BYJU'S EXAM PREP

AE/JE Foundation

Civil Engineering

Design of Steel Structures

Important Formula Notes

IMPORTANT FORMULAS ON DESIGN OF STEEL STRUCTURES

1. SAFE AND YIELD MOMENT OF RESISTANCE

1.1. Safe moment of resistance: $M = f \times z$

Where, f = stress in extreme fibre and z = section modulus

1.2. Yield Moment of Resistance (M_y): $M_y = f_y \times z$

Where, f_y = yield stress and z = section modulus

2. PLASTIC MOMENT CAPACITY

Plastic moment = M_p = C × lever arm = T × lever arm.

M_p = C × lever arm $= f_y \times \frac{A}{2} \times (\bar{y}_1 + \bar{y}_2)$

$$M_P = f_y\left[\frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)\right] = f_y z_P$$

Here, $z_P = \frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)$ = Plastic modulus

$\bar{y}_1, \bar{y}_1$ = Centroid distances of compression area and tension area from neutral axis.

3. IMPORTANT FACTORS IN PLASTIC DESIGN

3.1. Shape factor (SF) $= \frac{M_P}{M_y} = \frac{f_y z_P}{f_y z} \quad \Rightarrow SF = \frac{z_P}{z}$

3.2. Load factor (LF) $= \frac{M_P}{M} = \frac{f_y . z_P}{fz}$

$$\frac{f_y}{f} \times \frac{z_P}{z} = FoS \times SF = \text{load factor}$$

∴ Load factor = Factor of Safety × Shape factor

3.3. Factor of Safety (FOS)

Factor of safety for ductile material, $FOS = \frac{f_y}{f}$

Factor of safety for brittle material, $FOS = \frac{\text{Ultimate stress}}{\text{Working stress}}$

Shape factors for various cross-sections:

S.No.	CROSS SECTION	SHAPE FACTOR
1.	Rectangle	1.5
2.	Circle	1.7
3.	Rolled steel I section	1.12 to 1.14
4.	H section	1.5
5.	Diamond section	2
6.	Triangle	2.34

3.4. Calculation of Collapse Load

To form a mechanism, the number of plastic hinges required = n = D_s +1.

In the plastic analysis, the following conditions must be satisfied.

i) Equilibrium Condition: the equation of equilibrium should be satisfied.

ii) Mechanism Condition: sufficient number of plastic hinges must develop so that a part or entire structure must transform into a mechanism leading to collapse.

iii) Yield Condition: Bending moment at any section should not exceed plastic moment capacity (M_p) of the cross-section.

On the basis of the above 3 conditions, there are two methods:

i) Kinematic method or Kinematic theorem or upper bound theorem

a) It is the combination of equilibrium and mechanism conditions.

b) It also states that the collapse load formed by assuming a mechanism will always be greater than or equal to the true collapse load.

ii) Static method or Static theorem or lower bound theorem

a) It satisfies equilibrium and yield conditions.

b) It states that the collapse load formed based on any collapsed bending moment will always be less or equal to the true collapse load.

4. LOCATIONS WHERE PLASTIC HINGES CAN FORM

i) At maximum bending moment locations.

ii) At fixed supports and rigid joints.

iii) Wherever the cross-section changes.

iv) Under point loads in supported spans but not at a free end.

v) Whenever the material changes.

4.1. Principle of Virtual Work:

"when a body is in equilibrium, the total virtual work done by all forces is zero".

4.2. Collapse Load for Various Cases

i) Simply Supported beam

a) Point load at mid-length: Collapse load = $\frac{4M_P}{\ell}$ KN

b) Uniformly distributed load: Collapse load $= \frac{8M_P}{\ell^2}$ KN / m

c) Eccentric loaded point load: Collapse load = $\frac{M_P \ell}{ab}$ $\{\because a + b = \ell\}$

(ii) Fixed beam

a) Point load at mid-length: Collapse load $= \frac{8M_P}{\ell}$ KN

b) Uniformly distributed load:

⇒ When two hinges are formed simultaneously at the ends $\Rightarrow W_1 = \frac{12M_P}{\ell^2}$ KN / m

⇒ When the third hinge is formed at mid-span after the hinges form at the ends

Collapse load $= \frac{16M_P}{\ell^2}$ KN / m

c) Eccentric point load: Collapse load $= \frac{2M_P \ell}{ab}$ kN

(iii) Propped cantilever beam

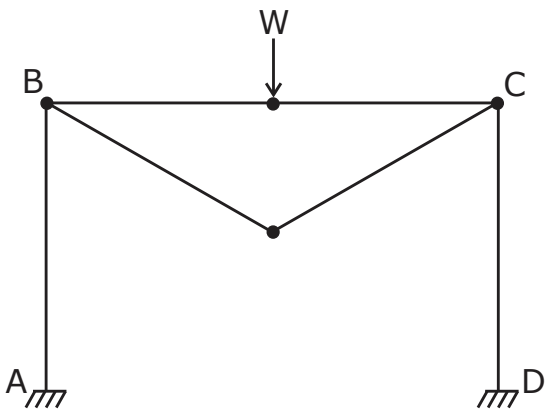
a) Point load at mid-length: Collapse Load = $\frac{6M_P}{\ell}$ kN

b) Uniformly distributed load: Collapse Load = $\frac{11.656M_P}{\ell^2}$ kN

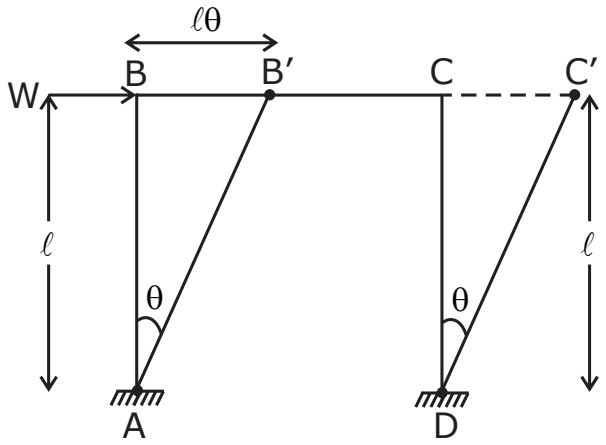
c) Eccentric point load: Collapse load $= \frac{M_P(\ell + b)}{ab}$ kN

4.3. Plastic Analysis of Frame

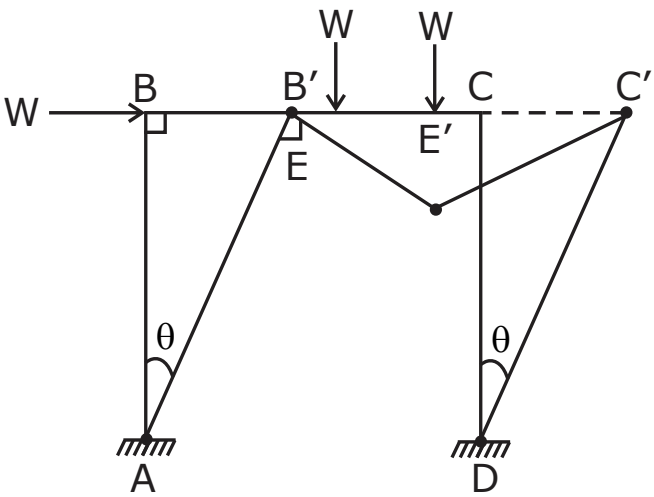
4.3.1. Beam Mechanism: the mechanism is formed only in one member of the portal frame,



4.3.2. Sway Mechanism: if the frame sways to the left or to the right.



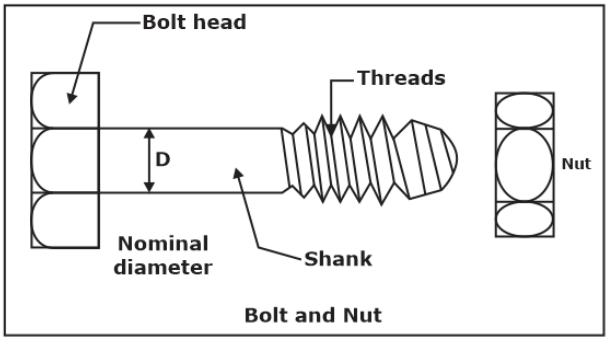
4.3.3. Combined Mechanism: It is formed by the combination of sway and beam mechanism.



DESIGN OF CONNECTIONS

1. **INTRODUCTION:** Different types of fasteners available are rivets, bolts, pins and welds.

2. **BOLTED CONNECTIONS**



Bolts are classified as:

(a) Unfinished or Black Bolts, (b) Finished (turned) bolts, (c) High strength friction grip (HSFG) bolts

2.1. Black Bolts: Black bolts are also referred to as ordinary, unfinished, rough, or common bolts. They are the least expensive bolts.

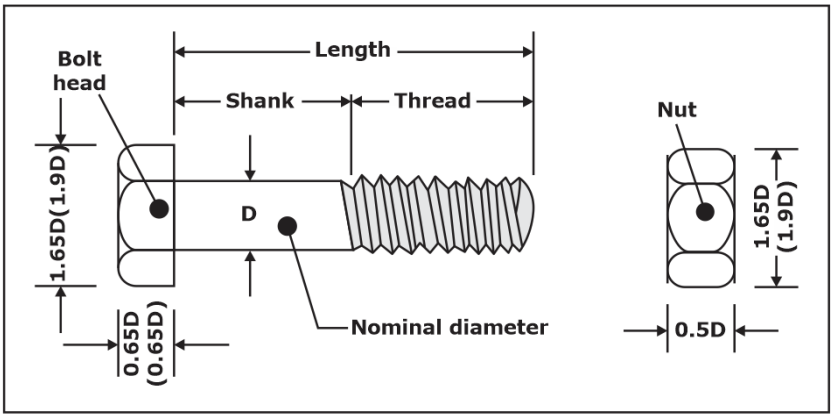


Fig. Hexagonal head black bolt and nut.

Figures in bracket are for high-strength bolts and nut

In steel construction, generally, bolts of property class 4.6 are used. In property class 4.6, the number 4 indicates 1/100th of the nominal ultimate tensile strength in N/mm^2 and the number 6 indicates the ratio of yield stress to ultimate stress expressed as a percentage. Thus, the ultimate tensile strength of a class 4.6 bolt is 400 N/mm^2 and yield strength is 0.6 times 400, which is 240 N/mm^2.

2.2. Turned Bolts (Close Tolerance Bolts): These are similar to unfinished bolts, with the difference that the shanks of these bolts are formed from a hexagonal rod. They are mainly used in special jobs (in some machines and where there are dynamic loads).

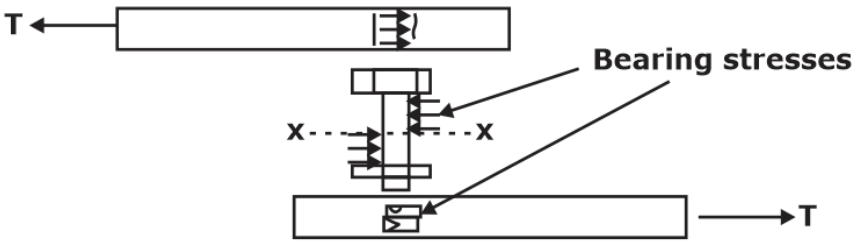
2.3. High Strength Friction Grip bolts (HSFG): The tension in the bolt ensures that no slip takes place under working conditions and so the load transmission from plate to the bolt is through friction and not by bearing. HSFG bolts are made from quenched and tempered alloy steels with grades from 8.8 to 10.9.

3. CLASSIFICATION OF BOLTS BASED ON METHOD OF LOAD TRANSFER

On the basis of load transfer in the connection, bolts may be classified as:

(a) Bearing type: Unfinished (black) bolts and finished (turned) bolts are bearing type since they transfer shear force from one member to other members by bearing.

(b) Friction grip type: HSFG bolts belongs to the friction grip type since they transfer shear by friction.



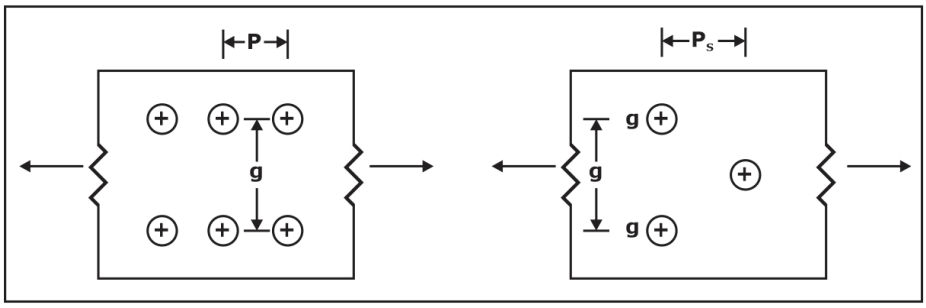
(a) Bearing connection

4. TERMINOLOGY IN BOLTED CONNECTION

1. Pitch of the Bolts (P): It is the centre to centre spacing of bolts in a row, measured in the direction of load.

2. Guage (g): It is the distance between the two consecutive bolts of adjacent rows and is measured at right angle to the direction of load.

3. Staggered Pitched (P_s): It is the centre to centre distance of staggered bolts measured in the direction of load.



4. Diameter of Bolt Hole: The diameter of the bolt hole is larger than the nominal diameter (shank diameter) of the bolt

5. Area of Bolt at Root (A_{nb}): Area of Bolt at root of the thread is less than at shank of the Bolt. For some common bolt sizes, $A_{nb} = 0.78 \times A_{sb}$ where, A_{sb} = Area of bolt at shank.

5. IS 800– 2007 SPECIFICATIONS FOR SPACING AND EDGE DISTANCE OF BOLT HOLES

(i) Pitch shall not be less than 2.5 d, where 'd' is the nominal diameter of the bolt.

(ii) Pitch shall not be more than

(a) 16t or 200 mm, whichever is less, in case of tension members.

(b) 12t or 200mm, whichever is less, in case of compression members where t is the thickness of the thinnest member.

(c) In the case of staggered pitch, the pitch may be increased by 50 per cent of the specified value provided the gauge distance is less than 75 mm.

(iii) In case of butt joints, the maximum pitch is to be restricted to 4.5d for a distance of 1.5 times the width of the plate from the butting surface.

(iv) The gauge length 'g' should not be more than 100+4t or 200mm whichever is less.

(v) Minimum edge distance shall be

(a) Less than 1.7 × hole diameter in case of sheared or hand flame cut edge.

(b) Less than 1.5 × hole diameter in case of rolled, machine flame cut, sawn and planed edges.

(vi) Maximum edge distance should not exceed.

(a) $12t\,\epsilon$, where $\epsilon = \sqrt{\frac{250}{f_y}}$ and t is the thickness of the thinner outer plate.

(b) 40 + 4t, where t is the thickness of thinner connected plate if exposed to corrosive in Environment.

(vii) Apart from the required bolt from the consideration of design forces, additional bolts called tacking fasteners should be provided as specified below.

(a)If the value of gauge length exceeds after providing design fasteners at maximum edge distances tacking rivets should be provided.

- At 32t or 300mm, whichever is less, if plates are not exposed to the weather.
- At 16t or 200mm, whichever is less if plates are exposed to the weather.

(viii) In the case of a member made of up two flats, or angles or tees or channels, tacking rivets are to be provided along the length to connect its components as specified below.

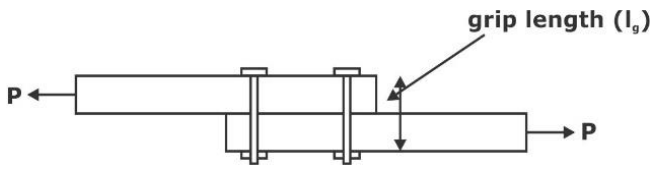
(a) Not exceeding 1000mm, if it is a tension member.

(b) Not exceeding 600mm, if it is a compression member.

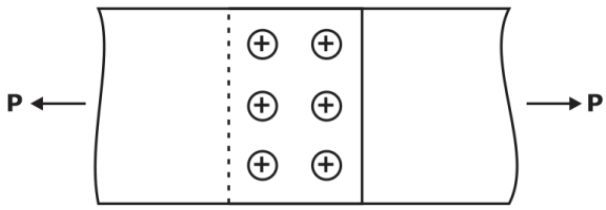
6. TYPES OF BOLTED CONNECTION

Bolted joints may be grouped into the following types.

6.1. Lap Joint: It is the simplest type of joint. In this, the plate to be connected overlaps one another.

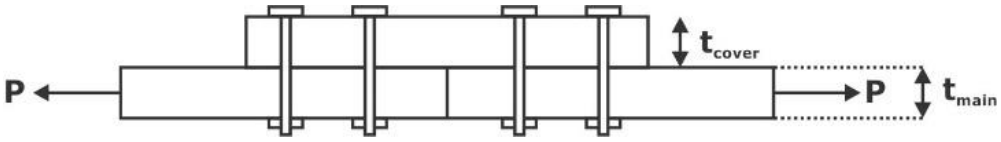


$l_g \not> 5d$, d = diameter of the bolt.



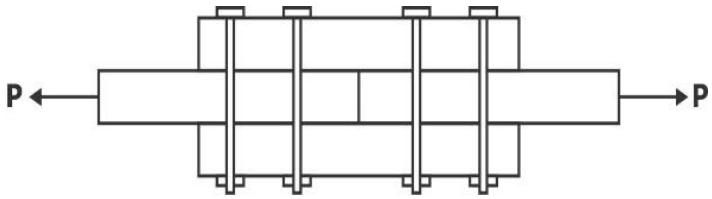
Lap Joint

6.2. Single Cover Butt Joint: Bolts in single cover butt joints are subjected to single shear and bearing.



$t_{cover} \geq t_{main}$

6.3. Double cover butt joint: Bolts in double cover butt joint are subjected to double shear and bearing.



sum of $t_{cover} \geq t_{main}$

7. **ASSUMPTIONS IN DESIGN OF BEARING BOLTS:** The following assumptions are made in the design of bearing (finished or unfinished) bolted connections:

1. The friction between the plates is negligible.
2. The shear is uniform over the cross-section of the bolt.
3. The distribution of stress on the plates between the bolt holes is uniform
4. Bolts in a group subjected to direct loads share the load equally
5. Bending stresses developed in the bolts in neglected.

8. **DESIGN TENSILE STRENGTH OF PLATES IN A JOINT**

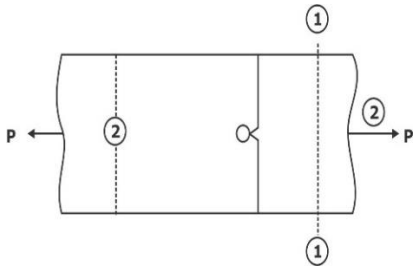
(i) Failure of bolts

(a) Shear failure of bolt, the bolt will get cut into 2 pieces.

(b) Bearing failure of bolt, the bolt will go out of shape.

(ii) Failure of plates

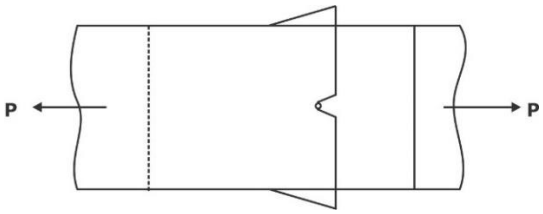
(a) Shear of plates, the cracks are developed parallel to the direction of force.



Shear stress at (2) – (2)

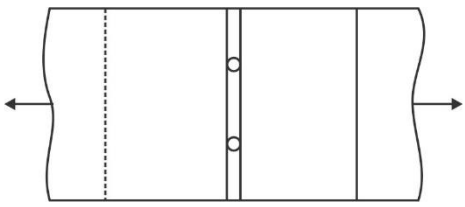
Tensile stress at (1) – (1)

(b) Splitting failure: It occurs due to diagonal tension in the plate at bolt level.



(c) Bearing failure: Bolt will push the plate forward. It occurs when the bearing strength of the plate is less.

(d)Tension failure or tearing failure of plate: Cracks are developed perpendicular to the direction of force.



Design strength of bolted joint: It is taken as the least value of

- Shear strength of all bolts in the joint (P_s)
- Bearing strength of all bolts in the joint (P_b)
- Tensile strength of the plate (P_t)

(i) Lap joint

Case (a): For entire width of plate.

P_s = shear strength of all bolts in the joint.

$$P_s = n \times \frac{\pi}{4} \times d^2 \times f_s$$

n = Total number of bolts in the joint.

d = Diameter of bolt.

f_s = permissible shear stress in the bolt.

$$f_s = \frac{f_u}{\sqrt{3} \times 1.25} \text{ (for LSM)}$$

P_b = bearing strength of all bolts in the joint.

$P_b = n \times d \times t \times f_b$

n = number of bolts in the joint

d × t = projected area of the bolt against the plate.

t = thickness of thinner plate.

f_b = permissible bearing stress in the bolt.

$$f_b = 2.5 \times k_b \times \frac{f_u}{1.25}$$

P_T = Tensile strength of plate.

$$P_T = Ag \times \frac{f_y}{1.1} \quad \text{(Based on gross area yielding at x.x)}$$

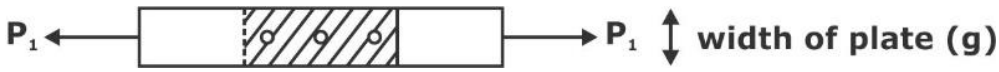
$$= A_{net} \times \frac{0.9 f_u}{1.25} \quad \text{(based on net area)}$$

Least value of P_s, P_b and P_t is taken as the strength of the joint.

Case (b): Strength of joint / gauge length

P_{S_1} = Shear strength of all bolts in shaded gauge length.

$$P_{S_1} = n_1 \times \frac{\pi}{4} \times d^2 \times f_s$$



n_1 = total number of bolts in shaded gauge length.

$n_1 = 3$

P_{b_1} = bearing strength of all bolts in shaded gauge length.

$$P_{b_1} = n_1 \times d \times t \times f_b$$

P_{t_1} = tensile strength of the plate/gauge length

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

$$= A_{net} \times \frac{0.9 f_u}{1.25}$$

(ii) Double Cover butt joint

P_s = shear strength of all bolts in the joint

$$P_s = n \times 2 \times \frac{\pi}{4} \times d^2 \times f_s$$

P_b = Bearing strength of all bolts in the joint.

$$P_b = n \times d \times t \times f_b$$

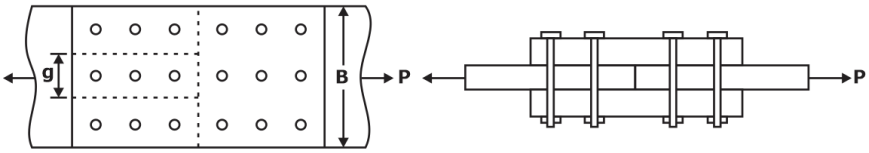
n = g

t = sum of cover plate thickness or thickness of thinner main plate, whichever is less.

P_t = tensile strength of plate.

$$= Ag \times \frac{f_y}{1.1}$$

$$= A_{net} \times \frac{0.9 f_y}{1.25}$$



(iii) Rivet value (R_v): It is the strength of a single bolt. It is taken as the least value of P_s and P_b of a single bolt.

(a) When bolt is in single shear

$$\left.\begin{aligned} P_s &= \frac{\pi}{4} \times d^2 \times f_s \\ P_b &= d \times t \times f_b \end{aligned}\right\} \text{whichever is less is } R_v$$

(b) If bolt is in double shear

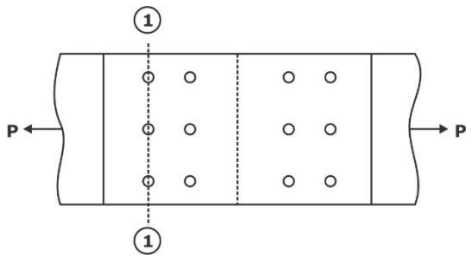
$$\left.\begin{aligned} P_s &= 2 \times \frac{\pi}{4} \times d^2 \times f_s \\ P_b &= d \times t \times f_b \end{aligned}\right\} \text{whichever is less}$$

$$\text{Number of bolts required at a joint} = n = \frac{\text{Factored load}}{\text{Rivet value}}$$

$$n = \frac{F}{R_v}$$

9. ARRANGEMENT OF BOLT

(i) Chain bolting:

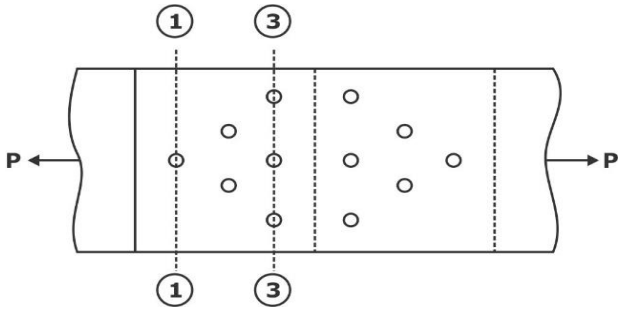


Here, net area = A_{net} = (B – 3d)t at 1 – 1

d = diameter of bolt hole

B = width of plate

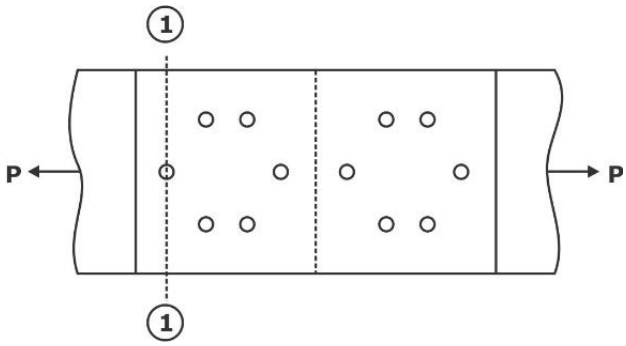
(ii) Diamond bolting



here, net area = A_{net} = (B – d)t at 1 – 1

So, Diamond riveting/bolting is more efficient than chain bolting.

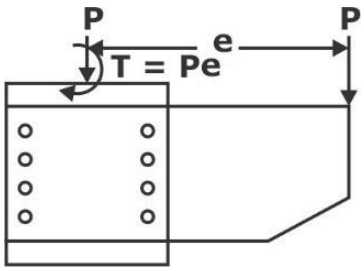
(iii) Mixed Bolting



In this arrangement, both cover plates and the main plate can take the maximum load.

10. ECCENTRIC BOLTED CONNECTION: If the centre of gravity of the bolt group does not lie on the line of action of the load, then it is called eccentric connection.

10.1. In-plane eccentricity: In this case, the bolt group and load are on the same plane but CG of bolt group does not lie on the line of action of the load.



There are two effects when the load is applied as shown above

→ direct load (P)

→ twisting moment (T) = Pe.

Direct shear force on bolt due to load P, $F_1 = \frac{P}{\Sigma A_i} \times A_i$

where, A_i = cross-section area of each bolt.

If the cross-sectional area is the same for all bolts, then, $F_1 = \frac{P}{nA_i} \times A_i = \frac{P}{n}$

where, n = number of bolts.

Shear force in the bolt due to twisting moment (T), $F_2 = \frac{Pe}{\Sigma A_i r_i^2} \times r_i A_i$

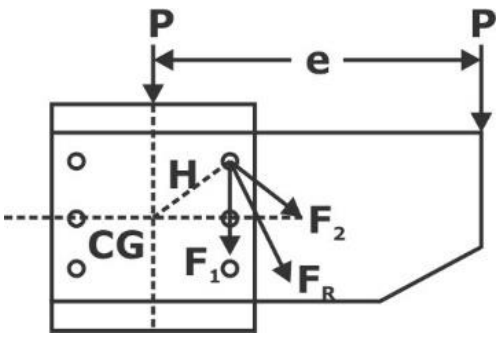
If all the bolts have the same area, $F_2 = \frac{Pe}{\Sigma r_i^2} \times r_i$

where, r_i = radial distance of each bolt from CG of bolt group.

Resultant shear force in the bolt, $F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

θ = the angle between F_1 and F_2

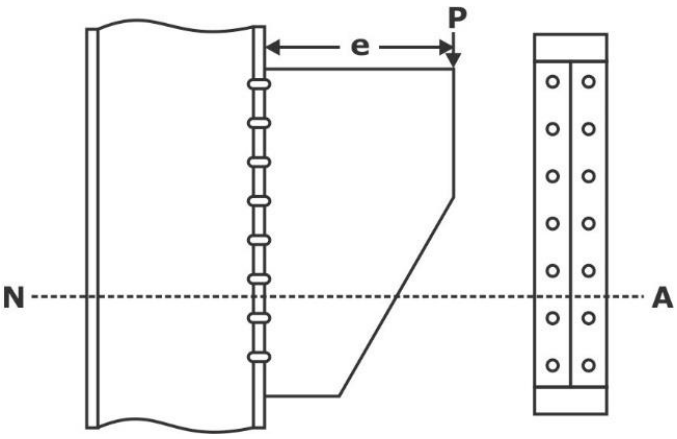
$F_R \not> R_v$ for safe bolting, R_V = rivet/bolt value.



If the bolts are of the same diameter, the most critically stressed bolt is the one for which 'r' is maximum and θ is minimum.

10.2. Out of plane eccentricity

Section is subjected to a direct shear force P and moment M = Pe.



On tension side, only bolt resists load but on compression side entire zone between the columns and the connecting angle resist the load.

It is assumed that the neutral axis (NA) lies at a height of $\frac{L}{7}$ of the depth of bracket, measured from the bottom edge of the angle.

Since bolts above NA are subjected to tension and shearing, the governing criteria to prevent failure of the bolt is given by

$$\left(\frac{P_{T,\,cal}}{P_T}\right)^2 + \left(\frac{P_{S,\,cal}}{P_S}\right)^2 \le 1$$

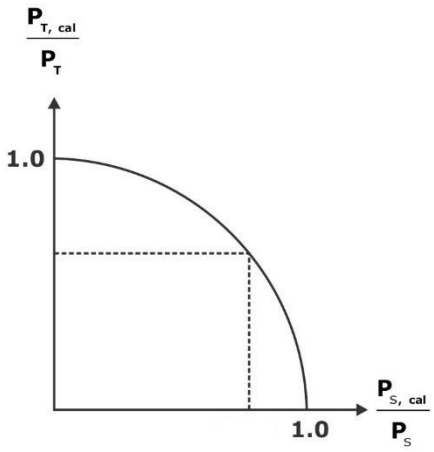
Here,

$P_{T,\,cal}$ = calculated factored tensile force in the bolt.

P_T = tension capacity of bolt

P_S = shear capacity of bolt

Interaction curve between P_T and P_S



11. DESIGN OF WELDED JOINT: There are three types of welded joints

(i) Butt weld

(ii) Fillet weld

(iii)Slot weld or plug weld

11.1. Butt weld: It is also called as groove weld. Depending upon the shape of the groove made for welding butt welds are classified as square butt weld, single v butt weld, double v butt weld, sing U butt joint, single J butt joint, single bevel butt joint.

11.2. Fillet Weld: It is a weld of approximately triangular cross section joining two surfaces approximately at right angles to each other in lap joint or corner joint. It is assumed that fillet weld always offer resistance in the form of shear only.

(i) Size of fillet weld:

(a) Size of normal fillet weld shall be taken as the minimum weld leg size.

(b) For deep penetration welds with penetration not less than 2.4 mm, size of weld is minimum leg size + 2.4 mm.

(c) For fillet welds made by semi automatic or automatic process with deep penetration more than 2.4 mm,

S = minimum leg size + actual penetration

(ii) Throat: It is the minimum dimension in fillet weld. Throat thickness = t_t = k × size of weld.

Where, k is a constant depends on angle between fusion faces.

Angle between fusion faces	k
60° — 90°	0.7
91° — 100°	0.65
101° — 106°	0.6
107° — 113°	0.55
114° — 90°	0.5

Fillet weld should not be used if the angle between fusion faces is less than 60° or more than 120°.

(iii) Minimum size of weld

The minimum size of weld specified is 3mm. To avoid the risk of cracking in the absence of preheating the minimum size specified are

Thickness of thicker plate	**Minimum size of weld**
< 10 mm	3 mm
10 mm	5 mm
20 mm – 32 mm	6 mm
32 mm – 50 mm	8 mm

(iv)Maximum size of weld

(a)At a square edge, Maximum size of weld = thickness of plate – 1.5 m

(b)At the round edge, Maximum size of weld $= \frac{3}{4} \times$ thickness of plate

(v) Effective length of weld: Welding length made is equal to effective length plus twice the size of the weld. Effective length should not be less than four times the size of the weld.

(vi) Lap joint: The minimum lap should be four times the thickness of thinner part joined or 40 mm whichever is more. The length of weld along either edge should not be less than the transverse spacing of welds.

(vii) Strength of weld: Load carrying capacity of weld

P = permissible shear stress in weld × effective area of weld.

$$P = f_s \times l_{eff} \times t_t$$

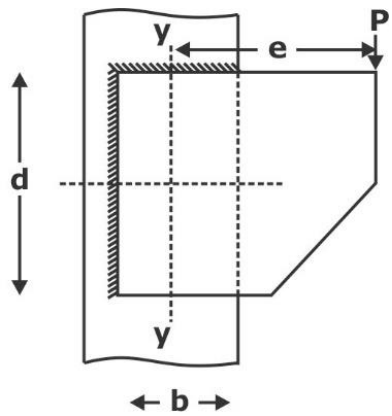
Where, f_s = permissible shear stress in weld $= \dfrac{f_u}{\sqrt{3} \times 1.25}$

11.3. Slot weld or plug weld: Slot welding is alone to increase the length of the weld.

Minimum width of slot = 3 t or 25 mm, whichever is less.

Here, t = thickness of plate in which slot is made.

12. ECCENTRIC WELDED CONNECTION: Plane of moment and the eccentric load, P is equivalent to a direct load P at the centre of gravity of the group of weld and a twisting moment, P × e.



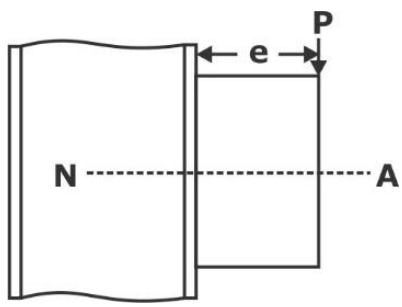
Due to direct load P, shear stress f_1, is developed and due to twisting moment, torsion shear stress f_2 is developed at A (this torsional shear stress is maximum if radial distance is maximum). Resultant due to f_1 and f_2 can be calculated as follows:

$$f_R = \sqrt{f_1^2 + f_2^2 + 2f_1 f_2 \cos\theta}$$

θ = angle between f_1 and f_2

Out of plate eccentricity:

The effect of eccentric load is equivalent to a direct load acting at centre of gravity of weld group and bending moment, P × e.



Neutral axis for the weld group lies at the centre of gravity of weld group because bending tensile stress and bending compressive stress are resisted by weld only.

Due to direct load, P vertical shear stress f_1 is developed and due to bending moment, bending tensile stress is developed.

Resultant stress is calculated as follows

$$f_R = \sqrt{f_1^2 + f_2^2} \le f_s = \frac{f_u}{\sqrt{3} \times 1.25}$$

TENSION MEMBERS

1. **LUG ANGLE:** Length of the end connection of a heavily loaded tension member may be reduced by using lug angles. By using the lug angle, there will be saving in the greatest plate.
   (i) The effective connection of the lug angle shall as far as possible terminate at the end of the member.
   (ii) The connection of lug angle to main angle shall preferably start in advance of the member to the gusset plate.
   (iii) Minimum of two bolts rivets or equivalent welds be used for attaching lug angle to gusset plate.
   (iv) The purpose of lug angle is to reduce the shear lag effect and to reduce the length of the connection to the gusset plate.
   (v) Shear lag factor , which takes care of loss of efficiency due to shear lag should not be less than 0.7.
2. **LONG JOINTS:** If the length of the joint is more than 15 d or 150 t_t where d is the diameter of bolt hole and t_t is the throat thickness.
3. **GRIP LENGTH:** grip length should not more than 5d, where d is the diameter of bolt hole. If the value of grip length is in the range of 5d to 8d, then P_s is multiplied by a reduction factor to take care of the additional stresses. But in case the value of grip length is more than 8d, then in that case the section must be redesigned.

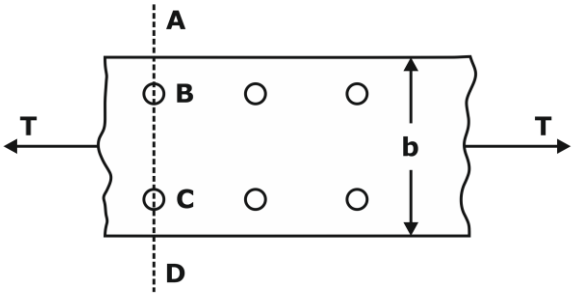
4. **SLENDERNESS RATIO LIMITS**

Type of member	Maximum value of slenderness ratio
(i) Pure tension member	400
(ii) Hanger bars	160
(iii) Pure compression member	180
(iv) Lacing member	145
(v) Tension member subjected to reversal of stresses due to live load	180
(vi) Pure tension member subjected to reversal of stress due to action of wind.	350

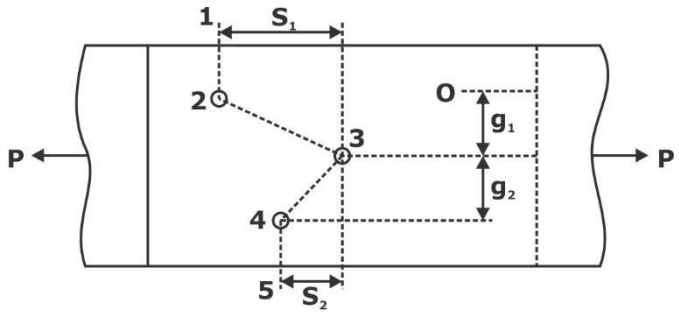
5. **NET SECTIONAL AREA:**

Case 1: Chain Bolting: Net area along the section ABCD, A_{net} = (B – nd)t

Where, n = number of bolt holes, d = diameter of bolt holes, t = thickness of plate.



Case 2: Staggered Bolting



Net Area along the section 1-2-3-4-5 is given by, $A_{net} = \left[B - nd + \frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2}\right] \times t$

If s_1=s_2, and g_1=g_2, $A_{net} = \left[B - nd + \frac{n's^2}{4g}\right] \times t$

Where, s = staggered pitch, g = gauge distance, n' = number of staggered pitches, n = number of holes in zig zag line

6. **LOAD CARRYING CAPACITY OF A TENSION MEMBER:** A tension member may fail in any of the following modes:

6.1. Gross Section Yielding: $T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$

Here, A_g = gross area of plate,

f_y = yield stress,

γ_{mo} = Partial factor of safety governed by yielding = 1.1

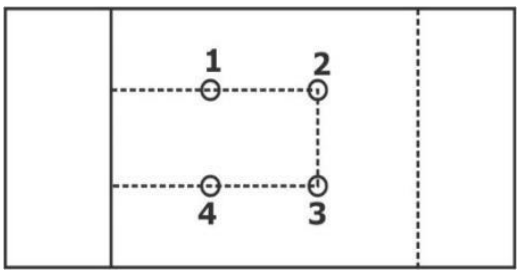
6.2. Net section Rupture: $T_{dn} = 0.9\frac{A_n f_u}{\gamma_{m1}}$

Here, A_n = Net area of plate,

f_u = Ultimate strength of plate

γ_{m1} = Partial factor of safety governed by ultimate strength = 1.25

6.3. Block shear failure:



Shear failure occurs along 1 – 2 and 3 – 4 whereas tension failure occurs along 2 – 3.

For shear yield and tension failure: $T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + 0.9\frac{A_{tn} f_u}{\gamma_{m1}}$

For tension yield and shear fracture: $T_{db2} = \frac{A_{tg} f_y}{\gamma_{m0}} + 0.9\frac{A_{vn} f_u}{\sqrt{3}\gamma_{m1}}$

Where,

A_{vg} = Gross area in shear along the line of action of force

A_{vn} = Net area in shear along the line of action of force

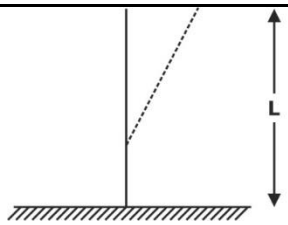
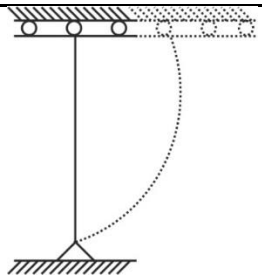
A_{tg} = Gross area in tension across the line of action of force

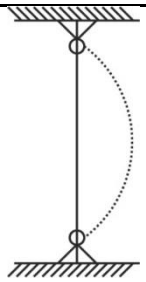
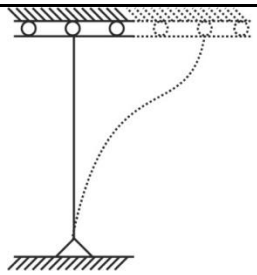
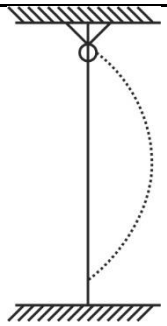
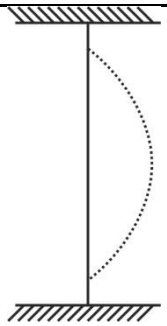
A_{tn} = Net area in tension across the line of action of force

COMPRESSION MEMBERS

1. EFFECTIVE LENGTH

(a) If end conditions can be assessed: Where the boundary conditions in plane of buckling can be assessed the effective length (KL) can be calculated as follows.

Boundary Condition				Diagram	Effective Length
At one end		At another end			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Free	Free		2.0 L
Restrained	Free	Free	Restrained		L

Restrained	Free	Restrained	Free		L
Restrained	Restrained	Free	Restrained		1.2 L
Restrained	Restrained	Restrained	Free		0.8 L
Restrained	Restrained	Restrained	Restrained		0.65 L

(b) Compression member in truss

- In case of bolted, riveted or welded trusses and braced frames, the effective length, KL, shall be taken as 0.7 to 1.0 times the actual length, depending upon the degree of end restraints provided.
- For buckling in the plane perpendicular to the plane of truss, the effective length may be taken as actual length.

(c) In frames: In frame analyses, if deformed shape is not considered the effective length depends upon stiffness of the members meeting at the joint.

2. EULER'S COLUMN FORMULA: Critical load on column, $P_{Cr} = \frac{\pi^2 E I_{min}}{L^2}$

Where, L = effective length, I_{min} = minimum moment of inertia = A r_{min}^2

r_{min} = minimum radius of gyration.

As per IS 800: 2007. The factored compressive load carrying capacity of the column,

$P_c = f_{cd} \times A_g$

Here, f_{cd} = design compressive stress, A_g = gross cross sectional area of the column.

3. **ANALYSIS OF STRUT:** To prevent the buckling of strut components between tack bolts, $\left(\frac{\ell_t}{r_{min}}\right)_{comp} \not> 40$

4. **DESIGN OF LACING:** If lacing members are used, then effective length of the column is increased by 5% in the calculations. Lacing members are designed as truss elements. The maximum slenderness ratio for lacing member is 145.

Angle of lacing with respect to vertical should not be less than 40° and more than 70°.

If $\theta > 70°$, force in the lacing member will be very high and it may buckle. If $\theta < 40°$, length of lacing member will be more and it may buckle.

To prevent buckling of the column component between lacing connection,

$\left(\frac{C}{r_{min}}\right) \not> 50 \not> 0.7 \times$ slenderness ratio of overall section

Where, C = overlap length $\not<$ 4t, r_{min} = minimum radius of gyration

According to IS 800 – 2007, the dimensions of the lacing bar are specified as follows

(i) Length of lacing bar (l_1): It is taken as the distance between inner welds or bolts.

(ii) Effective length of lacing bar (l_{eff}):

$l_{eff} = l_1$, for single lacing (with one or two bolts)

$l_{eff} = 0.7 l_1$, for double lacing.

$l_{eff} = 0.7 l_1$, for welded lacing.

(iii) Minimum thickness (t_{min}):

$t_{min} \not< \frac{l_{eff}}{40}$, for single lacing.

$t_{min} \not< \frac{l_{eff}}{60}$, for double lacing.

(iv) Minimum width (b_{min}): $b_{min} \not< 3\phi$

Where, ϕ = nominal diameter of bolt.

At the top and bottom, tie plates or batten plates are provided. They prevent distortion of built up column cross section due to twisting moment at the connection of the base plate with the column.

Force in the lacing system:

- The lacing shall be designed to resist transverse shear force, V = 2.5 % of column load (to take care of the eccentricity of axial loads and moments arising due to accidental lateral loads).
- The shear force, V is shared by parallel planes of lacing system equally. So transverse shear force on each lacing system is $\frac{V}{n}$. where, n is the number of parallel planes of lacing.
- If the column is subjected to bending also, V_t = bending shear + 2.5% column force.
- Laced members should not be subjected to eccentric loading because additional transverse shear force is developed in lacing system due to moment.

5. DESIGN OF BATTENED COLUMN: IS 800 – 2007 specifies the following rules for the design of battened column.

(i) Batten plates should be placed symmetrically.

(ii) At both ends batten plates should be provided.

(iii) The number of battens should be such that the member is divided into less than three bays. As far as possible, they should be spaced and proportioned uniformly through out.

(iv) Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded.

(v) By providing battens distance between the members of columns is so maintained that radius of gyration about the axis perpendicular to the plane of battens is not less than the radius of gyration about the axis parallel to the plane of the batten.

(vi) The effective slenderness ratio of battened columns shall be taken as 1.1 times the maximum actual slenderness ratio of the column, to account for shear deformation.

(vii) The vertical spacing of battens, measured as centre to centre of its end fastening, shall be such that the slenderness ratio of any component of the column over that distance shall be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its axis.

(viii) Battens shall be designed to carry the bending moment and shear forces arising from transverse shear force V_t equal to 2.5% of the total axial force.

(ix) In case columns are subjected to moment also, the resulting shear force should be found and then the design shear is sum of this shear and 2.5% of axial load.

(x) The design shear and moments for battens plates is given by

$V_b = \frac{V_t C}{NS}$ and $M = \frac{V_t C}{2N}$ at each connection here, V_t = transverse shear force as defined in 8 and 9.

Here,

C = distance between centre to centre of battens longitudinally.

N = Number of parallel planes.

S = minimum transverse distance between the centroid of the fasteners connecting batten to main

(xi) The effective depth of end battens (longitudinally), shall not be less than the distance between the centroids of main members.

(xii) The effective depth of intermediate battens shall not be less than $\frac{3}{4}$th of above distance.

(xiii) In no case the width of battens shall be less than twice the width of one member in the plane of the batten. It is to be noted that the effective depth of a batten shall be taken as the longitudinal distance between the outermost fasteners.

(xiv) The thickness of battens shall not be less than $\frac{1}{50}$ th of the distance between the inner most connecting lines of rivets, bolts or welds.

(xv) The length of the weld connecting batten plate to the member shall not be less than half the depth of batten plate. At least one third of the weld shall be placed at each end of this edge.

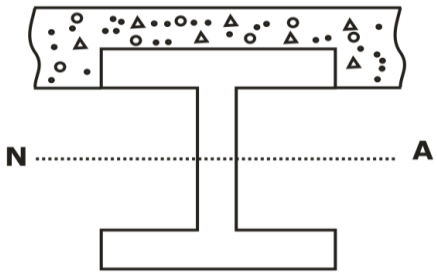
Beams

1. COMMON TYPES OF BEAMS

- Floor beam: A major beam of a floor system usually supporting joints in building.
- Girder: In buildings, girders are the same, as floor beams also a major beam in a structure.
- Grit: A horizontal member fastened to and spanning between peripheral columns of an industrial building to support wall cladding.
- Joist: A beam supporting floor construction but not major beams.
- Lintel: Beam member used to carry wall loads over wall openings for doors, windows etc.
- Purlin: A roof beam, usually supported by roof trusses.
- Rafter: A roof beam, usually supporting purlins.
- Spandrels: Exterior beams at floor level of buildings, which carry part of floor load and the exterior wall
- Stringers: Beam supporting stair steps (in case of buildings).
- Header: A beam at stair well openings.

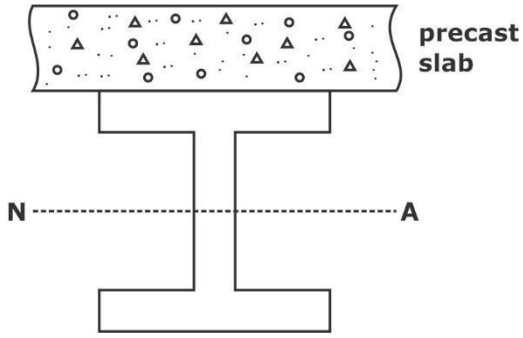
2. TYPES OF BEAMS BASED ON THE LATERAL SUPPORTS TO COMPRESSION FLANGES

(i) Laterally supported beam: If the compression flange of the beam is completely restrained against lateral movement, then it is called laterally supported beam.



Since moment of inertia about y-axis increases due to the slab, there is no possibility of the buckling of the compression flange. So, design bending compressive stress, f_{bd} is taken f_y(in LSM) or 0.66 f_y (in WSM).

(ii) Laterally unsupported beam: If the compression flange of the beam is not restrained against lateral movement then it is called laterally unsupported beam.



Since there is a possibility of buckling of the compression flange, design bending compressive stress is taken as $f_{bd} = X_{LT}\left(\frac{f_y}{1.1}\right)$.

Where, X_{LT} = reduction factor to take care of lateral torsional buckling of the beam.

3. CLASSIFICATION OF CROSS SECTIONS:

(i) Plastic section: It has the capacity to develop the plastic hinge and collapse mechanism

M_P = plastic moment capacity of cross section = $f_y \times Z_p$

M_y = Yield moment carrying capacity $= \frac{f_y}{1.1} z_P$

$M_o = f_{bd} \cdot z_p$

Where, $f_{bd} = \frac{f_y}{1.1}$ design stress

(ii) Compact section: It has the capacity to develop plastic hinge but does not have the capacity to form collapse mechanism.

$M_p = f_y \cdot z_p$

$M_u = f_{bd} \cdot z_p$

Here, $f_{bd} = \frac{f_y}{1.1}$

(iii) Semi-compact section: It has the capacity to develop yield moment only.

$M_y = f_y \cdot z$

$M_o = f_{bd} \cdot z$

(iv) Slender Section: These sections fail by buckling even before reaching yield stress.

$M_u = f_{bd} \cdot z$

4. **BENDING (FLEXURAL) STRENGTH:** Bending strength design of laterally supported beam is governed by yield stress and lateral or torsional buckling controls the design of laterally unsupported beams.

(a) Laterally supported beams:(For beam supported laterally against lateral -torsional buckling)

- Factored design moment at any section (M) ≤(Design bending strength of section M_d)
- When $d/t_w < 67\,\epsilon$ (No shear buckling in web)
- Nominal shear capacity (V_n) - Plastic shear strength of beam (V_p)
- Design shear strength $V_d = V_n/\gamma_{mo}$
- When $d/t_w > 67\,\epsilon$ (web of beam susceptible to shear buckling)

Case I: Low shear case (Factored design shear force $V \le 0.6\,V_d$)

Design bending strength $M_d = \beta_p Z_p f_y/\gamma_{mo}$

$\le 1.2\, Z_e f_y/\gamma_{mo}$ (For simply supported beams)

$\le 1.5\, Z_e f_y/\gamma_{mo}$ (For cantilever beams)

Where,

$\beta_b = 1.0$ (for plastic and compact sections)

$\beta_b = Z_e / Z_p$ (for semi compact sections)

β_b and Z_P elastic and plastic section modulus of the cross section

For slender sections:

$M_d = Z_e f'_y$ (f'_y Reduced design strength)

Case II: High shear case - (Factored design shear force $V > 0.6\,V_d$)

Design bending strength

$M_d = M_{dv}$ (M_{dv} - design bending strength under high shear)

For plastic or compact section

$$M_{dv} = M_d\beta(M_d - M_{fd}) \le \frac{1.2Z_e f_y}{\gamma_{mo}} \text{ where } \beta = \left(\frac{2V}{V_d} - 1\right)^2$$

M_d = Plastic design moment of the whole section neglecting high shear case and considering well buckling effect

V = Factored applied shear force

V_d = Design shear strength as governed by web yielding fort web buckling.

M_{fd} = Plastic design strength of area of cross section excluding shear area.

For Semi compact section

$M_{dv} = Z_e f_y/\gamma_{mo}$

(b) Laterally unsupported beams (For beam unsupported laterally against lateral torsional buckling)

- Beam with major axis bending and compression flange not restrained against lateral bending fail by lateral. Torsional buckling before attaining their bending strength
- The effect of lateral torsional buckling need not be considered when $\lambda_{LT} \le 0.4$ (where λ_{LT}- non dimensional effective slenderness ratio for lateral torsional buckling)

The bending strength of laterally unsupported beam is given by,

$M_d = \beta_b.Z_p.f_{cd}$

β_b= 1.0 (For plastic and compact sections)

= Z_e/Z_p (for semi compact sections)

Z_e = Elastic section modulus

Z_p = Plastic section modulus

f_{cd} = Design bending compressive stress= $X_{LT}.\frac{f_y}{\gamma_{mo}}$

X_{LT} = Bending stress reduction factor to account for lateral torsional buckling

$$X = \frac{1}{\phi_{LT} + \left((\phi_{LT}^2 - \lambda_{LT}^2)\right)^{0.5}} \leq 1.0$$

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

α_{LT} = Imperfection factor

= 0.21 (For rolled sections)

= 0.49 (Welded section)

λ_{LT} = Non dimensional slenderness ratio

$$= \sqrt{\beta_b Z_p.f_y / M_{cr}} \leq \sqrt{1.2Z_e.f_y / M_{cr}} = \sqrt{\frac{f_y}{f_{cr}.b}}$$

$$M_{cr} = \sqrt{\left(\frac{\pi^2 EI}{\lambda_{LT}^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{\lambda_{LT}^2}\right]}$$

= $\beta_b.Z_p.f_{cr.b}$

M_{cr} = The moment at which a beam fail by lateral buckling when subjected to uniform moment is called elastic critical moment.

5. SHEAR STRENGTH OF LATERALLY SUPPORTED BEAM

Design shear strength $V_d = \frac{A_v f_{yv}}{\sqrt{3}\gamma_{mo}}$

Where,

A_v = Shear area

f_{yv} = Yield strength of the web

The shear area may be calculated as given below :

For I and Channel Sections

(i) Major axis bending

Hot rolled – ht_w.

Welded –dt_w

(ii) Minor axis bending

Hot rolled or welded – $2bt_f$

For Rectangular Hollow Sections of Uniform Thickness

(i) Loaded parallel to depth (h) : $\frac{Ah}{(b+h)}$

(ii) Loaded parallel width (b) : $\frac{Ab}{(b+h)}$

(iii) Circular hollow tubes of uniform thickness – $\frac{2A}{\Pi}$

(iv) Plates and solid bars – A

Where,

A = Cross-section area:

b = Overall breadth of tubular section, breadth of I section flanges

d = Clear depth of web between flanges

h = Overall depth of the section

t_r = Thickness of the flange and

t_w = Thickness of the web.

6. DEFLECTION LIMIT

• Vertical deflection for simply supported span

Elastic cladding – span/240

Brittle cladding – span /300

• Vertical deflection for cantilever span

Elastic cladding – span/120

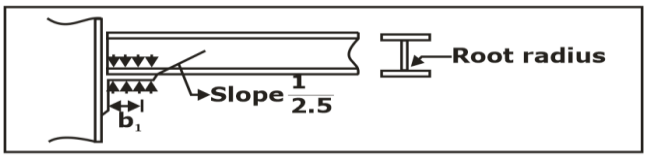
Brittle cladding – span /240

• Vertical deflection for purlins and grit

Elastic cladding – span/ 150

Brittle cladding – span/180

7. WEB CRIPPLING



The crippling occurs at the root of the radius. IS 800-2007 has accepted the following formula to find crippling of web: $F_w = (b_1 + n_c)t_w \times \frac{f_{yw}}{\gamma_{mo}}$

where,

b_1 = Stiff bearing length

n_c = Length obtained by dispersion through the flange to the web function at a slope 1 : 2.5 to the plane of flange .

$\Rightarrow n_c = 2.5\ t_f$

f_{yw} = Yield stress of the web

8. WEB BUCKLING

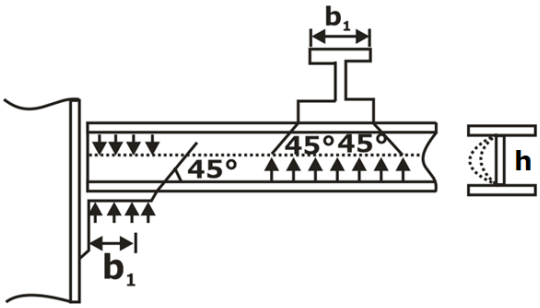
The web buckling strength of support will be

$F_{Wb} = B.t_w.f_{cd}$

f_{cd} = Allowable compressive stress

f_{cd} = Allowable compressive stress corresponding to the announced web strut according to buckling curve 'C

B = Length of steff portion of the bearing plus additional length is given dispersion 45° to the level of NA



Hence as per IS 800-2007. Effective web buckling is to be found based on the cross-section of web.

At support, $A = \left(b_1 + \frac{h}{2}\right)t_w$

Web buckling strength = $F_{cdw} = \left(b_1 + \frac{h}{2}\right)t_w f_c$

And f_c is the allowable compressive stress corresponding to the assumed web column of effective length = 0.7 d where, d is the web height.

At concentrated load

$$F_{cdw} = \left(b_1 + 2 + \frac{h}{2}\right)t_w f_c$$

$$F_{cdw} = (b_1 + h)t_w \times f_c$$

9. DESIGN OF BUILT UP BEAMS: If a single beam section could not withstand applied load, then we use built up beams. The factored moment carrying capacity can be expressed as

$$M_u = \left(\frac{f_y}{1.1}\right) \cdot z_P$$

$$(Z_P)_{required} = \frac{M_u}{\left(\frac{f_y}{1.1}\right)}$$

here, M_u = factored bending moment.

If cover plates are provided to increase moment carrying capacity, then

$$(Z_p)_{req} = (Z_p)_{beam} + (Z_p)_{plates}$$

$$Z_p = A_p \cdot d$$

Here, A_p = area of plate required on each side

$$A_p = \frac{(Z_P)_{req} - (Z_p)_{beam}}{d} \text{ (in LSM)}, \quad A_p = \frac{Z_{req} - Z_{beam}}{d} \text{ (in WSM)}$$

10. STEPS FOR BEAM DESIGN (LIMIT STATE DESIGN):

A. Design of laterally supported beams

1. Calculate the factored load and the maximum bending moment and shear force
2. Obtain the plastic section modulus required

$$Z_{req} = \frac{(M \times \gamma_{mo})}{fy}$$

Select a suitable section for the beam-ISLB, ISMB, ISWB or suitable built up sections (doubly symmetric only). (Doubly symmetric, singly symmetric and asymmetric- procedures are different)

3. Check for section classification such as plastic, compact, semi-compact or slender. Most of the sections are either plastic or compact. Flange and web criteria.

$$\frac{d}{t_w}, \frac{b}{t_f} \varepsilon = \sqrt{\frac{250}{fy} = 1}$$

4. Calculate the design shear for the web and is given by

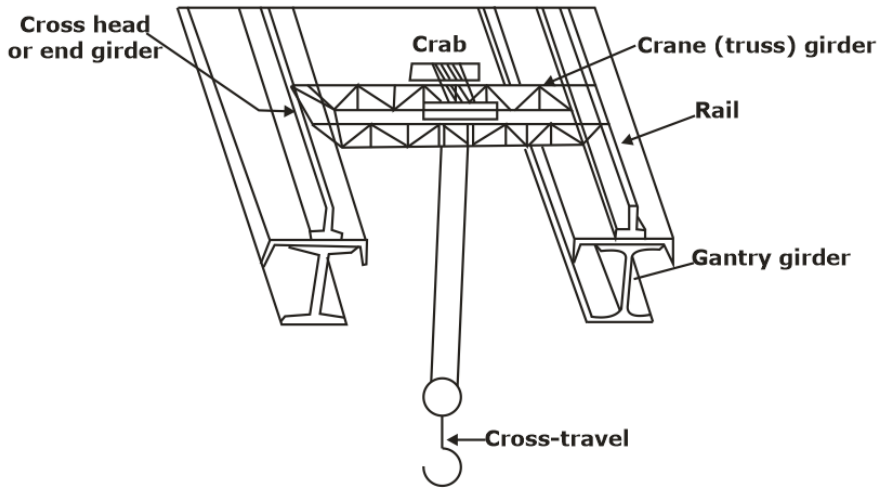
$$= V_{dp} = \frac{(Av \times fy)}{\sqrt{3} \times \gamma_{mo}} > V_d \text{ and } V < 0.6\ V_d$$

5. Calculate the design bending moment or moment resisted by the section (for plastic and compact) $M_d = \beta_p Z_P f_y / \gamma_{mo}$
6. Check for buckling
7. Check for crippling or bearing
8. Check for deflection

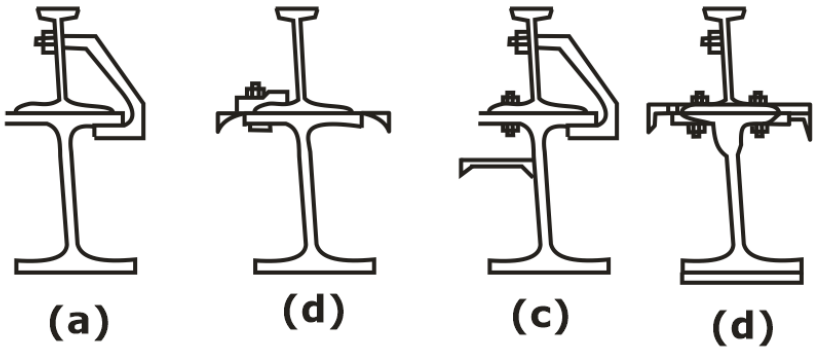
B. Design of laterally un-supported beams

1. Calculate the factored load and the maximum bending moment and shear force.
2. Design of ISB is by trial and error method. The design bending stress is significantly less which is to be assumed to start with. Assume slenderness ratio and Wu and get the corresponding critical bending stress and hence the corresponding design bending stress.
3. Determine the required plastic section modulus and select the section.
4. Determine the actual design bending stress of this selected section knowing in slenderness ratio which should be greater than that assumed previously. Otherwise revise the section.
5. Check for shear, buckling, crippling and deflection should be done Design bending strength can be calculated as per IS 800:2007.

11. Gantry Girder: A typical arrangement of a crane system is shown in Figure.



Different forms of gantry girder are as follows:

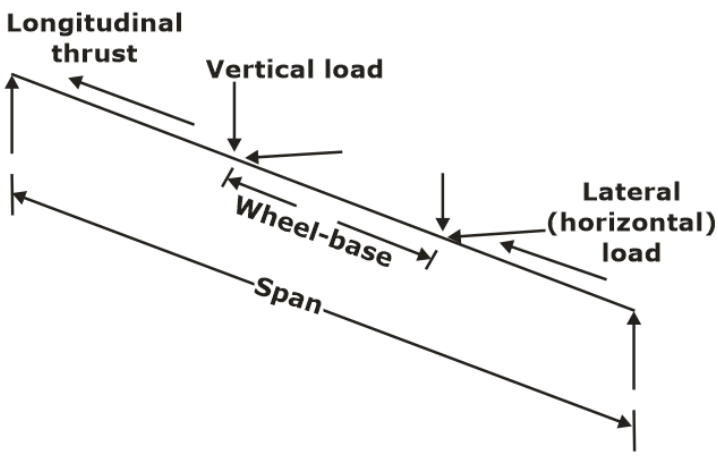


11.1. LOADS ON GANTRY GIRDER: These are subjected to three forces:

(i) The reaction from the crane girder, acting vertically downwards.

(ii) The longitudinal thrust, due to starting or stopping of crane, acting in the longitudinal direction.

(iii) The lateral thrust, due to starting or stopping of the crab acting horizontally, normal to the gantry girder.



Additional load on the structure due to Impact load:

Type of Load	Additional load
(a) Vertical forces transferred to the rails (i) For electrically overhead cranes (ii) For hand operated cranes	25% of maximum static wheel load 10% of maximum static wheel load
(b) Horizontal forces transverse to the rails (i) For electrically overhead cranes (ii) For hand operated cranes	10% of the weight of the crab and the weight lifted on the crane 5% of the weight of the crab and the weight lifted on the crane
(c) Horizontal forces along the rails	5% of the static wheel load

11.2. PERMISSIBLE DEFLECTION: The vertical deflection of a gantry girder should not exceed the values specified below:

(i) Where the cranes are manually operated - $\frac{L}{500}$

(ii) Where the cranes are travelling overhead and operated electrically up to 500 kN - $\frac{L}{750}$

(iii) Where the cranes are traveling overhead and operated electrically over 500 kN - $\frac{L}{1000}$

(iv) Other moving loads, such as charging cars, etc.- $\frac{L}{600}$

Where, L = span of gantry girder

12. PLATE GIRDER: It is a flexural member designed for bending which is used when I section are not sufficient to support the anticipated load.

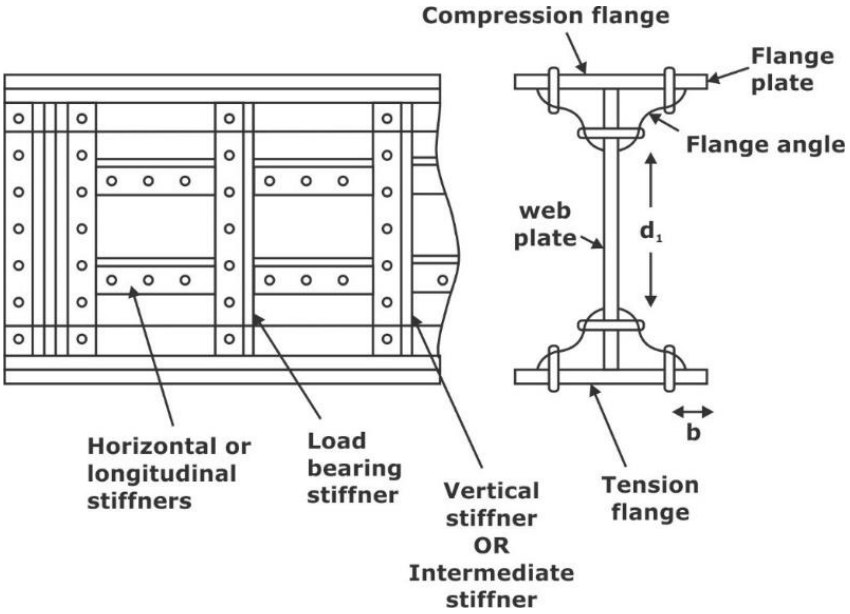
12.1. Elements of plate girder

(i) Web plate

(ii) Flange angles with or without flange cover plate

(iii) Stiffeners

(iv) Splices



12.2. General Considerations

- The flange angles must be unequal angles with longer legs connected to flange plates to get more bearing area.
- The size of flange angels should be such that, the should form at least $\frac{1}{3}$ of total flange area.
- The stiffeners are designed on the basis of following condition.

(i) If $\frac{d_1}{t_\omega} > 90$, vertical stiffeners are provided to prevent buckling of web due to diagonal compression. Vertical stiffeners are provided under point loads. Vertical stiffeners are provided under point loads. These stiffeners are called load bearing stiffeners. They prevent & web crippling or web crimpling or local buckling of web.

(ii) If $\frac{d_1}{t_\omega} > 200$, horizontal stiffeners are longitudinal stiffeners are provided above NA at a distance of 0.2 d_w (depth of web plate) from the compression flange. They prevent buckling of web due to bending compressive stress.

(iii) If $\frac{d_1}{t_w} > 250$, additional horizontal stiffeners are provided at neutral axis. These stiffeners prevent buckling of web between vertical stiffeners due to shear force.

(iv) If $\frac{d_1}{t_w} > 400$, then section must be redesigned.

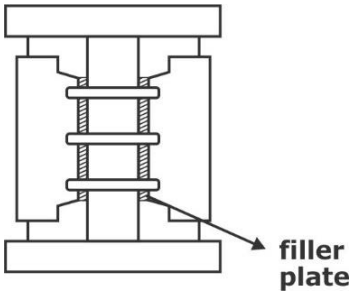
• At the supports, to prevent bending of flange plate and buckling of web plate due to support reaction and bearing stiffeners are provided. If the bearing stiffeners are the only means of providing torsional restraint, then they are also called torsional stiffeners. The other means of providing torsional restraint are extending the plate girder into the wall or using small piece of angle (web cleats).

• **Design of end bearing stiffeners**

End bearing stiffeners are designed as an imaginary column with both ends fixed whose effective length is $0.7l_1$.

Here,

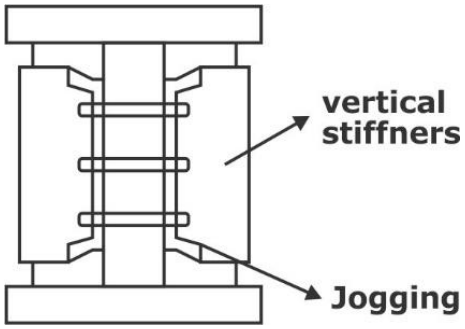
l_1 = length of bearing stiffener between flange angles.



Since bearing stiffeners are designed as columns they should be vertical and should not be jogged (jogging means bending i.e., they should not be bent to touch the web plate) and the gap between bearing stiffeners and web plate must be filled by filler plate. Since they act as a column, the end bearing stiffener should have a tight bearing between the flange angles.

• **Design of vertical stiffeners**

Vertical stiffeners are used to prevent buckling of web due to diagonal compression. These stiffeners are not designed as columns. So, the ends of stiffeners need not have tight bearing with the flange angle.



Since the vertical stiffeners are not designed as column they can be joggled (i.e. they can be bent to touch the web plate so that filler plates need not be provided)

Dimensions of vertical stiffeners are as follows:

(i) Spacing of vertical stiffeners (s)

$$s \not< 0.33\ d_1 \text{ and } s \not< 1.5\ d_1$$

here, d_1 = clear depth of web between toes of flange angles.

(ii) Lesser clear panel dimension $\not> 180\, t_w$

greater clear panel dimension $\not> 270\, t_w$

(iii) So, the spacing of vertical stiffeners increases, permissible shear stress in the web decreases due to buckling possibility of the web.

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