# Steel Structures

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# STEEL STRUCTURES

## INDEX

# 1 MATERIALS AND SPECIFICATIONS

## 1.1. Introduction

**Steel structure are built up with hot – rolled steel sections**

- Hot – rolled sections are made up of structural steel
- IS 800-2007: code of practice for use of structural steel in general building construction

**1. Types of structural steel:**

- IS 226 (Standard Quality)
- IS 2026 (Fusion welding Quality)
- IS 961 (High tensile steel)
- IS 1977 (Ordinary Quality)
- IS 8500 (Medium & high strength qualities)

**2. IS 226 (Standard Quality)**

- Most commonly used steel for general construction purposes of buildings, bridges, industrial structures, transmission line towers etc.
- Riveting, bolting can be done for all thickness but welding is permitted for thickness $\leq$ 20mm only
- Carbon content: 0.23 to 0.25%, Elongation: 23%
- Designated as: Fe 410 – S

**3. IS 2062: (Fusion Welding type):**

- Steel commonly used for general construction purpose, particularly suitable for structure subjected to dynamic loads and impact such as bridge decking, girders and crane girders.
- Designated as Fe 410 WA, Fe 410 WB, Fe 410 WC.
- Suitable for welding in all thickness.
- Carbon content (max) 0.20–0.25%, elongation: 23%

## 4. IS 961 (High tensile steel):

- Greater strength and atmospheric corrosion resistance
- Fe 570 – HT: For structure with fabrication by methods other than fusion Welding.
- Fe 540 – W – HT: For structures where fusion welding is involved.
- Carbon content $\approx$ 0.27% for Fe 570 – HT, Elongation = 20%

| Type of steel | Grade/ Classification | UTS (MPa) | Yield Strength (MPa) | | | Elongation Gauge $5.65\sqrt{S_0}$ |
|---|---|---|---|---|---|---|
| | | | Thickness (mm) | | | |
| | | | <20 | 20-40 | >40 | |
| Standard structure steel (Standard Quality steel IS 226 & Fusion welding Quality IS 2062) | E250 (Fe 410A) | 410 | 250 | 240 | 230 | 23 |
| | E250 (Fe 410B) | 410 | 250 | 240 | 230 | 23 |
| | E250 (Fe 410C) | 410 | 250 | 240 | 230 | 23 |
| | | | <16 | 16-40 | 41-63 | |
| Micro alloyed high strength steel IS 8500 | Fe 440 | 440 | 300 | 290 | 280 | 22 |
| | Fe 540 | 540 | 410 | 390 | 380 | 20 |
| | Fe 590 | 590 | 450 | 430 | 420 | 20 |

## 5. Physical Properties of structural steel:

| Physical Property | As per IS 800-2007 |
|---|---|
| Specific gravity | 7.85 |
| Unit mass of steel | $\rho_s = 7850 kg/m^3$ |
| Modulus of Elasticity | $E = 2 \times 10^5$ N/mm$^2$ |
| Modulus of Rigidity | $G = 0.769 \times 10^5$ N/mm$^2$ |
| Coefficient of thermal Expansion | $\propto = 12\times10–6/^{oC}$ |
| Poisson's ratio | $\mu = 0.30$ |

## 6. Various types of rolled structural steel section

- Rolled steel I-sections (beam section)
- Rolled steel channel sections
- Rolled steel Tee sections
- Rolled steel angle sections
- Rolled steel bars

- Rolled steel tubular sections
- Rolled steel flats
- Rolled steel plates
- Rolled steel sheets etc.

**I — sections:**

- Indian standard junior beam (ISJB)
- Indian standard light beam (ISLB)
- Indian standard medium weight beam (ISMS)
- Indian standard wide flange beam (ISWB)
- Indian standard heavy beam (ISHB)
- Indian column section (SC)
- An I section is designated by its depth and weight

**Eg:** An ISLB 500 at 735.75 N/m means, An I-section is 500 mm deep and self weight is 735.8 N per meter length.

**Channel Sections:**

- Indian standard junior channel (ISJC)
- Indian standard light channel (ISLC)
- Indian standard medium weight channel with sloping flange (MC)
- Indian standard medium weight channel with parallel flange (MCP)
- Indian standard gate channel (ISGC)
- Designated by its depth and weight

**Eg:** ISLC 350 at 380.63 N/m

**T Sections:**

- Indian standard rolled normal T – section (ISNT)
- Indian standard rolled deep legged T – (ISDT)
- Indian standard rolled slit light weight T – bars (ISLT)
- Indian standard rolled silt medium weight T – bars (ISMT)
- Indian standard rolled silt T – bars from H – section (ISHT)
- Designation ISNT 125 at 274 N/m

**Angle sections:**

- Indian standard equal angles, Indian standard unequal angles and Indian standard bulb angles
- Designated by abbreviation ISA along with lengths of both legs and thickness.

**Eg:** ISA 75x 75 x 6 mm

## 1.2. Introduction to Limit State

### 1. Analysis and Design

- Analysis refers to the determination of the axial forces, shear forces, bending moments, torsional moments etc acting on different members of a structure due to the applied loads and load combinations
- Design involves the selection of shape and size of the member and connection details of various members (beam to beam, beam to column, column to foundation etc) to resist all forces and moments determined in the analysis safely and economically

### 2. Design requirements of steel structure

- To fit for their purpose (Should sustain all anticipate loads expected on it and Should withstand all deformations during and after construction)
- Should be safe should be economical and durable

### 3. Uncertainties in design

The uncertainties affecting the safety of a structure are due to

- Uncertainty about loading (unfavorable deviation of the load from its characteristic value. Inaccurate assessment of the load, improper assessment of load effect etc)
- Uncertainty about material strength (Unfavorable deviation of material strength from its characteristic value)
- Uncertainty about structural dimensions (variation member sizes) due to fabrication tolerances.
- Uncertainty in the calculation of strength of the member

### 4. Limit States

- Its acceptable limit for the safety and serviceability requirement before failure occurs is called limit state.
- Limit states are basically two categories, strength and serviceability.
- It is basically statistical method have been used for determination of load and material properties with small probability of structure reaching limit state of strength and serviceability.

### 5. Types of limit states

| Limit State of strength | Serviceability Limit State |
|---|---|
| • Strength (yield, buckling)<br>• Stability against overturning and sway<br>• Fracture due to fatigue<br>• Plastic collapse<br>• Brittle Fracture | • Deflection<br>• Vibration<br>• Fatigue checks (including reparable damage due to fatigue)<br>• Corrosion<br>• Fire |

## 6. Characteristic Strength or Resistance

The characteristic resistance or strength of a material (such as steel) is defined as that value of resistance below which not more than a 5 percentage of test results may be expected to fall.

## 7. Characteristic Load

The characteristic load is that value of the load, which has an accepted probability (95 %) of not being exceeded during the life span of the structure.

8. The safety format used in Limit State Codes is based on probable maximum load and probable minimum strengths, so that a consistent level of safety is achieved.

9. The design requirements are expressed as follows:

$$\text{Design Action } (S_d) \leq \text{Design Strength } (R_d)$$

$S_d$ = Design value of internal forces and moments caused by the design Loads, $F_d$

$F_d$ = $\gamma_f$ x Characteristic Loads.

$\gamma_f$ = a load factor which is determined on Probabilistic basis

$\gamma_m$ = a material factor, which is also determined on a 'probabilistic basis' (uncertainties of element behavior and possible strength reduction due to manufacturing tolerances and imperfections in the material)

## 10. Partial safety factors (Load factors)

| Combination | Limit State of Strength | | | | | Limit state of Serviceability | | | |
|---|---|---|---|---|---|---|---|---|---|
| | DL | LL | | WU EL | AL | DL | LL | | LL |
| | | Leading | Accompanying (CL, SL etc.) | | | | Leading | Accompa-flying (CL etc.) | |
| **DL+LL+CL** | 1.5 | 1.5 | 1.05 | – | | 1.0 | 1.0 | 1.0 | – |
| **DL+LL+CL+WL/EL** | 1.2<br>1.2 | 1.2<br>1.2 | 1.05<br>0.53 | 0.6<br>1.2 | – | 1.0 | 0.8 | 0.8 | 0.8 |
| **DL+WL/EL** | 1.5<br>(0.9) | – | – | 1.5 | – | 1.0 | – | – | 1.0 |
| **DL+ER** | 1.2<br>(0.9) | 1.2 | – | – | – | – | – | – | – |
| **DL+LL+AL** | 1.0 | 0.35 | 0.35 | - | 1.0 | | | | |

*This value is to be considered when stability against overturning or stress reversal is critical Abbreviations:

| | | |
|---|---|---|
| DL= Dead Load, | LL= Imposed Load (Live Loads), | WL= Wind Load, |
| SL= Snow Load, | CL= Crane Load (Vertical/Horizontal), | AL= Accidental Load, |
| ER= Erection Load | EL= Earthquake Load. | |

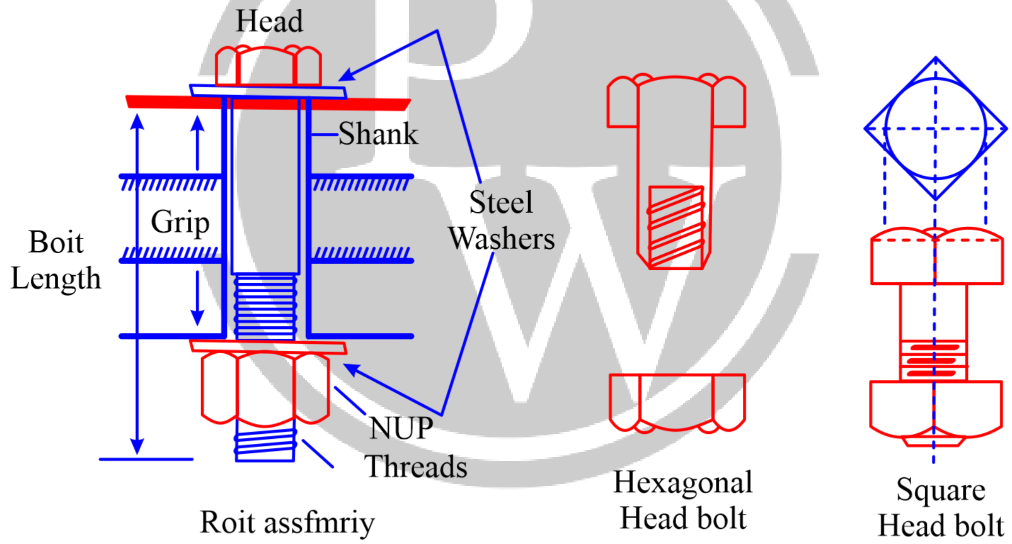
## 11. Partial safety factors (Strength or resistance factors)

| SI. No. | Definition | Partial Safety Factor | |
|---|---|---|---|
| 1 | Resistance, governed by yielding $\gamma_{mo}$ | 1.10 | |
| 2 | Resistance of member to buckling $\gamma_{mo}$ | 1.10 | |
| 3 | Resistance, governed by ultimate stress $\gamma_{ml}$ | 1.25 | |
| 4 | Resistance of connection $\gamma_{ml}$ | Shop Fabrications | Field Fabrications |
| | (i) Bolts-Friction Type, $\gamma_{mf}$ | 1.25 | 1.25 |
| | (ii) Bolts-Bearing Type, $\gamma_{mb}$ | 1.25 | 1.25 |
| | (iii) Rivets, $\gamma_{mr}$ | 1.25 | 1.25 |
| | (iv) Welds, $\gamma_{mw}$ | 1.25 | 1.50 |

# 2 BOLTED CONNECTIONS

## 2.1. Introduction

- A bolt may defined as metal pin with head at one end as shank threaded at other end to receive a nut. Steel washers are usually provided under bolt as well as under the nut to distribute the clamping pressure on the bolted member and to prevent the threaded portion of the bolt from bearing on the connected pieces.
- Bolts can be used for making end connection in tension and compression members. They can also be used to hold column bases in position.

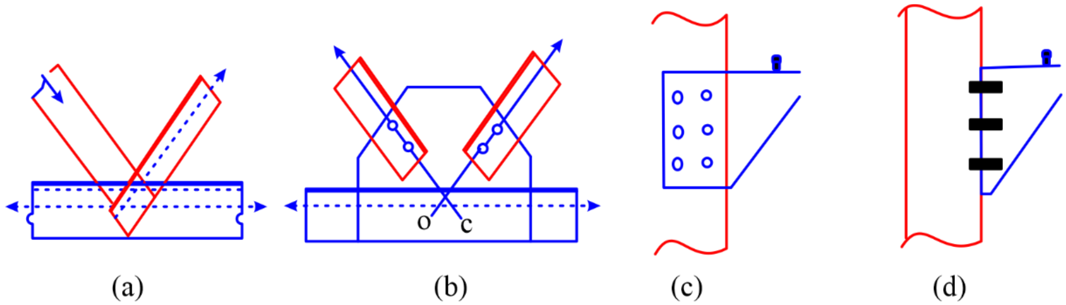



## 2.2. Classification of Bolted Connection

The bolted connections are classified based on geometry and loading conditions into three types namely

**(a) Classification based on the type of resultant force transferred:**

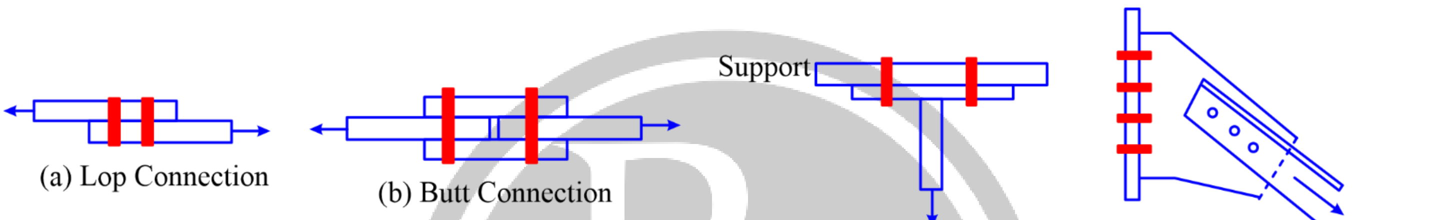
- Concentric connections (force transfer in tension and Compression member),
- Eccentric connections (in reaction transferring brackets) or moment resisting connections (in beam to column connections in frames).




Concentric Connection          Moment Connection

**(b) Classification based on the type of force experienced by the bolts:**

- Shear connections
- Tension connections and
- Combined shear and tension Connections




(a) Lop Connection

(b) Butt Connection

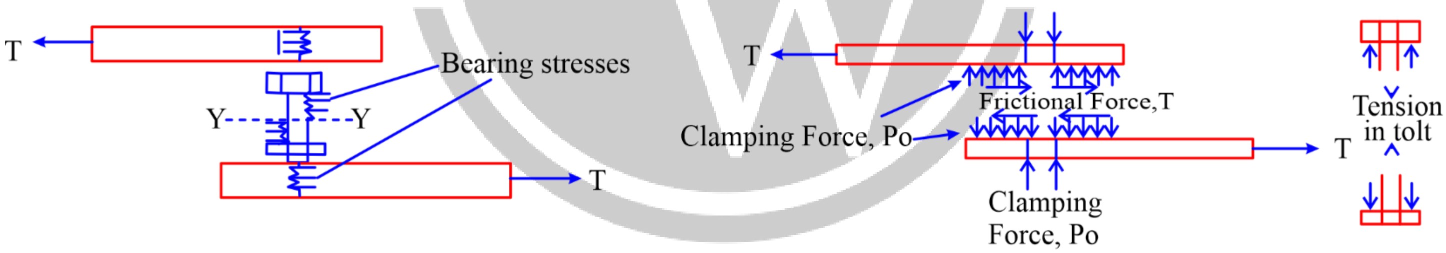
Shear Connection

Tension Connection

Tension plus shear Connection

**(c) Classification based on force transfer mechanism by bolts:**

- Bearing type (bolts bear against the holes to transfer the force)
- Friction type (force transfer between the plates due to the clamping force generated by the pre-tensioning of the bolts).




Bearing Connection

Tension Connection

## 2.3. Types of Bolts

They are several types of bolts used to connect structural members. Some of them are listed below:

**(a) Black Bolts or unfinished bolts**

- Black bolts are referred to as ordinary, rough or common bolts. They are least expensive bolts and are made of low carbon steels (mild steel) with square or hexagonal head. the diameter of the hole is about 1.5 to 2.0 mm larger than the bolt diameter for ease in fitting.
- They are primarily used in light structures under static loads such as small trusses, purlins, bracings. They are also used as temporary fasteners during erection where HSFG bolts or welding are used as permanent fasteners.

- These bolts are not recommended for connections subjected to impact load, vibration and fatigue.
- For bolt of a property class 4.6 represents the ultimate tensile strength is 400 N/mm2 and yield strength is 0.6 times 400 which is 240 N/$mm^2$.
- Ordinary bolted joints, the force transfer through interlocking and bearing of bolts and joint is called bearing type joint.

### (b) High Strength Friction Grip (HSFG) Bolts.

- High strength friction grip bolts are made from bars of medium carbon heat treated steel (high tensile steel). The bolt property class 10.9S and 12.9S are commonly used in steel connections.
- The HSFG bolts are available in sizes from 16mm to 36mm and are designated as M16, M20, M24 and M30.
- These bolts tightened (by torque wrenches) until they have very high tensile stresses, so that connected parts are clamped tightly together between the bolt head and nut, this permits load to be transferred
- primarily by friction not by shear.
- These bolts are most suitable for bridges where the stress reversal may occur or slippage is undesirable also for seismic loading and for fatigue load are ideal.
- High strength bolts have replaced rivets and black bolts are being used in structures, high raised building, bridges etc

## 2.4. Types of bolted joints

They are two types of bolted joints subjected to axial force (the loads are assumed to pass through the C.G of the group of bolts)

### (a) Lap joint

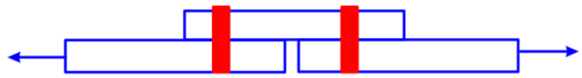
- The two members to be connected are overlapped and connected together such a joint is called lap joint.
- The load in a lap joint has eccentricity, as the centre of gravity of load in one member and centre of gravity of load in second member are not in a same line, therefore a couple formed which causes undesirable bending.

Double bolted lap joint

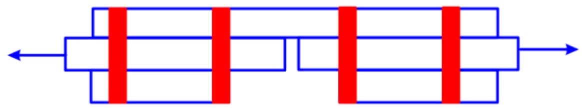
### (b) Butt joint

- Two members to be connected placed end to end. Additional cover plate/plates are provided on either one or both sides, called cover plates are placed and connected to main plate.
- If cover plate is provided on one side it is called single cover butt joint.
- If cover plate is provided on both sides of main plate, it is called a double cover butt joint.
- Double cover butt joint, eccentricity of a force doesn't exist and hence bending is eliminated, where as it exists in the case of lap joint.

- The shear capacity of bolt in double cover butt joint is double that of a bolt in a lap joint.



Single cover butt joint (Single bolted)



Double cover butt joint (Double bolted)
butt joint

## 2.5. Specifications of bolted joint

### (a) Diameter of bolt holes ($d_0$)

$d_0$ = Nominal diameter of bolt (d) + 1mm (for d = 12mm to 14mm)

$d_0$ = Nominal diameter of bolt (d) + 2mm (for d = 16mm to 24mm)

$d_0$ = Nominal diameter of bolt (d) + 3mm (for d ≥ 27 mm)

### (b) Pitch (P)

It is distance between centers of two consecutive bolts measured along parallel to direction of force or stress in a member. For wide plates pitch may also defined as the centre to centre distance of bolts measured along the length of member or the connection. When bolts are placed staggered the pitch will be referred to as staggered pitch

(i) Minimum pitch (P,,,i.)

2.5 × Nominal diameter of bolt (2.5d)

(ii) Maximum pitch (Pmax)

- 12t or 200mm whichever is less for compression member
- 16 t or 200mm whichever is less for tension member
- The distance between the centers of any two consecutive bolts should not exceed 32 t or 300mm whichever is less
- Maximum gauge should not more than 100 + 4t or 200 mm whichever is less
- 32t or 300mm whichever is less for tacking or stitch bolts (when plates are not exposed to weather)
- 16t or 200mm whichever is less for tacking or stitch bolts (when plates are exposed to weather)
- In case of two flats, angles, channels or tee section maximum pitch of tack bolts (In which tack or stitch bolts are to be provided along length to connect each of them)

- Not exceeding 600mm for compression members
- Not exceeding 1000mm for tension members.

### Gauge (g)

It is distance between adjacent bolt lines, or it is centre to centre distance between two consecutive bolts measured along the width of member or connection

### End distance(e)

It is the distance from centre of bolt hole to the nearest edge of member or cover plate in the direction of stress or force.

### Edge distance

It is the distance from centre of bolt hole to the nearest edge of member or cover plate at right angle to the direction of stress.

#### (i) Minimum edge distance

- Minimum edge distance ≈ 1.7 x diameter of hole for sheared or hand flame cut edges
- Minimum edge distance 1.5 x hole diameter incase of rolled, machine flame cut edges

#### (ii) Maximum edge distance

- Maximum edge distance to nearest edge of bolt hole to an edge of un stiffened part should not exceed

$$12t\varepsilon \text{ where } \varepsilon = \sqrt{\frac{250}{f_y}} \text{ where } t \text{ is}$$

thickness of thinner outside plate

- 40mm + 4t, where t is thickness of thinner outside plate (for corrosive Environments)

## 2.6. Failure of Bolted Connections

The failure of connections with bearing bolts in shear involves either bolt failure or the failure of the connected plates.

- Shearing failure of bolt
- Bearing failure of bolt
- Tension failure of bolt
- Bearing failure of plate
- Tearing failure of plate
- Block shear failure

## 2.7. Design strength of bearing type Bolts against Factored Shear Force

### a) Design shear strength of bolts ($V_{dsb}$)

- A bolt subjected to a factored shear force

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

$V_{nsb}$ = nominal shear capacity of a bolt and

$\gamma_{mb}$ = 1.25

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} b(n_n A_{nb} + n_s A_{sb}) \text{and} V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

Where $f$ub = ultimate tensile strength of the bolt

$n_n$ = number of shear planes with threads intercepting the shear plane

n, = number of shear planes without threads intercepting the shear plane

$A_{sb}$ = nominal plain shank area of the bolt

= $\pi d^2/4$

$A_{nb}$ = net tensile area at threads, (area corresponding to root diameter at the thread ≈ 0.78 $A_{sb}$)

For bolts in single shear, either $n_n$ or $n_s$, is one

For bolts in double shear the sum of $n_n$ and $n_s$, is two.

The nominal shear capacity of bolt for long joint is lesser and modified as

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

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb}) \beta_{lj} \beta_{lg} \beta_{pkg}$$

$\beta_{lj}$ = reduction factor for long joints,

In long joints, the distance between the first and the last bolt exceeding 15d in the direction of load, the nominal shear capacity $V_{nsb}$, shall be reduced by the factor, $\beta_{lj}$

$$\beta_{lj} = 1.075 \frac{l_i}{200d} \qquad (0.75 \le \beta_{li} \le 1.0)$$

$l_j$ is the length joints,

$\beta_{lg}$ is reduction factor for long grip lengths,

When grip length of bolts increases (if the grip length exceeds five times the nominal diameter), the bolt subjected to greater bending moment due to shear force acting on its shank

$$\beta_{lj} = \frac{8_d}{3d + l_g}$$

$1_g$ = grip length ($l_g$ should not greater than 8d)

$\beta_{pkg}$ = reduction factor for packing plates

When packing thickness is more than 6inm thick the shank of the bolts is subjected to bending which affects the nominal shear capacity of the bolt

$$\beta_{pkg} = 1.0 - 0.0125\, t_{pkg}$$

### (b) Design bearing strength of bolts and Plate ($V_{dpb}$)

A bolt bearing on any plate subjected to a factored shear force

$$V_{dpd} = \frac{V_{npd}}{\gamma_{mb}} Where\, \gamma_{mb} = 1.25$$

$V_{dpb}$ = bearing strength of a bolt,

$V_{dpb}$ = 2.5$k_b$ d t $f_u$

Where, $$k_b = smaller\ of\ \frac{e}{3d_0}, \left(\frac{p}{3d_0} - 0.25\right), \frac{f_{ub}}{f_u}\ and\ 01.0$$

$d_o$ = diameter of the bolt hole

$e$ & $p$ = end and pitch distances of the fastener respectively along bearing direction

$f_{ub}$ = ultimate tensile stress of the bolt

$f_u$ = smaller of the ultimate tensile stress of the bolt and the ultimate tensile stress of the plate

$d$ = nominal diameter of the bolt in mm

$t$ = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction.

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

### (c) Design Tensile strength of bolts (Tdb)

The nominal tensile capacity of bolt in tension is given by

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

$f_{ub}$ = ultimate tensile stress of the bolt

$f_{yb}$ = yield stress of the bolt

The bolt safe in tension if the factored tension force

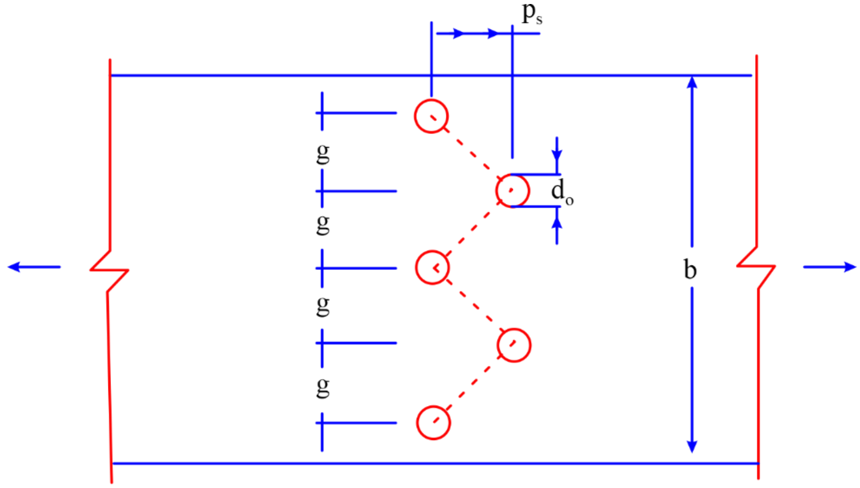
$$T_{nb} = \frac{T_{nb}}{\gamma_{mb}} where\, \gamma_{mb} = 1.25\, and\, \gamma_{mo} = 1.10$$

$$T_{nb} = 0.9 \frac{f_{ub} A_{nb}}{\gamma_{mb}} \leq \frac{f_{yd} A_{sb}}{\gamma_{mo}}$$

### (d) Design bolt strength or Design bolt value (Vdb)

It is least value of design strength of both in shear, design strength of both in bearing and design strength of bolt in tension (if exists).

### (f) Design Tensile strength of plate (Tdp)




Plates with Bolts Holes in Tension

The tensile strength of plate is given by

$$\text{Tensile strength of plate} = T_{dp} = 0.9 A_n \frac{f_u}{\gamma_{ml}}$$

Net area = $A_n$= $(b-nd_0)t$ for chain bolting

### (g) Design strength of a connection ($V_{dc}$)

The strength of a bolted connection is the minimum design strength based on strength of bolt in shear, in bearing, in tension (if exists) and minimum design strength of connected member against gross section yielding or net section rupture.

### Efficiency of the joint or percentage strength of joint (η)

Efficiency of the bolted joint (η) also called percentage strength of the joint is the ratio of design strength of joint to the design strength of main member expressed as percentage.

❑❑❑

# 3 WELDED CONNECTIONS

## 3.1. Introduction

### 1. Welding and Welded connections

- Welding is the process of joining two pieces of metal by creating a strong metallurgic bond between them by heating (fusion) or pressure or both. It is distinguished from other forms of mechanical connections, such as riveting or bolting, which are formed by friction or mechanical connections, such as riveting or bolting, which are formed by friction or mechanical interlocking.
- For steel structures metal or electric welding is generally used.

### 2. Advantages and disadvantages of welded connections

Welding offers many advantages over bolting and riveting.

- Welding enables direct transfer of stress between members eliminating gusset and splice plates necessary for bolted structures. Hence, the weight of the joint is minimum.
- In the case of tension members, the absence of holes improves the strength and efficiency of the section.
- It involves less fabrication cost compared to other methods due to handling of fewer parts and elimination of operations like drilling, punching etc. and consequently less labour leading to economy.
- Welding offers air tight and water tight joining and hence is ideal for oil storage tanks, ships etc.
- Welded structures also have a neat appearance
- Welded structures arc more rigid compared to structures with riveted and bolted connections.
- A truly continuous structure is formed by the process of fusing the members together.
- Generally welded joints are as strong or stronger than the base metal, thereby placing no restriction on the joints.
- Stress concentration effect is also considerably less in a welded connection.
- Some of the disadvantages of welding are that it requires skilled manpower for welding as well as inspection.
- Also, non-destructive evaluation may have to be carried out to detect defects in welds.
- Welding in the field may be difficult due to the location or environment. Welded joints are highly prone to cracking under fatigue loading.
- Large residual stresses and distortion are developed in welded connections.

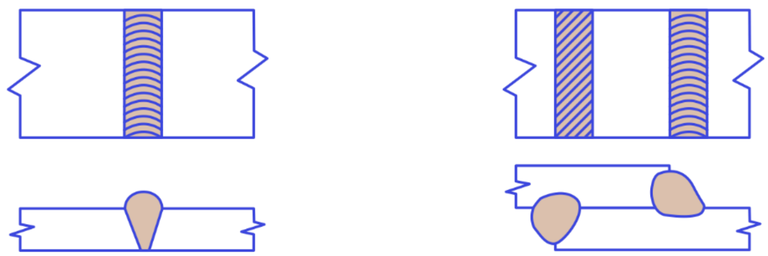
## 3. Types of welds

(a) Butt or Groove weld

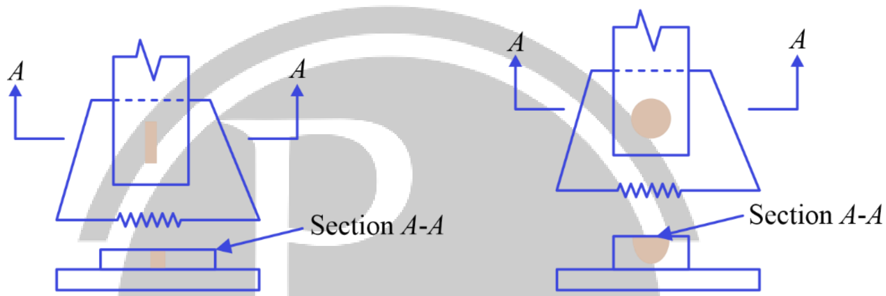
(b) Fillet or lap weld

(c) Slot weld

(d) Plug weld etc



(a) Groove welds (b) Fillet welds

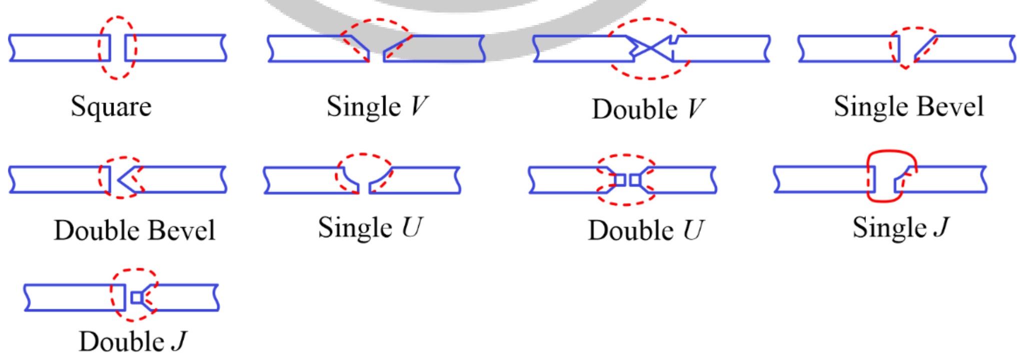



(c) Slot weld (d) Plug welds

## 4. Design of Butt (Groove) weld

### (a) Types of butt weld

Square butt weld, Single V butt weld, Double V butt weld, Double V butt weld, Single Bevel butt weld, Double bevel butt weld, Single U butt weld, Double U butt weld, Single J butt weld and Double J butt weld etc.




Types of Butt welds

### (b) Size of but t weld ($t_e$)

- The size of butt weld is specified by throat dimension and also called effective throat thickness ($t_c$)
- $t_e$ = as 5/8$^{th}$ thickness of thinner member in case of single $V$, Single $U$ and single bevel bat t joint (i.e for partially penetrated butt welds).

- $t_c$ = Thickness of thinner member in case of Double $V$, Double $U$ and Double bevel out joints (i.e for fully penetrated butt welds)

**(c) Effective Area**

- The effective area of butt weld is the product of effective throat thickness and the effective length of the butt weld.

**(d) Design strength of butt weld**

- **Design axial strength of butt weld**

The design strength of butt weld in tension or compression is governed by yield

$$T_{dw} = \frac{f_y L_w t_e}{\gamma_{mw}}$$

$f_y$ = smaller of yield stress of the weld ($f_{yw}$) and parent metal ($f_y$) in Mpa

$L_w$ = Effective length of weld in mm

$t_e$ = Effective throat thickness of weld in mm

$\gamma_{mw}$ = Partial safety factor [$\gamma_{mw}$ = 1.25 for shop welding and

$\gamma_{mw}$ = 1.50 for site (field) welding]

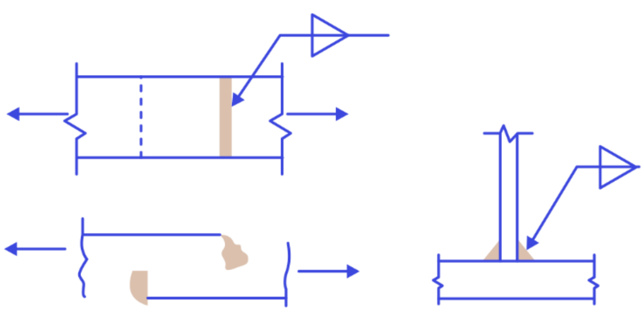
- **Design shear strength of butt weld**

The design strength of butt weld in shear is governed by yield

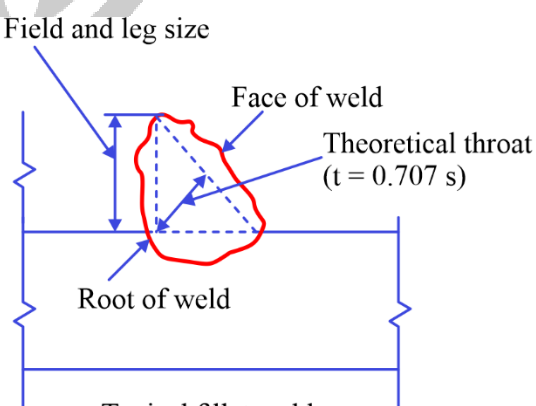
$$V_{dw} = \frac{f_{yw1} L_w t_e}{\gamma_{mw}}$$

$f_{ywl}$ = smaller of shear stress of the weld $\left(f_{yw}/\sqrt{3}\right)$ and parent metal $\left(f_y/\sqrt{3}\right)$ in Mpa

## 5. Design of Fillet weld



Typical fillet weld connections




Typical fillet weld

**(a) Size of fillet weld (s):** it is minimum leg length of cross section of fillet weld (in it distance from root to the toe of the fillet weld)

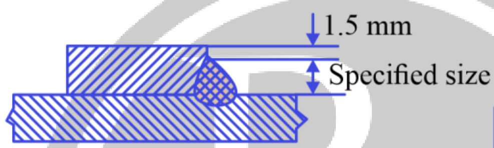
**(b) Minimum size of fillet weld ($s_{min}$)**

- The size of a fillet weld should not be less than 3 mm or more than the thickness of the thinner part joined

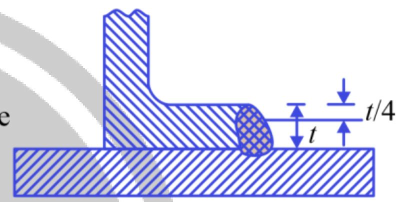
| Thickness of thicker part | | Minimum size of fillet weld s (mm) |
|---|---|---|
| **Over (mm)** | **Up to and including (mm)** | |
| – | 10 | 3 |
| 10 | 20 | 5 |
| 20 | 32 | 6 |
| 32 | 50 | 8 (First run) & 10 |

### (c) Maximum size of fillet weld ($s_{max}$)

- fillet weld is provided to square edges, the weld size should be at least 1.5 mm less than the edge thickness (i.e Thickness of thinner member – 1.5 mm)
- for the rounded toe of a rolled section, the weld size should not exceed ¾ thickness of the section at the toe (i.e. = $3/4^{th}$ thickness of rolled section at toe)




Fillet welds on square edge of plate




Fillet welds on round toe of rolled section

### (d) Effective throat thickness ($t_t$)

- It is perpendicular distance from right angle corner of fillet weld to the hypotenuse.
- Minimum throat thickness of fillet weld not less than 3 mm
- $T_t$ = K × Size of the weld (K = constant depends on angle between fusion faces
- Fillet welds are normally used for connecting parts whose fusion faces form angles between 60° and 120°.

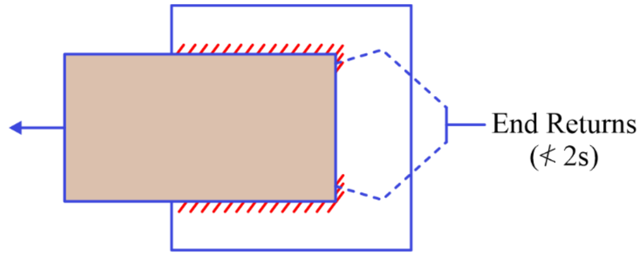
| Angle between fusion faces α | 60° – 90° | 91° – 100° | 101° – 106° | 107° – 113° | 114° – 120° |
|---|---|---|---|---|---|
| **Constant K** | 0.70 | 0.65 | 0.60 | 0.55 | 0.50 |

### (e) Effective length of fillet weld ($L_w$)

- It is actual length shown on the drawing
- $L_w$ = Overall length of weld (L) – 2 × size of fillet weld (2s)
- Minimum effective length not less than four times size of weld (4 × s) or 40 mm whichever is higher

### (f) End Return

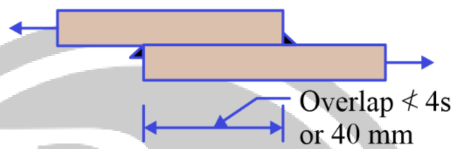
- The fillet weld terminating at the end or side of the member should be returned around the corner when over practicable for a distance not less than twice the weld size as shown in figure




- End returns are made twice the size of the weld to relive the high stress concentration at the ends
- End returns must be provided for welded joints, which are subjected to eccentricity, stress reversals or impact loads. This particularly important on tension end of parts carrying bending loads

### (g) Overlap

- The overlap of plates to be welded in lap joint should not less than four times thickness of thinner member (4t) or 40 mm whichever is more.




### (h) Intermediate fillet weld

Intermediate fillet weld: when length of fillet weld required transmitting a force less than the continuous fillet weld (where t is thickness of thinner plate)

- Clear spacing between intermediate filler weld should not more than
  - 12t or 200 mm whichever is less for compression member
  - 16t or 200 mm whichever is less for tension member

### (i) Design strength of fillet weld ($P_{dw}$)

The design shear of fillet weld $f_{wd} = f_{wn}/\gamma_{mw}$, $f_{wn}$ = Nominal shear strength of fillet weld = $f_u/\sqrt{3}$

The design strength of a fillet weld (based on the throat area)

$$P_{dw} = \frac{L_w t_t f_u}{\sqrt{3}\gamma_{mw}} = \frac{L_w (K.s) f_u}{\sqrt{3}\gamma_{mw}}$$

$L_w$ = Effective length of fillet weld in mm

$t_t$ = (K s) Effective throat thickness in mm

$s$ = size of weld in mm

$f_u$ = Smaller of ultimate strength of weld and parent metal in Mpa

$\gamma_{mw}$ = Partial safety factor

[$\gamma_{mw}$ = 1.25 for shop welding and $\gamma_{mw}$ = 1.50 for site (field) welding]

### (j) Reduction factor for long Joint ($B_{lw}$)

- If the maximum length $l_f$ of the side welds transferring shear along its length exceeds 150 times the throat size of the weld, $t_t$, the reduction in weld strength as per the long joint.

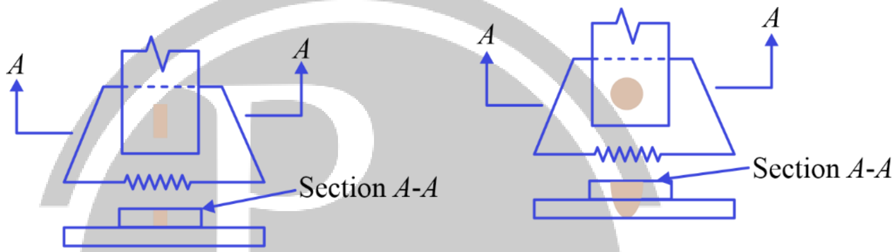
- The design capacity of weld ($f_{wd}$) reduced by a factor $\beta_{lw} = 1.2 - \dfrac{0.2 l_j}{150 t_t} \leq 1.0$

$l_j$ = Length of joint in the direction of force transfer

$t_t$ = throat size of the weld

### (k) Design of Plug (or) slot Welds

- Plug and slot welds are used most often to tie two parts together and in particular to reduce the unsupported dimensions of cover plates in compression. There may also be used for shear transmission.
- Plug and slot welds are used along with fillet weld, when sufficient welding length is not available along the edges of the members.
- A slot is cut in one of overlapping members and having welding metal is filled in slot if the slot is small and completely filled with weld metal. It is known as plug weld
- The following specifications are for design of plug on slot weld as per IS 816 — 1969.




Slot weld　　　　Plug weld

(a) The width and diameter of slot should not be less than three times the thickness of part which slot is formed or 25 mm which ever is greater.

(b) Corner at enclosed ends should be rounded with a radius not less than 1.5 times the thickness of upper plate or 12 mm which ever is greater.

(c) The distance between the edges of the plates and slot between edges of adjacent slot should not be less than twice the thickness of the upper plate.

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# 4 ECCENTRIC CONNECTIONS

## 4.1. Eccentric Bolted Connection

### (1) Introduction

Connections become complex when they have to transmit axial and shear forces in addition to bending moments or twisting moments, between structural members oriented in different directions.

### (2) Concentric load

A load is said to be concentric load, when its line of action passes through centre of gravity of bolt group or rivet group or weld group

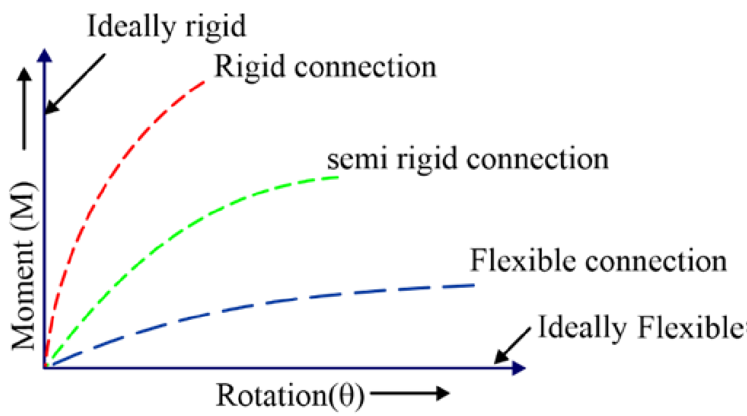
### (3) Eccentric load

A load is called eccentric load when its line of action does not pass through centre of gravity of bolt or rivet or weld group. Because of eccentricity additional moment is induced in the joint.

### (4) Beam to Column Connections

Beam to column connections can be classified as simple, semi-rigid and rigid depending on the amount of moment transfer taking place between the beam to the column.

- Simple connections are assumed to transfer only shear at some nominal eccentricity. Therefore such connections can be used only in non-sway frames where the lateral loads are resisted by some alternative arrangement such as bracings or shear walls.
- Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations. Rigid connections are necessary in sway frames for stability and also contribute in resisting lateral loads.



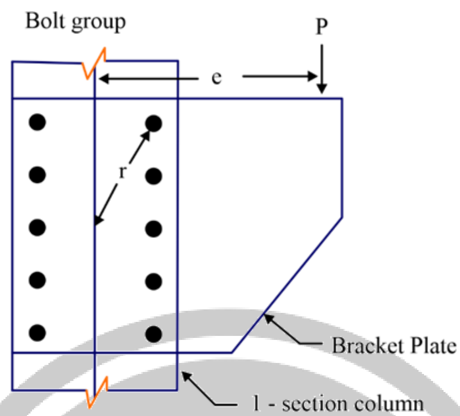

- A third type connection which resists end moments as well as permits relative rotation between the beam and column. The moment rotation relationship for different types of connection are shown in figure.

## (5) Analysis of Bolt Groups

In general, any group of bolts resisting a moment can be classified into either of two cases depending on whether the moment is acting in the shear plane (Bracket type connection-I) or in a plane perpendicular (Bracket type connection – II) to it.

## (6) Bracket Type Connection-I (Elastic Analysis)

### (a) Load or moment is lying in the plane of Bolt group




The eccentric load P may be replaced by concentric load and in plane moment (Twisting Moment) (P x e) acting on the joint.

Bolts are subject to direct concentric factored load (P) and twisting moment (M = p x e).

Direct vertical shear force in each bolt due to Direct axial load $f_a = P/n$ ($n$ – Number of bolts in connection)

Force in each bolt due to moment (M = P.e)

$$F_m = Per/\sum r^2$$

$n$ = Number of bols/rivets in a connection

$P$ = Eccentricity of a load

$r$ = Radial distance of bolt from C.G of bolt group

$$\sum r^2 = \sum x^2 + \sum y^2$$

The resultant of $F_a$ and $F_m$ act on the bolt is $F_R$ and θ is angle between $F_a$ and $F_m$, the resultant force $F_R$

$$F_R = \sqrt{F_a^2 + F_m^2 + 2F_aF_m\cos\theta}$$

- Critical bolt is that which is subjected to the maximum resultant force. Critical bolt which is farthest from C.G of the bolt group and nearest to the applied load line is most critical.
- For safety of connection FR $\leq$ design strength of the bolt ($V_{db}$).
- For design of joint approximate number of bolts in each vertical line $n^1 = \sqrt{\dfrac{6M}{mpV_{db}}}$

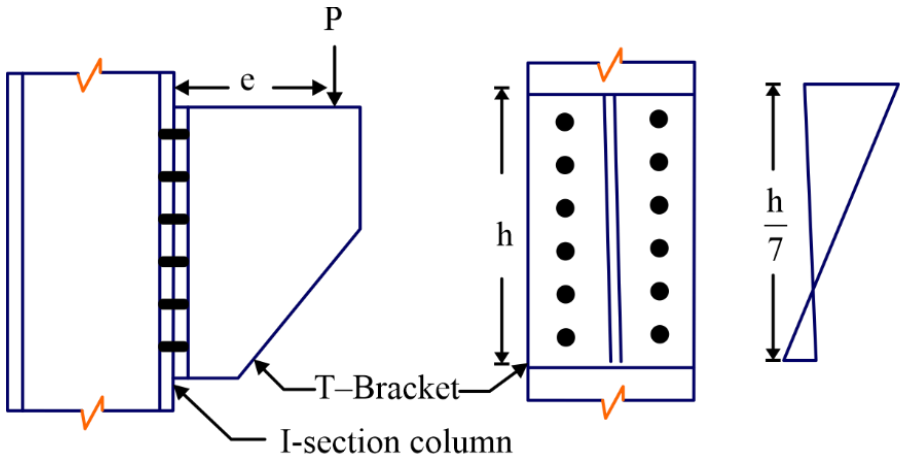
  $N_1$ – Number of bolts required per each vertical line

  $m$ – Number of lines in vertical

  $p$ – Pitch of the bolt

  $V_{db}$ – design strength of the bolt.

### (b) Load or moment is not lying in the plane of Bolt group




- The eccentric load P may be replaced by concentric load and moment $(P \times e)$ acting on the joint.
- Bolts are subjected to Direct concentric factored load $(P)$ and a bending moment $(M = P \times e)$
- Bolts are subjected to direct shear along with Tension due to moment
  Vertical shear force in each bolt due to
  Direct axial load $V_b = P/n$
  ( $n$ - Number of bolts in connection)
  Tensile force in any $i^{th}$ bolt due to moment

$$T_{bi} = \frac{M^1 y_i}{\sum y_t^2}$$

Moment of resistance provide by bolt in tension $M^1$

$$M^1 = \frac{M}{\left[1 + \frac{2h}{21} \sum y_i y_i^2\right]}$$

Tensile force in the extreme critical bolt

$$T_b = \frac{M^1 y_n}{\sum y_i^2}$$

$n$ - Number of bolts in connection
$e$ - Eccentricity of load
$h$ - Height of bracket
$y_1, y_2, \ldots, y_n$ - Distance of each bolts in tension from the axis of rotation

- For safety of connection is checked in combined shear and tension by using interaction equation

$$\left(\frac{v_b}{v_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

$V_b$ = Factored shear force on bolt,

$V_{db}$ = Design shear capacity

$T_b$ = Factored Tensile force on bolt,

$V_{db}$ = Design Tension capacity

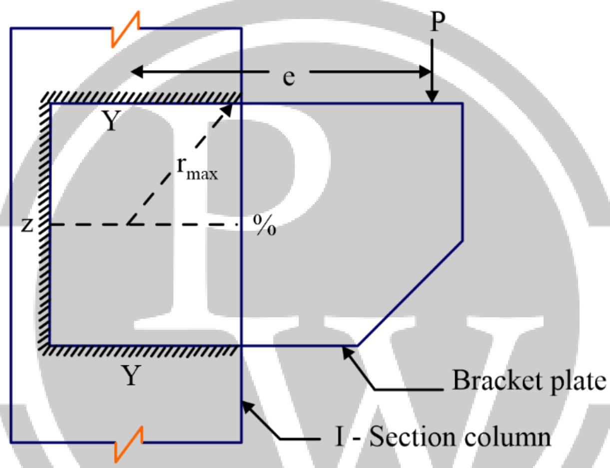
- For design of connection approximate no of bolts in each vertical line $n^1 = \sqrt{\frac{6M}{mpV_{db}}}$ m - No of bolt lines in vertical

  *p*- Pitch of bolt

  $V_{db}$ – Design strength of bolt

## 4.2. Eccentric Welded Connections

**(a) Bracket Type Connection – I (Fillet Weld) Load or moment is lying in the plane of fillet weld group**




Weld is subjected to Direct concentric load $(P)$ and a twisting moment $(M = P \times e)$

Direct vertical shear stress in the weld

$$q_1 = P/(d + 2b)t_t$$

d - Depth of the bracket

$t_t$ - Effective throat thickness of fillet weld $-Ks$

Maximum shear stress in weld at due to twisting moment (P.e)

$$q_2 = Per_{max} / I_p.$$

$I_p$ – Polar moment of inertia $-I_{xx} + I_{yy}$

*P*-Factored load

*e*-Eccentricity of the load

*r* - Radial distance of weld from C.G of weld group

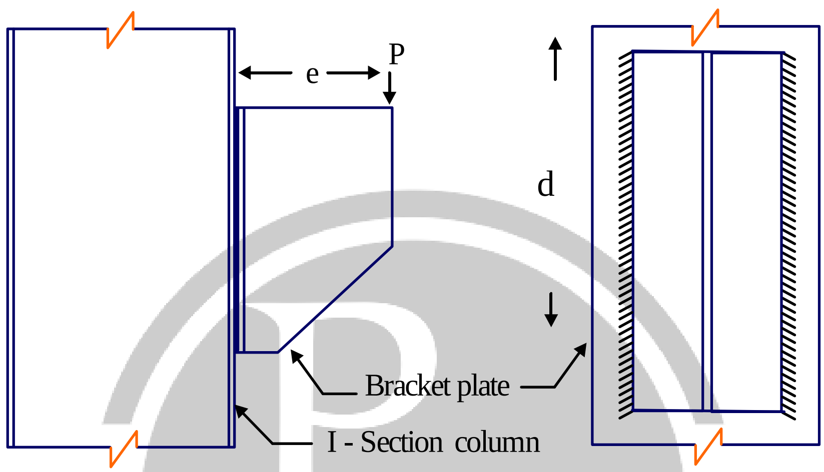
The resultant stress of $q_1$ and $q_2$ act in the weld is $q_R$ and $\theta$ is angle between $q_1$ and $q_2$, the resultant force $q_R$

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

For safety of connection the resultant shear stress $(q_R)_{max} \leq$ Design shear strength of weld $\left(f_{wd} = f_u / \sqrt{3}\gamma_{mw}\right)$

## (b) Bracket Type Connection – II (Fillet Weld)

### Load or moment is not lying in the plane of fillet weld group




Weld is subjected to Direct concentric load ($p$) and a twisting ($M = P \times e$)

Direct vertical shear stress in the weld

$$q_{1cal} = P/2d\, t_t$$

d – Depth of the bracket

$t_t$ = Effective throat thickness of weld (K s)

Bending stress in weld due to moment

(M = P, e)

$Q_{2cal} = P\, e\, (d/2) / [2\, (t_t\, d^3/12)]$

e – Eccentricity of the load

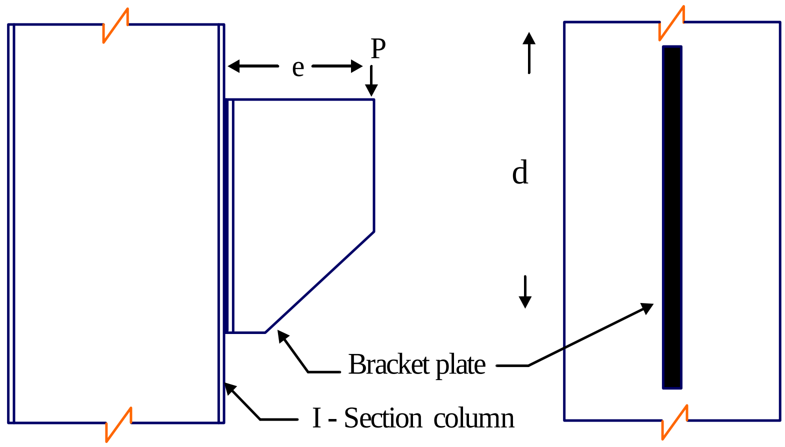
d – Depth of the bracket

- The combined stress of $q_{1cal}$ and $q_{2cal}$ act on the fillet weld is $f_e$

$$f_e = \sqrt{q_{1cal}^2 + q_{2cal}^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

- For safety of connection the resultant shear stress $(f_e)_{Max} \leq$ Design shear capacity of weld ($f_{wd} = f_u/\sqrt{3}\, \gamma_{mw}$)

**(c) Load or moment is not lying in the plane of butt weld group**




Direct vertical shear stress in the weld

$$q_{cal} = P/d\ t_e$$

d – Depth of the bracket

$t_e$ – Effective throat thickness of butt weld Bending stress in weld due to moment (P.e)

$f_{cal} = P\ e\ (d/2) / [(t_e d^3/12)]$

e – Eccentricity of the load

d – Depth of the bracket

The combined stress of $f_{cal}$ and $q_{cal}$ act on the butt weld is $f_e$

$$f_e = \sqrt{f_{cal}^2 + 3q_{cal}^2}$$

The combined bending and shear stress in the butt weld may be checked by the interaction formula

$$f_e = \sqrt{f_{cal}^2 + 3q_{cal}^2} \leq \frac{f_y}{\gamma_{m0}}$$

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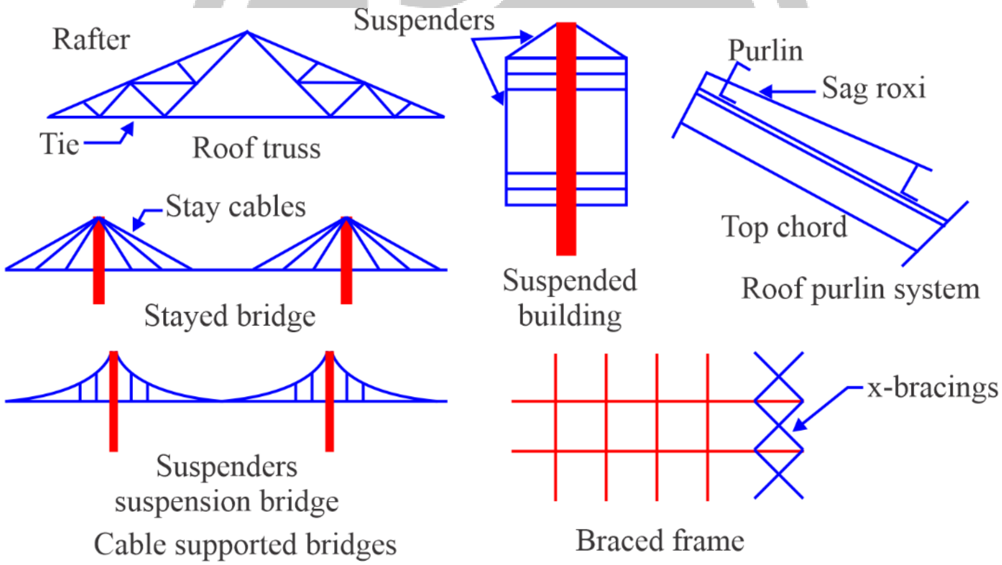
# 5 TENSION MEMBERS

## 5.1. Introduction

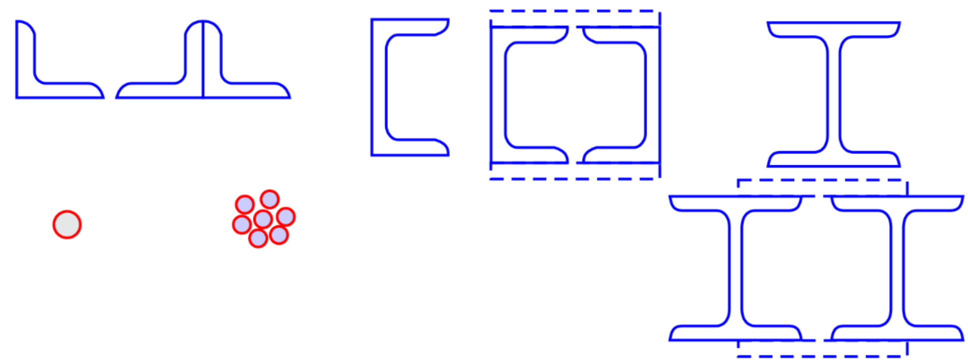
Tension members are linear members in which axial forces act so as to elongate (stretch) the member. A rope, for example, is a tension member. Tension members carry loads most efficiently, since the entire cross section is subjected to uniform stress. Unlike compression members, they do not fail by buckling. Ties of trusses, suspenders of cable stayed and suspension bridges, suspenders of buildings systems hung from a central core (such buildings are used in earthquake prone zones as a way of minimising inertia forces on the structure), and sag rods of roof purlins are other examples of tension members.
Tension members are also encountered as bracings used for the lateral load resistance. In X type bracings the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice versa. Hence, such members may have to be designed to resist tensile and compressive forces.




Tension Members in structures

## 5.2. Types of Cross-sections

The tension members can have a variety of cross sections. The single angle and double angle sections are used in light roof trusses as in industrial buildings. The tension members in bridge trusses are made of channels or I sections, acting individually or built-up. The circular rods are used in bracings designed to resist loads in tension only. They buckle at very low compression and are not considered effective. Steel wire ropes are used as suspenders in the cable suspended bridges and as main stays in the cable-stayed bridges.

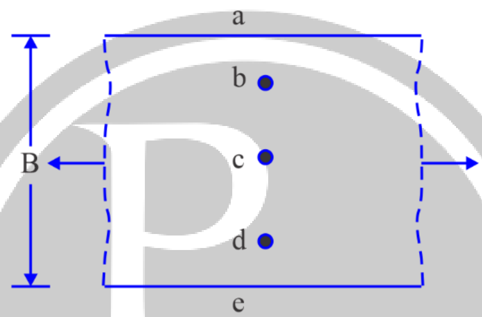


Cross Sections of tension members

## 5.3. Net Sectional Area (AD)

$A_n = A_g$ - Sectional area of holes

**(a) Plate with chain Bolting**

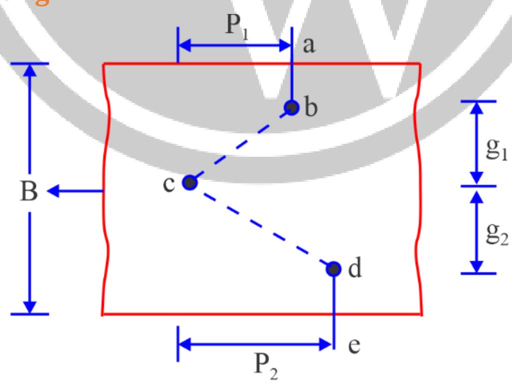



(Hole diameter $d_v$ and n = 3)

Net sectional area of plate with chain bolting along a-b-c-d-e

$A_n = (B - nd_o)$ *for chain bolting*

**(b) Plate with staggered(Zig-Zag) Bolting**




(Hole diameter $d_v$ and n = 3)

Net sectional area of plate with staggered bolting along a-b-c-d-e A = $= \left(B - nd_0 + \frac{P_1^2}{4g_1} + \frac{P_2^2}{4g_2}\right)t$ for staggered bolting

$A_g$ = Gross sectional area of plate
$A_n$ = Net sectional area of plate
B = Widtb of plate
n = Number of bolts
$d_o$ = Diameter of bolt hole
t = Thickness of plate

## 5.4. Types of failures in tension member

(a) Gross section yielding (Limit state of yielding in the gross section)
(b) Net section rupture (Limit state of fracture or rupture)
(c) Block shear failure

## 5.5. Design strength of tension member ($T_d$)

### (a) Design tensile strength based on gross section yielding ($T_{dg}$)

factored design tension (T) < $A_g f_y$

Design tension strength

$$T_{dg} = T/\gamma_{mo} = A_g f_y/\gamma_{mo}$$

### (b) Design tensile strength based on net section ruptnre ($T_{dn}$)

(i) A Tension member connected to other member or gusset plate by bolt or weld Factored design tension (T) < $A_n f_u$

Design tension strength

$$T_{dn} = T/\gamma_{mo} = 0.9 A_n f_u /\gamma_{m1})$$

(ii) A tearing strength of an angle section connected through one leg

Design tension strength

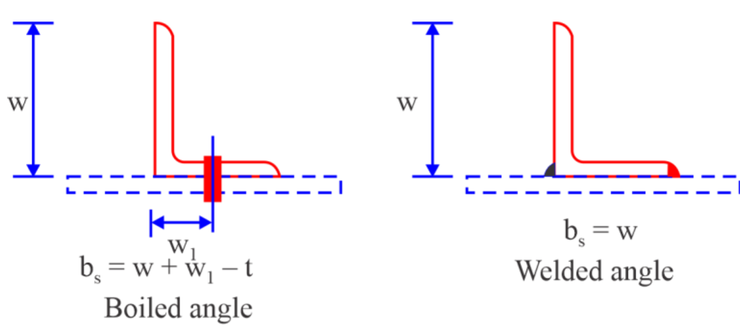
$$T_{dn} = 0.9\frac{A_{nc}f_u}{\gamma_{m_1}} + \beta\frac{A_{go}f_u}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076 (w/t) (f_y/f_u) (b/L_c)$$

$$\beta = \left(f_u \cdot \gamma_{mo} / f_y \cdot \gamma_{m1}\right) \text{ and } \beta \geq 0.7$$

$A_n$ = Net area of total cross section
$A_{nc}$ = Net area of connected leg
$A_g$ = Gross area of the outstanding leg
t = Thickness of the angle leg

$L_c$ = Length of end connection (Distance between the outermost bolt in the joint along the length or length of weld along load direction




Angles with End Connction

**Note:**

- Above equation is valid for double angles, channels, I-sections and other rolled sections connected one or more elements to an end gusset.
- For preliminary design of tension member IS800 code recommends following formula for design tearing strength of net section

$$T_{dn} = \alpha \frac{A_n f_u}{\gamma_{m_1}}$$

$$\begin{aligned} a &= 0.6 \text{ for one or two bolts } (n \leq S\ 2) \\ &= 0.7 \text{ for three bolts } (n = 3) \\ &= 0.8 \text{ for four or more bolts } (n \geq 3) \\ &= 0.8 \text{ for weld lengths} \end{aligned}$$

**(c) Design tensile strength based block shear (the block shear strength at end connection) ($T_{db}$)**

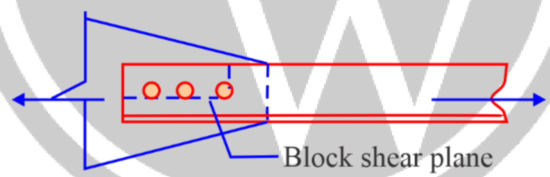
➢ **For plates**

(i) For shear yield and tension fracture

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

(ii) For shear fracture and tension yield

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




Block shear Failure

where. $A_{vg}$ and $A_{vn}$ = minimum gross and net area in shear along a line of transmitted force, respectively, and $A_{tg}$ and $A_{tn}$ = minimum gross and net area in tension from the hole to the toe of the angle, perpendicular to the line of force, respectively.

## 5.6. Design of Axially loaded Tension member

- In the design of a tension member, the design tensile force is given and the type of member and the size of the member have to be arrived at the type of member is usually dictated by the location where the member is used. In the case of roof trusses, for example, angles or pipes are commonly used. Depending upon the span of the truss, the location of the member in the truss and the force in the member either single angle or double angles may be used in roof trusses. Single angle is common in the web members of a roof truss and the double angles are common in rafter and tie members of a roof truss. Built-up members made of angles, channels and plates are used as heavy tension members, encountered in bridge trusses.

- The design process is iterative, involving choice of a trial section and analysis of its capacity.
- The net area required An to carry the factored design load T is $A_n = \dfrac{T}{\dfrac{0.9f_u}{\gamma_{m1}}}$ or $\dfrac{T}{\dfrac{\alpha\, f_u}{\gamma_{m1}}}$
- The net area increased by 25% – 40% to compute the gross cross sectional area calculated by $A_g$
- Gross area is also determined from its yield strength by $A_g = A_g = \dfrac{T}{\dfrac{f_y}{\gamma_{m0}}}$.
- Select a suitable rolled steel section to match with computed gross area.
- The number of bolts required to make the connection is calculated. These are arranged in a suitable pattern.
- The design tensile strength of trail section is calculated by considering
  - ➢ Strength in yielding of gross cross section
  - ➢ Strength in rapture of critical section and
  - ➢ Strength in block shear
- The design strength $T_d$ should be greater than factored design load
- The slenderness ratio of the member is checked as per IS 800.

## 5.7. Maximum or Limiting slenderness ratio (Stiffness Requirement)

- The tension members, in addition to meeting the design strength requirement, frequently have to be checked for adequate stiffness. This is done to ensure that the member does not sag too much during service due to self-weight or the eccentricity of end plate connections.
- Limitations on the slenderness ratio of members subjected to tension as per IS: 800

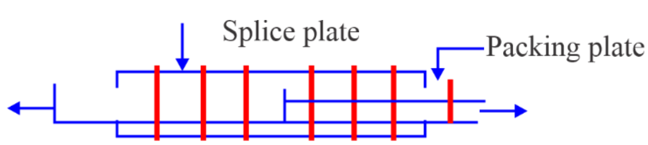
(a) A tension member in which reversal of direct stress due to other than wind seismic loading $l/r \le 180$.

(b) A member normally acting as a tie in roof truss or a bracing system but subjected to possible of reversal of stresses resulting from action of wind or earth quake forces $l/r \le 350$.

(c) For any other tension members $l/r \le 400$

### Tension Splice

- It is a joint for a tension member, tension splice is provided when Length of member required is lesser than available length from Indian rolling mills or factory or when two lengths of a tension member have different thickness (or cross section) are to be connected with filler plate
- Tension splices are provided on both sides of member joined in the form of cover plates.




Tension splice

- The strength of the splice plates and bolts/weld connecting them should have strength at least equal to design load
- The design shear capacity of bolt carrying shear through packing plate in excess of 6mm shall be decreased by a factor $\beta_{pkg}$

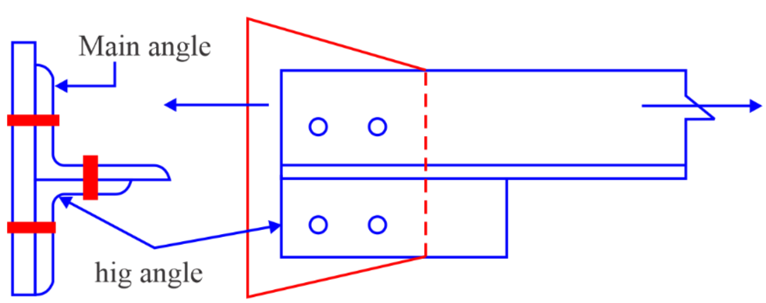
$$\beta_{pkg} = 1.0 - 0.0125\, t_{pkg}$$

(where $t_{pkg}$ = thickness of thicker packing plate)

## 5.8. Lug Angle

In order to increase the efficiency of the outstanding leg in single angles and to decrease the length of the end connections, sometimes a short length angle at the ends are connected to the gusset and the outstanding leg of the main angle directly,. Such angles are referred to as lug angles.

- Lug angle is short length of an angle (or channel) used at a joint to reduce the length of connection of heavily loaded
- tension member.
- By using lug angle there will be saving in gusset plate, but additional fosterers and angle member required, hence now days it is not preferred IS 800:2007 specifications for lug angle are
- The effective connection of lug angle shall as far as possible at the end of the connection
- The connection of lug angle to main member shall preferably start in advance of the member to gusset plate
- Minimum of two bolts or equivalent weld be used for attaching lug angle to gusset. If the main member is an angle
- The whole area of the member shall be taken as effective section rather than net effective section (whole area of the member is gross area less deduction for bolt holes)
- The strength of lug angle and fasteners connection to gusset plate or any other attachment should be at least 20% (10 %, if main member is channel) more than the force in outstanding leg.
- The strength of fasteners connection lug angle to main member shall be at least 40 % more than force carried by the
- outstanding leg (20 %, if main member is channel)




Angle tie with lug Angle

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# 6 COMPRESSION MEMBERS

## 6.1. Introduction

- A compression member is a structural member which is subjected to two equal opposite compressive forces applied at its ends. There are may types of compression members, the column being the best known. Top chords of trusses, bracing members, boom is another principle compression member in a crane and compression flanges of built up beams and rolled beams are all examples of compression elements. Columns are usually though of a as straight vertical members whose lengths are considerably greater than their cross-sectional dimensions.
- An initially straight strut or column, compressed by gradually increasing equal and opposite axial forces at the ends is considered first. Columns and struts are termed "*long*" or "*short*" depending on their proneness to buckling. If the strut is "short", the applied forces will cause a compressive strain, which results in the shortening of the strut in the direction of the applied forces. Under incremental loading, this shortening continues until the column "squashes". However, if the strut is "long", similar axial shortening is observed only at the initial stages of incremental loading. Therefore, as the applied forces are increased in magnitude, the strut becomes "*unstable*" and develops a deformation in a direction normal to the loading axis. The strut is in a "*buckled*" state.
- Bucking (mainly in members subjected to compressive forces) behaviour is thus characterized by deformations developed in a direction (or plane) normal to that of the loading that produces it.

## 6.2. General failures in axially loaded columns

- Very short columns subjected to axial compression fail by yielding or crushing.
- Very long columns fail by elastic buckling in the Euler mode.
- Intermediate columns generally fail by inelastic buckling.

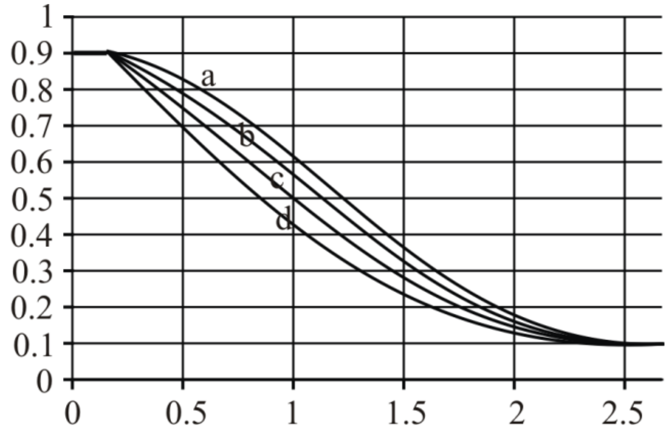
## 6.3. Design Compressive strength ($P_d$) of a member

Slenderness ratio (KL/$r$) and material yield stress ( $f_y$ ) are dominant factors affecting the ultimate strengths of axially loaded columns.

$$P_d = A_e f_{cd}$$

Where $A_e$ = Effective Sectional area.

$f_{cd}$ = Desing stress in compression.




**Column Buckling Curess as per IS: 800**

- Code also recommends following equation for estimating design axial compressive stress ($f_{cd}$) of axially loaded compression member, it considers the residual stress, initial imperfection and eccentricity of load

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

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

$\lambda$ = Non dimensional effective slenderness ratio = $\sqrt{\frac{f_y}{f_{cc}}} = \sqrt{f_y\left(\frac{KL}{r}\right)^2 / \pi^2 E}$

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

KL/*r* = Effective slenderness ratio

$\alpha$ = Imperfection factor

| Buckling class | a | b | c | d |
|---|---|---|---|---|
| $\alpha$ | 0.21 | 0.34 | 0.49 | 0.76 |

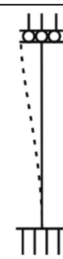
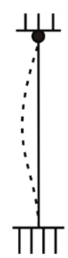
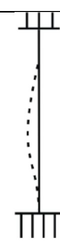
Stress reduction factor account for residual stresses =

$$\chi = \frac{1}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

___ = Partial safety factor for material strength.

## 6.4. Effective length of Columns (KL)

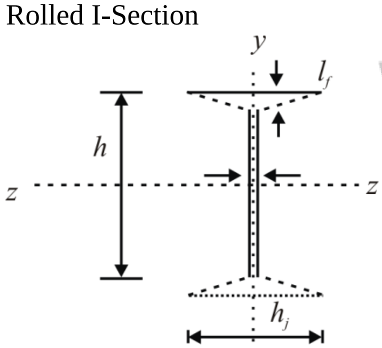
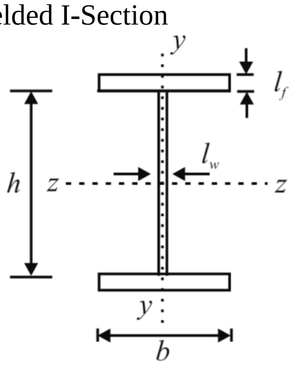
The effective length KL, is calculated from the actual length (L) of the member, considering the rotational and relative translational boundary conditions at the ends.

| Boundary Conditions | | | | Schematic representation | Effective length |
|---|---|---|---|---|---|
| At one end | | At the other end | | | |
| Translation | Rotation | Translation | Rotation | | |
| Restrained | Restrained | Free | Free |  | 2.0 L |
| Free | Restrained | Restrained | Free |  | |
| Restrained | Free | Restrained | Free |  | 1.0 L |
| Restrained | Restrained | Free | Restrained |  | 1.2 L |
| Restrained | Restrained | Restrained | Free |  | 0.8 L |
| Restrained | Restrained | Restrained | Restrained |  | 0.65 L |
| L is the unsupported length of the compression member | | | | | |

## 6.5. Effective length for angle struts

| Type | Section | Effective length | |
|---|---|---|---|
| | | In plane of gusset | Perpendicular to gusset |
| Continuous angles (top or bottom chord of trusses) | Single angle or double angle | 0.7 L to 1.0 L | 1.0 L |
| Discontinuous angles | Single angle connected with one bolt<br>Single angle connected with more than one bolt are equivalent weld | 0.85 L | 1.0 L |
| Discontinuous angles | Double angles placed on either side of the gusset plate<br>Double angles placed on same side of the gusset plate | 0.70 L to 0.85 L | 1.0 L |

## 6.6. Buckling Class of Cross Sections

| Cross Section | Limits | Bucking about axis | Bucking Class |
|---|---|---|---|
| Rolled I-Section   | $h/b_f > 1.2 = t_f \leq 40$ mm<br>40 mm $< t_f \leq 100$ mm | z-z<br>y-y<br>z-z<br>y-y | a<br>b<br>b<br>c |
| | $h/b_f \leq 1.2$ :<br>$t_f \leq 100$ mm<br>$t_f > 100$ mm | z-z<br>y-y<br>z-z<br>y-y | b<br>c<br>d<br>d |
| Welded I-Section   | $t_f \leq 40$ mm<br>$t_f > 40$ mm | z-z<br>y-y<br>z-z<br>y-y | b<br>c<br>c<br>d |
| Hollow Section | Hot rolled | Any | a |
| Channel, Angle, T and Solid Sections | | Any | c |
| Built-up Member | | Any | c |

## 6.7. Design of Compression Member

- Desing stress in compression ($f_{cd}$) in the member is to be assumed.
- For angles struts $f_{cd}$ = 90 N/mm$^2$
- For rolled steel beam sections $f_{cd}$ = 135 N/mm$^2$
- Column with heavy factored load $f_{cd}$ = 200 N/mm$^2$
- Effective cross sectional area required for factored load $P_d$; $A_c = P_d/f_{cd}$
- Select a suitable section to give effective area required and calculate minimum radius of gyration ($r_{min}$) for selected section
- The effective length of the column is calculated based on end conditions and the slenderness ratio is computed ($KL/r_{min}$), which should be less than permissible slenderness ratio.
- For the estimated slenderness ratio, the design compressive stress $f_{cd}$ and design compressive strength of section $P_d$ which should be higher than factored load. If not repeat above steps.

## 6.8. Maximum Slenderness Ratio (Stiffness Requirement)

The IS: 800 impose the following limitations on the slenderness ratio of members subjected to compression.

(a) A member carrying compressive loads from dead and imposed loads $l/r \leq 180$.

(b) A member subjected to compressive forces resulting only from combination with wind/earth quake actions $l/r \leq 250$.

(c) Compression flange of a beam restrained against torsional buckling $l/r \leq 300$.

## 6.9. Built up Columns

(a) Used when rolled steel sections do not provide required section and ara or large radius of gyration of column section is required to different directions built up section.

(b) It usually provided either lacing or batten system for built up columm.

## 6.10. Lacing and Battening for built up Compression Members.

(a) The different components of built up section are placed in such a way that the built up section has same radius of gyration about both axes. (i.e. $r_{zz} = r_{yy}$)

(b) The different components of built up sections are connected together so that they act as a single column.

(c) Lacing and battening systems are used to connect the members.

(d) Lacing is generally preferred in case of eccentric loads. For axially loaded members, battening is preferred.

## 6.11. Lacing System

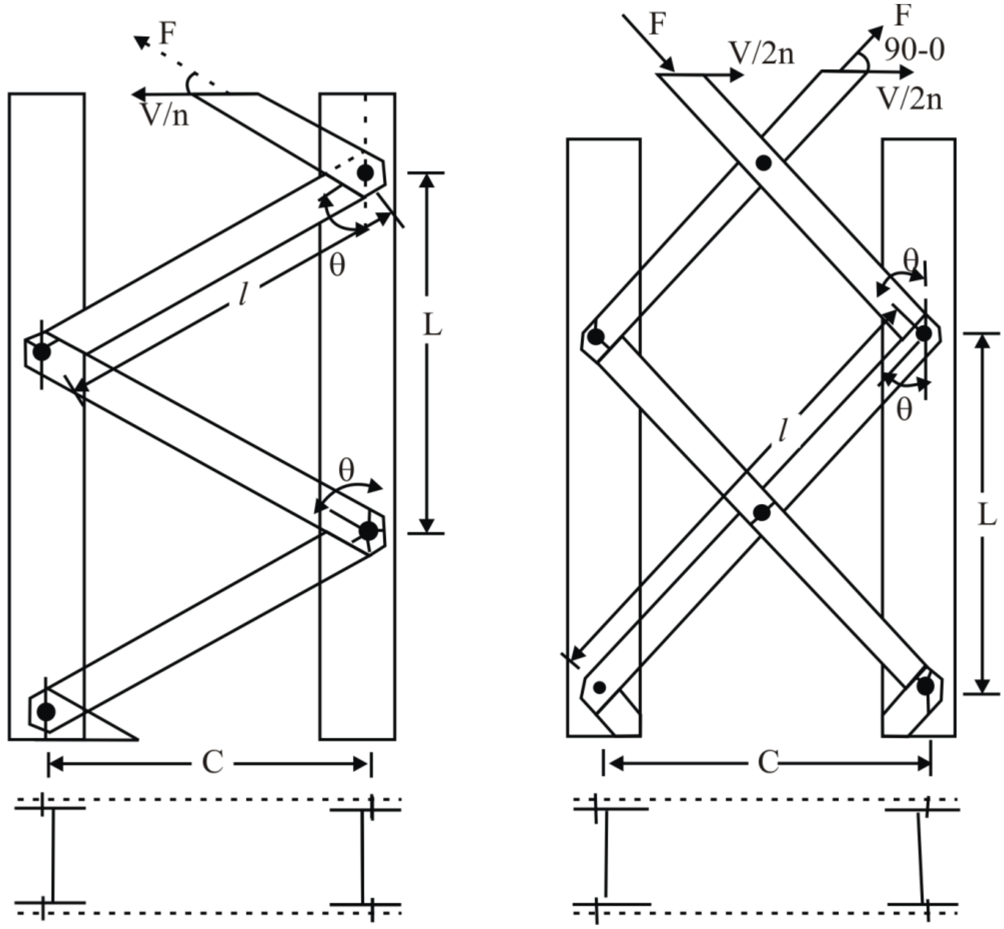
### General Specification

- Flat bars, angle, channel and tubular sections are used for lacing.
- Lacing system should not be varied throughout the length of the member.

- The single – laced system on opposite sides of the main components should be in the same direction so that one be the shadow of the other.
- Tie plates should be provided at the ends of the lacing system and at points where lacing systems are interrupted.

## Design Specifications

- The effective slenderness ratio of laced column should be increased by 5% to account for shear deformations due to unbalance horizontal forces.
- Angle of Inclination (θ) of lacing system with longitudinal axis should be between 40° and 70°. (i.e. $40° \leq \theta \leq 70°$ )
- Effective slenderness ratio of lacing bar should not exceed 145.




- **Effective length of lacing member ($l_e$)**

  For single lacing (Bolted) $l_e = l$

  - For double lacing Bolted at end $le = 0.7\ l$
  - For welded lacing $le = 0.7\ l$

- For Bolted or welded lacing system, $L/r^c$ min $\leq 50$ or $\leq 0.7$ times KL/r of member as a whole, whichever is less.

  Where $r^c_{min}$ = minimum radius of gyration of the components of compression member

- Minimum width of lacing bar in Bolted connection.

| | | | | |
|---|---|---|---|---|
| Shank diameter of the bolt (d) inmin | 22 | 20 | 18 | 16 |
| Width of lacing bar | 65 | 60 | 55 | 50 |
| Width of lacing bar is approximately 3x nominal shank diameter of bolt. | | | | |

- Minimum thickness of lacing bar
  - $t_{min} = l/40$ for single lacing
  - $t_{min} = l/60$ for double lacing

  Where $l$ = length of the lacing bar
    - The lacing should be designed to resist a transverse shear of 2.5% design column load (V = 2.5% of design axial load)
    - The lacing should be designed to resist additional shear due to bending if the compression member carries bending.
    - For single lacing, the force (Design compression or Design tensile) in each lacing bar,

$$F = \frac{V}{N\sin\theta};$$

$$F = \frac{V}{2\sin\theta} \quad \text{for double lacing (N = 2)}$$

$$F = \frac{V}{4\sin\theta} \quad \text{for double lacing (N = 4)}$$

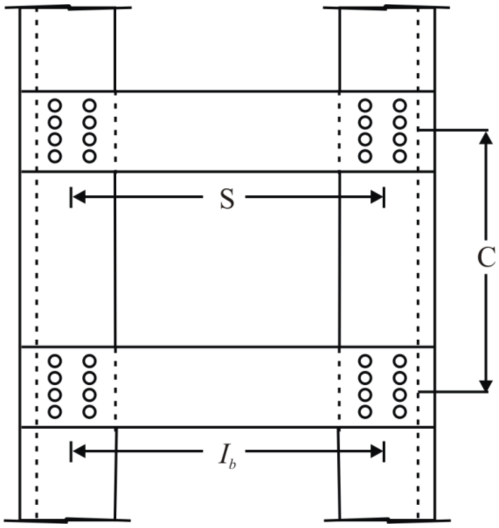
## 1.12. Batten System

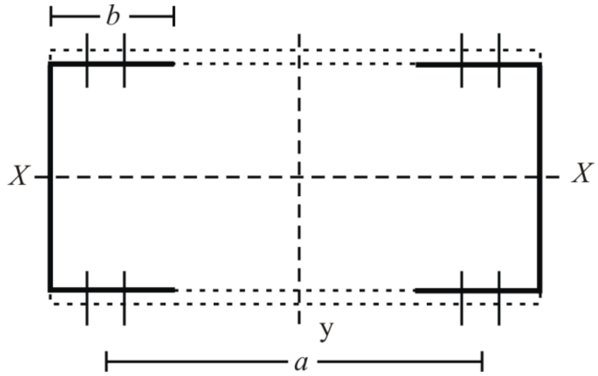
### General Specification

- The no. of battens should be such that the member is divided into not less than three parts longitudinally (i.e. minimum 4 batten plates or minimum two intermediate battens and two end battens)
- Flat plates are used for battens.
- Effective length of battened column should be increased by 10%.

### Design Specifications

- Spacing of battens 'C' is such that, the slenderness ratio of the lesser main component, $\frac{C}{r_{min}^{c}} \ngtr 50$ or 0.7 KL/r of the member as a whole about Z-Z axis (parallel to battens), whichever is less.




Where $r_{min}^{c}$ = minimum radius of gyration of component.

S = transverse distance between centroids of bolt group or rivet group

C = Spacing of battens

$l_b$ = transverse distance between centroids of inner end bolt group or rivet group.

- Battens should be designed to carry bending moment and shear forces arising from transverse shear (V) 2.5% of total design axial load on member.

Longitudinal shear on batten, $V_1 = \dfrac{VC}{NS}$

Moment of batten, $M = \dfrac{VC}{2N}$

Where N = No. of parallel plates of battens = 2 in the above figures.

- Thickness of batten, $t > l_b / 50$ ($l_b$ = length of batten plate)
- Effective Depth of batten

$d$ > (3a/4) for intermediate batten

$d$ > a for end batten

$d$ > 2b for any batten

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# 7 COLUMN BASES & COLUMN SPLICES

## 7.1. Introduction

The design compressive stress in a concrete footing is much smaller than it is in a steel column. So it becomes necessary that a suitable base plate should be provided below the column to distribute the load from it evenly to the footing below. The main function of the base plate is to spread the column load over a sufficiently wide area and keep the footing from being over stressed.

## 7.2. Types of Column bases

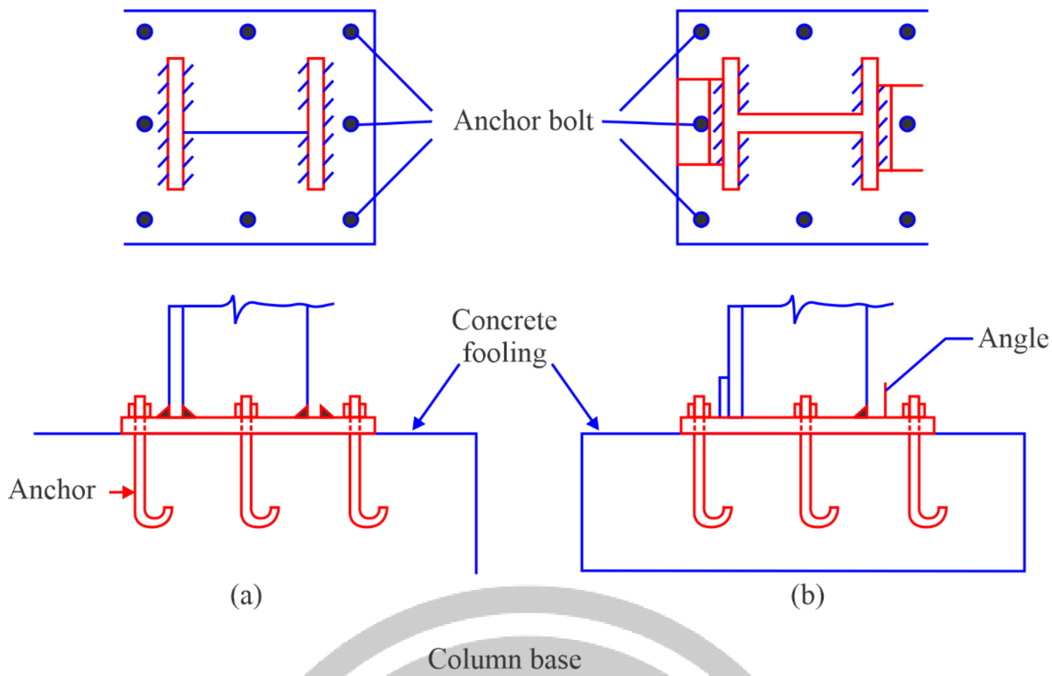
For a purely axial load, a plain square steel plate or a slab attached to the column is adequate. For small columns these plates will be shop-welded to the columns, but for larger columns, it may be necessary to the plates separately and set them to the correct elevations. For this second case the columns are connected to the footing with anchor bolts that pass through the lug angles which have been shop-welded to the columns. When there is a large moment in relation to the vertically applied load a gusseted base may be required. This is intended to allow the lever arm from the holding down bolts to be increased to give maximum efficiency while keeping the base plate thickness to an acceptable minimum.

- Slab base
- Gusseted base

## 7.3. Slab Base (Concentrically Loaded Columns)

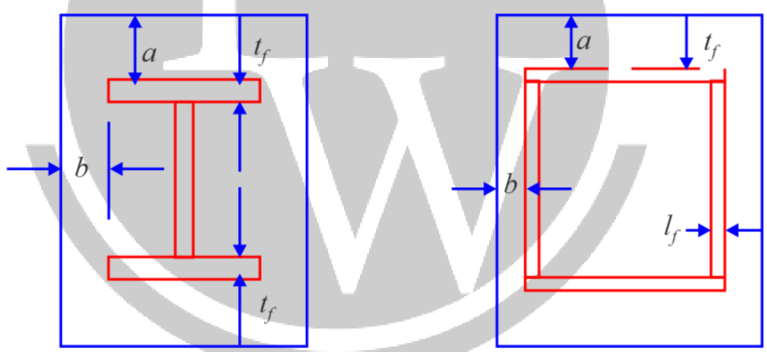
- The design compressive stress in a concrete footing is much smaller than it is in a steel column.
- The main function of the base plate is to spread the column load over a sufficiently wide area and keep the footing from being over stressed.
- For a purely axial load, a plain square steel plate or a slab base attached to the column is adequate.
- When there is a large moment in relation to the vertically applied load a gusseted base may be required.
- The minimum thickness, $t_s$, of rectangular slab bases, supporting columns under axial compression shall be

  For I, H, channel, box or RHS




Base plate thickness $t_s = \sqrt{\dfrac{2.5w\left(a^2 - 0.3b^2\right)\gamma_{m0}}{f_y}} > t_f$

(Thickness of column flange)




$w$ = pressure in N/mm$^2$ on underside of plate assuming a uniform distribution.

$a$, $b$ = larger and smaller projection of the slab base beyond the column,

Maximum allowable bearing strength = 0.45 $f_{ck}$ (where $f_{ck}$ = cube strength of concrete)

**Note:** A reduced value of 0.45 $f_{ck}$ is used against maximum of 0.60 $f_{ck}$ as recommended by the code $f_{yp}$ = design strength of plate, but not greater than 250 N/mm$^2$ divided by $\gamma_m$ .

**Design procedure:**

- Assume a suitable grade of concrete. The bearing strength of concrete is 0.45 $f_{ck}$ reactive
- Area of slab base (A)

$$= \frac{\text{Factored column load (p)}}{\text{Bearing strength of concrele}\left(0.45\, f_{ck}\right)}$$

- If square base plate is provided,

  Side of square base plate = $L = B = \sqrt{A}$

  If projections of base plate beyond the column faces are a & b are kept equal $(D + 2b)(b_f + 2a) = A$

  $L$ = Length of base plate in mm

  $B$ = Width of base plane in mm

  $a$ = Bigger projection of base plate beyond column in mm

  $b_s$ = smaller projection of base plate beyond column in mm

  $D$ = Width of the flange of column

- The intensity of up to end bearing pressure is under slab base is N/mm$^2$.

$$w = \frac{P}{A_0}$$

  $Ag$ = Area of slab base plate is provided

- Thickness ts of the slab base

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}} > t_f$$

- Holding down 2 or 4 in number and of 20 mm diameter are usually provided, when base is subjected to only axial compressive load, two bolts will be enough.

- Thickness of the slab base

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}}$$

$$= \sqrt{\frac{2.5\times 8(125^2 - 0.3\times 125^2)\times 1.10}{250}} = 31.02\text{ mm} \geq t_f$$

  Thickness of slab base $t_s$ = 31.02mm

## 7.4. Gusseted Bases

- Column with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength.
- Gusseted base keeps the base plate thickness to be minimum.
- The thickness of Gusseted base is computed by equating the moment at critical section (i.e at root of gusset angle) and equating to moment of resistance of gusset.
- Design bending strength at the critical section $M_d = 1.2\, f_y\, Z/\gamma_{mo}$
- Thickness of gusseted base $\quad t = c\sqrt{2.75\frac{w}{f_y}}$
- Where, $c$ = cantilever projection of base plate
  $f_y$ = yield stress of steel in N/mm2
  $t$ = Aggregate thickness of base plate for bolted gusseted base and the thickness of base plate for welded base plate.

## 7.5. Column Splice

- A joint is required in the length of column member is called column splice.
- When the length of the column is more than the length of column section is available.
- In multi-storied building the section column required for various floors may be different.

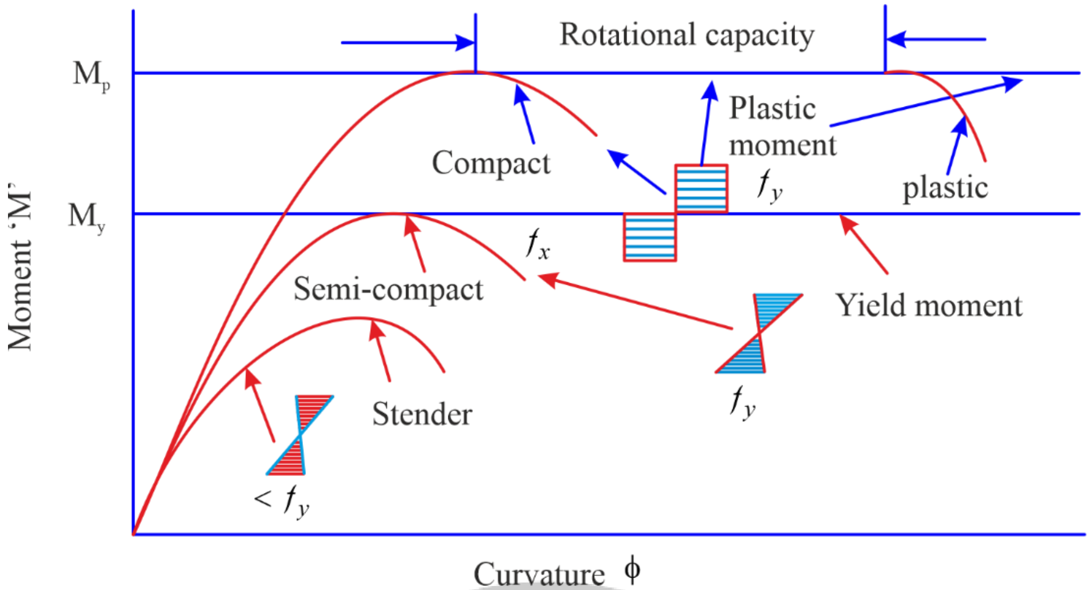
### Specifications of Column Splice

- When the ends of compression member are faced (machined) for complete bearing over whole area they should be splice to whole the connected members accurately in position and this is tension if any bending pressure.
- When such members are not faced (Machine) for complete bearing splice should designed to transmit all forces to which they are subjected
- Column splices are designed as a short column.

# 8 BEAMS

## 8.1. Introduction

1. A structural member subjected to transverse loads (loads perpendicular to tis longitudinal axis) is called as beam. Beams may be classified as
   - **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 carry wall loads over wall opening 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 steps step (in case of buildings).
   - **Header:** A beam at stair well openings.
2. Classifications of sections (depending on yield moment, plastic moment and rotational capacities, the four classes of section are the plastic, the compact, the semi compact and slender sections)
   - **Plastic section:** Cross sections which can develop plastic hinges and have rotation capacity required for failure of the section by formation of plastic mechanism are called plastic section.
   - **Compact section:** Cross sections which can develops plastic moment resistance but have inadequate plastic hinge rotation capacity for formation of a plastic machine before buckling are called as compact section
   - **Semi compact section:** Cross sections in which the extreme fibre compression can reach yield stress, but can’t develop the plastic moment of resistance due to local buckling are called semi compact section.
   - **Slender section:** Cross sections in which elements buckle locally even before attaining of yield stress are called as slender sections.




## 8.2. Bending (Flexural) strength

Supported 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) $\leq$ Design bending strength of section $M_d$)
- When $d/t_w < 67\ \varepsilon$ (No shear buckling web)
- Nominal shear capacity ($V_n$) – Plastic shear strength of beam ($V_p$)
- Design shear strength $V_d = V_n / \gamma_{mo}$
- When $d / t_w > 67\varepsilon$ (web of beam susceptible to shear buckling)

**Case I:**

Low shear case – (Factored design shear force $V \leq 0.6\ V_d$)

Design bending strength $M_d = \beta_b Z_p f_y/\gamma_{mo} \leq 1.2\ Z_e\ f_y/\gamma_{mo}$ (For simply supported beams) $\leq 1.5\ Z_e\ f_y/\gamma_{mo}$ (For cantilever beams)

Where $\beta_b = 1.0$ (for plastic and compact sections $\beta_b = Z_e/Z_p$ 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

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

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

$V$ = Factored applied shear force

$V_d$ = Design shear strength as governed by web yielding (or) web backing.

$M_{fd}$ = 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 neckline need to be considered when $\lambda_{LT} \le 0.4$ (where $\lambda_{LT}$ - no dimensional effective slenderness rating for lateral torsional buckling)

The bending strength of laterally unsupported beam is given by

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

$\beta_b$ = 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 $= \chi_{CT} \cdot \frac{fy}{rmo}$

$\chi_{LT} \cdot =$ Bending stress reduction factor to account for lateral torsional buckling.

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

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

$\alpha_{LT} =$ Imperfection factor

= 0.21 (For rolled sections)

= 0.49 (Welded section)

$\lambda_{LT} =$ No dimensional slenderness ratio $= \sqrt{\beta_b Z_p . f_y / M_{cr}} \le \sqrt{1.2 Z_e . f_y / M_{cr}} = \sqrt{\frac{f_y}{f_{cx} . b}}$

$$M_{cr} = \sqrt{\left(\frac{\pi^2 EI}{L_{LT}^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{L_{LT}^2}\right]} = \beta_b . Z_p . f_{cr.b}$$

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

## 8.3. Shear Strength of Laterally Supported Beam

Design shear strength of section

$$V_d = A_v f_{yw} / \sqrt{3}\gamma_{mo}$$

$A_v$ – shear area

$f_{yw}$ – yield strength of web Shear area $A_v$.

### (a) For I – section and channel section

- Major axis bending:

  Hot rolled – $A_v = h\ t_w$

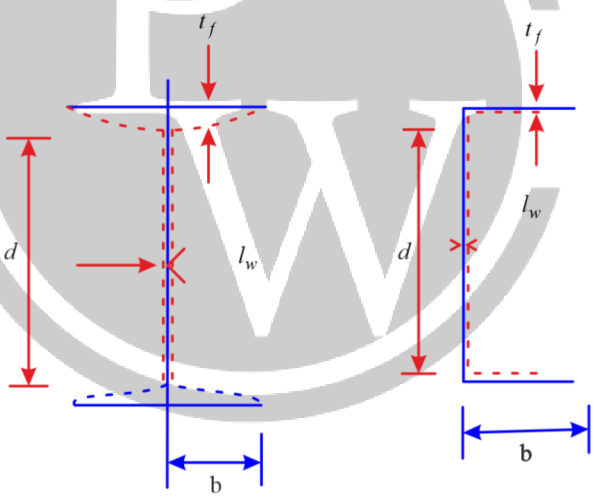
  Welded – $Av = d\ t_w$

- Minor axis bending :

  Hot rolled or welded – $A_v = 2\ b\ t_f$

### (b) For rectangular hallow sections of uniform thickness

- Load parallel to depth (h) $A_v = Ah/(b + h)$
- Load parallel to width (b) $A_v = Ab/(b + h)$




Rolled chamels Rolled Beam and

## 8.4. Deflection Limit

- Excessive deflections may lead to crack in the plaster or ceilings and may damage the martial attached (or) supported by the beam. This limits the maximum deflection. To the value given below
- Other reasons for limiting the deflections are

  (a) Excessive deflection may create problems for roof drainage

  (b) These may cause undesirable twisting and distortion of connections and connected members and lead to high secondary stresses.

- Deflection can be reduced by increasing the depth of beam section, reducing the spam, providing greater and restraints.
- Vertical deflection for cantilever span

  Elastic cladding – span/240

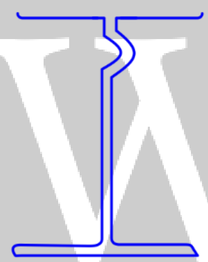
  Brittle cladding – span/240
- Vertical deflection for purlins and grit

  Elastic cladding – span/150

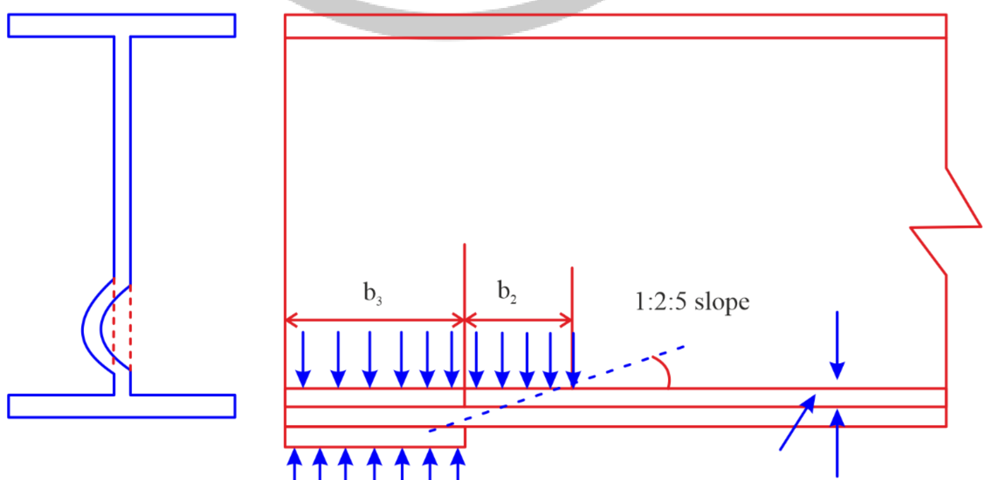
  Brittle cladding – span/180

## 8.5. Web Crippling

- Loads and reactions concentrated along a short length of flange of beam are resisted by compressive stresses in the web which vary with distance from the load.
- Stress concertation occurs at the junction of the web and the flange. As a result, large bearing is developed joint below the concentrated loads consequently the web near the portion of stress concentration tends to fold over the flange. This type of local bucking phenomena is called crimpling or crimpling of the web.
- Web crippling is buckling of web caused buy the compressive force delivered through flange to keep the bearing stresses within permissible limits. The concentrated load should be transferred flange. To the web sufficiently large bearing area



Web Crippling



The bearing strength is $F_w = \frac{A_e.f_{yw}}{\gamma_{mo}}$ $f_w$ = yield design strength of the web $A_e$ = effective area of the web = $b_1t_w$ . The angel of dispersion of the load is assumed to be 1:2.5

Bearing length $b_1$ = b =2$n_1$ (under concentrated load) $b_1$ = b + $n_1$ (under reaction at support) $n_1$ = 2.5 ($t_f$ + $R_1$)

- The bearing strength calculated should be greater than the concentrated load (it will safe for buckling)
- To eliminate web criplling, bearing stiffeners are to be provided.
- To provide thicker webs

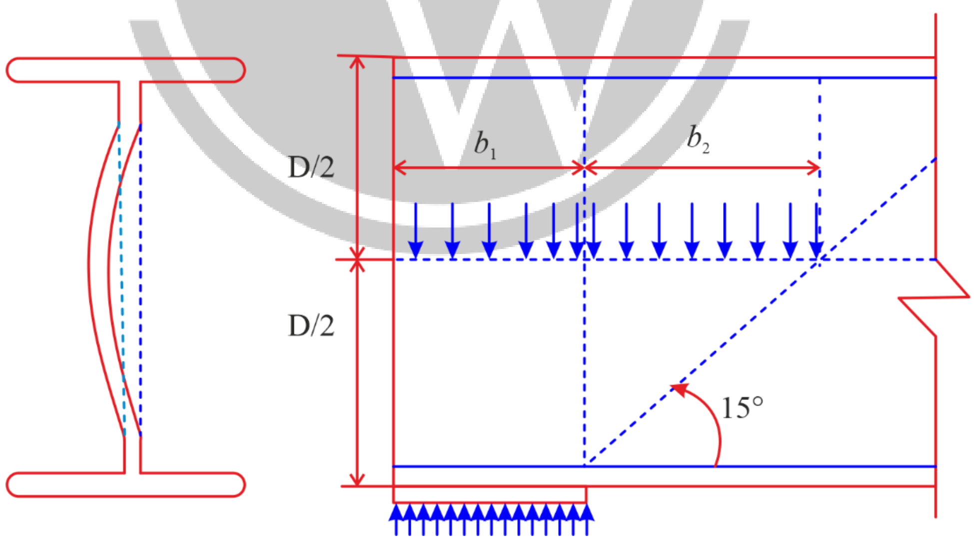
## 8.7. Web Buckling

- The web in rolled steel section behaves like a column when placed under concentrated loads. The web is await thin therefore subjected to buckling stress for the web acting as a column. The buckling of coloumn is much influenced by the restraints provided for the flanges.
- Vertical buckling of web is not a problem with rolled beam sections, this possibility exits in the thin webs of deep plate girder.
- The maximum diagonal compression occurs at NA will be inclined to 45° to it The web buckling strength of support will be

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

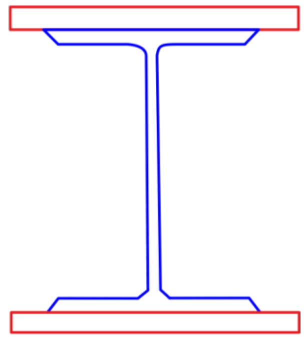
Fcd = Allowable compressive tress corresponding to the announced web strut according to buckling curve 'C

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




## 8.8. Built – up beams (Plated Beams)

- Used when long spans and heavy loads, large B.M are generated when available, rolled beam sections don't provide sufficient strength to resist the external B.M.
- When the depth of the beam may be restricted due to head room requirement



Built - up beam

$I_a = I_{p\ req} - I$ (Divide by 'h/2)

$$\frac{I}{h/2} = \frac{I_{p\ req}}{h/2} - \frac{I}{h/2}$$

$$Z_a = Z_{p.req} - Z_p;$$

$$Z_a = A_a.h \Rightarrow A_a = Z_a / h$$

h – Overall depth of I – section

$Z_p$ = plastic modulus of I – section = $M_{fy}$

M = factored bending moment

$Z_a$ = Plastic section modulus of plates required

$I_{p\ req}$ = plastic moment of inertia of the section to resist the total bending moment

I = Moment of inertia of the desirable beam section available

$I_a$ = Additional moment of inertia of the section required from plate section

M.I of plates to be calculated

$$A_a = B \times t_{fp}.$$

B = Width of flange cover plate

$T_{fp}$ = Thickness of flange cover plate

$M_{cr}$ = Elastic critical moment corresponding to lateral – torsional buckling of beam.

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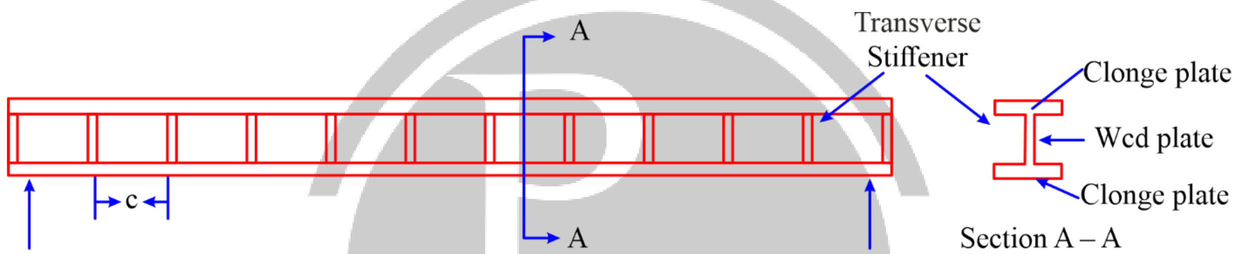
# 9 PLATE GIRDER

## 9.1. Introduction

A fabricated plate girder is employed for supporting heavy loads over long spans. The bending moments and shear forces produced in such girders are well beyond the bending and shear resistance of rolled steel girders available. In such situations the designer has the choice of one the following solutions:




- Use two or more regularly available sections, side-by-side. (my not satisfy deflection limitation)
- Use a fabricated plate girder, (within limits) to choose the size of web and flanges, or use a steel truss

## 9.2. Elements of Plate Girder

- Web plate
- Flange angles with or without flange plates for bolted/riveted plate girder.
- Stiffeners
  - Vertical or transverse or stability stiffeners
  - Horizontal or longitudinal stiffeners
  - Load or end bearing stiffeners
- Splices
  - Flange splice
  - Web splice

## 9.3. Economical Depth of Plate Girder

$$d = [M\,k/f_y]^{13} \quad (\text{where } k - d/t_w)$$

Where, d – depth of the web and

$t_w$ – thickness of web

If $d/t_w \leq 67\ \varepsilon_w$; (may be designed as ordinary beam)

[where $\varepsilon_w = \sqrt{(250/f_{yw})}$]

$f_{yw}$ – yield stress of wed

## 9.5. Minimum Thickness of Web Plate

Minimum web plate thickness should meet serviceability and compression flange bucking criterion.

Minimum web plate thickness based on serviceability requirement

(a) When transverse stiffeners are not provided,

- $d/t_w \leq 200\ \varepsilon$ (Web connected to flanges along both longitudinal edges)
- $d/t_w \leq 90\ \varepsilon$ (Web connected to flanges along both longitudinal edge only)

(b) When transverse stiffeners are provided,

- $d/t_w \leq 200\ \varepsilon_w$ for $3d \geq c \geq d$
- $c/t_w \leq 200\ \varepsilon_w$ for $0.74\ d \leq c \leq d$
- $d/t_w \leq 200\ \varepsilon_w$ for $c \leq 0.74\ d$

(C – spacing of transverse stiffener)

(c) When transverse stiffeners and longitudinal stiffeners at one level only provided (0.2d from compression flange)

- $d/t_w \leq 250\ \varepsilon_w$ for $2.4\ d \geq c \geq d$
- $c/t_w \leq 250\ \varepsilon_w$ for $0.74\ d \leq c \leq d$
- $d/t_w \leq 340\ \varepsilon_w$ for $c \leq 0.74\ d$

(d) When transverse stiffeners and one longitudinal stiffener provided one at 0.2 d from compression flange and second longitudinal stiffener at N. A

- $d/t_w \leq 250\ \varepsilon_w$ Where $\varepsilon_w = \sqrt{250/fy}$

## 9.6. Minimum Web Thickness based on Compression Flange Bucking Requirement

(a) When transverse stiffeners are not provided

- $d/t_w \leq 345\ \varepsilon_f^2$

[Where $\varepsilon_f = \sqrt{(250/f_{yf}}$]

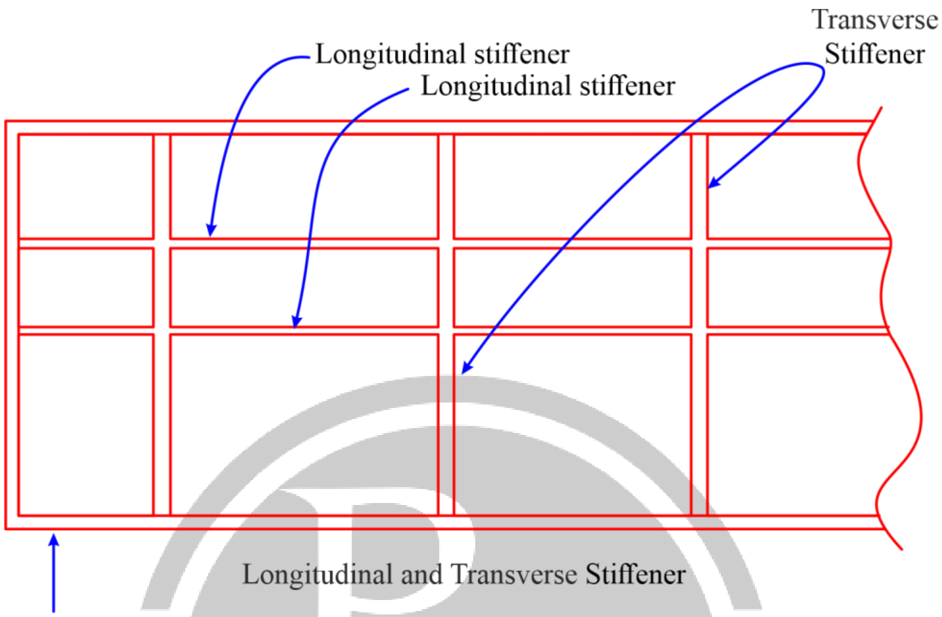
(b) When transverse stiffeners are provided

- $d/t_w \leq 345\ \varepsilon_f^2$ for $c \geq 1.5\ d$
- $d/t_w \leq 345\ \varepsilon_f$ for $c \geq 1.5\ d$

## 9.7. Stiffeners

### (a) Transverse (stability or vertical) stiffener

- Intermediate transverse stiffeners increase buckling resistance of web against shear




Longitudinal and Transverse Stiffener

- Angle section are provided for riveted or bolted plate girders and flat or plate section for welded plate girders
- Spacing of intermediate stiffeners depends on thickness of web
- To avoid local buckling of transverse stiffener, outstand from face of web should not exceed 20 $t_q$ ($t_q$ – Thickness of stiffener)
- Minimum moment of inertia of transverse stiffener

  If $c/d \geq \sqrt{2}$ $I_s \geq 0.75\ d\ t_w^3$

  If $c/d \geq \sqrt{2}$ $I_s \geq 1.5\ d^3\ t_w^3 / c^2$

- Intermediate stiffener not subjected to external loading should connected to the web so as withstand to shear (in kN/mm) of not less than $t_w^2/5\ b_s$ kN/mm ($b_s$– outstand of stiffener)

### (b) Longitudinal (Horizonal) Stiffener

- Longitudinal stiffener increases bucking resistance of web against bending
- These are provided between transverse stiffener
- Minimum moment of inertia of first longitudinal stiffener provided at 0.2d from compression flange $I \geq c\,t_w^3$
- Minimum moment of inertia of second Longitudinal stiffener provided at N.A
- $I \geq d_2\, t_w^3$ ($d_2$ – 2 × distance from compression flange to N.A)

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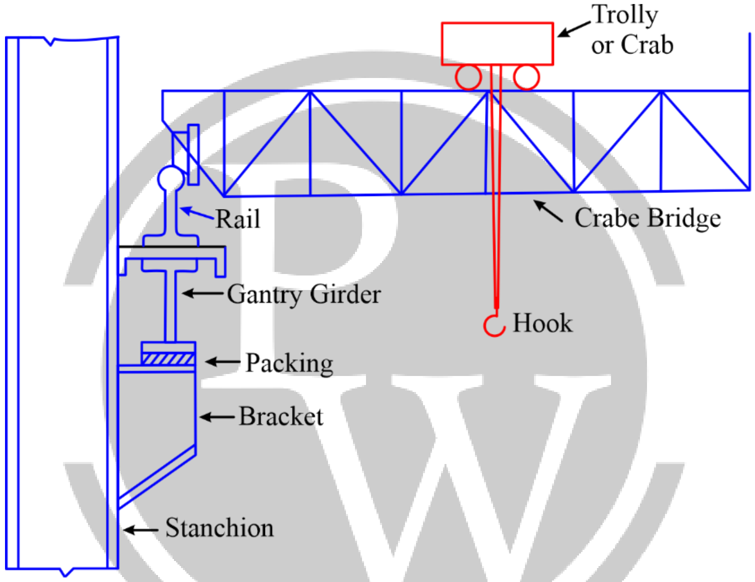
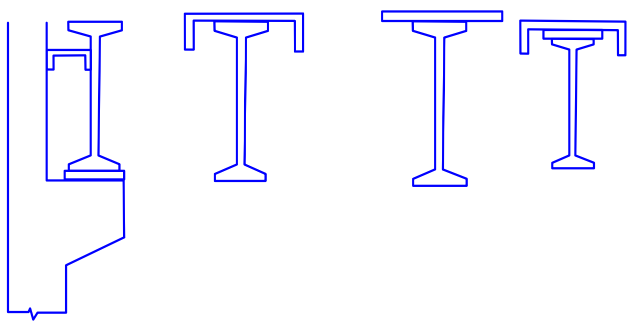
# 10 GRANTY GIRDER

## 10.1. Gantry Girders

1. These are laterally unsupported beams provided in industrial building to carry cranes.

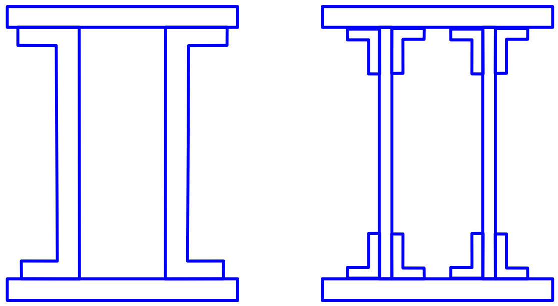



2. Gantry girders are subjected to vertical, lateral and longitudinal
   - The lateral thrusts are caused due to sudden stopping or starting of the crab and these lateral forces act normal of the crab and these lateral forces act normal to the rails.
   - Longitudinal loads are caused due to stopping or starting of the crane girder, and produce a thrust along the rails.
3. Section for a Gantry Girder



- The compression flange of an I section is reinforced with a channel to increase the lateral stability.

- When greater laterally stability and torsional Rigidity are required in case of heavy lateral forces, a box type of girder is adopted.



4. As the loads are applied on the gantry girder suddenly, additional stresses are induced in the girder due to the impact effect.

**Additional loads for structures subjected to impact:**

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

## 10.2. Limiting Deflections

The vertical deflection of a gantry girder should not exceed the values specified below:

- Where the cranes are manually operated $L/500$
- Where the cranes are overhead traveling and operated electrically up to 500 kN $L/750$
- Where the cranes are overhead traveling and operated electrically over 500 kN $L/1000$
- Other moving loads, such as charging cars, etc. $L/600$

Where $L$ = Span of the gantry girder.

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# 11 ROOF TRUSSES

## 11.1. Introduction

1. All joints are assumed to be hinged and loads applied at the joints only.

2. All members of a roof trusses are subjected to axial forces

$$\text{Pitch} = \frac{\text{Rise}}{\text{Span}} \quad \text{Slope} = \frac{\text{Rise}}{\frac{(\text{Span})}{2}}$$

$\therefore$ $$\text{Slop} = 2 \times \text{pitch}$$

3. Pitch depends on type of roofing material, ventilation and light requirement

Small Pitch : <1/12

Medium Pitch : 1/12 to 1/5

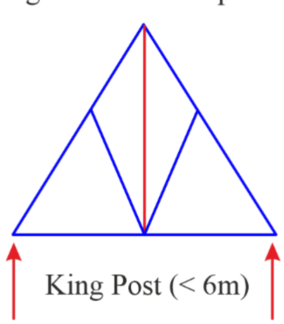
Large Pitch : >1/12

4. Common pitches for different roof sheets.

Pitch 1/6 for G I Sheets; 1/12 for AC sheets; ¼ for snow load with wind load

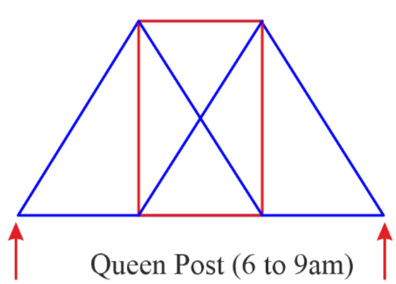
## 11.2. Types of Trusses

King Post Truss is upto 6m;



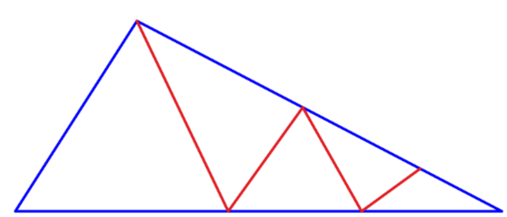
King Post (< 6m)

Pratt truss is 6 to 30m;

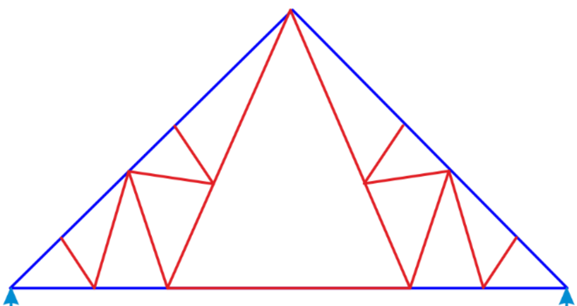


Queen Post (6 to 9am)

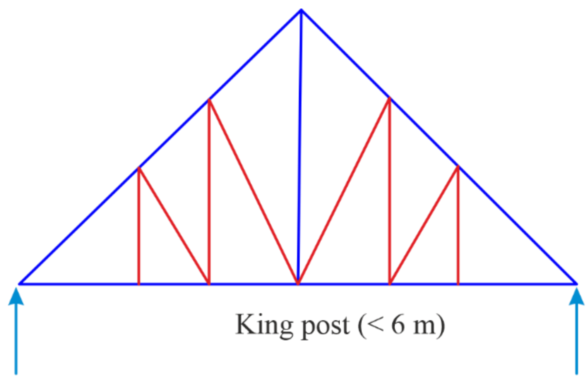
Queen post Truss is 6 to 9m



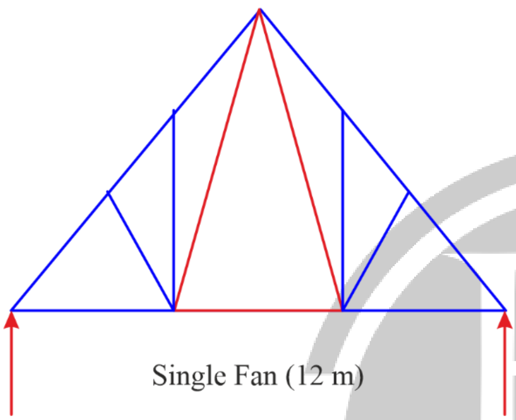
Saw tooth truss (5 to 8m)



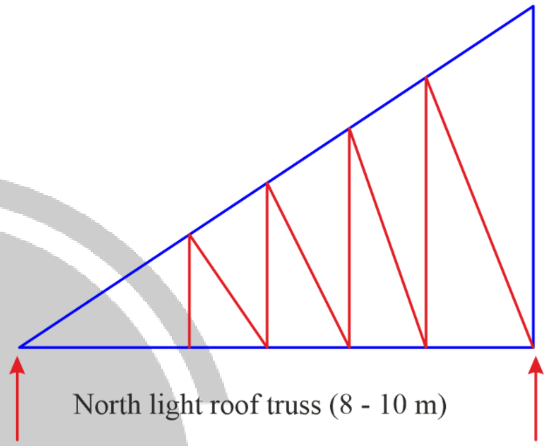
Compound Fink (10 - 20 m)
Or French truss



King post (< 6 m)

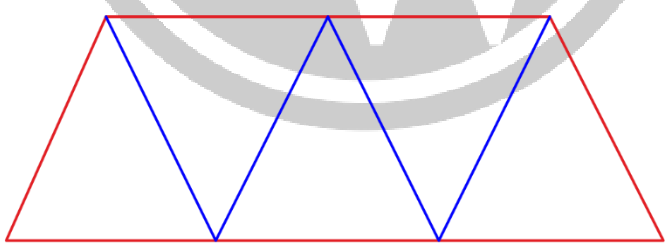


Single Fan (12 m)

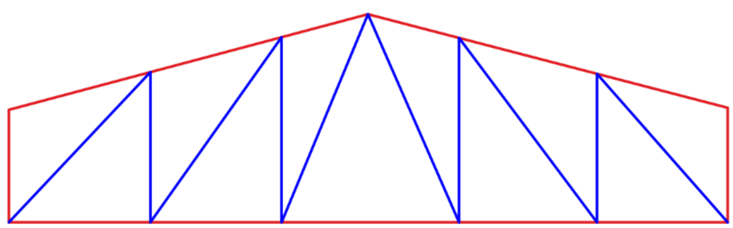


North light roof truss (8 - 10 m)

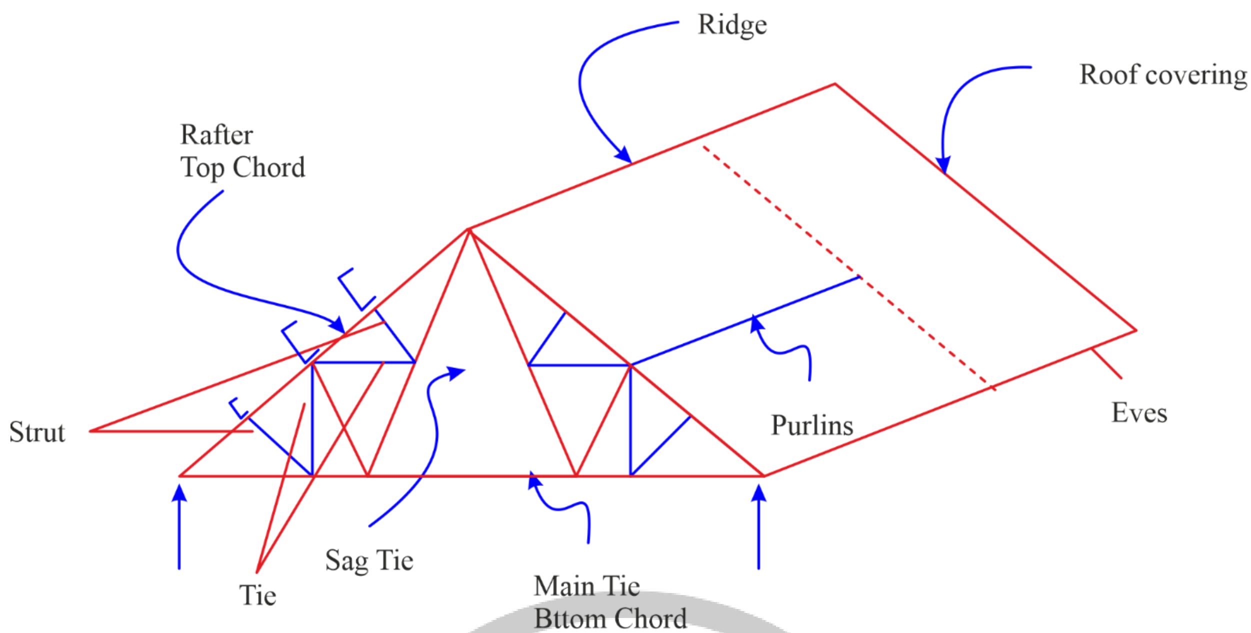
- The type of truss depends on span and pitch
- Warren truss is used for small pitch.
- Pratt truss is used for medium pitch.
- Fink truss is used for large pitch.



Warren truss



Pratt truss




**Spacing of Roof Trusses:**

Economical spacing 1/3 to 1/5 of span. For economical spacing, cost of trusses = 2 × cost of purlins + cost of covering

## 11.3. Elements of Roof Truss

**Principle Rafter**

The top chord members of a roof truss are called 'Principal rafters'. They support the purlins. They are mainly compression members and may be subjected to shear and bending moment if the purlins are not placed at nodal points.

- **Struts:** The member carrying compressive forces in a roof truss are called struts.
- **Ties:** Members carrying tensile forces.
- **Main Tie:** The bottom chord member. It is usually in tension and takes compression if reversal of loads occurs due to wind load.

  **Principle rafter:** The top chord members of a roof truss are called 'Principal rafters'. They support the purlins. They are mainly compression members and may be subjected to shear and bending moment if the purlins are not placed at nodal points.
- **Ridge line and eves:** The top line of the roof truss is called the ridge line and the bottom edge of roof surface is called eves.
- **Purlins:** These are Members subjected to transverse loads and rest on the rafters of roof trusses. They support sheetings that carries roof covering. They are horizontal beams spanning between the two adjacent trusses. Spacing generally varies from 2m to 3m.
- **Sag tie:** To reduce deflection and moment due to self weight.
- **Common Rafter:** These are provided only if the spacing of purlins are larger than the available lengths of sheeting. Rafters are inclined beams supported on the purlins.

**Shoe Angle:** It is the supporting angle provided at the junction of top and bottom chords of truss. The reaction of the truss is transferred to the column through the shoe angle. It is supported on the base plate.

- **Base Plate & Anchor Bolts:** Base plate supports the shoe angle of the truss through bolts. Base plate is anchored to the column or wall through anchor bolts. Anchor bolts can take both downward as well as upward reactions from the truss.
- **Bay:** It is the distance between adjacent trusses.
- **Rise:** It is the distance from the highest point to the line joining supports.
- **Sag Rods for Purlins:** Sag rods are installed in the plane of slope connecting the purlins. The sag rods reduce the span for bending about the weaker axis (XY) caused by the tangential components of the load. Generally two lines of sag rods are provided in each bay which are connected to the ridge purlins.
- **Lateral bracing of end trusses:** Bracing is required to resist horizontal loading (such as that due to wind etc). The Bracing for roof trusses and supporting columns provide stiff rigid structure. Bracing is provided to the last two trusses on either side of the shed both at top chord and bottom chord levels. Similarly, the last two supporting columns at either end are to be vertically braced.
- Bracings are not required if the truss is supported on masonry walls and if strong end gable walls and if strong end gable walls are provided.
- The bracings are provided in such a manner that their diagonals form angles about 45° with the load to be carried.

## 11.4. Loads on Roof Trusses:

Dead Load, Live Load, Wind Load and Snow Loads

**(a) Dead Load:**

(i) Roof Covering

(ii) Purlins weight

(iii) Self weight truss 100 to 150 N/m$^2$ on plan area

**(b) Imposed load (L.L):** For slopes ≤10°, live load is 1500 N/m$^2$ if access is provided. 750 N/m$^2$ if access is not provided.

**For roofs sloping > 10°**

(i) For roof membrane, sheets or purlins: 750 N/m$^2$ less 20 N/m$^2$ for every degree increase in slope over 10° subjected to a minimum of 400 N/m$^2$.

(ii) For member supporting the roof members and roof purlins, such as trusses, beams, girder etc, 2/3 of load in (a).

(iii) Self weight truss 100 to 150 N/m$^2$ on plan area

**(c) Snow load:** 25 N/m$^2$ per cm depth of snow.

If > 50° slope, snow load need not be considered

**(d) Wind load:** Design wind pressure,

$$P_Z = 0.6\,V_z^2$$

$P_z$ = design wind velocity in m/sec at a height, z

$V_z = k_1 \cdot k_2 \cdot k_3 \cdot V_b$

$V_b$ = basic wind speed in m/s at a height 10 m at the locality. It is to be obtained from the code. The code divides the country into six zones for wind velocity calculations.

$k_1$ = probability or risk factor

$k_2$ = terrain, height and structure size factor.

$k_3$ = topography factor.

The design wind pressure on a roof is determined by combination of external wind pressure and internal wind pressure.

**(i) External wind pressure:** It depends on slope. The external wind pressure in term of basic wind pressure 'p' on roofs when wind is normal to ridge is as follows.

**(ii) Internal wind pressure:** It depends upon permeability of the structure. For different permeability of buildings, the internal air pressure in terms of basic wind pressure 'P' is given below.

| Type of Buildings | Internal Pressure |
|---|---|
| 1. Nero permeability, no openings (Multistoried building with panel wall sand no opening) | 0 |
| 2. Normal permeability (upto 5%)<br>(Flow of air commonly afforded by structure through open windows and doors). | ±0.2 P |
| 3. Medium openings (5% - 20% openings) | ±0.5 P |
| 4. Large opening (Area of opening > 20% of total wall area) (Hangers and sheds) | ±0.7 P |

## 11.5. Design of Purlin

- Purlins are beams provided over trusses to support roofing between adjacent trusses. Channels and angles sections are commonly used as purlins.
- Wind forces is assumed to act normal to roof truss and gravity loads pass through the centre of gravity of the purlin section hence purlin sections is subjected to twisting, in addition to bending such bending is called unsymmetrical bending.
- Purlin may be designed as a continuous or simply supported beam. IS 800 recommends the purlins to be designed as continuous beam subjected to bi-axial bending

## Design Procedure

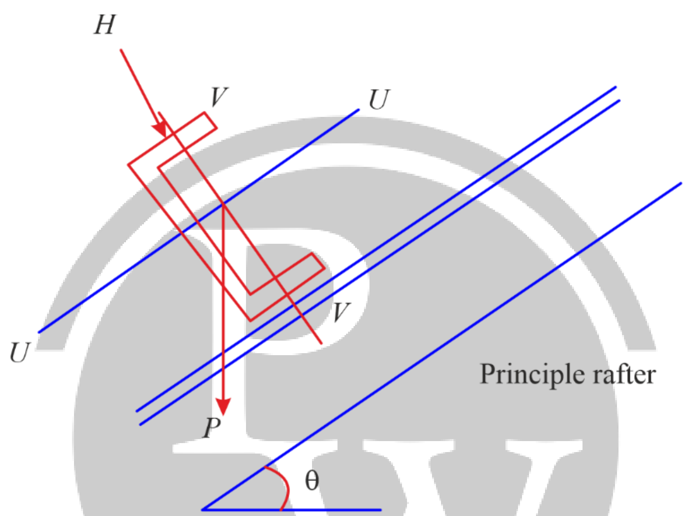
- The gravity load $P_1$ due to sheeting and live load etc.

  Load etc to wind $H_1$ and hence loads are multiplied by load factors

  P = Factored along V-V axis = $\gamma f (P_1 \cos\theta + H)$ in kN

- Maximum bending moments

$$M_{UU} = \frac{Pl}{10} \quad M_{VV} = \frac{Hl}{10}$$




$l$ = Span of the purlin c/c distance between adjacent trusses

- Purlin are subjected to bi-axial bending and require trail and error method for their design
- Design capacities of the section

$$M_{dz} = Z_{pz} \cdot \frac{f_y}{\gamma_{mo}} \le 1.2\, Z_{ez} \cdot \frac{f_y}{\gamma_{mo}}$$

$$M_{dy} = Z_{py} \cdot \frac{f_y}{\gamma_{mo}} \le \gamma_f \cdot Z_{ey} \cdot \frac{f_y}{\gamma_{mo}}$$

(If $\frac{Z_p}{Z_y} > 1.2$ then $\gamma_f$ is used)

For safety $M_{dz}$ $M_{zz}(M_{uu})$ & $M_{dy} < M_{yy}$ $(M_{vv})$

(i) The local capacity of the section is checked using following interaction equation $\frac{M_{zz}}{M_{dz}} + \frac{M_{yy}}{M_{dy}} \le 1.0$

(ii) The deflection of purlin is calculated which should be less than deflection limit. (Span of purlin/180 for brittle cladding and span or purlin/150 for elastic cladding)

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