotted holes and 0.7 for long slotted holes.

F_0 = Proof load in tension $= A_{nb} * f_o$ (A_{nb} is net area of bolt at thread, and f_o is proof stress =0.7*ultimate tensile stress of bolt

- Note:- If joint is long the nominal shear capacity of the bolt is reduced by reduction factor β_{lj}

Tensile Strength of Bolts $\Rightarrow$ A bolt is subjected to a factorized tensile force, T_b shall satisfy $T_b \leq T_{db}$

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

$$T_{nb} = 0.9 f_{ub} A_{nb} < f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma m0}$$

γ_{m0} = partial safety factor for material resistance governed by yielding=1.10

f_{ub} = ultimate tensile strength of bolt f_{yb} = yield stress of bolt.

Bolt Subjected to Combined Shear and Tension

A bolt required to resist both design shear force (V_{sd}) and design tensile force (T_b) at the same time shall satisfy:

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

where

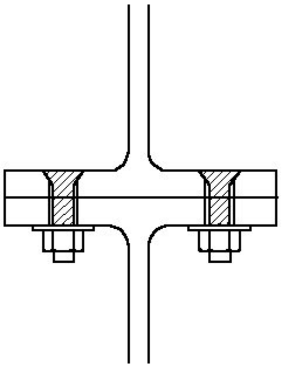
V_{sb} =factored shear force acting on the bolt, V_{dsb} =design shear capacity

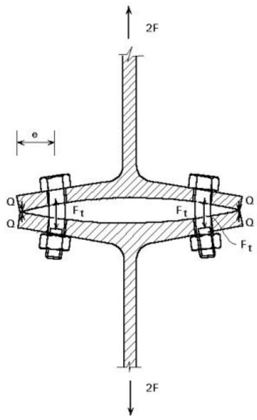
T_b =factored tensile force acting on the bolt, and T_{db} =design tension capacity

Prying Effect

- The prying forces in bolts result from any eccentricity between one of the loaded members and a bolt connecting it to another member; as the joined members try to separate, the members' area away from the loaded section react against each other producing a prying force (as in a crowbar) which must be transferred by the bolt in addition to the original load.
- IF the flanges are flexible, then flanges separates from one another and the center of gravity of the of the compressive forces will moves towards the edges of the flange and bolt tension will increase but if the flanges are rigid, then no separation of flanges and hence the shifting of forces will not occur and there will be no prying action

- The maximum value of the prying force will be reached when only the corners of the flange remain in contact with the other connected parts and in such a case the prying force will shift to the tip of the flange.
- The value of prying force can be kept small by using thick plate





10.4.7 Where prying force, Q as illustrated in Fig. 16 is significant, it shall be calculated as given below and added to the tension in the bolt.

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

where

l_v = distance from the bolt centreline to the toe of the fillet weld or to half the root radius for a rolled section,

l_e = distance between prying force and bolt centreline and is the minimum of either the end distance or the value given by:

$$l_e = 1.1 t \sqrt{\frac{\beta f_o}{f_y}}$$

where

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,

f_o = proof stress in consistent units, and

t = thickness of the end plate.

Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

Working Stress Design

Shear Stress in a Bolt $(f_{sb} < f_{asb})$

Actual shear stress in a bolt $f_{sb} = \frac{V_{sb}}{A_{sb}}$ V_{sb} = Actual shear force in working load

The allowable shear stress in a bolt $f_{asb} = \frac{0.6V_{nsb}}{A_{sb}}$ (A_{sb} =nominal plain shank area of bolt)

Bearing Stress in Bolt

Bearing Stress in a Bolt $(f_{pb} < f_{apb})$

Actual Bearing stress in a bolt $f_{pb} = \frac{V_{sb}}{A_{sb}}$ V_{pb} = Actual bearing force in working load

The allowable Bearing stress in a bolt $f_{apb} = \frac{0.6V_{npb}}{A_{pb}}$ (A_{pb} =nominal plain area of bolt)

Tensile Stress in Bolt

Tensile Stress in a Bolt $(f_{tb} < f_{atb})$

Actual Tensile stress in a bolt $f_{tb} = \frac{V_{tb}}{A_{tb}}$ V_{tb} = Actual Tensile force in working load

The allowable Tensile stress in a bolt $f_{atb} = \frac{0.6V_{ntb}}{A_{tb}}$ (A_{tb} =nominal plain area of bolt)

- (IS 800,11.1.4)In load combinations involving wind or seismic loads, the permissible stresses in steel structural members may be increased by 33 percent. For anchor bolts (used in foundation) and construction loads this increase shall be limited to 25 percent. Such an increase in allowable stresses should not be considered if the wind or seismic load is the major load in the load combination (such as acting along with dead load alone).

- Strength and Efficiency of joint is minimum strength based on strength of bolt in shear and bearing.

  Efficiency of the joint is $\mathbf{N} = \frac{\textbf{Strength of boltedjoint per pitch length}}{\textbf{Strength of solid plate per pitch length}}$

- **Thickness of cover plate should not be less than 5/8 of main plate.**

Types of Connections

(A) Simple (Flexible) Connection ⟹These connection only transfer only shear to the column and undergo considerable deformation at the joint and used in small building where strength is more important than stiffness.

(B) Rigid Connection ⟹These connection transfer moment to the column and are assumed to undergo negligible deformation at the joint and used in high rise building.

(C) Semi-Rigid Connection ⟹These connection are in between the Flexible and Rigid Connection

Example1 Find the strength of 20mm diameter bolt of grade 4.6 to join plate of thickness 12mm (a) Lap joint (b) single cover butt joint cover plate 10mm (c) double cover butt joint 8mm thick. Net tensile area for 20mm dia bolt is 245mm^2.

Solution

(a) Lap joint The strength of bolt in single shear, $V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 245 * \frac{400}{\sqrt{3}*1.25} * 10^{-3} = 45.26\text{KN}$

The strength of bolt in bearing $V_{dpd} = \frac{2.5k_b dtf_u}{\gamma_{mb}}$

$d_0 = 22mm\ e = 33mm$ and pitch =50mm

K_b is least of $\frac{e}{3d_0} = \frac{33}{2*22} = 0.5$ or $\frac{P}{3d_0} - 0.25 = \frac{50}{3*22} - 0.25 = 0.5$ or $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$ Hence $K_b = 0.5$

$$V_{dpb} = 2.5 * 0.5 * 20 * 12 * \frac{410}{1.25} * 10^{-3} = 98.4KN$$

The strength of the bolt will be minimum of shear and bearing and is 45.26KN

(b) Single cover butt joint

Shear strength will be same as above = 45.26KN

The strength of bolt in bearing (the thickness in bearing will be least of aggregate thickness of cover plate and the minimum thickness of the main plates. Hence t=10mm)

$$V_{dpd} = \frac{2.5k_b dtf_u}{\gamma_{mb}} = 2.5 * 0.5 * 20 * 10 * \frac{410}{1.25} * 10^{-3} = 82KN$$

The strength of the bolt is minimum of shear and bearing and is 45.26KN

(c) The strength of bolt in double shear $V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 2 * 245 * \frac{400}{\sqrt{3}*1.25} * 10^{-3} = 90.52KN$

The strength of bolt in bearing (the thickness in bearing will be least of aggregate thickness of cover plate and the minimum thickness of the main plates. Hence t=12mm)

$$V_{dpd} = \frac{2.5k_b dtf_u}{\gamma_{mb}} = 2.5 * 0.5 * 20 * 12 * \frac{410}{1.25} * 10^{-3} = 98.4KN$$

The strength of the bolt is minimum of shear and bearing and is 90.52KN

Example2 for the example1 part (c) if two plate 10mm and 18mm thick are to be joined by double cover butt joint. Find the shear strength of bolt

Solution

the bolt will be in double shear and since the two plates to be joined are of thickness 10mm and 18mm, packing plate of thickness 8mm will be provided. As the thickness of packing plate is more than 6mm, the shear strength of the joint will have to be reduced by the use coefficient $\beta_{pkg} = 1 - 0.0125 * t_{pkg} = 1 - 1.0125 * 8 = 0.9$

$$V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} \beta_{pkg} = 2 * 245 * \frac{400}{\sqrt{3} * 1.25} * 0.9 * 10^{-3} = 81.47KN$$

Example3 A tie member is connected to a gusset plate with 6 bolts, 20mm diameter of grade 4.6 along the direction of force in the member. If the bolts were provided at a pitch of 75mm and at an edge distance of 40mm. what percentage of shear strength will be reduced.

Solution length of the joint is $l_j = 5 * 75 = 375 > 15 * 20 = 300mm$

hence reduction factor $B_{ij} = 1.075 - \frac{l_j}{200d} = 1.075 - \frac{375}{200*20} = 0.98125$

Percentage reduction in shear capacity $(1 - 0.98125) * 100 = 1.875\%$

Example4 A joint is made with 8 bolt of diameter 16mm of grade 4.6 to connect angle of thickness 8mm. IF the tensile force is 141.42KN and shear force is 141.42KN. Check the section is safe in shear.

Grade of steel f_u =410MPa and Grade of bolt $f_{ub} = 400MPa$ and $f_{yb} = 240MPa$

For 16mm dia bolt $A_{nb} = 157mm^2$ and $d_h = d_0 = 18mm$

Solution

Shear in each bolt $V_{sb} = \frac{141.42}{8} = 17.68KN$ Tension in each bolt $T_b = \frac{141.42}{8} = 17.68KN$

Strength of the bolt in single shear $V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\, \gamma_{mb}} = 157 * \frac{400}{\sqrt{3} * 1.25} * 10^{-3} = 29KN$

Strength of bolt in Tension $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$ $T_{nb} = 0.9 * f_{ub} A_{nb} = 0.9 * 400 * 157 * 10^{-3} = 56.52KN$

T_{nb} should not be greater than $f_{yb} \frac{\gamma_{mb}}{\gamma_{m0}} A_{sb} = 240 * \frac{1.25}{1.10} * 201 * 10^{-3} = 54.82KN$

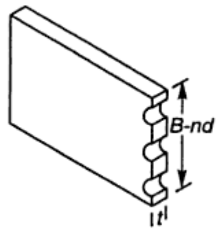
$$T_{db} = \frac{54.82}{1.25} = 43.85KN$$

Check

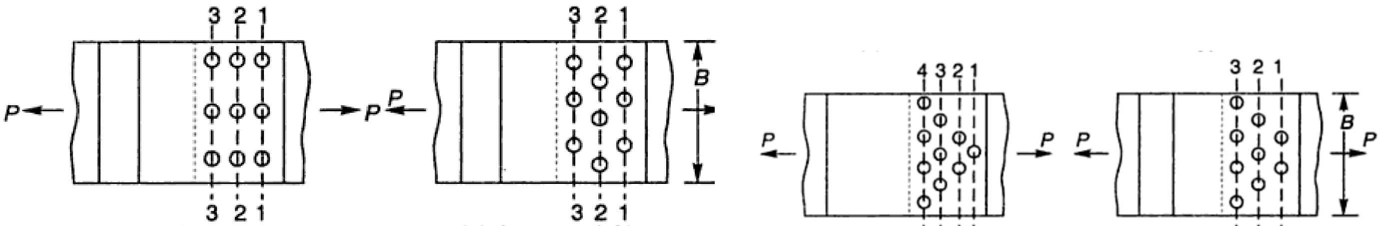
$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1$$

$\left(\frac{17.68}{29}\right)^2 + \left(\frac{17.68}{43.82}\right)^2 = 0.53 < 1$ Hence safe.

Types of Riveting



Diamond or Staggered riveting is better than chain as the net area is less at 1-1 hence more strength.



(a) Chain (b) Staggered Chain (c) Diamond (d) Staggered Diamond

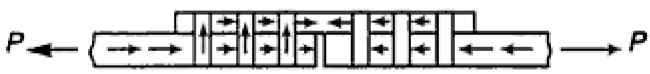
Diameter of the rivets ∅=6.01$\sqrt{t}$ (Unwin formula, it always gives higher value.) t is minimum thickness of the plate.

It is economical and advisable to use small number of larger diameter rather than larger number of small diameter

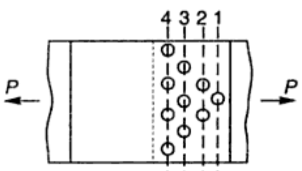
Tensile Strength of Plate

If Tensile load is more than tensile strength of plate, the plate will fail in tension.

The important concept for tensile strength of plate is net area, more then net area more is the tensile strength.

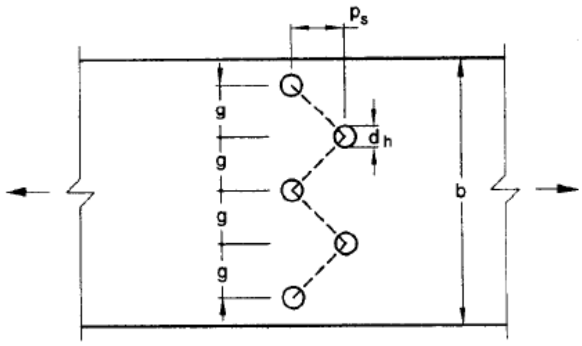


(a)



(c) Diamond

Diamond pattern is most efficient and economical since the net area reduces proportionally, as shown in diamond pattern at section 1-1 net area (of main plate) is more hence more force is resisted by main plate at 1-1 than at section 2-2 and 3-3, the critical section of main plate is 1-1 but the critical section for cover plate is 3-3.



The tensilse strength of plate $T_{nd} = 0.9A_n \frac{f_u}{\gamma_{m1}}$

Where ,

A_n is effective net area of the plate given by

$A_n = (B - nd_h)t$ for chain bolting

$A_n = (B - nd_h + \sum \frac{p_i^2}{4g})t$ for staggered bolting.

f_u is ultimate stress in MPa

γ_{m1}=1.25

Pin Connection

- Pin connections are provided are provided when hinged joints are required.
- Pin connections reduces the secondary stresses.
- Pinned connection can't resist longitudinal tension stresses.
- Only one pin is present in connection, forces acing on a pin are generally greater than bolt.

Shear Capacity

(a) If rotation is not required and the pin is not intended to be remove, $V = 0.6f_{yp}A$

(b) If rotation is required and the pin is intended to be remove, $V = 0.5f_{yp}A$

Bearing Capacity

(a) If rotation is not required and the pin is not intended to be remove, $V = 1.5f_{yp}dt$

(b) If rotation is required and the pin is intended to be remove, $V = 0.8f_{yp}dt$

Flexural Capacity (Flexure is most critical in case of pin)

(a) If rotation is not required and the pin is not intended to be remove, $M_u = 1.5f_{yp}Z$

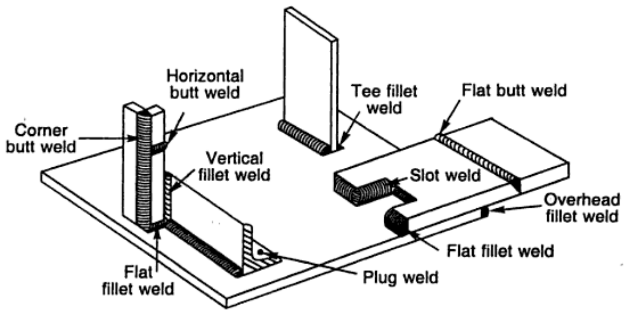
(b) If rotation is required and the pin is intended to be remove, $M_u = 1.0f_{yp}Z$

Welding Connections

Advantage of Welding

(a) Welding connection gives are more efficient use of material and produces one piece member.
(b) The speed of fabrication and erection is fast.
(c) Welding saves weight, hence reduces the cost as plates are reduced or eliminated.
(d) No deduction for holes, thus can carry more tensile load.
(e) Welded joints are better for fatigue loads, impact loads and vibrations.
(f) Welding is best for making rigid connection

Types of Welding



Fillet and butt weld (groove weld) are most frequently used. The butt weld stronger than the fillet weld. Butt welds are provided when the members to be joined are lined up. Fillet welds are provided when two members to be joined are in different planes

Process of Welding

The most common process is arc welding i.e fusion process. The bond between the metals is procduced by reducing the surfaces to be joined to a molten state and then allowing the molten metal to solidify. When the molten metal solidifies union is completed. The temperature required to molten the metal is around 3600^0.

The testing of welds is done through

(a) Magnetic Particle Method
(b) Ultrasonic Method
(c) Dye Penetration Method
(d) Radiography (use of X-rays or gamma rays but used only for groove welds only)

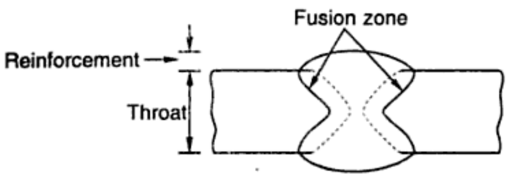
Assumption in Welded Joints

(a) The weld connection are isotropic and homogenous.
(b) The parts connected by the weld are rigid and their deformations are neglected.
(c) Only stress generated by external loading is considered.

Butt Welding (Groove welding)

- Reinforcement make the groove welding stronger in case of static loading but in dynamic loading stress concentration occur in reinforcement leading to early failure. The size of reinforcement should not be greater than 3mm.
- The size of weld is specified by throat dimension (The **throat** of the weld is the distance from the center of the face to the root of the weld.)
- In case of complete penetration of the groove weld the effective throat thickness is taken as thickness of the thinner member jointed but if complete penetration is not achieved the effective throat thickness is taken as 5/8 of thickness of the thinner member.
- The total area of the groove welding $\mathrm{A} = \mathrm{t_e} * \mathrm{l_w}$

  where t_e is effective throat thickness and l_w is length of the weld.



- Strength of Groove weld in tension or compression $\mathrm{T_{dw}} = \frac{\mathrm{f_y * l_{w*} t_e}}{\gamma_{\mathrm{mw}}}$

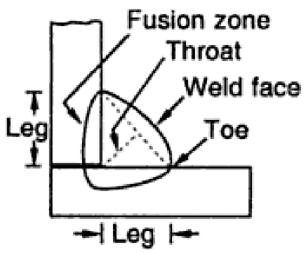
  Where, $\mathrm{f_y}$ is smaller of yield stress of the weld and the parent metal in MPa

  γ_{mw} is partial safety factor 1.25 for shop welding and 1.5 for site welding.
- Strength of butt weld in shear $\mathrm{V_{dw}} = (\mathrm{f_{yw1}} * \mathrm{l_w} * \mathrm{t_e}) / \gamma_{\mathrm{mw}}$

  $\mathrm{f_{yw1}}$ is smaller shear stress of weld and the parent metal $(\mathrm{f_y}/\sqrt{3})$

Fillet Weld

- The size of the fillet weld is specified by dimension of its leg (the leg length if the distance from the root to the toe of the fillet weld.

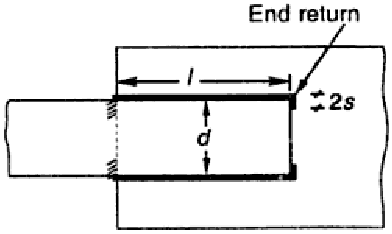


- Maximum size of the weld = thickness of thinner member -1.5mm.
- Minimum size of the filled weld is based on thickness of thicker member. For thicker member thickness 0-10mm the minimum size of the weld is 3mm, 10-20mm the minimum size of the weld is 5mm, 20-32mm the minimum size of the weld is 6mm, 32-50mm the minimum size of the weld is 8mm first run and total thickness is 10mm. It should not be less than 3mm and shall not exceed 0.7t (t is thickness of thinner member)
- The overlap of plates for fillet weld in a lap joint should not be less than four times thickness of thinner member or 40mm whichever is more.

- Effective throat thickness it is the shortest distance from the root of the fillet weld to the face of the weld.
- Effective throat thickness =K x size of the weld (K is constant which depend on the angle between fusion face)

Angle between fusion face	60^0-90^0	91^0-100^0	101^0-106^0	107^0-113^0	114^0-120^0
K	0.7	0.65	0.6	0.55	0.50

- Effective length in no case should be less than 4S. (S is the size of the weld)
- Actual length of the weld is equal to effective length plus 2S
- Effective Area= Effective length x throat thickness



- A fillet weld is designed to fail by shear at an angle of 45^0 through throat.
- Intermittent fillet welds minimum length =4S or 40mm whichever is higher and t he clear spacing should not be more than 12t or 200mm for compression and should not be more than 16t or 200mm for tension.
- The intermittent weld should not be provided for dynamic loading
- If the weld joint exceed 150 times the throat size of the weld, the reduction in the weld as per long joint $\beta = 1.2 - \frac{0.2l_j}{150*t_t} \leq 1.0$ where, l_j is length of joint and t_t= throat size of the weld.
- Effective area of fillet weld = effective length x effective throat thickness.
- **Strength of fillet weld** $P_{dw} = l_w * t_t * \frac{f_u}{\sqrt{3}} * \frac{1}{\gamma_{mw}} = l_w * kS * \frac{f_u}{\sqrt{3}} * \frac{1}{\gamma_{mw}}$

  f_u is smaller of ultimate stress of the weld and the parent metal in MPa
- Stress in weld due to Axial force(P) is given by $f_a = \frac{P}{t_t * l_w}$
- Stress in weld due to Shear force(Q) is given by $q = \frac{Q}{t_t * l_w}$
- **Combination of Stresses (Axial and Shear)**

  Equivalent Stress $f_e = \sqrt{(f_a^2 + 3q^2)} < \frac{f_u}{\gamma_{mw}\sqrt{3}}$
- **Combination of Stresses (Bending, Bearing and Shear)**

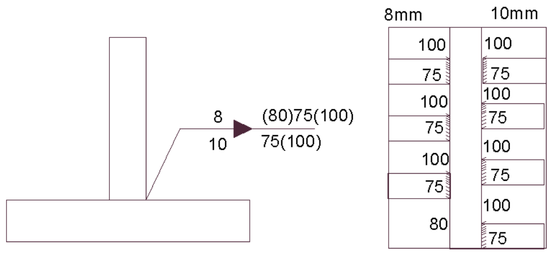
  Equivalent Stress $f_e = \sqrt{f_b^2 + f_{br}^2 + f_b * f_{br} + 3q^2}$

  Note No need to check for combined stress for butt weld if they are axially loaded or the sum normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50% percent of shear strength.

Working Stress Design

- Actual Compressive or tensile or shear stress of a weld should be less than or equal to $f_{aw} = \frac{0.6f_u}{\sqrt{3}}$
- Actual stress in the throat of fillet weld should be less than $f_{aw} = 0.4f_y$

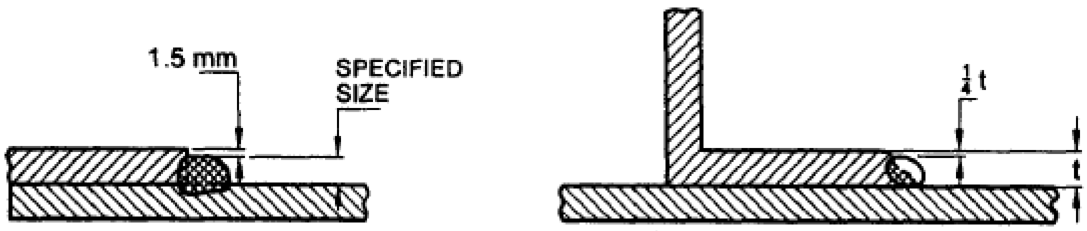
Welding Notation



Extra Clauses of IS800

10.5.8.1 Where a fillet weld is applied to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness.

10.5.8.2 Where the fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed 3/4 of the thickness of the section at the toe.



10.5.8.4 When fillet welds are applied to the edges of a plate, or section in members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thickness of the plate or section,

10.5.8.5 End fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5t

10.7 Minimum Design Action on Connection

1) Connections in rigid construction — a bending moment of at least 0.5 times the member design moment capacity

2) Connections to beam in simple construction — a shear force of at least 0.15 times the member design shear capacity or 40 kN, whichever is lesser

3) Connections at the ends of tensile or compression member — a force of at least 0.3 times the member design capacity

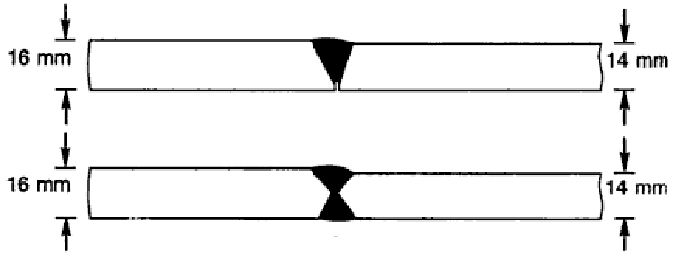
4) Splices in members subjected to axial tension — a force of at least 0.3 times the member design capacity in tension

5) Splices in members subjected to axial compression —. The fasteners shall be sufficient to transmit a force of at least 0,15 times the member design capacity in axial compression. In addition, splices located between points of effective lateral support shall be designed for the design axial force,

P_d plus a design bending moment, not less than the design bending moment $M_d = (P_d l_s)/1000$

where, l_s, is the distance between points of effective lateral support.

Example5 Two plates 16mm and 14mm thickness are to be joined by a groove weld (a) single V groove (b) Double V groove. The joint is subjected to a factored tensile force of 430KN. Effective length of the weld is 175mm, Check the connection. Fe410 $f_y = 250\text{MPa}$ and shop welding.



Solution

(a) For single V groove, throat thickness $t_e = \frac{5}{8}t = \frac{5}{8} * 14 = 8.75\text{mm}$

Strength of weld $T_{dw} = \frac{l_w t_e f_y}{\gamma_{mw}} = 175 * 8.75 * \frac{250}{1.25} * 10^{-3} = 306.25\text{KN} < 430\text{KN}$ hence unsafe.

(b) For double V groove $t_e = 14\text{mm}$

Strength of weld $T_{dw} = \frac{l_w t_e f_y}{\gamma_{mw}} = 175 * 14 * \frac{250}{1.25} * 10^{-3} = 490\text{KN} > 430\text{KN}$ hence safe.

Example6 A tie member in a truss girder is 250mm x 14mm in size. It is welded to a 10mm thick gusset plate by a fillet weld. The overlap of the member is 300mm and the weld size is 6mm. Determine the design strength of the weld. Fe410 $f_y = 250\text{MPa}$ and shop welding

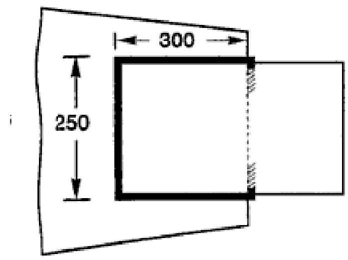
Solution

Effective length of the weld $l_e = 2 * 300 + 250 = 850\text{mm}$

Effective throat thickness $t_t = \text{KS} = 0.7 * 6 = 4.2\text{mm}$

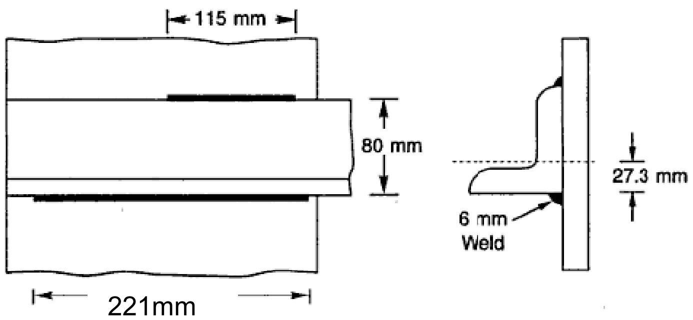
Design Strength of the weld

$$P_{dw} = l_w * kS * \frac{f_u}{\sqrt{3}} * \frac{1}{\gamma_{mw}} = 850 * 4.2 * \frac{410}{1.25\sqrt{3}} * 10^{-3} = 676KN$$



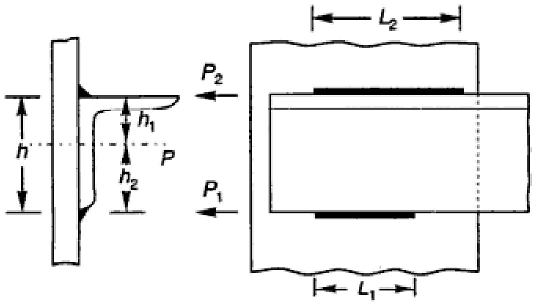
Example7 A tie member consisting of an ISA 80mm x 50mm x 8mm is welded to a 12mm thick gusset plate at site. Design weld to transmit load equal to the design strength of the member =222.27KN. Fe410 $f_y = 250\text{MPa}$

Use 6mm weld



Solution

The center of gravity of weld should coincide with the centroid of the section. If the section is symmetrical than welding is also done in symmetrically but if the member is unsymmetrical like angle, the length of the longitudinal fillet weld are kept different on the two sides as shown in figure.



(a)

Taking the moment about the line passing through length L_1.

$$P_2 h - Ph_2 = 0 \implies P_2 = \frac{Ph_2}{h} \text{ and } P_1 = \frac{Ph_1}{h}$$

Once the load P_1 *and* P_2 are known, the fillet weld length can be designed.

$$P_1 = 222.27 * \frac{80 - 27.3}{80} = 146.42\text{KN}$$

$$P_2 = 222.27 * \frac{27.3}{80} = 75.85\text{KN}$$

Size of the weld is given as 6mm and effective throat thickness will be $0.7 * 6 = 4.2mm$

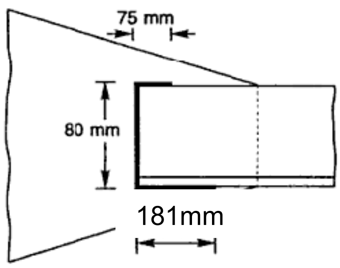
The design strength of the weld,

$$P_{dw} = l_w * kS * \frac{f_u}{\sqrt{3}} * \frac{1}{\gamma_{mw}} = 146.42 * 10^3 = l_{w1} * 4.2 * \frac{410}{1.5\sqrt{3}} \implies l_{w1} = 221mm$$

Similarly

$$75.85 * 10^3 = l_{w2} * 4.2 * \frac{410}{1.5\sqrt{3}} \implies l_{w2} = 114.44mm$$

Example8 Design the fillet weld for the angle section of example6 is weld is to be done on its three sides.



Solution Total weld length is $l_{w1} + l_{w2} + 80\text{mm}$

Load taken by 80mm $P_{dw1} = 80 * 4.2 * \frac{410}{1.5\sqrt{3}} * 10^{-3} = 53$

Remaining load $222.27 - 53 = 169.27\text{KN}$ this load will be distributed in top and bottom.

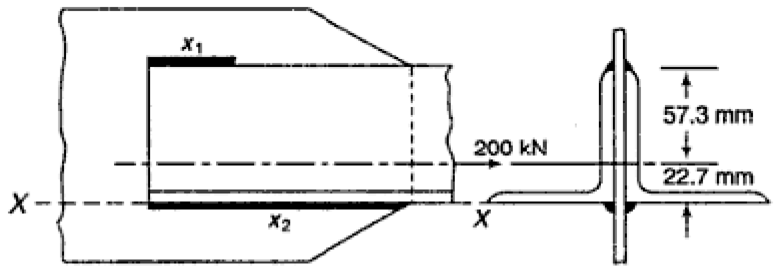
$$P_1 = 169.27 * \frac{80 - 27.3}{80} = 111.50\text{KN}$$

$$P_2 = 169.27 * \frac{27.3}{80} = 57.76\text{KN}$$

$$111.50 * 10^3 = l_{w1} * 4.2 * \frac{410}{1.5\sqrt{3}} \Longrightarrow l_{w1} = 181mm$$

$$57.76 * 10^3 = l_{w2} * 4.2 * \frac{410}{1.5\sqrt{3}} \Longrightarrow l_{w2} = 75mm$$

Example 9 A tie member of a truss consists of double angle section, each 80mm x 80mm x 8mm welded on opposite side of the a 12mm gusset plate as shown in the figure. Find the length of the weld for workshop welding if factored tensile force is 200KN. Size of the weld is 6mm



Solution

Double angle section, so load will divide in each angle equally hence load in each angle 100KN

$$P_1 = 100 * \frac{80 - 27.3}{80} = 65.875\text{KN}$$

$$P_2 = 100 * \frac{27.3}{80} = 34.1\text{KN}$$

$$65.875 * 10^3 = l_{w1} * 4.2 * \frac{410}{1.25\sqrt{3}} \Longrightarrow l_{w1} = 82.82mm$$

$$34.1 * 10^3 = l_{w2} * 4.2 * \frac{410}{1.25\sqrt{3}} \Longrightarrow l_{w2} = 42.87mm$$

TENSION MEMBER

A structural member subjected to axial tension is called tension member or a tie. The member & connection are so arranged that eccentricity in connection and bending stresses are not developed.

A tension member may fail in any of the following mode:-

1) **Gross section yielding** :- Considerable deformation of the member in longitudinal direction may take place before it fractures, making the structure unserviceable hence, we must also consider yielding on gross section.

2) **Net section rupture**:- The fracture of the member occurs when the net section of member reaches the ultimate stress.

(A) Gross section yielding

$$\left[\boldsymbol{T_{dy} = \frac{f_y \quad A_y}{1.1}}\right]$$

* When a tension member is subjected to a tensile force although the net x – section yield first, the deformation within the length of connection will be smaller than deformation in the remainder of the tension member. It is because the net section exist within the small length of member, most of the length of member will have an unreduced x – section, so attainment of yield stress on the gross area will result in larger total elongation.

(B) Net Section Rupture

$$\left[\boldsymbol{T_{dn} = \frac{0.9\, f_u}{\gamma_{m1}} \times An}\right]$$

An: Net effective area of x – section.

$$\boldsymbol{An = B - ndo + \sum_{i=1}^{m} \frac{p_i{}^2}{4g_i}}$$

n = No. of bolt holes.

d_o = dia. of bolt holes.

m = no. of inclined lines

(i) In angles with single leg connected

* Force transferred to one leg by end connection locally gets transferred as tensile stresses over the entire x – section by shear. The connected leg will have higher stresses than the outstanding leg thus as one part of the angle lags behind the other in stress, the process is known as shear lag.

=> Shear leg reduces the effectiveness of outstanding leg, thus unequal angles are connected through the longer legs.

=> The code has recommended following relations for computation of design rupture strength , (T_{DN})

TENSION MEMBER

$$T_{DN} = \frac{0.9\, fu\ A_{nc}}{\gamma_{m1}} + \frac{\beta f_y\ A_{y0}}{\gamma_{m0}}$$

Where, $\beta = 1.4 - 0.076\ \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{l_c}$

$\leq \frac{0.9\, f_u/\gamma_{m1}}{f_y/\gamma_{m0}}$

≥ 0.7

A_{nc} ⟶ net section area of connected leg.

Ago⟶ gross section area of outstanding leg

w ⟶ width of outstanding leg.

t ⟶ thickness of angle.

bs⟶ shear leg width.

Infact "bs" is calculated from the farthest point of outstanding leg to nearest bolt or weld line in the connected leg.

(fig.)

l_c ⟶ length of connection / length of weld along the load direction

(fig.)

=> As the above equation has 't' & 'l_c' terms which can't be determined unless design is complete hence, for preliminary sizing , the rupture strength of the net section may be adopted as

$$\left[T_{dn} = \frac{\alpha fu\ An}{\gamma_{m1}}\right]$$

where α is the reduction factor considering shear lag and depends upon the length of connection i.e, on the no. of bolts per line in the direction of applied load.

$\alpha = 0.6\ for\ one\ two\ bolts$

$\alpha = 0.7$ for three bolts

$\alpha = 0.8$ for four or more bolts.

$\alpha = 0.8$ for welded connection

Note – 1

* From the above expression it is evident that the shear lag effect can be reduced by increasing the length of the end connection.

TENSION MEMBER

→ The T_{DN} has been calculated under the assumption that when the connected leg reaches the ultimate state outstanding leg reaches the yield state.

(ii) Strength When both leg connected

When the both the logs are connected. The angles can be open of can be open up and treated As a plate.

$A_{n-1} = (l_1 + l_2 - t - do)t$

$A_{n\ 2-2} = \left(\beta - 2\ do + \frac{p^2}{4g}\right) t$

$A_n = \min\left(A_{n\ 1-1,}\ A_{n\ 2-2}\right)$

(3) block Shear failure

$$\left.\begin{aligned} T_{db} &= \frac{f_y\ A_{vg}}{\sqrt{3}\,\Upsilon_{mo}} + \frac{0.9\ f_u\ A_{tn}}{\Upsilon_{m1}} \\ T_{db} &= \frac{0.9\ f_u\ A_{vn}}{\sqrt{3}\Upsilon_{m1}} + \frac{f_y\ A_{tg}}{\Upsilon_{mo}} \end{aligned}\right\}$$

Minimum of the two is taken as design strength

In case of welded angle tension member, A_{tn} & A_{vn} should be replaced by A_{tg}& A_{vg} respectively.

* Design tensile strength (T_d)

T_d= $\min\{T_{dg}, T_{dn}, T_{db}\}$.

=> ***Slenderners ratio***

$$\boldsymbol{\lambda = \frac{L_{eff}}{r_{minm}}}$$

- Although there is no stability problem in tension member yet maximum slenderness ratio (λ) is limited to safeguard against the buckling due to load reversals during transportation, erection.
- The other reason to limit the slenderness ratio in tension member is to limit the lateral deflection and prevent vibration.

=> **Maximum slenderness ratio for tension member**

Description	Type equation here.
1. A tension member in which reversal of direct stress due to loads other than wind or earthquake.	180
2. A member normally acting a tension member but in which reversal of loading is due to wind or earthquake.	350
3. A member always in tension except pretensioned member	400

TENSION MEMBER

Lug Angles

- Lug angles is the short length angle section used at joints to connect the outstanding leg of a member there by reducing the length of a joint.
- Bolts connecting the outstanding leg of the main member with the lug angle , should start in advance than all other bolts. This will ensure that force in the outstanding leg is effectively transfer to the lug angle.
- When lug angle are used to connect the angles member, the whole area of the member are effective, no shear lag effect
- Atleast two bolts should be used to attach the leg angles to gusset plate.

(Designing IS code Recommendations)

=> When angle member are main member

(a) Lug angle & the connection with its gusset plate should be ≥ 1.2 (force in outstanding leg of main member

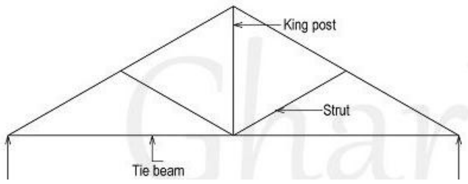
(b) Connection of outstanding log of main member with the lug angle should be designed for a force $\geq$ 1.4 times (Force in outstanding leg)

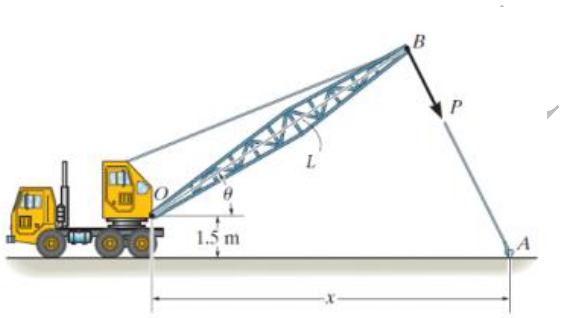
=> When the channel members are member

(a) Lug angles & their connection with gusset plate should be designed for a force ≥ 1.1 (force in outstanding leg)

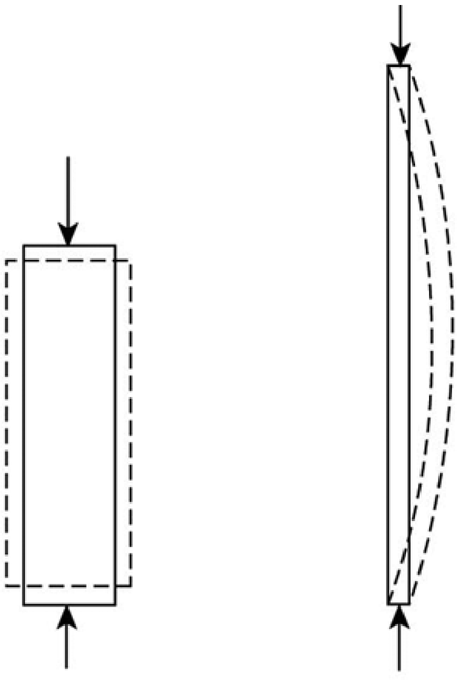
(b) Connection of lug angle with the outstanding leg of main member should be designed for a force ≥ 1.2 (force in outstanding leg)

- A compression member is a structural member which is subjected to two equal and opposite compressive forces.
- Ideal compression member is perfectly straight and load passes through center of gravity of the member but in real life it's impossible to achieve a perfectly axially loaded compression member.
- Compression member is also known as Column, stanchion or post (vertical member), continuous or discontinuous strut (inclined member used in truss),Boom (principal compression member in a crane)





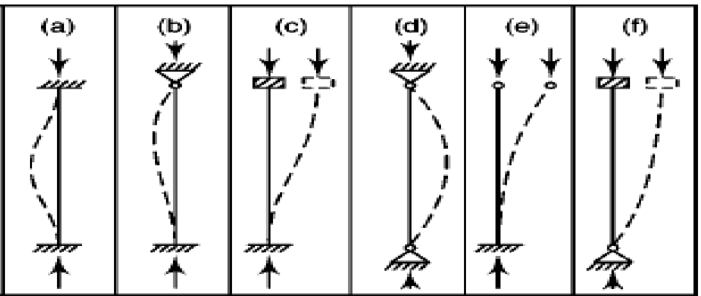
- For this course, we assume Hooks law to be valid and condition of stable equilibrium. (when buckling is involved the compression member is in unstable equilibrium and complex relationship between stress and strain.
- In tension member bending moment lead to straightening of member, in compression member, bending moment leads to buckling hence, compression member with bending moment is more critical than tension member.
- Failure of compression member can be due to (1) Crushing or Yielding (2) Buckling.
- In Yielding failure, member is absolutely straight, concentric loading, member is homogenous but in buckling compressive loading causes column to deflect or buckle slightly. This deflection increases the eccentricity and thus the bending moment. This may go on to a point where bending moment increases so much that column fail in buckle.
- Types of Column (1) Long Column, fail by elastic buckling. (2) Intermediate Column by yielding and buckling (3) Short Column by crushing or yielding.(column shorten in the direction of the loading)



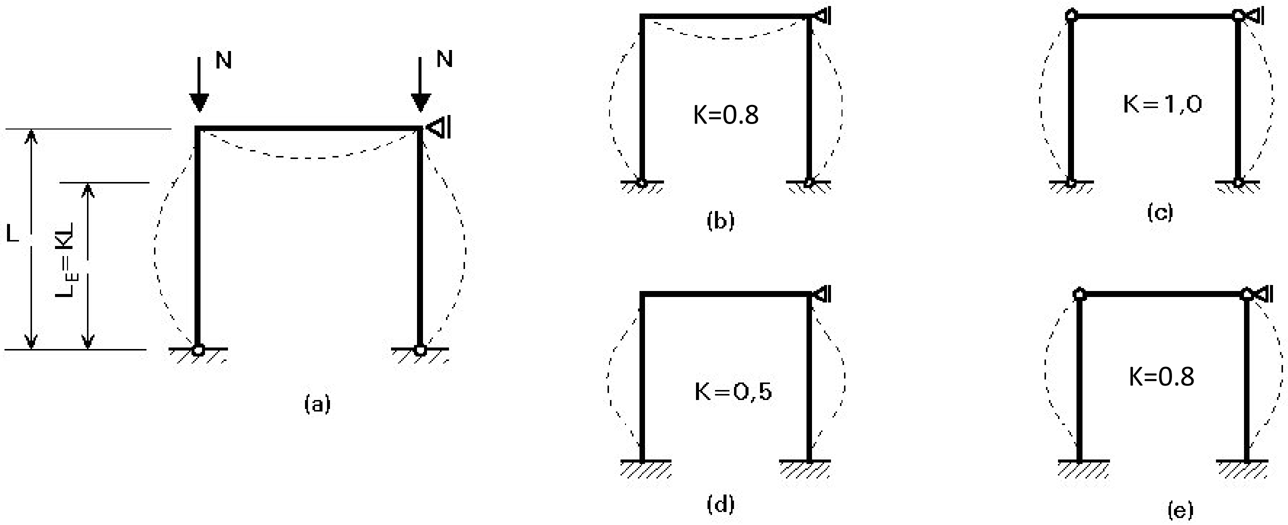
(a) Short column (b) Long column

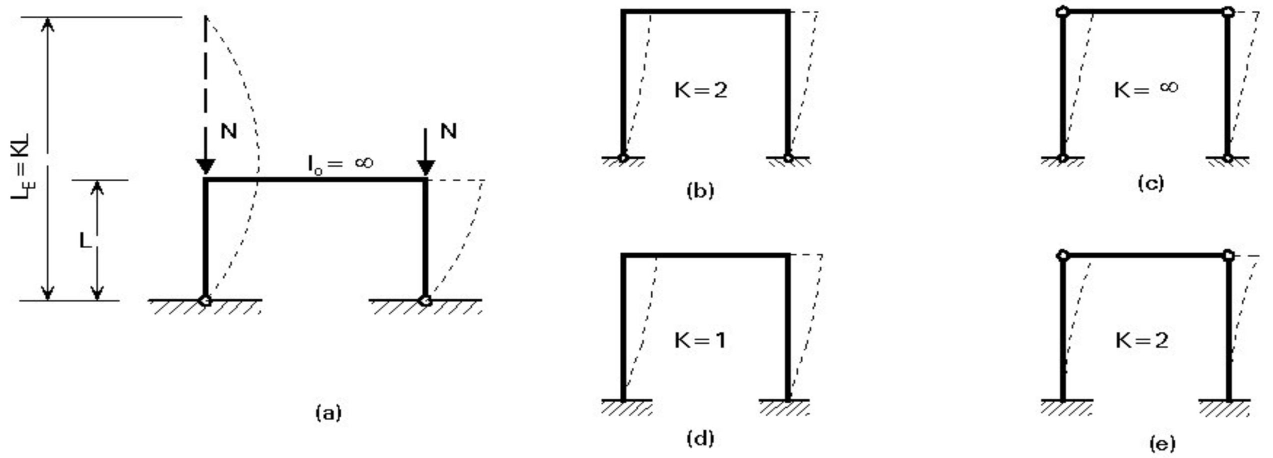
Effective Length

- It is the distance between point of contraflexure, (it depend on end condition) $Le = KL$
- The smaller the effective length of the particular compression member, the smaller is the chances of lateral buckling and the more is the load taking capacity.

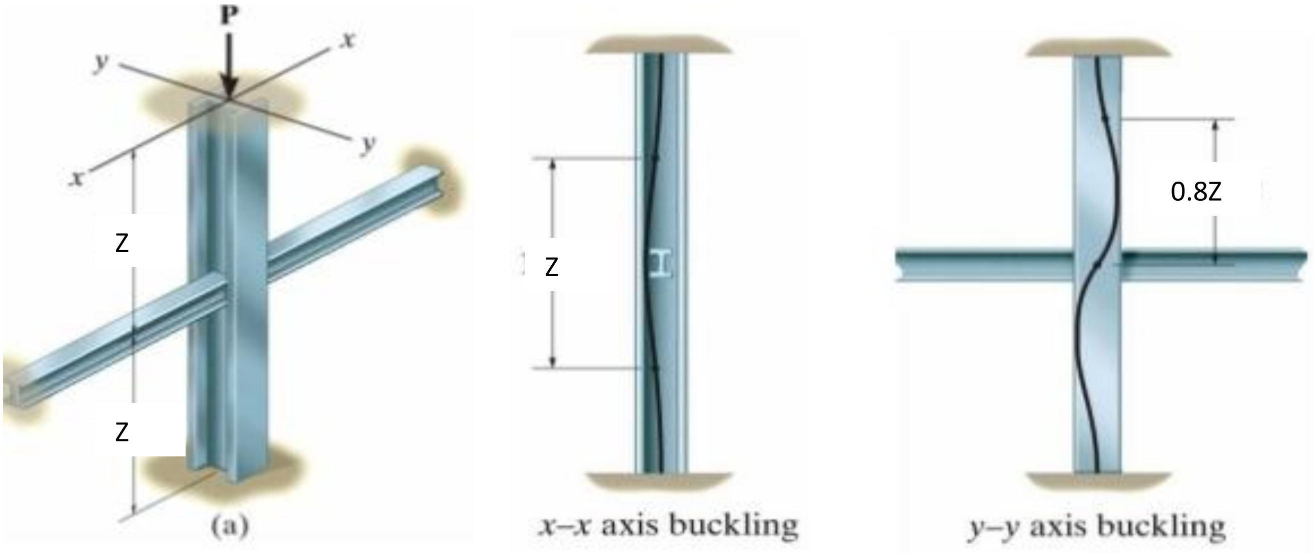


Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended K value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.0	2.0
End condition code		Rotation fixed, translation fixed				
		Rotation free, translation fixed				
		Rotation fixed, translation free				
		Rotation free, translation free				





- Major and Minor Axis ⟹ For the given I section, $I_{xx} > I_{yy}$, so buckling is possible about Y-Y axis, hence translation restriction in perpendicular direction is applied to the weak axis of the cross section. Now $L_{eff}, x - x = 0.5 * 2z = z \; and \; L_{eff}, y - y = 0.8 * z$



- IS Code Method for finding Effective length (ANNEX D) based on Wood curve

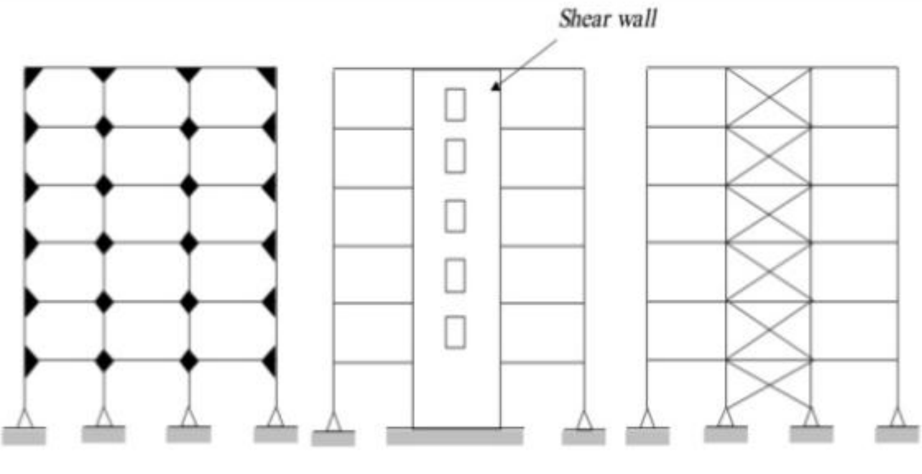
For non-sway frames (braced frames):

$$K = \frac{[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2]}{[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2]}$$

For sway frames (moment-resisting frames):

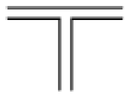
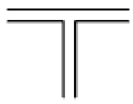
$$K = \left\{\frac{[1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2]}{[1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2]}\right\}^{0.5}$$

⟹ A non sway frames is frame in which lateral sway is prevented, this can be done by proving diagonal bracing in the frame, providing shear wall or making the joints rigid.



(a) Moment resisting frames (b) Shear wall frames (c) Braced frames

- Effective Length of Angle Struts

Type	End connection	Effective length
Discontinuous single angle	(a) One bolt or rivet	Distance between the centre of end fastening (as per IS 800-1984)
	(b) Two or more bolts or Rivets or equivalent Welding	0.85 of distance between node points (as per IS 800-1984)
Discontinuous double angle, stitched together by bolts or welding at regular intervals	(a) Same side of gusset 	
	(i) One bolt or rivet	Distance between centre of end fastenings
	(ii) Two bolts or rivets or equivalent welding	0.7–0.85 of distance between nodes
	(b) Both sides of gusset	
	(i) One bolt or rivet 	Distance between centre of end fastenings
	(ii) Two bolts or rivets or equivalent welding	0.7-0.85 of distance between nodes
	In a plane perpendicular to that of end gusset —for both (a) and (b)	Distance between centre of end fastening
Continuous angles (e.g., top and bottom chords of trusses, tower legs)	(a) Continuous	0.7–1.0 of distance between nodes
	(b) In a plane perpendicular to the plane of truss	Distance between centres of nodes

SLENDERNESS RATIO

- The tendency of a member to buckle is measured in slenderness ratio (λ) $\lambda = \frac{l_{eff}}{}$
- The column buckle about the axis having maximum slenderness ratio or we can say having minimum radius of gyration.
- $r = \sqrt{\frac{I}{A}}$ and $r_{min} = \sqrt{\frac{I_{min}}{A}}$ (A idealized section is one have same, moment of inertia is all direction.)
- The radius of gyration can be increased by spreading the material of section away from its axis.
- Round Tubes are excellent sections for compression member, since they have same radius of gyration in all direction.
- Single angle sections least radius of gyration is less than channels or I section, hence they should be avoided as much as possible.
- Double angle back to back are most suited for trusses, also two angles can also be used in the form of star (cruciform section) and box section can also be used.
- Two channel placed face to face provide a larger value of radius of gyration as compared to channel placed back to back and separated apart for the for the same spacing, thus two channels fact to face are ideal section for compression member .

Maximum Limit of Slenderness Ratio

Table 3 Maximum Values of Effective Slenderness Ratios

Sl No. (1)	Member (2)	Maximum Effective Slenderness Ratio (KL/r) (3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
iii)	A member subjected to compression forces resulting only from combination with wind/earthquake actions, provided the deformation of such member does not adversely affect the stress in any part of the structure	250
iv)	Compression flange of a beam against lateral torsional buckling	300
v)	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake forces[1]	350
vi)	Members always under tension[1] (other than pre-tensioned members)	400

Local Buckling

- Most Structural member are composed of flat plate element i.e they can be regarded as combination of individual plate element connected together to form the shape, while considering the stability in compression the stability of these component plate must also be considered.



- Local Buckling adversely affects the load carrying capacity of columns and beams due to reduced stiffness and strength of the buckled plate.

- The local bucking can be prevented by adopting higher thickness of elements i.e by controlling width to thickness ratio.

- When elastic analysis or Plastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling.

Four Classes of section are defined as per IS800

a) Class 1 (Plastic) — Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The width to thickness ratio of plate element shall be less than that specified under Class 1
(Plastic), in Table 2.
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 thickness 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 2.
c) Class 3 (Semi-compact) — Cross-sections, 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 thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact), in Table 2.
d) Class 4 (Slender) — Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under Class 3 (Semi-compact), in Table 2. In such cases, the effective 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

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4)

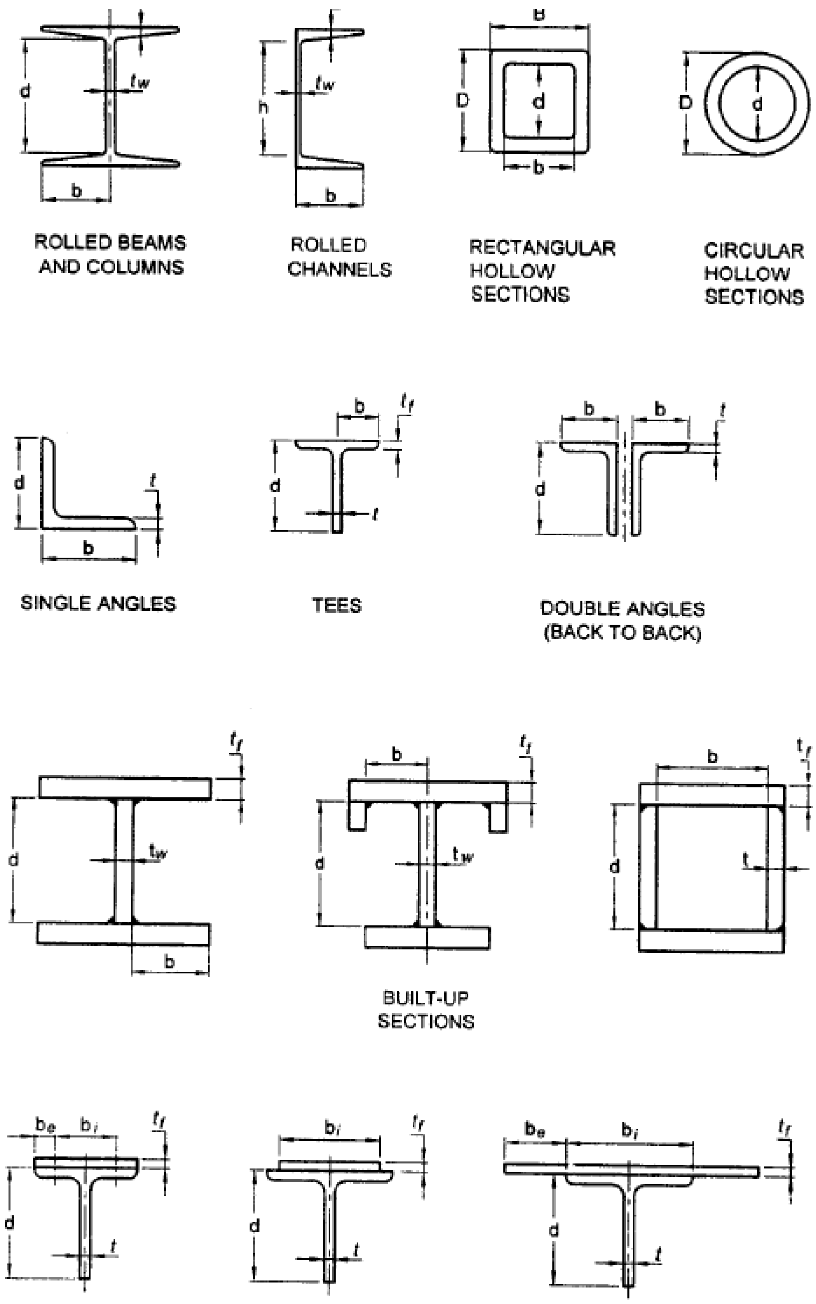
Compression Element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)			(2)	(3)	(4)	(5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\leq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross-section.
2 $\varepsilon = (250 / f_y)^{1/2}$.
3 Webs shall be checked for shear buckling in accordance with **8.4.2** when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Fig. 2, **3.7.3** and **3.7.4**).
4 Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.
5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overll section}}$$

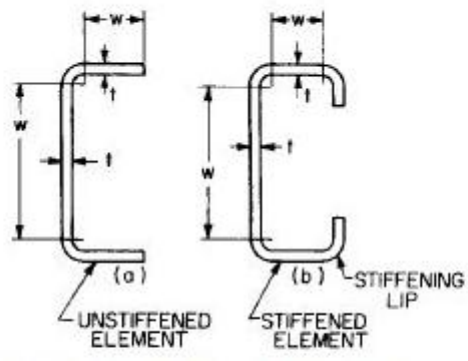


COMPOUND ELEMENTS

b_i — Internal Element Width

b_e — External Element Width

FIG. 2 DIMENSIONS OF SECTIONS

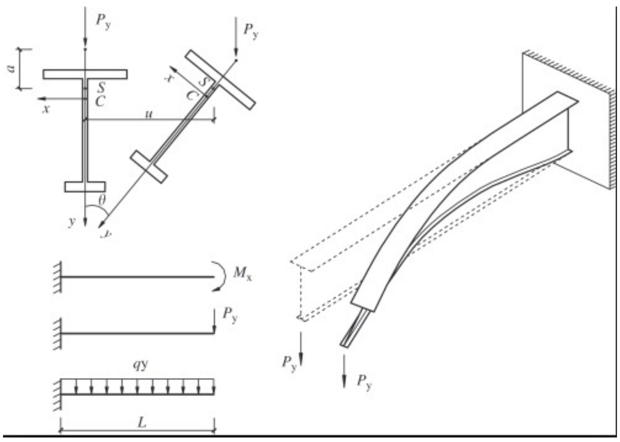


- As per IS 801, the maximum (b/t) for compression elements is
  (A) Stiffened elements with one longitudinal edge connected to a flange =60
  (B) Stiffened elements with both longitudinal edge connected to other stiffened elements =500
  (C) Unstiffened compression elements=60

- Flexural Buckling (Euler Buckling) ⟹It is the bending of compression member about axis having largest slenderness ratio usually minor principal axis, in this type of buckling member undergo translation without rotation.

- Torsional Buckling ⟹ This type of buckling occur by twisting about the longitudinal axis and this occur when the torsional rigidity of the member is smaller than bending rigidity. It can only occur with double symmetrical cross section with very slender cross sectional area, hence standard hot rolled member are not susceptible to torsional buckling. The star shape angle (cruciform shape) is most vulnerable to this type of buckling In this case the flange under compression is free to move laterally and also twist. The buckling will be seen in the compression flange of a simply supported beam.

- Flexural Torsional Buckling ⟹ In this type of buckling, member bends and twists simultaneously. This type of failure occurs only with unsymmetrical cross section with one axis of symmetry and with no axis of symmetry.

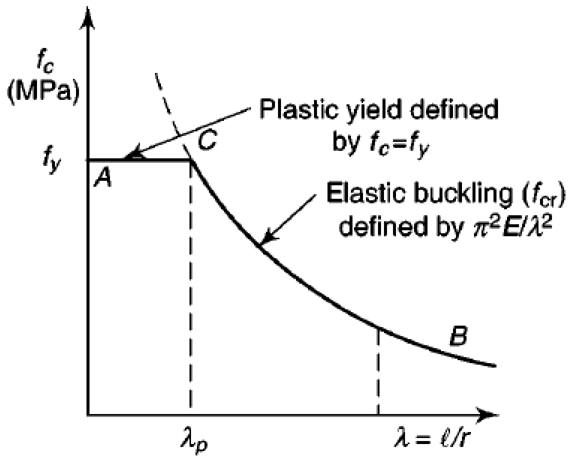


Elastic & Inelastic Buckling

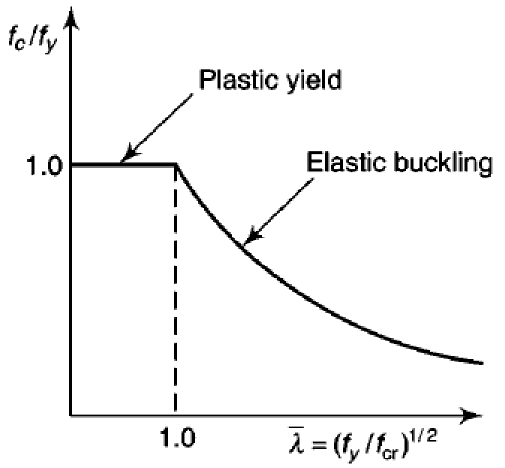
A compression member is usually subjected to two modes of failure (A) Material Failure (yielding) (B)Geometric Failure (slender column) , a column will either govern in yielding failure or be slender i-e fail in elastic buckling.

Elastic buckling occurs before yielding of material of which the member is made.

And in case if the material of the member yields first followed by buckling in the inelastic zone , it is called plastic buckling.



Strength curve
(a)



Strength curve in a non-dimensional form
(b)

Compressive Strength

- As per IS800, 2007 compressive strength depend on (a) effective length (b) Yield strength of the steel (c) on local buckling. IS800, 1984 uses Mercant Rankine Formula but in that local Buckling was not considered.
  - Based on Perry Robertson approach (British code BS-5950)

$$P_d = A_c f_{cd}$$

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

where

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

λ = non-dimensional effective slenderness ratio

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

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(KL/r\right)^2}$$

- IS800 Propose multiple column curves a,b,c,d in non-dimensional form (Based on Perry Robertson approach), These curves are for different cross section (Table-10 of IS-800), the direction in which buckling can occur and the fabrication process (hot rolled, welded or cold formed)
- The imperfection factor α , increases with imperfection.

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

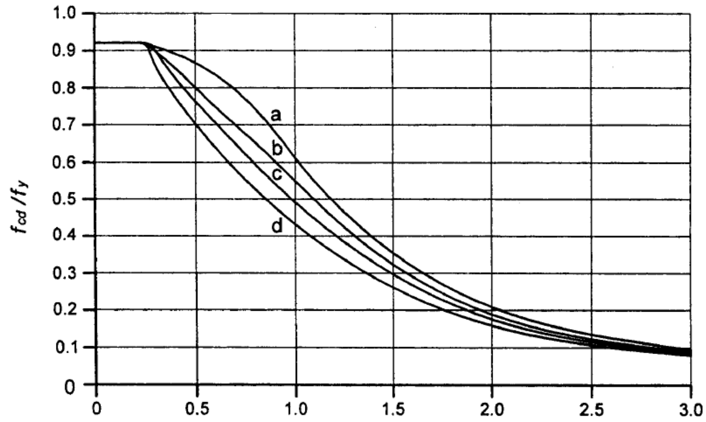


FIG. 8 COLUMN BUCKLING CURVES

$$\lambda = \sqrt{fy/fcc}$$

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z y-y	a b
	$40 \leq$ mm $< t_f \leq 100$ mm	z-z y-y	b c
	$h/b_f \leq 1.2$: $t_f \leq 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \leq 40$ mm	z-z y-y	b c
	$t_f > 40$ 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 $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections		Any	c
Built-up Member		Any	c

- For same member Buckling curve is different for different axis
- Curve a for hollow (tubular section) and Curve c solid circular section and both curve members are initially straight and free from eccentric loading but residual stresses are more in Curve c than in Curve a. (hollow section have uniform temperature variation but solid section have non uniform variation)
- The most important is curve c for any axis for channel, Angel,T and solid section.

- If you use formula as given in code than Table 8(a) to 8(d) can be used and if you can want to use only table than table 9(a) to 9(d) can be used.

Example1 Design axial load of the column section ISMB400, height of the column is 3.5m and both end pinned.

f $\overline{mm^2}$

Sectional Properties $Depth\ of\ the\ section, h = 400mm,\ \ Flange\ width, b = 140mm, t_f = 16mm,$

$$t_w = 8.9mm, A = 7846mm^2, r_{z-z} = 161.5mm\ and\ r_{y-y} = 28.2mm$$

Solution First step is to find the Buckling Curve using Table 10,

$$\frac{h}{b} = \frac{400}{140} = 2.86 > 1.2 \quad and\ t_f = 16 < 40mm$$

Hence for z-z axis it will be curve (a) and for y-y axis it will be curve (b)

Strength about z-z axis

$$\lambda_z = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{f_y * \left(\frac{KL_t}{r_z}\right)^2 / (\pi^2 E)} = \sqrt{250 * \frac{\left(\frac{3500}{161.5}\right)^2}{3.14^2 * 2 * 10^5}} = 0.2439$$

From Table 7 curve (a), $\alpha_z = 0.21$

$$\emptyset = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(0.2439 - 0.2) + 0.2439^2] = 0.534$$

$$(\mathrm{f_{cd}})_{z-z} = \frac{\frac{\mathrm{fy}}{\gamma \mathrm{mo}}}{\emptyset + (\emptyset^2 - \lambda^2)^{0.5}} = \frac{\frac{250}{1.1}}{0.534 + (0.534^2 - 0.2439^2)^{0.5}} = 225.2\,\frac{N}{mm^2}$$

Similarly for y-y axis

$$\lambda_y = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{f_y * \left(\frac{KL_t}{r_z}\right)^2 / (\pi^2 E)} = \sqrt{280 * \frac{\left(\frac{3500}{28.2}\right)^2}{3.14^2 * 2 * 10^5}} = 1.3968$$

From Table 7 curve (b), $\alpha_y = 0.34$

$$\emptyset = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.34(1.3968 - 0.2) + 1.3968^2] = 1.679$$

$$(\mathrm{f_{cd}})_{y-y} = \frac{\frac{\mathrm{fy}}{\gamma \mathrm{mo}}}{\emptyset + (\emptyset^2 - \lambda^2)^{0.5}} = \frac{\frac{250}{1.1}}{1.679 + (1.679^2 - 1.3968^2)^{0.5}} = 87.06\,\frac{\mathrm{N}}{\mathrm{mm}^2} < 225.2\,\frac{N}{mm^2}$$

$$P_d = A_e * f_{cd} = 7846 * 87.06 = 683.07 * 10^3 N$$

OR

We can directly find $(f_{cd})_{y-y}$ using Table 9

$\lambda_{y-y} = \frac{KL_y}{r_y} = \frac{3500}{28.2} = 124.11$ for $f_y = 250\ and\ Class(b)$ using Table 9 and Interpolation

$$f_{cd} = 91.7 - \frac{91.7 - 81}{10} * 4.1 = 87.3\,\frac{N}{mm^2}$$

Single Angle Struts

- The compressive load in angle can be transferred by two ways (A) concentrically loaded to its centroid through end gusset OR (B) eccentrically loaded by connecting one of its leg to a gusset plate.
- The compressive load taking capacity will be calculated same as explanted above for case (A) i.e concentrically loaded to its centroid through end gusset
- But, if load is acted on connecting leg than the method will be different as explained below.

7.5.1.2 *Loaded through one leg*

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e, as given below:

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

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

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

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio $(250/f_y)^{0.5}$.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

Example2 ISA 150x150x12 Strut with length3m connected with (A) one Bolt (B) Two bolt at each end (C) welded at each end. Find the load taking Capacity. Given Area=3459mm^2, $r_{vv} = 29.3mm$, $\alpha = 0.49$

Solution Using Curve (c), $Table - 7 \quad \alpha = 0.49, (imperfection\ factor)$

(A) One Bolt

Table 12, $K_1 = 0.75, K_2 = 0.35, K_3 = 20$ $\qquad \varepsilon = \sqrt{\frac{250}{250}} = 1$ and Length $L = 3000mm$

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{3000}{29.3}\right)}{1 * \sqrt{\frac{\pi^2 * 2 * 10^5}{250}}} = 1.1523 \quad \text{Clause (7.5.12)}$$

$$\lambda_\emptyset = \frac{\frac{b_1 + b_2}{2t}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{150+150}{2*12}}{1 * \sqrt{\frac{\pi^2 * 2 * 10^5}{250}}} = 0.1407$$

$$\lambda_e = \sqrt{K_1 + k_2\lambda^2_{vv} + k_3\lambda^2_\emptyset} = \sqrt{0.75 + 0.35 * 1.1523^2 + 20 * 0.1407^2} = 1.2692$$

$$\emptyset = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.49(1.2692 - 0.2) + 1.2692^2] = 1.5672$$

$$(f_{cd})_{y-y} = \frac{\frac{fy}{\gamma_{mo}}}{\emptyset + (\emptyset^2 - \lambda^2)^{0.5}} = \frac{\frac{250}{1.1}}{1.5674 + (1.5674^2 - 1.2692^2)^{0.5}} = 91.38\frac{N}{mm^2}$$

$$P_d = A_e * f_{cd} = 3459 * 91.38 = 316.1 * 10^3 N = 316.1KN$$

(B) Two Bolt & Welded

Table 12, $K_1 = 0.2\ , K_2 = 0.35, K_3 = 20$ $\qquad \varepsilon = \sqrt{\frac{250}{250}} = 1$ and Length $L = 3000mm$

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{3000}{29.3}\right)}{1 * \sqrt{\frac{\pi^2 * 2 * 10^5}{250}}} = 1.1523 \quad \text{Clause (7.5.12)}$$

$$\lambda_\emptyset = \frac{\frac{b_1 + b_2}{2t}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{150+150}{2*12}}{1 * \sqrt{\frac{\pi^2 * 2 * 10^5}{250}}} = 0.1407$$

$$\lambda_e = \sqrt{K_1 + k_2\lambda^2_{vv} + k_3\lambda^2_\emptyset} = \sqrt{0.2 + 0.35 * 1.1523^2 + 20 * 0.1407^2} = 1.03$$

$$\emptyset = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.49(1.03 - 0.2) + 1.03^2] = 1.2338$$

$$(f_{cd})_{y-y} = \frac{\frac{fy}{\gamma_{mo}}}{\emptyset + (\emptyset^2 - \lambda^2)^{0.5}} = \frac{\frac{250}{1.1}}{1.2338 + (1.2338^2 - 1.03^2)^{0.5}} = 118.8\frac{N}{mm^2}$$

$$P_d = A_e * f_{cd} = 3459 * 118.8 = 316.1 * 10^3 N = 410.9KN$$

Double Angel Struts

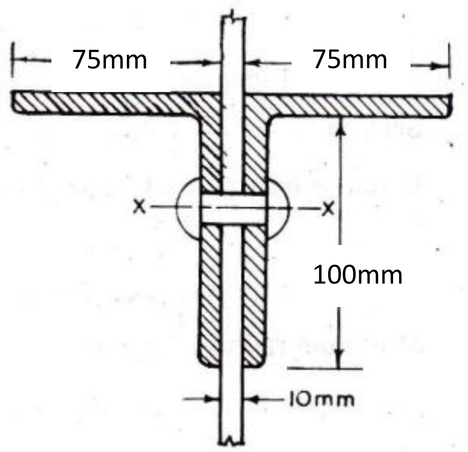
- For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centers of intersections.
- Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed same as single angel struts.
- Compression members composed of two angles, channels, or tees back-to-back in contact or separated by a small distance, shall be connected together by riveting, bolting or welding so that the ratio of most unfavorable slenderness of each member between the intermediate connections is not greater than 40 or 0.6 times the most unfavorable ratio of slenderness of the strut as a whole.
- In no case shall the ends of the strut be connected together with less than two rivets or bolts or their equivalent in welding, and there shall be not less than two additional connections in between, spaced equidistant along the length of the strut. Where the members are separated back-to-back, the rivets or bolts through these connections shall pass through solid washers or packing in between. Where the legs of the connected angles or the connected tees are 125 mm wide or more, or where webs of channels are 150 mm wide or over, not less than two rivets or bolts shall be used in each connection, one on line of each gauge mark.
- Compression members composed of two angles, channels, or tees back-to-back in contact or separated by a small distance, shall be connected together by riveting, bolting or welding so that the ratio of most unfavorable slenderness of each member between the intermediate connections is not greater than 40 or 0.6 times the most unfavorable ratio of slenderness of the strut as a whole, whichever is less.

Example3 A discontinuous strut of length 4m consist of two unequal angles ISA100x75x8 and connected by 10mm thick Gusset plate by its longer leg. Find the compressive strength if the angles are connected

(A) Opposite side of the Gusset Plate (B) Same side of Gusset Plate

Sectional Properties $Area = 1336mm2, r_x = 34.4mm, r_y = 21.8, r_{uu} = 34.8, r_{vv} = 15.9, I_x = 131.6 * 104, I_y = 63.3 * 104mm4, C_x = 31.0mm\ and\ C_y = 18.7mm$

Solution (A) Opposite side of the Gusset Plate



$$A' = 2 * 1336 = 2672, \qquad I' = 2 * I_x \qquad r' = \sqrt{\frac{2I}{2A}} = 31.4\,mm$$

$$I' = 2 * \left[I_y + A\left(C_y + \frac{t_g}{2}\right)^2\right] * 2 = 2\left[63.3 * 10^4 + 1336 * \left(18.7 + \frac{10}{2}\right)^2\right] = 276.68 * 10^4\,mm^4$$

$$r_y' = \sqrt{\frac{I'}{A}} = \sqrt{\frac{276.68 * 10^4}{2672}} = 32.18$$

$$r'_{min} = minimum\ of\ r'\ or\ r' = 31.4$$

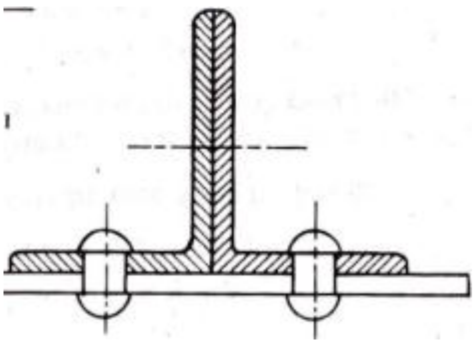
$$L_{eff} = 0.85 * L = 0.85 * 4000 = 3400mm$$

$$\lambda = \frac{L_{eff}}{r_{min}} = \frac{3400}{31.4} = 108.28 < 180$$

For Buckling Curve C, Table 9(C) $f_{cd} = 107 - \frac{107 - 94.6}{10} * 8.28 = 96.73$

$$P_d = A_e * f_{cd} = 2672 * 96.73 = 258.46KN$$

(A) Same side of the Gusset Plate



$$A' = 2 * 1336 = 2672, \qquad I' = 2 * I_y \qquad r' = \sqrt{\frac{2I}{2A}} = 21.8\,mm$$

$$I' = 2 * [I_x + A(C_x)^2] = 2[131.6 * 10^4 + 1336 * (31)^2] = 519.98 * 10^4\,mm^4$$

$$r_x' = \sqrt{\frac{I'}{A}} = \sqrt{\frac{519.98 * 10^4}{2672}} = 44.11$$

$$r'_{min} = minimum\ of\ r'\ or\ r' = 21.8$$

$$L_{eff} = 0.85 * L = 0.85 * 4000 = 3400mm$$

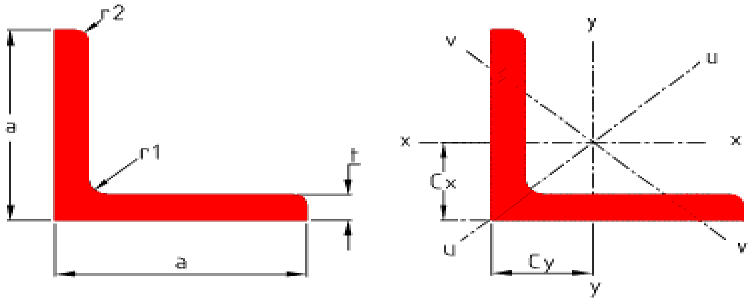
$$\lambda = \frac{L_{eff}}{r_{min}} = \frac{3400}{31.4} = 155.96 < 180$$

For Buckling Curve C, Table 9(C) $f_{cd} = 59.2 - \frac{59.2 - 53.3}{10} * 5.96 = 55.68$

$$P_d = A_e * f_{cd} = 2672 * 55.68 = 148.78KN$$

<u>Example4</u> A discontinuous strut of length 3m consist of two equal angles ISA 90x90x6 and connected by 10mm thick Gusset plate at both the legs to form a star cross section. Find the strength if it is connected to form a star cross section.

Sectional Properties:- $Area = 1047mm^2, r_x = r_y = 27.7, r_{uu} = 35, r_{vv} = 17.5, I_x = 131.6 * 10^4, I_y = 63.3 * 10^4 mm^4, C_x = C_y = 24.2mm$



Solution $A = 2 * 1047 = 2094mm^2$

$$I' = 2 * \left[I_x + A\left(C_x + \frac{t_g}{2}\right)^2\right], \quad r' = \sqrt{\frac{I_x}{A}} \Rightarrow \sqrt{\frac{2*\left(I_x + A\left(C_x + \frac{t_g}{2}\right)^2\right)}{2*A}} = \sqrt{r^2 + \left(C_x + \frac{t_g}{2}\right)^2}$$

Since, section is symmetrical $r' = r' = \sqrt{27.2^2 + \left(24.2 + \frac{10}{2}\right)^2} = 40.25$

$$r_v' = \sqrt{r^2 + \left(C_v + \frac{t_g}{2cos45}\right)^2} \quad C_v = \frac{C_y}{cos45} = \sqrt{2}C_y$$

$$r_v' = \sqrt{r^2 + 2\left(C_y + \frac{t_g}{2}\right)^2} = \sqrt{17.5^2 + 2 * \left(24.2 + \frac{10}{2}\right)^2} = 44.85mm$$

$$r_u' = r_u = 35mm$$

Minimum radius of gyration from r', r', r', r' *is* r' *and the value is* $35mm$

$$\lambda = \frac{L_e}{r_{min}} = \frac{2550}{35} = 72.16 < 180$$

From Table 9(c), $f_{cd} = 152 - \frac{152-136}{10} * 2.86 = 147.42Mpa$

$$P_d = 147.42 * 2094 = 308.7KN$$

For Design of tack Welding the IS code Clause 7.8.1 $(\lambda_e)_{welding} \leq 0.6\lambda$ *or* 40 *which ever is less*

$$= 0.6 * 72.86 = 43.72 > 40, \textit{hence take } 40$$

$$\lambda_e = \frac{S}{r_v} = 40 \Longrightarrow S = 40 * 17.5 = 700mm$$

Load taking capacity of weld=2.5% of 308.7=7.71KN

Assume weld size 5mm, then length of the weld will $L = \frac{\frac{P}{t_e * f_y}}{\sqrt{3} * \gamma_{mw}} = \frac{7.71 * 10^3 * \sqrt{3} * 1.25}{0.7 * 5 * 410} = 11.63 \text{ or } 12mm$

Bending and Axial Compression

Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships:

$$\frac{P}{P_{dy}} + K_y \frac{C_{my} M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} \leq 1.0$$

$$\frac{P}{P_{dz}} + 0.6 K_y \frac{C_{my} M_y}{M_{dy}} + K_z \frac{C_{mz} M_z}{M_{dz}} \leq 1.0$$

where

$C_{my,} C_{mz}$ = equivalent uniform moment factor as per Table 18;

P = applied axial compression under factored load;

$M_{y,} M_z$ = maximum factored applied bending moments about y and z-axis of the member, respectively;

$P_{dy,} P_{dz}$ = design strength under axial compression as governed by buckling about minor (y) and major (z) axis respectively;

M_{dy}, M_{dz} = design bending strength about y (minor) or z (major) axis considering laterally unsupported length of the cross-section (*see* Section 8);

$K_y = 1 + (\lambda_y - 0.2) n_y \leq 1 + 0.8\, n_y$;

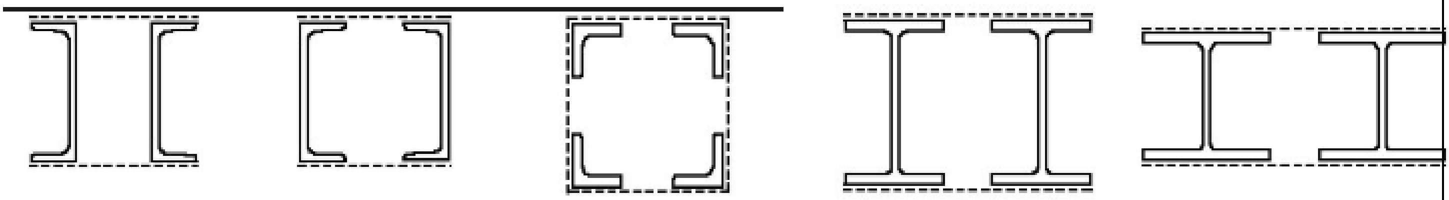
$K_z = 1 + (\lambda_z - 0.2) n_z \leq 1 + 0.8\, n_z$; and

$$K_{LT} = 1 - \frac{0.1 \lambda_{LT}\, n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1\, n_y}{(C_{mLT} - 0.25)}.$$

Built-Up Section

- Built-up Sections are used when (A) Load is very heavy and rolled steel sections are not adequate to take up the loading.(B) To have equal radius of gyration in both the direction i.e to increase the least radius of gyration to maximum $2Izz = 2\left[Iyy + A\left(\frac{s}{2} + Cyy\right)^2\right]$ if 's' is the spacing between section. (C) For customized special shape and size as required.

- Built-up Column always Buckle in Class (c)



Example5 Find the spacing between the two channel ISMC300 placed back back such that minimum radius of gyration is maximum.
Sectional Properties are $Area = 4564mm^2, r_{z-z} = 118.1mm, r_y = 26.1mm$, $I_{z-z} = 6362.6 * 10^4$,

$$I_y = 310.8 * 10^4 mm^4, C_{y-y} = 23.6mm$$

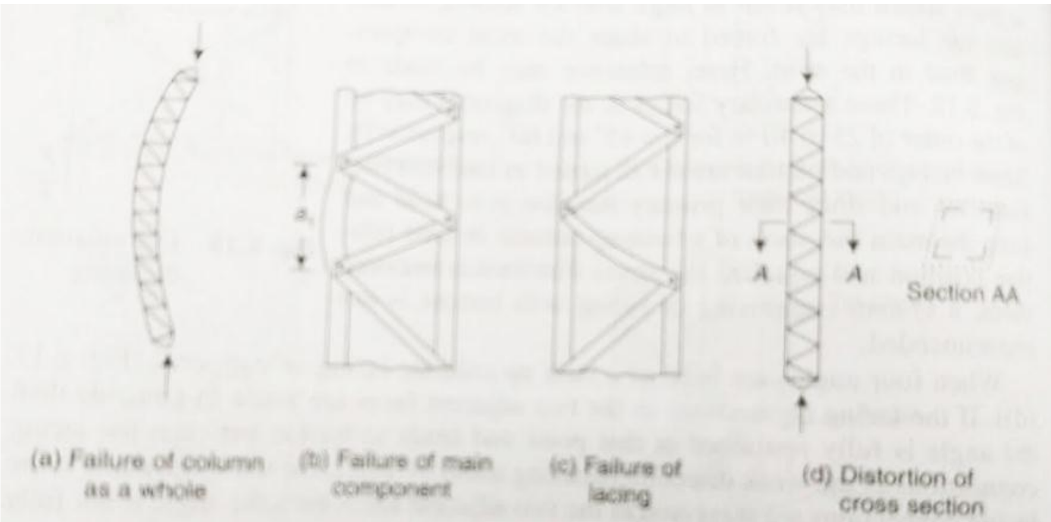
Solution $2I_{zz} = 2\left[I_{yy} + A\left(\frac{s}{2} + C_{yy}\right)^2\right]$

$$2 * 6362.6 * 10^4 = 2\left[310.8 * 10^4 + 4564\left(\frac{s}{2} + 23.6\right)^2\right]$$

$$s = 183.1mm$$

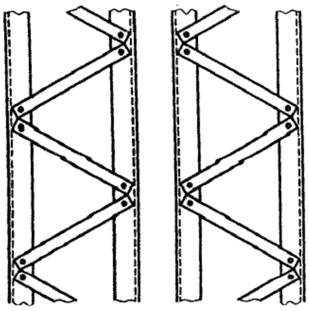
Lacing System

- Lacing system is used in built-up section to make both the section monolithic and not to take the axial load. Due to Transeverse shear force, lacing system is acted by Compression and Tension both.
- Lacing can be angle, channel, Tube, Flat Plate.
- Decision of Single lacing or double lacing system depend on distribution of force.
- For eccentric loading, Lacing System is preferred over Batten system
- Two Tie plate or Batten must be used at both the ends of the lacing and at points where lacing system are interrupted.
- Failure of Lacing System (a) Failure of the column as a whole (b) Failure of main compenent (c) Failure of Lacing (d) Distortion of cross section



- C-7.6.1.3⟹ Except for tie plates as specified in 7.7, double laced systems (see Fig. 10B) and single laced systems (see Fig. 10A) on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut (see Fig. 10C), unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

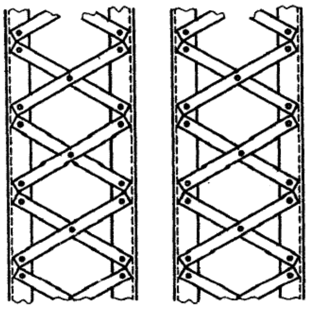
- C-7.6.1.4 ⟹ Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.



LACING ON FACE A LACING ON FACE B

PREFFERED LACING ARRANGEMENT

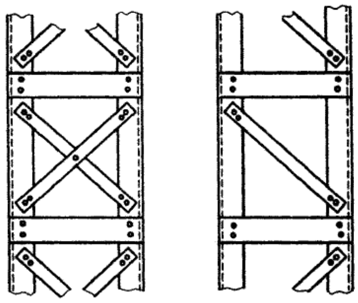
10A Single Laced System



LACING ON FACE A LACING ON FACE B

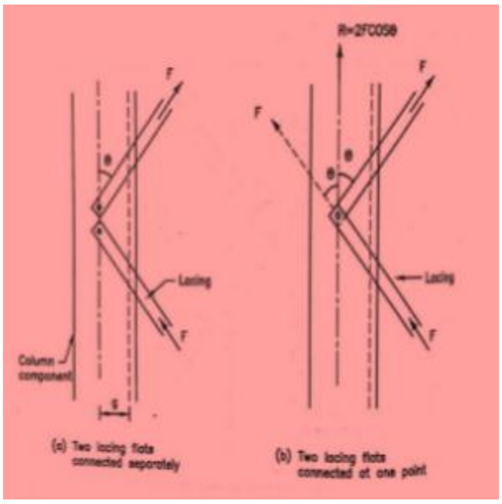
PREFFERED LACING ARRANGEMENT

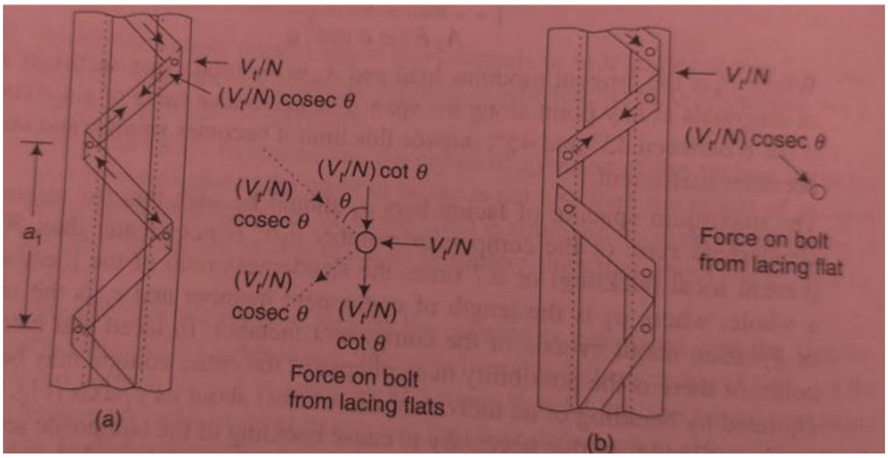
10B Double Laced System



10C Double Laced and Single Laced System Combined with Cross Numbers

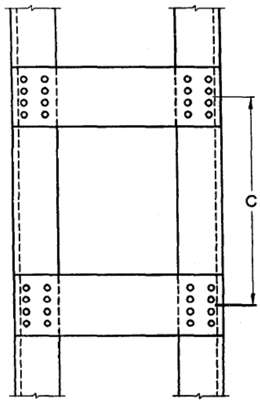
- C-7.6.1.5 ⟹ The effective slenderness ratio $\left(\frac{KL}{r}\right)_e$ of laced columns shall be taken as 1.05 times the $\left(\frac{KL}{r}\right)_o$, the actual maximum slenderness ratio, in order to account for shear deformation effects.
- C-7.6.2 ⟹ The minimum width of lacing bar shall be three times the nominal diameter of the end bolt/rivet.
- C-7.6.3 ⟹ The thickness of flat lacing bar shall not be less than 1/40 of its effective length for single lacing and 1/60 of the effective length for double lacing. (Rolled sections can be used)
- C-7.6.4 ⟹ The angle of inclination shall not be less than 40^{o} nor more than 70^{o} to the vertical axis of built up section.
- C-7.6.5 ⟹ The maximum spacing of lacing bars, whether connected by bolting, riveting or welding, shall also be such that the maximum slenderness ratio of the components of the main member $\left(\frac{a_1}{r_1}\right)$, between consecutive lacing connections is not greater than 50 or 0.7 times the most unfavorable slenderness ratio of the member as a whole, whichever is less, where a_1 is the unsupported length of the individual member between lacing points, and r_1, is the minimum radius of gyration of the individual member being laced together
- C-7.6.6.1 ⟹ The lacing shall be proportioned to resist a total transverse shear, V_t, at any point in the member, equal to at least 2.5 percent of the axial force in the member and shall be divided equally among all transverse lacing systems in parallel planes.
- C-7.6.6.3 ⟹ The slenderness ratio, $\frac{KL}{r}$, of the lacing bars shall not exceed 145. In bolted/riveted construction, the effective length of lacing bars for the determination of the design strength shall be taken as the length between the inner end fastener of the bars for single lacing, and as 0.7 of this length for double lacings effectively connected at intersections. In welded construction, the effective lengths shall be taken as 0.7 times the distance between the inner ends of welds connecting the single lacing bars to the members.

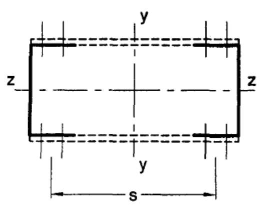




Batten System

- Batten plates are placed perpendicular to axis of column i.e if column axis is vertical then batten are place horizontal.
- C-7.7.1.1 Compression members composed of two main components battened should preferably have the, individual members of the same cross-section and symmetrically disposed about their major axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of the batten.
- 7.7.1.3 The battens shall be placed opposite to each other at each end of the member and at points where the member is stayed in its length and as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length from centre-to-centre of end.





- C-7.7.1.4 The effective slenderness ratio $\left(\frac{KL}{r}\right)_e$ of battened columns, shall be taken as 1.1 times the $\left(\frac{KL}{r}\right)_0$, the maximum actual slenderness ratio of the column, to account for shear deformation effects.
- C-7.7.2.1 Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force V_1 equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens. Battened member carrying calculated bending moment due to eccentricity of axial loading, calculated end moments or lateral loads

parallel to the plane of battens, shall be designed to carry actual shear in addition to the above shear. The main members shall also be checked for the same shear force and bending moments as for the battens.

$$\text{Transverse Shear } V_1 = 2.5\%\ of\ axial\ load$$

$$\text{Longitudinal Shear } V_b = \frac{V_1 C}{NS}$$

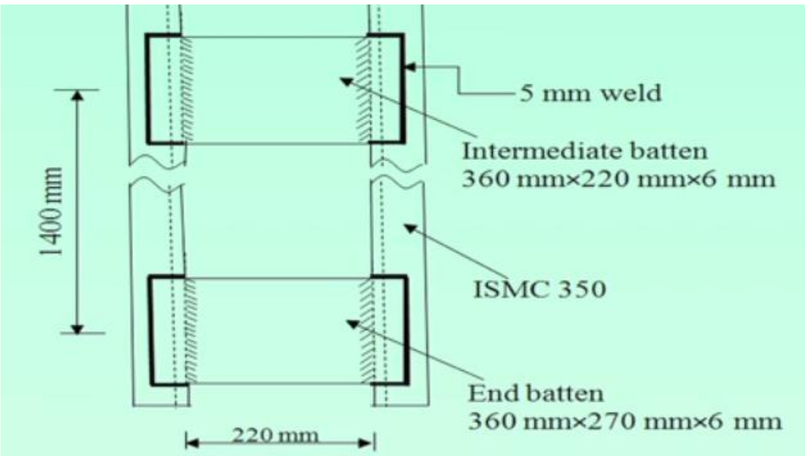
$$\text{Moment on Batten } M = \frac{V_1 C}{2N}$$

where,
C is distance between center to center of Batten longitudinally.
S is transverse distance between the centroid of the rivet/bolt.

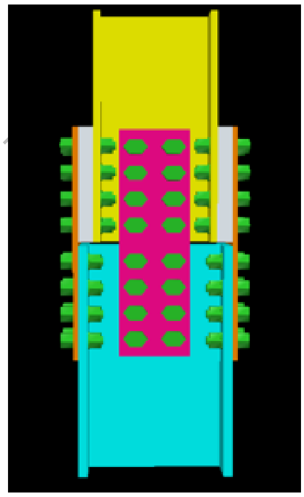
- C-7.7.2.2 The thickness of the batten plate $t_e \geq \frac{1}{50} * distance\ between\ inner\ most\ line\ of\ rivets.$

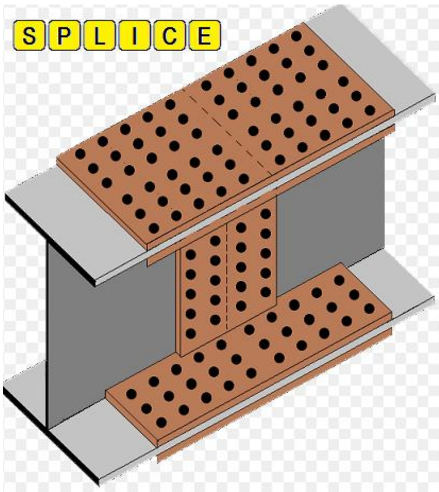
C-7.7.2.2 The Effective depth of End Batten plate $d \geq$ perpendicular distance between centroid of main member
The Effective depth of Intermediate batten plate $d \geq \frac{3}{4} *$ perpendicular distance between centroid of main member
The effective in no case the depth of any batten be less than twice the width of one member.
(Effective depth for bolt connection is center to center distance between extreme bolts, overall depth =effective depth+2(edge distance))
(Effective depth for welding connection is end to end distance between edges, overall depth=effective depth)

- C-7.7.3 The slenderness ratio of the batten $\lambda = \frac{C}{r_{min}} \leq 50$ or 0.7 times the slenderness ratio of the member as a whole.
- C-7.7.4.1 (Welded Connection) The overlap distance shall not be less than 4 times thickness of batten & length of the weld connection shall not be less than half the depth of the batten plate
  Length of welding is measured in longitudinal direction only.
  The size of the batten is less in welded connection than in bolted/riveted connection.



COMPRESSION SPLICES





- When available length of member in market is less than require then we use splices or when the size of the section changes then also we require splices (near the ground floor of multi story building)
- When the ends of the member are machined i.e complete bearing over then whole area, spliced are used to connect the member accurately in position (50% of the total load) and to resist tension when bending is present.
- Axail load due to super imposed load $P_{u1} = \frac{P_u}{4}$
- Axial load due to Bending $P_{u2} = \frac{M_u}{Lever\,Arm}$ (lever are is equal to distance between c/c of splice)
- When the ends of the member are not machined i.e not faced for complete bearing, splices should be designed to transmit all the forces to which the members are subjects.
- Web splices are designed to take the complete shear force either machined member or not.
- Splices are placed at point of contraflexure and splices are treated as short column hence they fail by crushing not buckling and we can use yield stress as f_{cd}

Example Find the thickness of the flange splice for ISMB 300 with flange width 250mm, Axial load 500KN, Bending Moment=40KN-m

Solution $P_{u1} = \frac{500}{4} = 125\ and\ P_{u2} = \frac{40 \times 10^3}{306} = 130.72$

$Total\ force = 130.72 + 125 = 255.72$KN

$$f_{cd} * A = 255.72 \Longrightarrow A = \frac{255.72 \times 10^3}{250} = 1022.9mm^2$$

$thickness, t = \frac{1022.9}{250} = 4.09$, provide 6mm plate.

BEAMS

- One of the frequently used structural members is a beam whose main function is to transfer load principally by means of flexural or bending action.

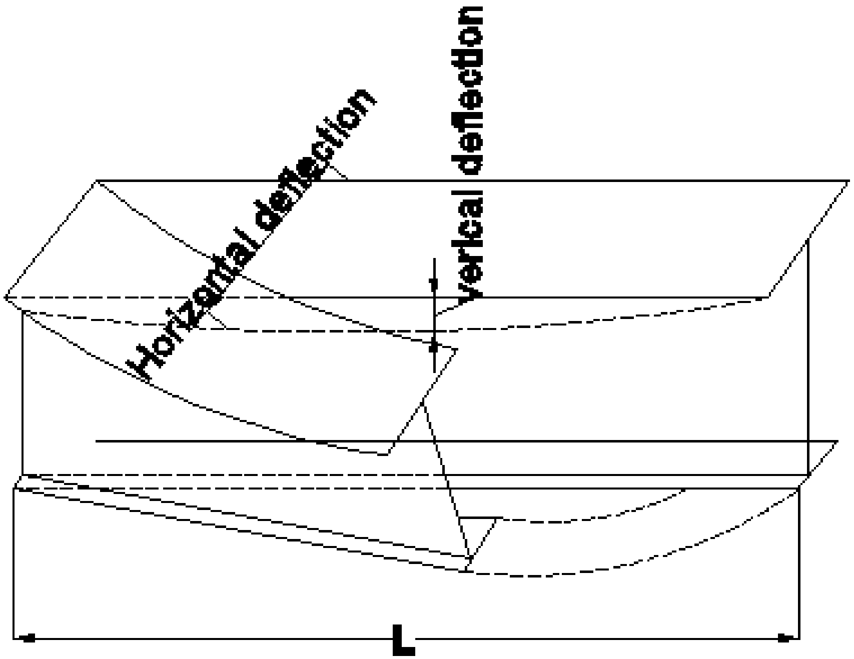
- In a structural framework, it forms the main horizontal member spanning between adjacent columns or as a secondary member transmitting floor loading to the main beams.

- Normally only bending effects are predominant in a beam except in special cases such as crane girders, where effects of torsion in addition to bending have to be specifically considered.

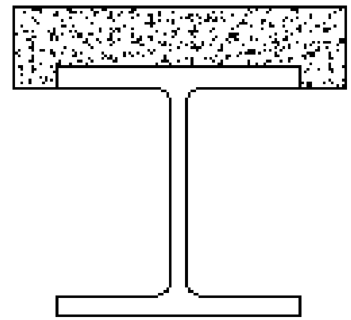
LATERALLY SUPPORTED BEAM

- When the lateral support to the compression flange is adequate, the lateral buckling of the beam is prevented and the section flexural strength of the beam can be developed.
- The strength of I-sections depends upon the width to thickness ratio of the compression flange.
- When the width to thickness ratio is sufficiently small, the beam can be fully plastified and reach the plastic moment, such sections are classified as compact sections.
- However provided the section can also sustain the moment during the additional plastic hinge rotation till the failure mechanism is formed. Such sections are referred to as plastic sections.

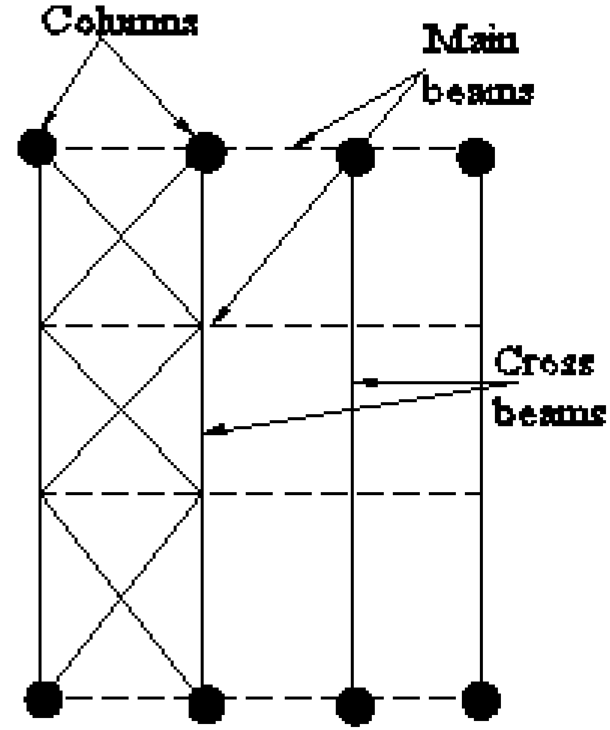
LATERALLY SUPPORTED BEAM



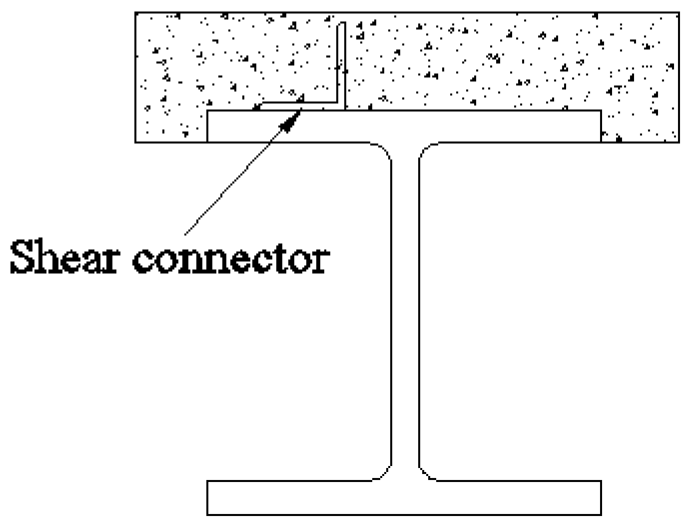
(a) Buckling of compression f



(b)



(d)

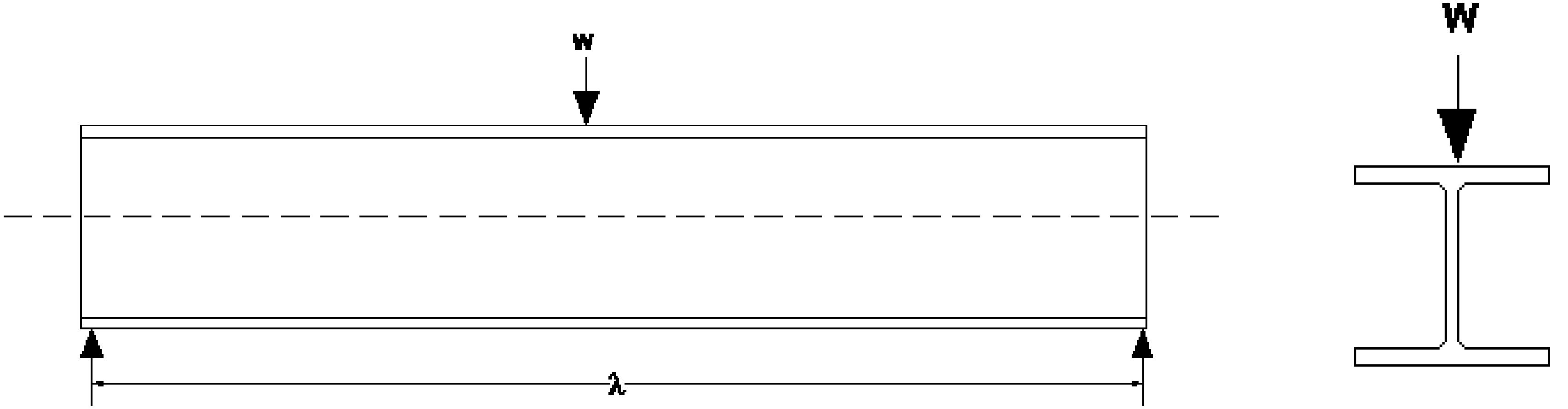


(c)

- When the compression flange width to thickness ratio is larger, the compression flange may buckle locally before the complete plastification of the section occurs and the plastic moment is reached.
- Such sections are referred to as non-compact sections.
- When the width to thickness ratio of the compression flange is sufficiently large, local buckling of compression flange may occur even before extreme fibre yields.
- Such sections are referred to as slender sections.

LATERALLY UNSUPPORTED BEAMS

- Under increasing transverse loads, a beam should attain its full plastic moment capacity.



Two important assumptions have been made therein to achieve the ideal beam behaviour.

They are:

- The compression flange of the beam is restrained from moving laterally; and
- Any form of local buckling is prevented.

1.Design a continuous beam of spans 4.9 m, 6 m, and 4.9 carrying a uniformly distributed load of **32.5 kN/m and the beam is laterally supported.**

<u>Factored load calculation</u>

Factored uniformly distributed load = 1.5 x 32.5 = 48.75 kN/m

The bending moment and shear force distribution are shown below

Maximum bending moment = 146.25 kN m

Maximum shear force = 146.25 + 146.25 = 292.5 kN

Plastic section modulus required

$$z_p = \frac{M x\, \gamma_{mo}}{f_y} = \frac{146.25 x 10^6 x\ 1.10}{250} = 643.5 x 10^3\ mm^3$$

Selection of suitable section

Choose a trial section of ISLB 350 @0.495 kN/m.

Overall depth *(h) = 350 mm*

Width of flange *(b)= 165 mm*

Thickness of flange (t_f) *= 11.4 mm*

Depth of web (d) = $h - 2(t_f + R) = 350 - 2(11.4 + 16) = 295.2$ *mm*

Thickness of web (t_w) *= 7.4 mm*

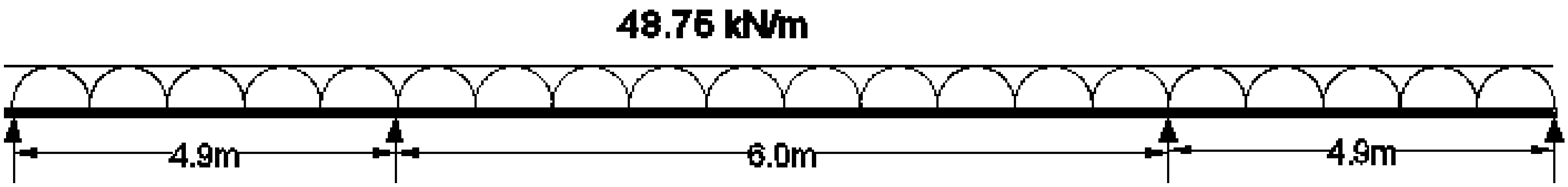
Moment of inertia about major axis *I, = 13158.3 x lo4 mm4*

Elastic section modulus $(Z_e) = 75\ 1.9 \times 10^3 mm^3$

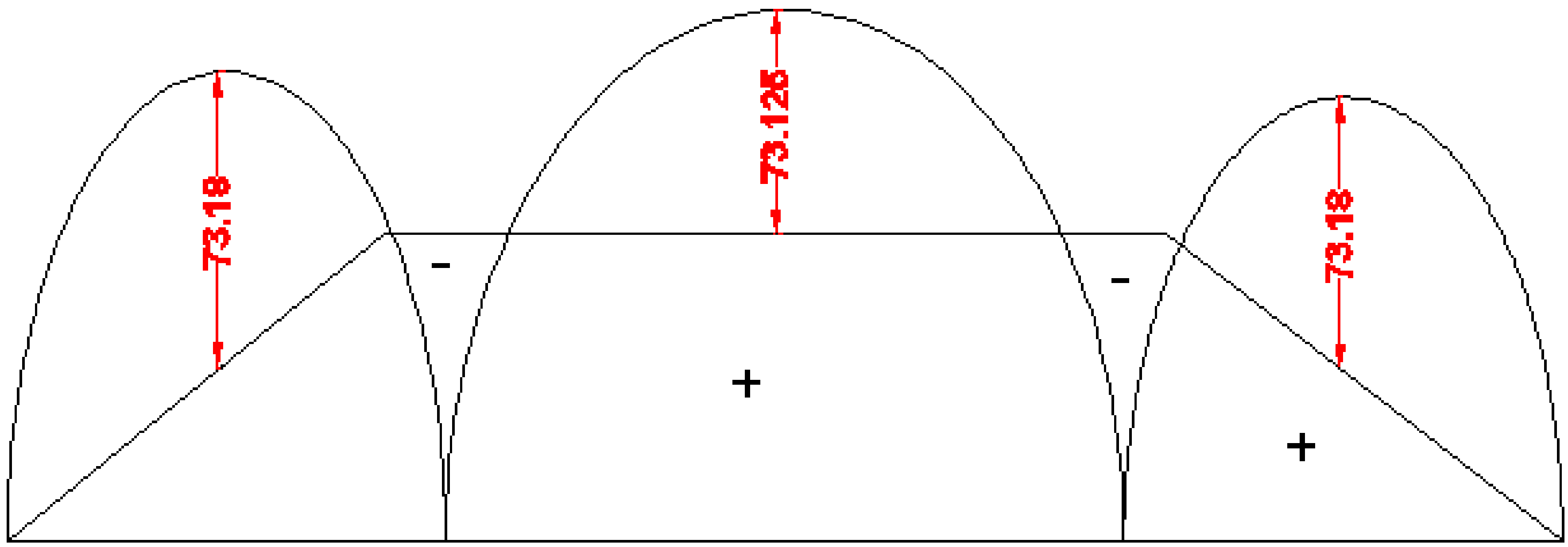
Plastic section modulus $(Z_p) = 851.11 \times 10^3\ mm^3$

Section classification

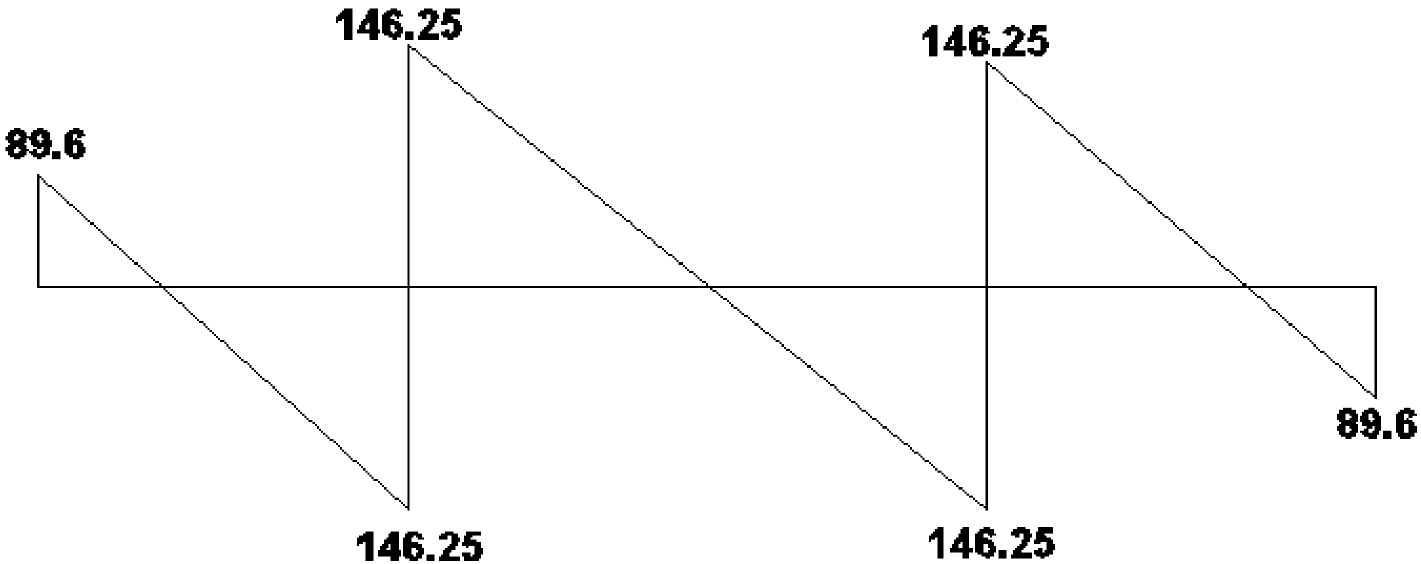
$$\frac{b}{t_f} = \frac{82.5}{11.4}$$



BEAM LOADING (a)



BENDING MOMENT DIAGRAM (b)



SF DIAGRAM (c)

$$\frac{b}{t_f} = \frac{295.2}{7.4} = 39.9 < 84$$

Hence the section is plastic.

Check for shear capacity of section

$$V_d = \frac{f_y}{m_o \times \sqrt{3}} \times h \times t_w = \frac{250}{1.1 \times \sqrt{3}} \times 350 \times 7.4 = 340 \text{ kN}$$

$0.6\ v_d = 204$ kN < 292.5 kN

This shows a high shear condition.

Check for moment capacity of the section [Eqn 6.8(a)]

$M_{dv} = M_d - \beta\ (M_d - M_{fd}) \leq 1.09 \times Z_e \times f_y$

where M_{fd} is the plastic design strength of the area of cross section excluding the shear area.

$$\beta = \left[2\ x \left(\frac{v}{v_d}\right) \text{x } 1\ \right]^2 = \left[2\ x \left(\frac{292.5}{340}\right) \text{x } 1\ \right]^2$$

Calculation of section modulus of flange

$$Z_{fd} = Z_p - A_w y_w$$

$$= 851.11 \times 10^3 - \left(350 \times 7.4 \times \frac{350}{4}\right)$$

$$= 624.485 \times 10^3 \text{ mm}^3$$

Therefore, $M_{fd} = \dfrac{Z_{fd}\ x\ f_y}{\gamma_{mo}}$

$$= \frac{624.485\ x 10^3 x 250}{1.10}$$

$$= 141.93\ kNm$$

Moment capacity of the section

$$M_d = \frac{Z_p\ x\ f_y}{\gamma_{mo}} = \frac{851.11\ x 10^3 x 250}{1.10}$$

$$= 193.43 \text{ kNm}$$

therefore, $M_{dv} = 193.43 - 0.52(193.43 - 141.93)$

$$= 165.65 \text{ kN m} < \frac{1.2\ x\ Z_e\ x\ f_y}{\gamma_{mo}} = \frac{1.2\ x\ 751.9\ x\ 10^3 x 250}{1.10}$$

$$= 205.06 \text{ kN m} > 146.25 \text{ kN m}$$

Hence the section is safe.

2. *Design a laterally unrestrained beam to carry a uniformly distributed load* ***of 30 kN/m. The beam is unsupported for a length of 3 m and is simply placed*** *on longitudinal beams at its ends.*

Calculation of load

Factored load = 1.5 x 30 =45 kN/m

Calculation of bending moment and shear force

$$\text{BM} = \frac{wl^2}{8} = \frac{45\ x\ 3^2}{8} = 50.625 \text{ kN.m}$$

$$\text{SF} = \frac{wl}{2} = \frac{45\ x\ 3}{2} = 67.5\text{kN}$$

Initialization of section

Assume λ = 100; $\frac{h}{t_f} = 25$ and hence from table 14, $f_{cr,b}$ = 291.31 N/mm^2

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}} = \sqrt{\frac{250}{291.31}} = 0.926$$

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

$$= 0.5[1 + 0.21(0.926 - 0.2) + 0.9262] = 1.005$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + [\Phi^2{}_{LT} - \lambda^2{}_{LT}]^{0.5}} \leq 1.0$$

$$= \frac{1}{1.005 + [1.005^2 - 0.926^2]^{0.5}} = 0.716 \leq 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{mo}} = \frac{0.716\ x\ 250}{1.10} = 162.7\ \text{N/mm}^2$$

Therefore required z of section = $\frac{50.625\ x\ 10^6}{162.7}$

= $311.1 \text{x} 10^3 \text{mm}^3$

Choose a section of ISMB 225 @ 0.3 12 kN/m.

Overall depth ***(D) = 225 mm***

Width of flange ***(b_f) = 110 mm***

Thickness of flange *(t_f) = 11.8 mm*

Thickness of web *(t_w) = 6.5 mm*

Depth of web (d) = ***D - 2(t_f + R) = 225 - 2(11.8 + 12) = 177.4 mm***

Moment of inertia about major axis I_{zz} = $3440\ x10^4\ mm^4$

Moment of Inertia about minor axis I_{yy} = $218\ x\ 10^4\ mm^4$

Elastic section modulus (Z_{ez}) = $305.9\ x\ 10^3\ mm^3$

Plastic section modulus (Z_{ey}) = $348.27\ x\ 10^3\ mm^3$

Minimum radius of gyration (r_y) = 18.6 mm

Section classification

Outstand of compression flange = (1 10/2)/11.8 = 4.66 < 9.4

Web with neutral axis at mid depth = 177.4/6.5 = 27.3 < 84

Therefore the section is plastic.

Calculation of lateral-torsional buckling moment

$M_{cr} = \sqrt{\frac{\pi^2 E I_y}{(KL)^2}\left(GI_t + \frac{\pi^2 E I_w}{(KL)^2}\right)}$ (from clause 8.2.2.1)/p-54

$G = \frac{E}{2(1+\mu)} = \frac{2\ x\ 10^5}{2(1+0.3)}$ $= 76.923 \times 10^3$ N/mm²

$I_t = \sum \frac{b_i t_i^{\,3}}{3} = \frac{2x110x11.8^3}{3} + \frac{(225 - 2x11.8)x6.5^3}{3}$

$= 138.926 \times 10^3 mm^3$

$I_w = (1-\beta_f)\ \beta_f\ I_y\ h^2_f$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$h_f = 225-11.8 = 213.2$ mm

$I_w = (1-0.5) \times 0.5 \times 218 \times 10^4 \times 213.2^2 = 24.77 \times 10^9 mm^6$

$$M_{cr} = \sqrt{\frac{\pi^2 x 2 x 10^5 x 218 x 10^4}{3000^2}\left(76.923 x 10^3 + 138.926 x 10^3\right) + \frac{\pi^2 x 2 x 10^5 x 24.77 x 10^9}{2}}$$

$= 87.79 kNm$

$$\lambda_{LT} = \sqrt{\frac{Z_p f_y}{M_{cr}}} = \sqrt{\frac{348.27 x 10^3 x 250}{87.79 x 10^6}} = 0.9959$$

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2{}_{LT}]$$

$$\alpha_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2{}_{LT}]$$

$$\alpha_{LT} = 0.21$$

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2{}_{LT}]$$

$$\chi_{LT} = \frac{1}{[\Phi^2{}_{LT} + \lambda^2{}_{LT}]^{0.5}} = 0.6685 \leq 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{mo}} = \frac{0.6685\text{x}250}{1.10} = 151.93\text{N/mm}^2$$

$$M_d = Z_p \mathrm{f_{bd}} = 348.27\mathrm{x}10^3\mathrm{x}151.93 = 52.91\mathrm{kNm}$$
$$> 50.625\mathrm{kNm}$$

Calculation of shear capacity of section

$$\mathrm{V_d} = \frac{f_y}{\gamma_{mo}\sqrt{3}}\,\mathrm{x\ D\ xt_w} = \frac{250}{1.10\ \mathrm{x}\sqrt{3}}\,\mathrm{x}\ 225\ \mathrm{x}\ 6.5$$

=191.kN

0.6 ***V_d= 115 kN*** > ***67.5 kN***

Calculation of deflection

$$\delta_b = \frac{5wl^4}{384EI},\ w = 30kN/m$$

$$\delta_b = \frac{5\ x\ 30\ x\ 3000^4}{384\ x\ 2\ x\ 10^5 x\ 3440x10^4} = 4.6mm$$

Allowable deflection = $\dfrac{l}{300} = \dfrac{3000}{300} = 10\text{mm}$

Hence the section is safe against deflection.

Check for web buckling:

Assuming that longitudinal beams are of the same size,

$$A_b = (b_1 + n_1)t_w = 4.6mm$$

$$b_1 = \frac{(b_f - t_w)}{2} = \frac{110 - 6.5}{2} = 51.75mm$$

$$n_1 = \frac{D}{2} = \frac{225}{2} = 112.5mm$$

A_b = (51.75+112.5) x 6.5 = 1067.6mm^2

r_{min} = $\sqrt{\frac{I}{A}} = \sqrt{\frac{1184}{336.4}} = 1.88mm$

λ = $\frac{l_{eff}}{r_{min}} = \frac{0.7x177.4}{1.88} = 66.18$

therefore, f_{cd} = 158.36N/mm^2(from table 9c of the code)

Strength of the section against web buckling = 158.36 x 1067.6

= 169.07 **kN** > **67.5 kN**

Check for web bearing:

$F_w = (b_1 + n_2)t_w f_y/\gamma_{mo}$

$b_1 = 51.75$ mm

$n_2 = 2.5(t_f + R) = 2.5(11.8+12) = 59.5$mm

$F_w = (51.75+59.5) \times 6.5 \times 250/(1.10 \times 10^3) = 164.35$kN > 67.5 kN

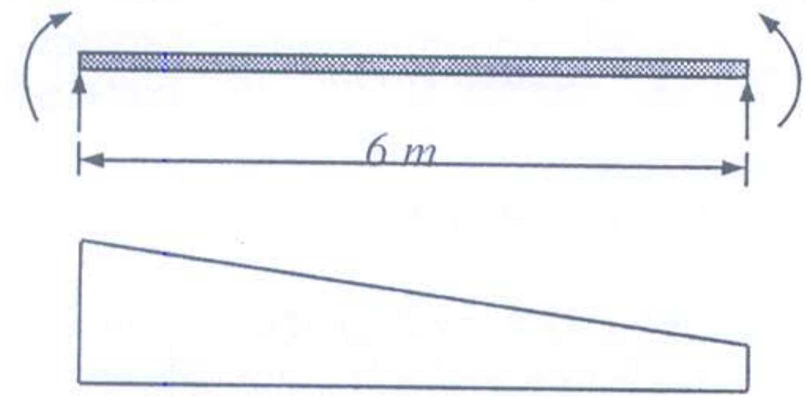
Hence the section is safe against web bearing.

PROBLEMS

3.A simply supported beam of span 6m is subjected to end moments of 202 kN.m (clockwise) and 112 kN.m (anticlockwise) under factored applied loading. Check whether ISMB-450 is safe with regard to lateral buckling.

Design check

For the end conditions given,it Is assumed that the beam is simply supported in a vertical plane,and at the ends the beam is fully restrained against lateral Deflection and twist with



B.M Diagram

no rotational restraints in plan at its ends.

Section classification of ISMB 450

The properties of the section are:

Depth, h = 450mm

Width, b = 150 mm

Web thickness,t_w = 9.4 mm

Flange thickness, t_f = 17.4 mm

I_y = 834 X 10^4 mm^4

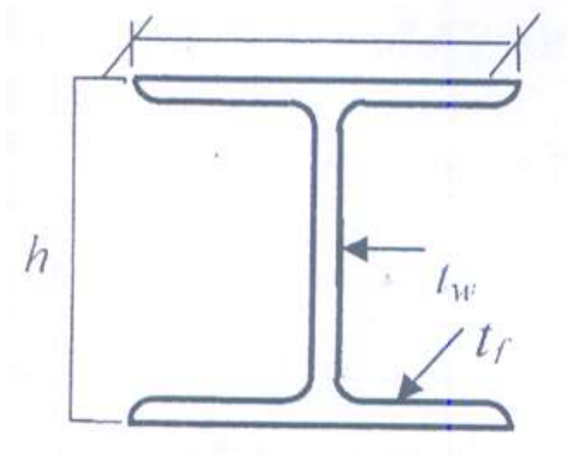
Depth between fillets,d = 379.2 mm

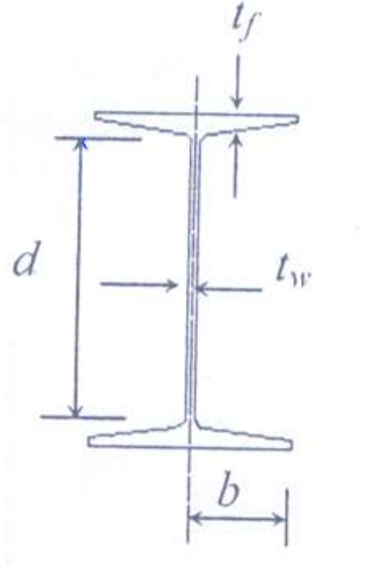
Radius of gyration about minor axis,

r_y = 30.1 mm

Plastic modulus about major axis,

z_p = 1533.36 X 10^3 mm^3





Rolled Steel Beams

Assume f_y = 250 N/mm^2 , E = 200000 N/mm^2 , γ_m = 1.10

<u>Type of section</u>

Flange criterion:

b= B/2 = 150/2 = 75mm

b/t_f = 75.0 /17.4 = 4.31

$b/t_f = 9.4\varepsilon$ where $\varepsilon = \sqrt{250/f_y}$

Hence , O.K

Web criterion:

d/t_w = 379.2/9.4 = 40.3

$d/t_w < 84\,\varepsilon$

Hence, O.K

Since, $b/t_f = 9.4\varepsilon < d/t_w < 84\,\varepsilon$,the section is classified as '*plastic*' *Table 3.1(section 3.7.2 of I.S 800)*

Check for lateral torsional buckling :

Check for slenderness ratio:

Effective length criteria:

With ends of compression flanges fully restrained for torsion at support but both the flanges are not restrained against warping, Effective length of simply supported beam, $L_{LT} = 1.0\ L$

Where L is the span of the beam. (*Table 8.3 of I.S.800*)

Hence, $L_{LT} = 1.0 \times 6.0\ M = 6000mm$, $L_{LT} / r = 6000/30.1$
$= 199.33$

Since the moment is varying from 155 k-Nm to 86 k-Nm, there will be moment gradient .So for calculation f_{bd} , critical moment , M_{cr} is to be calculated

Now, critical moment

$$M_{cr} = C_1 \frac{\pi^2 EI}{(KL)^2} \left\{\left[\left(\frac{K}{K_w}\right)^2 \frac{I_w}{I_y} + \left(\frac{GI_t (KL)^2}{\pi^2 EI_y} + (C_2 y_g - C_3 y_1)^2\right]^{0.5} - (C_2 y_g - C_3 y_t)\right\}$$

Where ,

C_1, C_2 , C_3= factors depending upon the loading and end restraint conditions

K, K_w = effective length factors of the unsupported length accounting for boundary conditions at the end lateral supports, Here,both K and K_w can be taken as 1.0 and

y_g = y distance between the point of application of the load and the shear centre of the cross-section and is positive when the load is acting towards the shear centre from the point of application

$y_j = y_s - 0.5 \int A(z^2 - y^2) y \, dA / I_z$

y_s = coordinate of the shear centre with respect to centroid, positive when the shear centre is on the compression side of the centroid.

Here,for plane and equal flange I section,

$y_g = 0.5 \times h = 0.5 \times 0.45 = 0.225$ M =225mm.

$y_j = 1.0(2\beta_f - 1)h_y/2.0$ (when $\beta_f \leq 0.5$)

h_y = distance between shear centre of the two flanges of the cross-section) = $h - t_f$

Here, $\beta_f = 0.5$ and $h_y = h - t_f = 450 - 17.4 = 432.6$mm

Hence, $y_j = 1.0 \times (2.0 \times 0.5 - 1) 432.6/2.0 = 0$ and $y_s = 0$

$I_t = \sum b_i t_i^3$,for open section

$= 2 \times 150 \times 17.4^3 + (450 - 2 \times 17.4) \times 9.4^3$

The warping constant, I_w is given by,

$I_w = (1- \beta_f)\ \beta_f\ I_y\ h_y^2$ for I sections mono-symmetric about weak axis,

$=(1-0.5) \times 0.5 \times 834 \times 10^4 \times 432.6^2 = 39019265.46 \times 10^4 mm^6$

Modulus of rigidity ,$G = 0.769 \times 10^5\ N/mm^2$

Here, $\psi = 86/155 = 0.555$ and $K = 1.0$ for which,

$C_1 = 1.283$, $C_2 = 0$ and $C_3 = 0.993$

Hence, critical moment

$$M_{cr} = C_1 \frac{\pi^2 EI}{(KL)^2} \{[(\frac{K}{K_w})^2 \frac{I_w}{I_y} + (\frac{G I_t (KL)^2}{\pi^2 E I_y} + (C_2 y_g - C_3 y_1)^2]^{0.5} - (C_2 y_g - C_3 y_t)\}$$

$$=1.283\ \frac{\pi^2 x 200000 x 834 x 10^4}{(1.0\ x\ 6000)^2} \{[\left(\frac{1}{1}\right)^2 \frac{39019265 x 10^4}{834 x 10^4} + \frac{0.769 x 10^5 x 192.527 x 10^4 x 6000^2}{\pi^2 x 200000 x 834 x 10^4}]^{0.5}$$

$= 357142.72 \times 10^3$ N-mm.

Calculation of f_{bd}:

Now $\lambda_{LT} = \sqrt{\beta_b z_p f_y / M_{cr}} = \sqrt{1.0 \times 1533.36 \times 10^3 \times 250 / 357142.72 \times 10^3}$

$= 1.036$ *(clause 8.2.2 of I.S 800)*

For which, $\Phi_{LT} = 0.5 \times [1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$= 0.5 \times [1 + 0.21(1.036 - 0.2) + 1.036^2] = 1.124$

For which , $\chi_{LT} = \dfrac{1}{\{\Phi_{LT}[\Phi^2{}_{LT} - \lambda^2{}_{LT}]^{0.5}\}}$

$= \dfrac{1}{\{1.124[1.124^2 - 1.036^2]^{0.5}\}}$

$f_{bd} = \chi_{LT} f_y / \gamma_{mo} = 0.641 \times 250 / 1.10 = 145.68 \text{ N/mm}^2$

Hence, $M_d = \beta_b z_p f_{bd}$ = 1.0 x 1533.36 x 145.68 / 1000

= 223379.88/1000 ~ 223.38 kN-m.

Max. Bending moment M_{max} = 202 kN-m

Hence, $M_d > M_{max}$ = (223.38 > 202)

Therefore,ISMB 450 is adequate against lateral torsional buckling for the applied bending moments.

(ii) If the ISMB 450 is subjected to a central load producing a maximum factored moment of 202kN.m ,check whether the beam is still safe

For this problem with zero bending moments at the supports and central max bending moment being 202kN-m.

For the valur of K = 1.0, C_1 = 1.365; C_2 =0 .553 and C_3 = 1.780

$$M_{cr}=C_1\frac{\pi^2EI}{(KL)^2}\{[(\frac{K}{K_w})^2\frac{I_w}{I_y}+(\frac{GI_t(KL)^2}{\pi^2EI_y}+(C_2y_g - C_3y_1)^2]^{0.5} -$$

$$(C_2y_g - C_3y_t)\}$$

$$= 1.365\,\frac{\pi^2x2x10^4x834x10^4}{(1.0\ x\ 6000)^2}\{[\left(\frac{1}{1}\right)^2\frac{39019x10^9}{834x10^4}+\frac{0.769x10^5x192.527x10^4x6000^2}{\pi^2x2x10^5x834x10^4}]^{0.5}$$

$$-0.553x225\}$$

$=310158.31x10^3$N-mm

Calculation of f_{bd}:

Now, $\lambda_{LT} =\sqrt{\beta_b z_p f_y/M_{cr}} = \sqrt{1.0x1533.36x10^3x250/310158.31x10^3}$

$= 1.112$ *(clause 8.2.2 of I.S 800)*

For which, $\Phi_{LT} = 0.5 \text{ x } [1+\alpha_{LT}(\lambda_{LT} - 0.2)+ \lambda_{LT}^2]$

$= 0.5 \text{ x } [1 + 0.21(1.112-0.2)+1.112^2] = 1.214$

For which , $\chi_{LT} = \dfrac{1}{\{\Phi_{LT}[\Phi^2{}_{LT} - \lambda^2{}_{LT}]^{0.5}\}}$

$$= \frac{1}{\{1.214[1.214^2 - 1.112^2]^{0.5}\}}$$

$f_{bd} = \chi_{LT}\, f_y / \gamma_{mo}$ = 0.588 x 250 /1.10 = 133.64 N/mm^2

Hence, $M_d = \beta_b z_p f_{bd}$ = 1.0 x 1533.36 x 133.64 / 1000

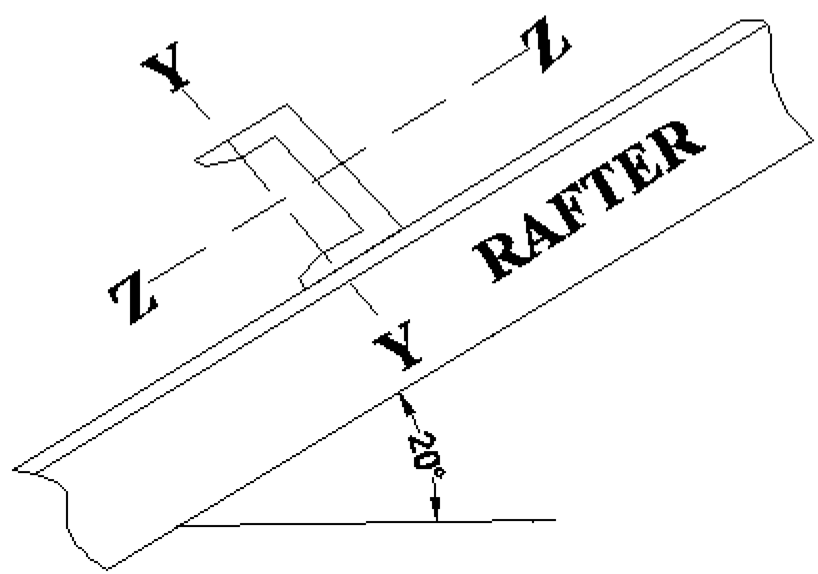
= 204918.23/1000 ~ 204.92 kN-m.

4. Design a purlin on a sloping roof truss with the dead load of 0.15 kN/ m2 (cladding and insulation), a live load of 2 kN/m2 and wind load of 0.5 kN/m^2(suction). The purlins are 2 m centre to centre and of span **4 m, simply supported on** a rafter at a slope of 20 degrees (see Fig).

(a) Provide channel section purlin

(b) Provide channel purlin with a sag rod at mid span

(c) Provide angle purlin



Solution:

Load calculation

Dead load = 0.15 x 2 = 0.3 kN/m

Live load = 2 x 2 = 4 kN/m

Wind load = 0.5 x 2 = 1 kN/m (suction)

wd, = 0.3 X COS 20" = 0.282 kN/m

Wi, = 4 x cos 20" = 3.76 kN/m

W,, = -1 kN/m

W_{iy} = 4 x sin 20" = 1.37 kN/m

w_{dy} = 0.3 X Sin 20" = 0.103 kN/m

Note that W_{wy} *is zero as wind pressure is perpendicular to the surface on which it* acts, i.e., normal to the rafter.

Factored load combination:

Z-direction:

WL + DL + LL = (1.2 x 1 .0) + (1.2 x 0.282) + (1.2 x 3.76) = 6.0552 kN/m

DL + LL = (1.5 x 0.282) + (1.5 x 3.76) = 6.063 kN/m

Y-direction:

DL + LL = (1.5 x 0.103) + (1.5 x 1.37) = 2.21 kN/m

Bending moment and shear force calculation:

M_z = 6.063 x4^2/8 = 12.126 kN m

M_y = 2.21 x 4^2/8 = 4.42 kN m

F_z = 6.063 x 4/2 = 12.126 kN

F_y = 2.21 x 4/2 = 4.42 kN

(a) <u>Channel section purlin</u>

Assume an ISMC 200 channel.

Plastic section modulus required

$$= \frac{M_z \ x\ \gamma_{mo}}{f_y} + 2.5\ x\ \frac{d}{b}\ x\ \frac{M_y\ x\ \gamma_{mo}}{f_y}$$

$$= \frac{12.126x10^6x1.10}{250} + 2.5\,x\,\frac{200}{75}\,x\,\frac{4.42\,x10^6x1.10}{250}$$

$= 183 \text{ x } 10^3 \text{ mm}^3$

Choose a channel section ISMC 200 @ 0.22 kN/m with plastic section modulus of

Z_{pz} = 211.25 x l0^3 mm^3 and Z_{py} = 40.716 x l0^3 mm^3.

<u>Section Properties:</u>

Cross sectional area $A = 2821\ mm^2$

Depth of the section $h = 200\ mm$

Width of flange $b = 75\ mm$

Thickness of flange $t_f = 11.4\ mm$

Thickness of web $t_w = 6.1\ mm$

Depth of web $d = h - 2(9+ R) = 200 x 2 (11.4 + 11) = 155.2\ mm$

Elastic section modulus $Z_{ez} = 181.7 x l0^3\ mm^3$

Elastic section modulus $Z_{ey} = 26.3\ x\ l0^3\ mm^3$

Plastic section modulus $Z_{pz} = 2\ 11.25\ x\ 10^3\ mm^3$

Plastic section modulus $Z_{py} = 40.716 \text{ x } l0^3 \text{ mm}^3$

Moment of inertia $I_{zz} = 1830\ x\ l0^4\ mm^4$

Moment of inertia $I_y = 14\ 1\ x\ 10^4\ mm^4$

Section classification:

$$\frac{t}{b_f} = \frac{75}{11.4} = 6.58 < 9.4$$

$$\frac{d}{t_w} = \frac{155.2}{6.1} = 25.44 < 42$$

Hence the section is plastic.

Calculation ***of shear capacity of the section*** Z-direction

$$v_d = \frac{f_y}{\gamma_{mo} x \sqrt{3}} x\ h\ x\ t_w = \frac{250}{1.1\ x \sqrt{3}}\ x\ 200\ x\ 6.1 = 160.18\ kN$$

$0.6V_d$ = 96 kN > 12.126 kN

Y-direction

Shear capacity = $\frac{250}{11.1\ x \sqrt{3}}\ x\ 2\ x\ 75\ x\ \frac{11.4}{10^3} = 224.4\ kN > 4.42kN.$

Note that in purlin design, the shear capacity is usually high relative to the shear force.

Design capacity of the section

$$M_{dz} = \frac{z_{pz}\ x\ f_y}{\gamma_{mo}} = \frac{211.25\ x\ 10^3}{1.1\ x\ 10^6} = 48kN.m$$

$$\leq \frac{z_{pz}\ x\ f_y}{\gamma_{mo}} = \frac{1.8x\ 181.7\ x\ 10^3 x 250}{1.1\ x\ 10^6} = 49.55kN.m$$

Hence, M_{dz} = 48 kN.m > 12.126 kN.m

$$M_{dy} = \frac{z_{py}\ x\ f_y}{\gamma_{mo}} = \frac{40.716\ x\ 10^3 x 250}{1.1\ 0} = 9.25kN.m$$

$$\leq \frac{r_f\ x\ ze_y x f_y}{\gamma_{mo}} = \frac{1.5x\ 26.3\ x\ 10^3 x 250}{1.1\ x\ 10^6} = 8.96kN.m$$

Since the ratio z_p / z_e is greater than 1.2,the constant in the preceding equation is replaced by the ratio of γ_f =1.5 ,Hence

M_{dy} = 8.96 kN.m > 4.42 kN.m

Overall member strength (local capacity)

To ascertain the overall member strength, the following interaction equation should be satisfied.

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{12.126}{48} + \frac{4.42}{8.96} = 0.75 \leq 1$$

Hence,the overall member strength is satisfactory

Check for deflection

$$\delta = \frac{5wl^4}{384EI} = \frac{5\ x\ 3.76\ x4000^4}{384x2x10^5x1830x10^4}$$

Allowable deflection $= \frac{l}{180} = \frac{4000}{180} = 22.22mm$

(Table 6 of I.S 800)

Hence, the section is safe.

Check for wind suction:

The effect of wind suction has not been considered till now; it can become critical in some situations. It has to be combined with dead load

Factored wind load $W_z = 0.9 \text{ x } 0.282 – 1.5 \text{ x } 1 =-1.246\text{kN/m}$

$W_y = 0.9 \text{ x } 0.103 =-0.0927\text{kN/m}$

Buckling resistance of section

Equivalent length l_e = 4 m

Moment = M_z = w1^2/8 = -1.246 x 4^2/8 = -2.492 kN m

My = 0.0927 x 42/8 = 0.1854 kN m

The value of M_z *is much lower than the value* 12.126 kN m *earlier, but the negative* sign indicates that the lower flange of the channel is in compression and this flange is unrestrained. Hence the buckling resistance of the channel must be found.

$$M_{cr} = \sqrt{\frac{\pi^2 EIy}{(KL^2)}}\left(GI_t + \frac{\pi^2 EIw}{(KL^2)}\right)$$

$$G = \frac{E}{2(1+\mu)}) = \frac{2X10^5}{2(1+0.3)} = 76.923X10^3 N/mm^2$$

$$I_t = \sum \frac{b_i t_i^{\,3}}{3} = \left[\frac{2X75x11.4^3}{3} + \frac{(200-11.4)x6.1^3}{3}\right] = 88346.77mm^4$$

$$I_w = (1-\beta_f)\ \beta_f\ I_y\ h^2{}_f$$

$$h_f = 200\text{-}11.4 = 188.6\text{mm}$$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$$I_w = (1\text{-}0.5) \times 0.5 \times 141 \times 10^4 \times 188.6^2$$

$$= 1.2538 \times 10^{10}\text{mm}^6$$

$$M_{cr} = \sqrt{\left[\frac{\pi^2 x 2x10^5 x141x10^4}{4000^2}(76.923x10^4 x88346.7 + \frac{\pi^2 x2x10^5 x1.2538x10^{10}}{4000^2}\right]}$$

$$= 38.09 \text{ kN m}$$

$$\lambda_{LT} = \sqrt{\frac{\beta_b z_p f_y}{M_{cr}}}$$

$$= \sqrt{\frac{1.0\ x\ 211.25x10^3 x250}{38.09x10^6}} = 1.1775$$

$$\Phi_{LT} = 0.5[1+\alpha_{LT}\ (\lambda_{LT} - 0.2) + \lambda^2_{LT}]$$

$$= 0.5[1+0.21(1.1775-0.2)+1.1775^2]$$

$$= 1.296$$

$$\chi_{LT} = \sqrt{\frac{1.0}{\Phi_{LT} + [\Phi^2{}_{LT} - \lambda^2{}_{LT}]^{0.5}}} \leq 1.0$$

$$= \sqrt{\frac{1.0}{1.296 + [1.296 - \lambda^2{}_{LT}]^{0.5}}} \leq 1.0$$

$$=0.544 \leq 1$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.544 \text{ X } 250}{1.10} = 123.71 \text{ N/mm}^2$$

$$M_{dz} = z_p f_{bd}$$

$$= 211.25 \text{ x } 10^3 \text{ x } 123.71$$

$$= 26.13 \text{ kNm} > 2.492 \text{ kNm}$$

The buckling resistanve M_{dy} of the section need not be found out,because the purlin is restrained by the cladding in the z-plane and hence instability is not considered for a moment about the minor axis .

Overall member strength

To ascertain the overall member buckling strength,the following interaction should be satisfied .

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{2.492}{26.13} + \frac{0.1854}{8.96} = 0.097 < 1$$

Hence the overall member strength is satisfactory.

- It has to be noted that the maximum buckling moment occurs at the centre of the beam and the maximum shear force at the supports.
- Hence it is not necessary to check the moment capacity in the presence of shear force.
- Also purlins are not normally checked for web bearing and crippling

as the applied concentrated loads are low (note the low value of Shear force)

(*b*) *Channel section purlin with one sag rod at mid span*

Since the channel section purlin is provided with a sag rod at mid – span,the bending moment in the y- direction will be reduced considerably .

$M_y = 2.21 \times 4^2 / 32 = 1.105 \text{kN m}$

$M_z = 12.126 \text{ kN m}$

Required section modulus = $(M_z \times \gamma_{m0}/f_y) + 2.5(d/b)(M_y \times \gamma_{m0}/f_y)$

Assuming ISMC 100 with d = 100mm and b = 50 mm,

Required Z = $(12.126 \times 10^6 \times 1.1/250)$

$= 77.66 \times 10^3 \text{mm}^3$

Provide ISMC 150 with following section properties

Depth of section h = 150mm; r_y = 22 mm

Width of flange b = 75 mm

Thickness of flange t_f = 9.0 mm

Thickness of web t_w = 5.7 mm

Elastic section modulus $z_{ez} = 105\text{x}10^3 mm^3$

Elastic section modulus $z_{ey} = 19.5 \text{ x } 10^3 mm^3$

Plastic section modulus

$$z_{pz}=119.5 \text{ x } 10^3 mm^3 > 77.66 \text{ x } 10^3 mm^3$$

Moment of inertia $I_{pz}=788 \text{ x } 10^4 mm^3$

Section classification

$$b/t_f = 75/9.0 = 8.33 < 9.4$$

d/t_w =[150-2(9.0 + 10)]/5.7 = 19.65 < 42

Hence the section is plastic.Shear capacity is not being checked since the shear foece is small and hence the section will be adequate.

Design capacity of the section

$M_{dz} = (z_{pz} \text{ x } f_y/\gamma_{m0})$

$= (119.83 \text{ x } 10^3 \text{ x } 250/1.1 \text{ x } 10^6) = 27.23 \text{kN m}$

$\leq (1.2 \text{ x } z_{ez} f_y/\gamma_{m0}) = [(1.2 \text{ x } 105 \text{ x } 10^3 \text{ x } 250)/(1.1 \text{ x } 10^6)]$

$= 28.63$

$Z_{py} = 2t_f\, b^2{}_f/4 + (h- 2t_f)\, t^2{}_w /4 = 2 \text{ x } 9.0 \text{ x } 75^2 /4 + (150\text{-}2 \text{ x } 9.0)$

$5.7^2/4 = 26384.6 \text{ mm}^3$

$M_{dy} = (z_{py}f_y/\gamma_{m0})$

$= (26384.6 \text{ x } 250/1.1 \text{ x } 10^6) = 6.0$ kN m

$\leq (1.5 \text{ x } z_{ey}f_y/\gamma_{m0}) = 1.5 \text{ x } 19.5 \text{x } 10^3 \text{ x } 250)/(1.1 \text{ x } 10^6)]$

$= 6.6$ kN m

Hence the section is safe.

Overall member strength

For overall member srength ,the following interaction equation must be satisfied.

$(M_z/M_{dz}) + (M_y/M_{dy}) \leq 1.0$

$(12.126/27.23) + (1.105/6.0) = 0.629 < 1.0$

Hence the member strength is satisfactory.

Check for deflection

$$\delta = (5wl^4/\ 384\mathrm{EI}) = (\ 5 \times 3.76 \times 4000^4\ ) /(384 \times 2 \times 10^5 \times 788 \times 10^4)$$

$$= 7.95 \text{ mm} < 22.22 \text{ mm}$$

Hence the section is safe.

Check for wind suction

From part (a) , $M_z = 2.492$ kN m

$M_y = 0.0927 \times 4^2/32 = 0.0464$ kN m

$f_{cr} =[\ 1473.5/\ (KL/r_y)/(h/t_f\ )]^2\}^{0.5}$

$KL/r_y = 4000/22 = 181.8$

$h/t_f = 150/9.0 = 16.67$

Thus, $f_{cr} = (1473.5/11.8)^2\ \{1+(1/20)\ [181.8/16.67]^2\}^{0.5}$

$= 173.1\ N/mm^2$

$f_{bd} = 120.0\ N/mm^2$ (from table 13a of the code)

$M_{dz} = Z_{pz}\, f_{bd} = 119.82 \times 10^3 \times 120.0/10^6 = 14.38$ kN m

Overall buckling strength

For overall buckling strength,the following interaction equation should be satisfied.

$$(M_z / M_{dz}) + (M_y / M_{dy}) = (2.492/14.38) + (0.0464/6.0)$$

$$= 0.18 < 10$$

Hence the overall buckling strength is satisfactory.

Hence by using one sag rod ,it was possible to reduce the section from ISMC 200 to ISMC 150 (about 25% reduction in weight).

(c) *Angle Section Purlin (as per BS 5950-1:2000)*

From part (a) M_z = 12.126 kN m; W_p = (1.0 +0.282+3.76) x 4

=20.168 kN

Moment at working load = 12.126 /1.5 = 8.084 kN m

Let us assume that bending about z-z axis resists the vertical loads and the horizontal component is resisted by the sheeting.

Design strength f_y = 250 Mpa

Applied moment = moment capacity of single angle

$8.084 \times 10^6 = 250 \times Z_{ez}$

Required $Z_{ez} = 8.084 \times 10^6 /250 = 32.33 \times 10^3 \text{ mm}^3$

Provide ISA 150 x 75 x 10 angle @ 0.17 kN/m,

With $Z_{ez} = 51.9 \times 10^3 \text{ mm}^3 > 20.168 \times 4 \times 10^6 / 1800 = 10^3 \text{ mm}^3$

$= 44.817 \times 10^3 mm^3$

$d/t = 150/10 = 15.0 > 10.5$ but < 15.7

The section is *semi – compact.*

Leg length perpendicular to plane of cladding

= 4000/45 = 88.88 mm < 150 mm

Leg length parallel to plane of cladding

= 4000/60 = 66.66 mm < 75 mm

Deflection need not be checked in this case.

The most indeterminate structure of steel frame collapse due to plastic deformation. The plastic analysis is based on ultimate load the structure may support just before just before its plastic collapse.

As shown in Fig.1 'P' increases such that the stress in member increases from 0 to σ_y, hence when 'P' is 0, $\Delta = 0$, when P is such that stress in member is σ_y, $\Delta = \frac{PL}{AE}$ or $\Delta = \frac{\sigma_y L}{E}$. Now further increasing 'P' but stress in bar will be σ_y, and bar will reach in plastic range hence deflection > $\frac{\sigma_y L}{E}$ but stress will be σ_y.

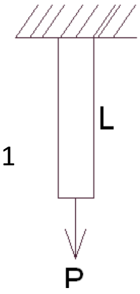
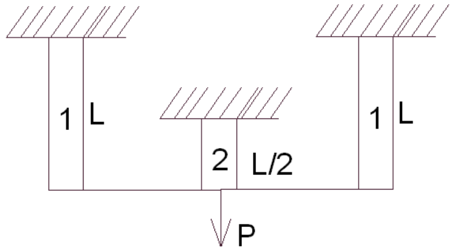


Fig.1

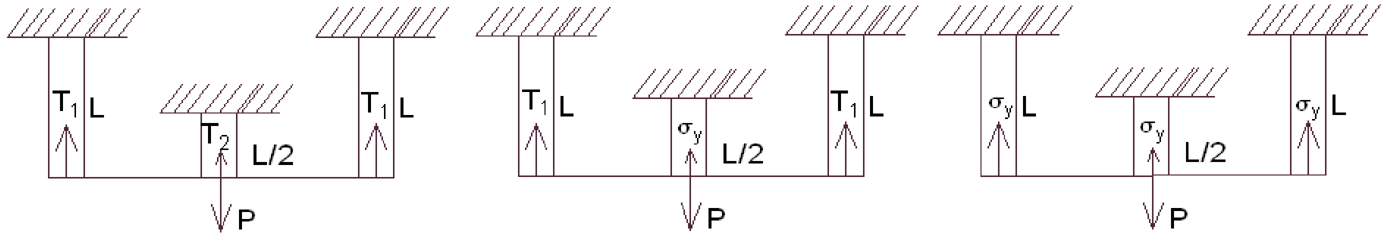
As shown in Fig.2 three bars of same material is fixed with rigid joint. And a load 'P' is applied at center.

$$2T_1 + T_2 = P \quad \Delta_1 = \Delta_2 = \frac{T_1 L}{AE} = \frac{T_2 L}{2AE} \Rightarrow T_1 = \frac{T_2}{2} \quad \text{hence } T_2 > T_1$$



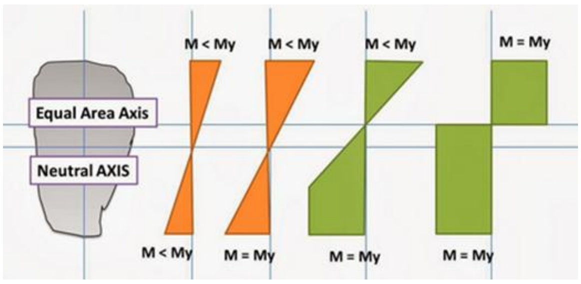
When 'P' will start increasing T_2 will yield first and after certain value, bar 2 will not take any load but if load 'P' further increases than that extra load will be taken up by outer bar and finally when both the bars yield than Pull 'P' is equal to $\sigma_y * 3 * A$ (maintaining equilibrium condition)

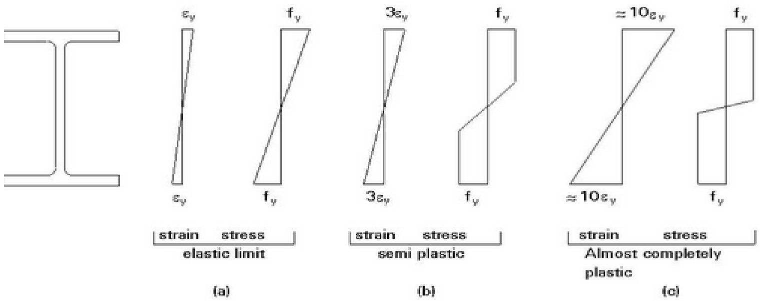
Further increasing the load 'P' the deflection increased unobstructed.

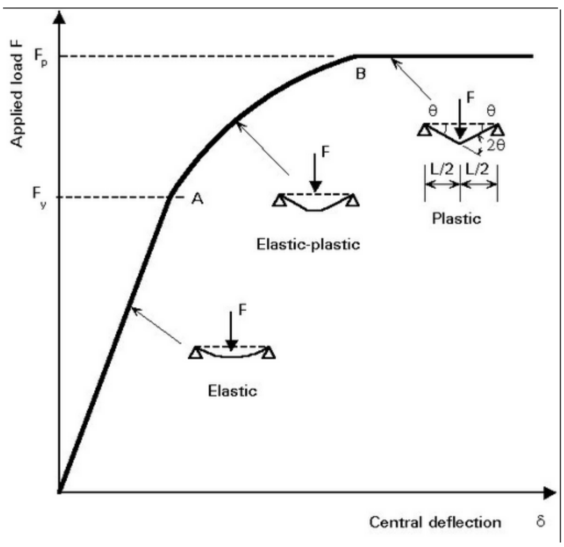


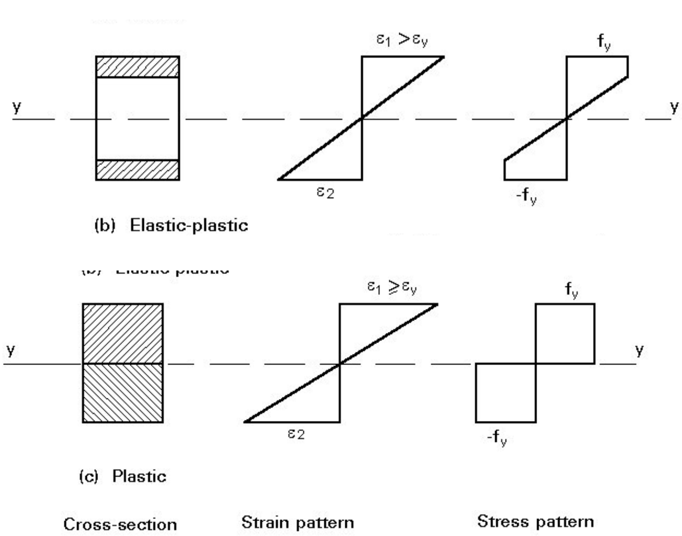
Plastic Bending of Beam

(1) $\frac{M}{I} = \frac{\sigma}{y}$ (2) $\frac{M_1}{I} = \frac{\sigma}{y}$ (3) $\frac{M_2}{I} = \frac{\sigma}{y}$ (4) $\frac{M_3}{I} = \frac{\sigma}{y}$ N.A will shift either upward or downward to make C=T. Hence change in N.A and the new position of N.A will be named as Equal Axis area.









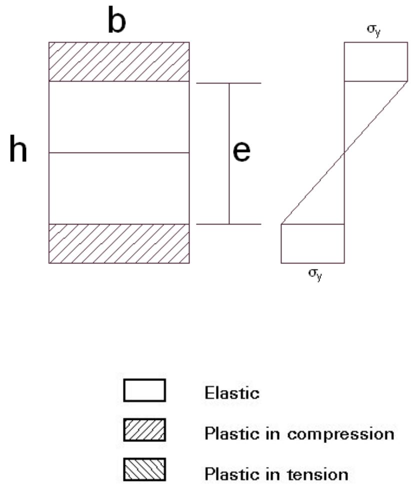
Equal Axis Area is the line which divides the section into two equal area.

$$M_p = \sigma_y\left(\frac{A}{2}*y_1+\frac{A}{2}*y_2\right)=\frac{\sigma_y A}{2}(y_1+y_2)=\sigma_y*Z_p$$

Z_p = Plastic modulus of the section $=\frac{A}{2}(y_1+y_2)$ **Z_p depends upon type of section (geometry)**

If section is symmetrical about x & y then Equal Area Axis is same as Neutral Axis.

Example1. Find the moment taking capacity of the section which is in elastic plastic range.



Solution moment taking capacity of elastic zone M_1

Average stress $\frac{\sigma_y}{2}$, area of elastic zone $=\frac{e}{2}*b$, lever arm about N.A=$\frac{2}{3}*\frac{e}{2}$

$$M_1 = 2*\frac{\sigma_y}{2}*\left(\frac{e}{2}*b\right)*\left(\frac{2}{3}*\frac{e}{2}\right)=\frac{\sigma_y be^2}{6}$$

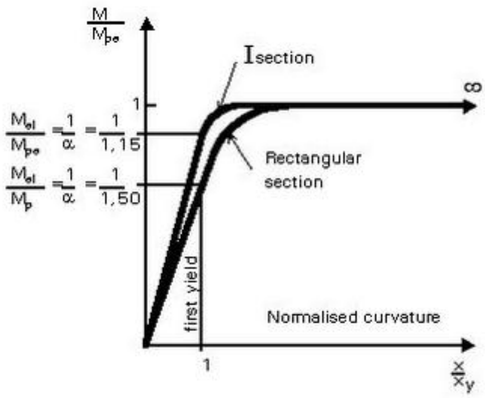
Moment resisted by plastic zone M_2

Average stress σ_y, area of elastic zone $=(\frac{h}{2}-\frac{e}{2})*b$,

lever arm about N.A=$\frac{h}{2}-\left(\frac{h}{4}-\frac{e}{4}\right)=\frac{h}{4}+\frac{e}{4}$

$$M_2 = 2*\sigma_y*\left(\frac{h}{2}-\frac{e}{2}\right)*b*\left(\frac{h}{4}+\frac{e}{4}\right)=\sigma_y*b*\frac{h^2-e^2}{4}$$

Total moment $M_1+M_2=\sigma_y*\left(\frac{bh^2}{4}-\frac{be^2}{12}\right)$



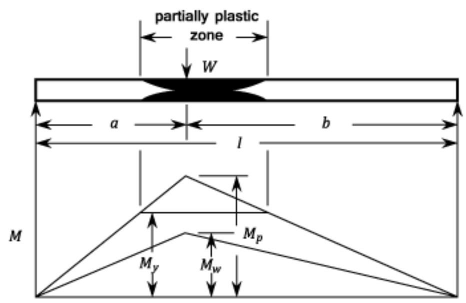
When section is completely yielded or plasticized. The curvature at the section become infinite and the section continue to rotate at constant moment. Now the section is not in condition to take any further moment hence section behave like a hinge.

The plastic hinge is defined as a yield zone due to bending in a structure member, at which the infinite rotation can take place at a constant moment M_p.

Length of plastic hinge for rectangular section with point load.

By gradually increasing 'W' we get $M_p=\frac{WL}{4}$ $M_y=\frac{\sigma_y bh^2}{6}$ and

$M_p=\frac{\sigma_y bh^2}{2}$ $\Rightarrow M_y=\frac{2}{3}M_p$ …….(1)



From bending moment diagram $\frac{M_p}{M_y}=\frac{\frac{L}{2}}{\frac{L}{2}-\frac{x}{2}}$ …….(2)

Substituting the value of $\frac{M_p}{M_y}$ from (1) in (2) we get $\frac{3}{2}=\frac{\frac{L}{2}}{\frac{L}{2}-\frac{x}{2}}$ $\Rightarrow x=\frac{L}{3}$ and variation is parabolic.

Similarly for rectangular section with U.D.L $x=\frac{L}{\sqrt{3}}$ and shape is linear

For I section with point load at center $x=\frac{L}{8}$ and shape is parabolic.

Shape Factor $S = \frac{Z_p}{Z}$ it is a geometric property and totally depend on section

1. Rectangle $Z = \frac{bh^2}{6}$ and $Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{bh}{2}\left(\frac{h}{4} + \frac{h}{4}\right) = \frac{bh^2}{4}$ $S = \frac{Z_p}{Z} = 1.5$
2. Diamond section $Z = \frac{bh^2}{24}$ & $Z_p = \frac{bh^2}{12}$ $S = 2$
3. Triangular section Z=2.34 4. Circular section S=1.70 5. Hallow section $S = \frac{\frac{16}{2\pi}\left(1-\left(1-\frac{2t}{d}\right)^3\right)}{1-\left(1-\frac{2t}{d}\right)^4}$ when d>>t

S=1.27 5.S for I section $I_{xx} = 1.12$ and $I_{yy} = 1.55$

Shape factor $S = \frac{Z_p}{Z} = \frac{M_p}{M_y} = \frac{\emptyset_p}{\emptyset_y}$ $\emptyset_p = \left(\frac{1}{R}\right)_p$ and $\emptyset_y = \left(\frac{1}{R}\right)_y$ R is radius of curvature

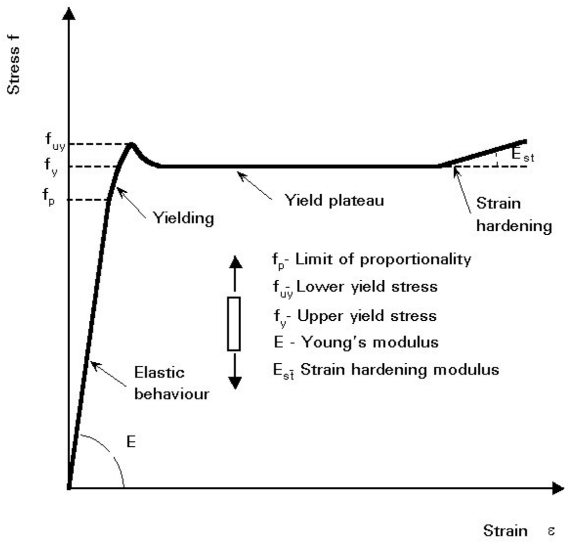
Load Factor $Q = \frac{W_c}{W_w} = \frac{\text{Collape Load}}{\text{Working Load}} = \frac{M_p}{M_1}$ $\frac{M_p}{M_y} = \frac{\sigma_y * z_p}{z * f_1}$ $\frac{\sigma_y}{f_1} = \text{Factor of Safety}$

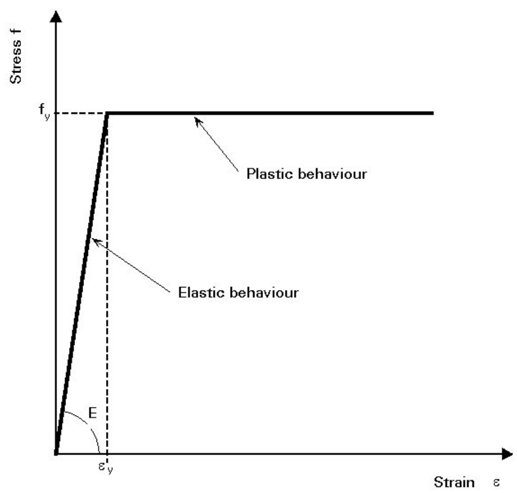
Hence, Load Factor $Q = \text{Shape Factor} * (\text{Factor of Safety}) = S * (F.S)$

For gravity loading $\frac{\sigma_y}{f_1} = \text{Factor of Safety} = 1.65$ hence $(Q)_{Isection} = 1.12 * 1.65 = 1.85$

For wind load $\frac{\sigma_y}{1.33 f_1} = \text{Factor of Safety} = 1.24$ hence $(Q)_{Isection} = 1.12 * 1.24 = 1.4$

Hence, for plastic analysis collapse load is 1.85 times the working load for gravity loading for wind loading this is 1.4 times the working load for I section.



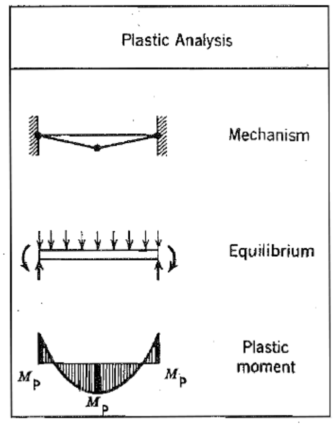


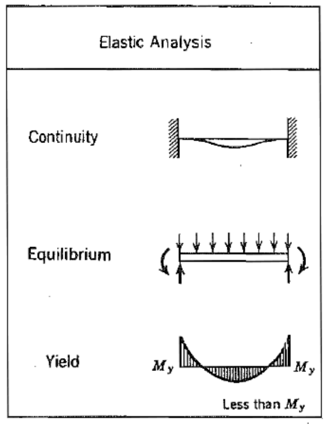
3

Fundamental condition for plastic analysis

For Elastic analysis we have three conditions (1) Equilibrium condition (2) Continuity condition or compatibility condition (3) Limiting stress condition

For Plastic analysis we have three conditions (1) Equilibrium condition (2) Mechanism condition (3) Plastic moment condition





When any elastic body is subjected to a system of load and deformation takes place, and the resistance is set up against the deformation then the elastic body is known as structure. In contradict to this, if no resistance is set up in the body against the deformation, than it is known as mechanical mechanism.

If a structure is having a indeterminacy of r, then the indeterminate structure becomes a determinate structure on the formation of r number of the plastic hinges. If on additional hinge is formed, after the structure has become a determinate one, than a mechanism is formed. Thus (r+1) number of plastic hinges are necessary to convert a structure into a mechanism.

Complete Collapse ⇒ If number of hinges are exactly r+1 than collapse will be know as Complete collapse.

Over Complete Collapse ⇒ If number of hinges are more than r+1 than collapse will be known as over complete collapse.

Partially Collapse ⇒ If number of hinges are less than f+1 than collapse will be know as Partially collapse.

Type of Mechanism

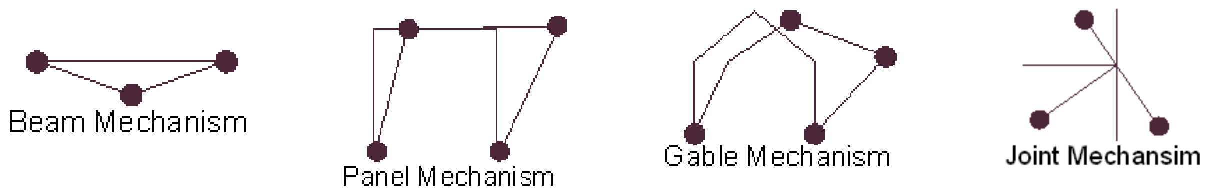
(1) Beam Mechanism

(2) Panel or Sway Mechanism

(3) Gable Mechanism

(4) Joint Mechanism

(5) Combination of any above.



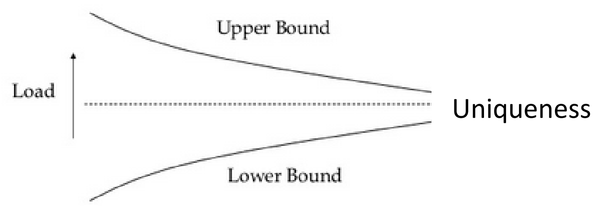
Number of Independent Mechanism = Number of Possible Hinges – Static Indeterminacy.

Equilibrium Condition Till mechanism is not formed $\Sigma F_x = 0, \Sigma F_y = 0 \;\; \Sigma M = 0$

Plastic Condition the bending moment at any section should not be more than the fully plastic moment of the section.

Static Theorem or Lower Bound Theorem for collapse not to occur $W \leq W_c$

Kinematic Theorem or Upper Bound Theorem for any mechanism $W \geq W_c$



Uniqueness Theorem or Combined Theorem For failure in plastic$W = W_c$

if structure is having a r indeterminacy, than at formation of r hinges the structure will determinate, at r+1 hinge formation structure will turn to a mechanism, (complete collapse), and at r+2 hinge formation structures will be over complete collapse.

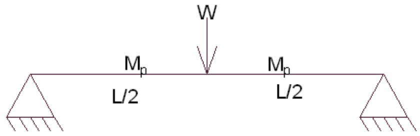
External work done is (1) for point load= $W * Deflection\ under\ the\ load$

(2) For UDL = Intensity of loading X Area of collapse mechanism diagram under the loading.

Internal work done =Plastic moment capacity of the section X cumulative change in angle.

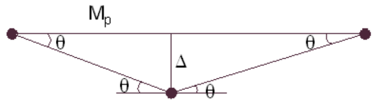
Example 1. Find the collapse load for the given structure.

Solution First find the position of no of possible hinges , in the given question the number of possible hinge is at under the load.



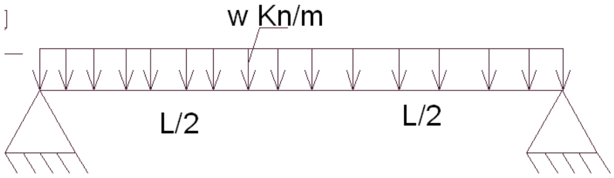
The indeterminacy of the structure is 0, for complete collapse we need 1 hinge at B.

By method of virtual work, Internal work done = External work done.

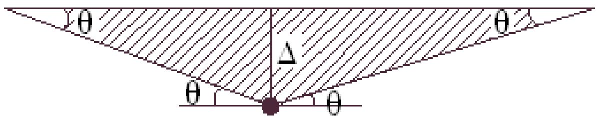


$M_p * (\theta + \theta) = W * \Delta \qquad \Delta = \frac{L}{2} * \theta$ hence,

$$M_p * 2\theta = W * \frac{L}{2} * \theta \Rightarrow W_c = \frac{4M_p}{L}$$



Example2 Find the collapse load for the given structure.



External work done $= W * \frac{L}{2} * (\frac{L}{2}\theta)$

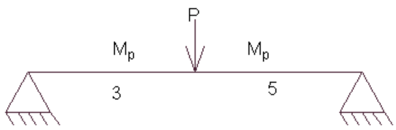
Internal Work done$= M_p(\theta + \theta + \theta + \theta) = 4M_p\theta$

$\Rightarrow 4M_p\theta = W * \frac{L}{2} * (\frac{L}{2}\theta)$

$M_p = \frac{W*L*\theta}{16}$

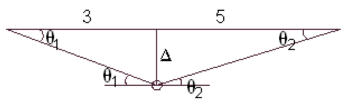
Example3 Find the collapse load for the given structure

Solution $3\theta_1 = 5\theta_2$



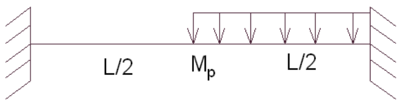
Internal work done $M_p(\theta_1 + \theta_2) = M_p\left(\theta_1 + \frac{3}{5} * \theta_1\right) = \frac{8M_p\theta_1}{5}$

External work done $= P * \Delta = P * 3\theta_1$

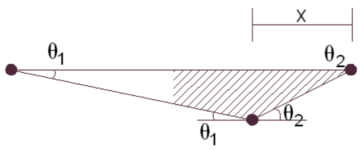


$$P * 3L\theta_1 = \frac{8M_p\theta_1}{5} \Rightarrow P = \frac{8M_p}{15L}$$

Example4 Find the collapse load for the given structure.



Solution $\Delta = \theta_1(L - x) = x\theta_2 \Rightarrow \theta_2 = \frac{\theta_1(L-x)}{x}$



External work done $= W * \left(\left(\frac{1}{2} * x * x * \theta_2\right) + \left(\frac{1}{2} * (L - x) * (L - x) * \theta_1\right) - \left(\frac{1}{2} * \frac{L}{2} * \frac{L}{2} * \theta_1\right)\right)$

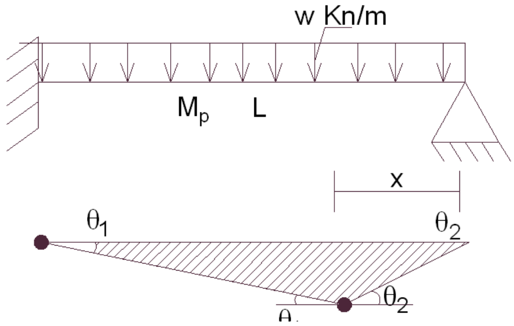
$W * \left(\left(\frac{1}{2} * x * x * \frac{\theta_1(L-x)}{x}\right) + \left(\frac{1}{2} * (L - x) * (L - x) * \theta_1\right) - \left(\frac{1}{2} * \frac{L}{2} * \frac{L}{2} * \theta_1\right)\right)$........(1)

Internal work done $= M_p(\theta_1 + \theta_2 + (\theta_1 + \theta_2)) = M_p(\theta_1 * 2 + 2\theta_2) = M_p\left(\theta_1 * 2 + 2\frac{\theta_1(L-x)}{x}\right)$.........(2)

$$M_p\left(\theta_1 * 2 + 2\frac{\theta_1(L-x)}{x}\right) = W * \left(\left(\frac{1}{2} * x * x * \frac{\theta_1(L-x)}{x}\right) + \left(\frac{1}{2} * (L - x) * (L - x) * \theta_1\right) - \left(\frac{1}{2} * \frac{L}{2} * \frac{L}{2} * \theta_1\right)\right)$$

$M_p = \left(-0.75W * x + \frac{2W}{L} * \frac{x^2}{2}\right)$.........(3)

For M_p to be max $\frac{dM_p}{dx} = \left(-075W + \frac{2W}{L} * \frac{2x}{2}\right) = 0 \Rightarrow x = 0.375L$

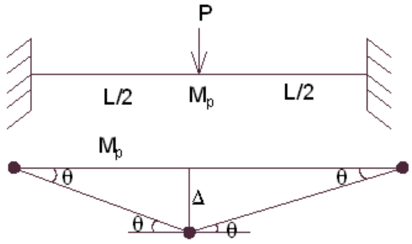


Substituting the value of x in (3)

$$M_p = \frac{WL}{14.22} \quad \text{or } W = \frac{14.22M_p}{L}$$

Example5 Find the position of the internal hinge for the propped cantilever as shown in Fig.

Solution the internal hinge will form at a distance of $x = 0.414L$ from the hinge end. And $W = 11.656\left(\frac{M_p}{L}\right)$



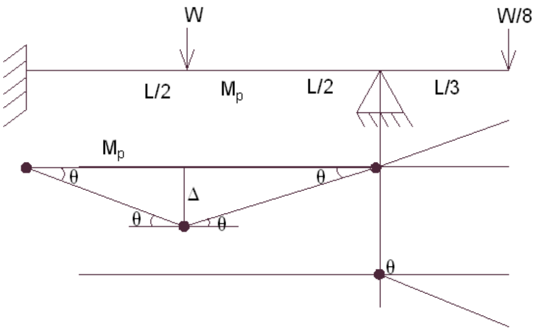
Example6 Find the collapse load for the given structure.

Solution Internal work done $M_p(\theta + \theta + \theta + \theta) = 4M_p$

External work done $= P * \Delta = P * \frac{L}{2}\theta$

$$P * \frac{L}{2}\theta = 4M_p \;\Rightarrow P = \frac{8M_p}{L}$$

Example7 Find the collapse load for the given structure.



Solution Two failure is possible

Failure 1.$W_1 * \frac{L}{2} * \theta - \frac{W}{8} * \frac{L}{3}\theta = (M_p * \theta + M_p * 2\theta)$

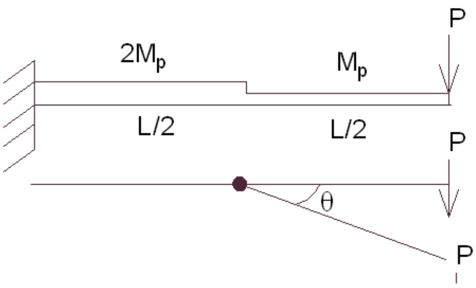
The work done by W/8 will be –ve as deflection is opposite of force.

$$W_1 = 6.545\frac{M_p}{L}$$

Failure 2$\frac{W_2}{8} * \frac{L}{3} * \theta = M_p * \theta \;\Rightarrow W_2 = \frac{24M_p}{L}$

From the above the two value the least will the collapse load $W = \frac{6.545M_p}{L}$

Example8 Find the collapse load for the given structure

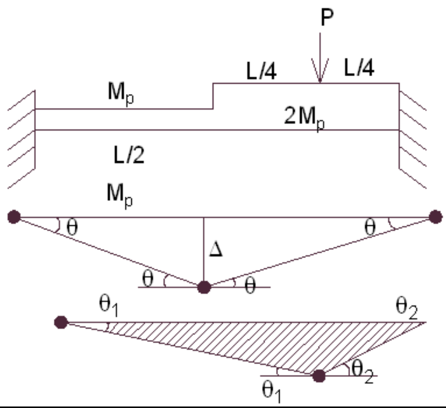


Solution Failure 1. $\frac{P_1L}{2} * \theta = M_p\theta \Rightarrow P_1 = \frac{2M_p}{L}$

Failure 2 $\quad P_2L\theta = 2M_p\theta \;\Rightarrow P_2 = \frac{2M_p}{L}$

From the above the two value the least will the collapse load $P = \frac{2M_p}{L}$

Example9 Find the collapse load for the given structure



Solution Failure 1

$$P_1 * \frac{L}{4} * \theta = (M_p\theta + 2M_p\theta + M_p(\theta + \theta) \Rightarrow P_1 = \frac{20M_p}{L}$$

Failure 2 $\quad \frac{3}{4}\theta_1 = \frac{1}{4}\theta_2 \Rightarrow \theta_1 * 3 = \theta_2$

$$P_2 * \frac{3}{4}L\theta = (M_p + 2M_p(\theta + 3\theta) + 2M_p * 3 \Rightarrow P_2 = \frac{20M_p}{L}$$

The collapse load for the beam $P_2 = \frac{20M_p}{L}$

Important Points to Understand

- Plastic Hinges are formed at point have greatest curvature and after having plastic hinge at a section there is distribution of moment at other section having more curvature till the time mechanism is formed.
- Since plastic analysis is used outside the hooks law hence we can't apply principle of superposition thus deformation is not directly proportional to load anymore.
- Between two plastic hinges, the portion in between will behave as rigid member not elastic.
- The basic assumption, planes sections remain plane after and before the bending, shear deformation is ignored is valid for plastic analysis also.