THEORY & OBJECTIVE

DESIGN OF STEEL STRUCTURES

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STEEL STRUCTURES INTRODUCTION

THEORY

1.1 OBJECTIVE OF DESIGN

The objective of design is the achievement of an acceptable probability that structures will perform satisfactorily for the intended purpose during the design life. They should sustain all the loads and deformations during construction and use and have adequate resistance to accidental loads and fire with an appropriate degree of safety.

1.2 METHODS OF DESIGN

1.2.1 Working Stress Method as per IS 800 : 1984

The stresses used in practical design are termed as working stresses or safe working stresses. These should never exceed the permissible stresses listed in table.

The permissible stresses are some fraction of the yield stress of the material.

It is defined as the ratio of the yield stress to the factor of safety.

Permissible Stresses in Steel Structural Members

S.No.	Types of stress	Notation	Permissible Stress (MPa)	Factor of Safety
1.	Axial tensile stress	σ_{at}	$0.6\ f_y$	1.67
2.	Maximum axial compressive stress	σ_{ac}	$0.6\ f_y$	1.67
3.	Bending tensile stress	σ_{bt}	$0.66\ f_y$	1.515
4.	Maximum bending compressive stress	σ_{bc}	$0.66\ f_y$	1.515
5.	Average shear stress	τ_{va}	$0.4\ f_y$	2.5
6.	Maximum shear stress	τ_{vm}	$0.45\ f_y$	2.22
7.	Bearing stress	σ_p	$0.75\ f_y$	1.33
8.	Stress in slab base	σ_{bs}	185	–

1.2.2 Limit State Method as per IS 800 : 2007

Limit State method should be used to design structure and its elements as per IS 800 : 2007. The design strength is the ultimate strength. Where the limit state method cannot be conveniently adopted, working stress method can be used.

1.3 Loads and Forces

For the purpose of designing any element, member or a structure, the following loads and their effects shall be taken into account, where applicable, with partial safety factors and combinations :

- Dead loads; *[DL]*
- Imposed loads; (Live load, crane load, snow load etc.) *[IL]*
- Wind loads *[WL]*
- Earthquake loads *[EL]*
- Erection loads *[ER]*
- Accidental loads such as those due to blast *[AL]*
- Secondary effects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components, eccentric connections.

1.3.1 Load Combinations

The following load combinations with appropriate load factors may be considered in designing

- Dead load + Imposed load
- Dead load + Imposed load + Wind or Earthquake load
- Dead load + Wind or Earthquake load
- Dead load + Erection load

1.4 Basis of Classification of Cross Sections as per IS 800-2007

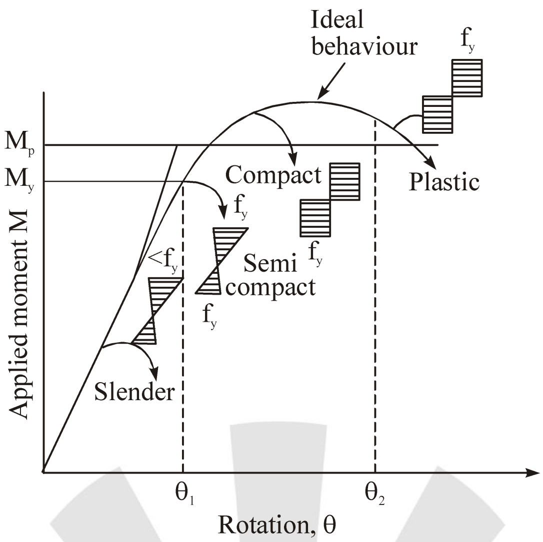
When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling to enable the redistribution of bending moment required before formation of failure mechanism.

The plate elements of a cross-section may buckle locally due to compressive stresses.

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

On the above basis, four classes of sections are defined as follows:

- **Semi-compact :** Cross-sections, in which the extreme fibre in compression can reach, yield stress, but cannot develop the plastic moment of resistance, due to local buckling.
- **Slender :** Cross-sections in which the elements buckle locally even before reaching yield stress.



Moment-rotation behaviour of the four classes of cross-sections.

- **Plastic :** Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism.
- **Compact:** Cross-sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism.

1.4.1 Geometric Properties of Cross-section

IS 800-2007 gives the concept of the gross and effective cross-sections of a member.

- The properties of the gross cross-section shall be calculated from the specified size of the member or read from appropriate table.
- The effective cross-section of a member is that portion of the gross cross-section that is effective in resisting the stresses.

1.5 Basis for Limit State Design of Steel Structures

In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. It shall also satisfy the serviceability requirements, such as limitations of deflection and vibrations and shall not collapse under accidental loads such as from explosions or impact or due to consequences of human error to an extent not originally expected to occur.

The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. The objective of design is to achieve a structure that will not become unfit for use with an acceptable target reliability.

In other words, the probability of a limit state being reached during its lifetime should be very low. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

1.6 Limit State Design Classifications

Limit states are the states beyond which the structure no longer satisfies the performance requirements specified. The limit states are classified as :

- Limit State of Strength
- Limit State of Serviceability

Deflection Limit as per IS 800:2007

Type of building	Deflection	Design Load	Member	Supporting	Maximum deflection
Individual Buildings	Vertical	Live Load / Wind load	Purlins and Girts	Elastic cladding Brittle cladding	Span/150 Span/180
		Live load	Simple span	Elastic cladding Brittle cladding	Span/240 Span/300
		Live load	Cantilever span	Elastic cladding Brittle cladding	Span/120 Span/150
		Live load / Wind load	Rafter supporting	Profiled metal sheeting Plastered sheeting	Span/180 Span/240
		Crane load (manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation over 50t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50t)	Gantry	Crane	Span/1000
	Lateral	No cranes	Column	Elastic cladding Masonry/Brittle cladding Crane (absolute)	Height/150 Height/240 Span/400
		Crane + wind	Gantry (lateral)	Relative displacement between rails supporting crane	10 mm
		Crane + wind	Column/frame	Gantry (Elastic cladding, pendent operated) Gantry (Brittle cladding; cab operated)	Height/200 Height/400
Other Buildings	Vertical	Live load	Floor and roof	Elements not susceptible to cracking Elements susceptible to cracking	Span/300 Span/360
		Live load	Cantilever	Elements not susceptible to cracking Elements susceptible to cracking	Span/150 Span/180
	Lateral	Wind	Building	Elastic cladding Brittle cladding	Height/300 Height/500
		Wind	Inter storey drift		Storey height / 300

Deflections are to be checked for the most adverse but realistic combination of service loads and their arrangement, by elastic analysis, using a load factor of 1.0.

Where the deflection due to dead load plus live load combination is likely to be excessive, consideration should be given to pre-camber the beams, trusses and girders. Generally for spans greater than 25 m, camber approximately equal to the deflection due to dead loads plus half the liveload, may be used.

Section 13 (As per IS 800) of code gives the guidelines for fatigue design but does not consider the effect of following,

- Corrosion fatigue
- low cycle (high stress) fatigue
- Thermal fatigue
- stress corrosion cracking
- effect of high temperature
- effect of low temperature

Code states that fatigue assessment is not normally required for building structures except in the following members.

- Those supporting lifting or rolling loads
- Those subjected to repeated stress cycles from vibrating machine
- These subjected to wind induced oscillations for a large number of cycles in life
- Those subjected to crowd induced oscillations of a large number of cycles in life.

For the purpose of design against fatigue, code classifies different details (of members and connections) under different fatigue classes.

1.7.5 Brittle Fracture

As with fatigue, brittle fracture will rarely occur in building constructions. Such fracture is the sudden failure of the material under service condition, caused by low temperature of sudden change in stress.

Since thick material is more prone to brittle fracture than thin material, limiting thickness are often prescribed by the codes for the various members.

❑❑❑

CONNECTIONS

THEORY

2.1 Basis of Design

Connections (or structural joints) may be classified according to the following parameters:

- Method of fastening such as rivets, bolts, and welding — connections using bolts are further classified as bearing or friction type connections
- Connection rigidity — Simple, rigid (so that the forces produced in the members may be obtained by using an indeterminate structural analysis), or semi-rigid

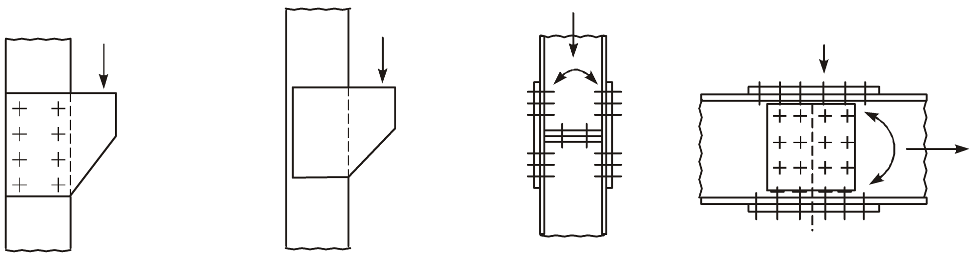
It is desirable to avoid connection failure before member failure due to the following reasons.

- A connection failure may lead to a catastrophic failure of the whole structure.
- Normally, a connection failure is not as ductile as that of a steel member failure.
- For achieving an economical design, it is important that connectors develop full or a little extra strength then the members that it is joining.

According to the IS code, based on connection rigidity, the joints can be defined as follows:

2.1.1 Rigid Connections

Rigid connections develop the full moment capacity of connecting members and retain the original angle between the members under any joint rotation, that is rotational movement of the joint will be very small on these connections.

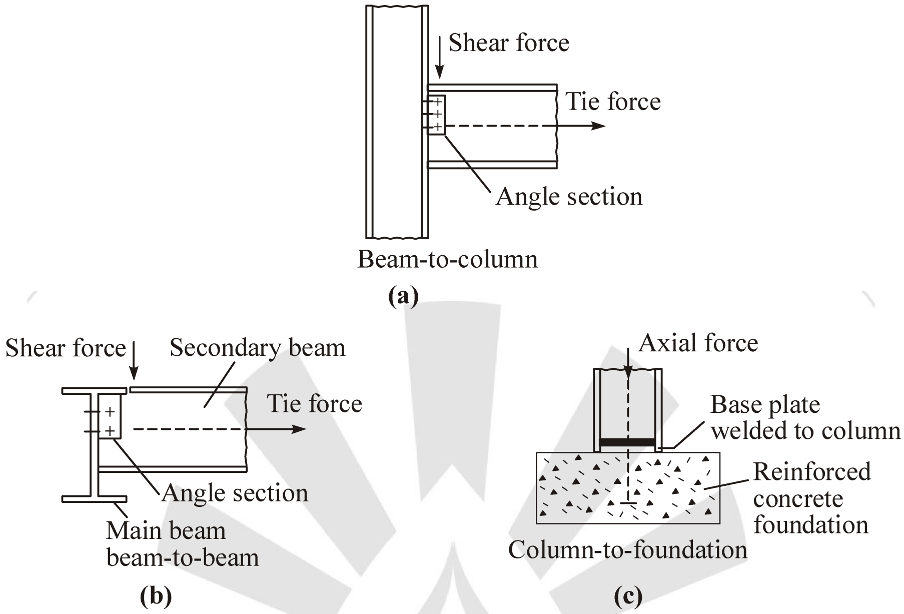


(a) (b) (c) (d)

Example of 'rigid' connections (Martin and Purkiss 1992)

2.1.2 Simple Connections

In simple connections no moment transfer is assumed between the connected parts and hence are assumed as hinged (pinned).



Examples of 'pinned' connections (Martin and Purkiss 1992)

- The rotational movement of the joint will be large in this case. Actually, a small amount of moment will be developed but is normally ignored in the design. Any joint eccentricity less than about 60 mm is neglected.

2.1.3 Semi-Rigid Connections

Semi-rigid connections may have sufficient rigidity to hold the original angles between the members and develop less than the full moment capacity of the connected member.

- In reality, all the connections will be semi-rigid. However, for convenience we assume some of the them as rigid and some as hinge.

2.2 Connection

The following three types of connections may be made in steel structures :

- Riveted
- Bolted
- Welded

2.2.1 Riveted Connections

Riveting is a method of joining together pieces of metal by inserting ductile metal pins called rivets into holes of pieces to be connected and forming a head at the end of the rivet to prevent each metal piece from coming out.

- Rivet holes are made in the structural members to be connected by punching or by drilling. The size of rivet hole is kept slightly more (1.5 to 2.0 mm) than the size of rivet.

- After the rivet holes in the members are matched, a red hot rivet is inserted which has a shop made head on one side and the length of which is slightly more than the combined thicknesses of the members to be connected.
- Then holding red hot rivet at shop head end, hammering is made.
- It results in to expansion of the rivet to completely fill up the rivet hole and also into formation of driven head.
- Desired shapes can be given to the driven head.
- The riveting is done may be in the workshops or in the field.

Riveting has the following disadvantages :

- High level of noise pollution.
- Needs heating the rivet to red hot.
- Inspection of connection is a skilled work.
- Removing poorly installed rivets is costly.
- High labour cost

Production of weldable quality steel and introduction of high strength friction grip bolts have replaced use of rivets.

Design procedure for riveted connections is same as that for bolted connection except that the effective diameter of rivets may be taken as rivet hole diameter instead of nominal diameter of rivet

IS 800-2007 do not discuss riveted connection, it is consider in IS 800:1984

2.2.2 Bolted Connections

A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut.

Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the nut at the thread ends.

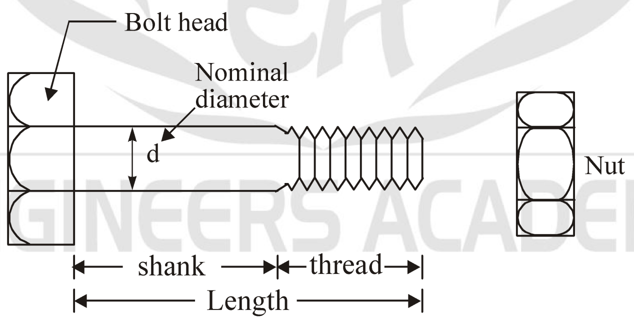


Fig. : Bolt and Nut

2.3 Classification of Bolts

Bolts are classified as :

- Unfinished (black) bolts
- Finished (turned) bolts
- High strength friction grip (HSFG) bolts

OBJECTIVE SHEET

1. Factor of safety adopted by IS: 800-1984 while arriving at the permissible stress in axial compression is

(a) 2.00 (b) 1.00

(c) 1.67 (d) 1.50

2. Maximum size of a fillet weld for a plate of square edge is

(a) 1.5 mm less than the thickness of the plate

(b) one half of the thickness of the plate

(c) thickness of the plate itself

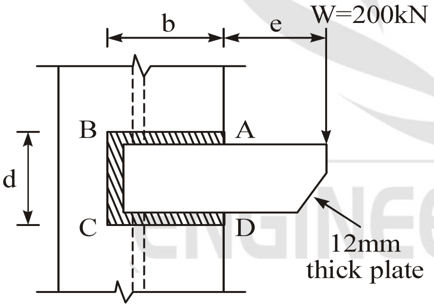
(d) 1.5 mm more than the thickness of the plate

3. In section, shear centre is a point through which, if the resultant load passes, the section will not be subjected to any

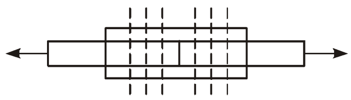
(a) Bending (b) Tension

(c) Compression (d) Torsion

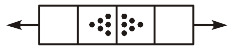
4. A 12mm bracket plate is connected to a column flange as shown in the figure below. The bracket transmits a load of W = 200 kN to the column flange. A 10mm fillet weld is provided along AB, BC and CD. If e = 350 mm, b = 200 mm and d = 600mm, verify if the size of the weld provided is adequate. Allowable shearing stress in the fillet weld can be taken to be 108 MPa.

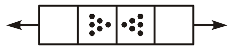


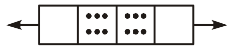
5. Identify the most efficient but joint (with double cover plates) for a plate in tension from the patterns (plan views) shown below, each comprising 6 identical bolts with the same pitch and gauge.

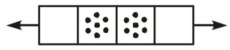


Common elevation

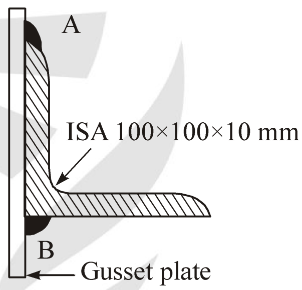
(a) 

(b) 

(c) 

(d) 

6. ISA 100 × 100 × 10mm (Cross sectional area = 1908mm^2) is welded along A and B (Refer to figure in the below question) such that the lengths of the weld along A and B are l_1 and l_2 respectively. Which of the following is a possibly acceptable combination of l_1 and l_2



(a) l_1 = 60 mm and l_2 = 150 mm

(b) l_1 = 150mm and l_2 = 60mm

(c) l_1 = 150mm and l_2 = 150 mm

(d) Any of the above, depending on the size of the weld

7. Rivet value is defined as

(a) lesser of the bearing strength of rivet and the shearing strength of the rivet

(b) lesser of the bearing strength of rivet and the tearing strength of thinner plate

(c) greater of the bearing strength of rivet and the shearing strength of the rivet

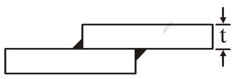
(d) lesser of the shearing strength of the rivet and the tearing strength of thinner plate

ANSWERS AND EXPLANATIONS

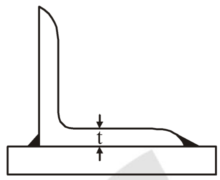
1. ***Ans. (c)***

$\therefore \quad \text{F.O.S} = \dfrac{1}{0.6} = 1.67$

2. ***Ans. (a)***



s_{max} = t – 1.5mm
For square edge

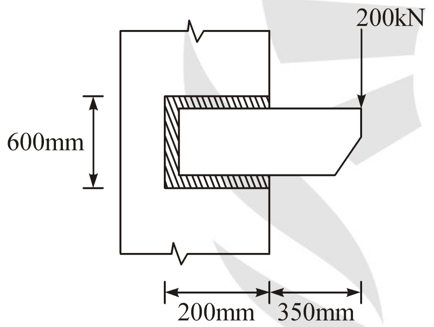


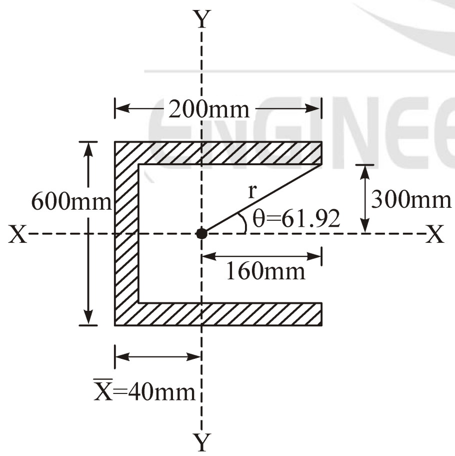
s_{max} = 3/4 × t
For round edge

where, t is thickness of thinner member.

3. ***Ans. (d)***

4. Throat fillet,





t = 0.7 × size = 0.7 × 10 = 7 mm

Let CG of weld be at $\bar{x}$ from left edge as shown

$$\therefore \quad \bar{x} = \frac{(600\times 0)+(2\times 200\times 100)\times 7}{(600+200+200)\times 7}$$

$= 40$ mm

For weld,

$$I_x = \frac{7\times 600^3}{12} + 2\times \left[\frac{200\times 7^3}{12} + 200\times 7\times 300^2\right]$$

$= 378 \times 10^6$ mm^4

For weld, I_y

$$= 2\left[\frac{7\times 200^3}{12} + 200\times 7\times (100-40)^2\right] + \left[\frac{600\times 7^3}{12}\right] + 600\times 7\times 40^2 = 26.15\times 10^6 \text{mm}^4$$

$$I_p = I_x + I_y = 404.15\times 10^6 \text{mm}^4$$

$$p_s = \frac{200\times 10^3}{(600+2\times 200)\times 7}$$

$= 28.57$ N/mm^2

$$p_b = \frac{M.r}{I_p}$$

$$M = (200\times 10^3) * (350+160)$$

$= 102\times 10^6$ N.mm

$$r = \sqrt{300^2+160^2} = 340\text{mm}$$

$$p_b = \frac{M.r}{I_p} = \frac{102\times 10^6\times 340}{404.15\times 10^6}$$

$= 85.8$ M/mm^2

$$p_r = \sqrt{P_s^2 + P_b^2 + 2P_sP_b\cos\theta}$$

3 TENSION MEMBERS

THEORY

- A structural member subjected to two pulling (tensile) forces applied at its ends is called a Tension Member.
- The member and connections are so arranged that eccentricity in the connection and bending stresses on the member are not developed.

3.1 BASIS OF DESIGN

In Working Stress Method

The permissible stress in axial tension on the net effective area of the sections should not exceed 0.60 f_y. the permissible stress in tension has been worked out after applying a factor of safety of 1.65, which is against yielding for tension members for buildings.

$$\sigma_{at} = 0.6\ f_y$$

where σ_{at} = stress in axial tension in MPa

f_y = yield stress of steel in MPa

In Limit State Method

The factored design tension T, in the members shall satisfy the following requirement

$$T < T_d$$

where T_d = Design strength of member.

- The design strength of member under axial tension T_d is the least of the design strength due to yielding of gross section, T_{dg} rupture of critical section, T_{dn} and block shear, T_{db}.

3.2 DESIGN STRENGTH DUE TO YIELDING OF GROSS SECTION

In Limit State Method

The design strength of members under axial tension T_{dg} as governed by yielding of gross-section is given by

$$T_{dg} = f_y\ A_g/\gamma_{m0}$$

where f_y = Yield strength of the material in MPa

A_g = Gross area of cross-section in mm^2

γ_{m0} = Partial safety factor for failure in tension by yielding = 1.10

3.3 Design Strength Due to Rupture of Critical Section

3.3.1 Net Effective Area

It is defined as the gross area (A_g) of the section minus the deductions (A_d) made for any loss of material. Thus. $A_n = A_g - A_d$.

3.3.2 Diameter of Bolt Hole

According to IS 800-2007 diameter of Bolt hole is to be taken as larger than nominal dia. of Bolt. Following table gives the diameter of holes for Bolts.

Clearances for Fastener Holes

S.No.	Nominal Size of Fastener, d mm	Size of the hole =Nominal diameter of the fastener +clearances mm			
		Standard clearance in diameter and width of slot	Over size clearance in diameter	Clearance in the length of the slot	
				Short slot	Long slot
1.	**2.**	**3.**	**4.**	**5.**	**6.**
A.	12-14	1.0	3.0	4.0	2.5 d
B.	16-22	2.0	4.0	6.0	2.5 d
C.	24	2.0	6.0	8.0	2.5 d
D.	Larger than 24	3.0	8.0	10.0	2.5 d

3.3.3 Rupture Strength of A Plate

The design strength in tension of a plate T_{dn} as governed by rupture of net cross-sectional area A_n, at the hole is given by

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

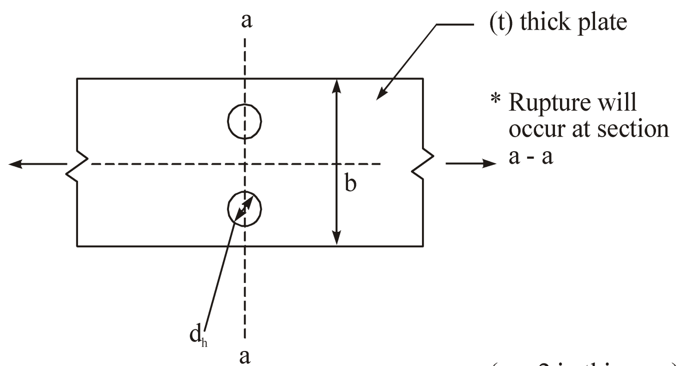
γ_{m1} = Partial safety factor for failure in tension by rupture = 1.25

f_u = Ultimate stress of the material in MPa

A_n = Net effective area of the member

3.3.4 Calculation of Net Effective Area (A_n)

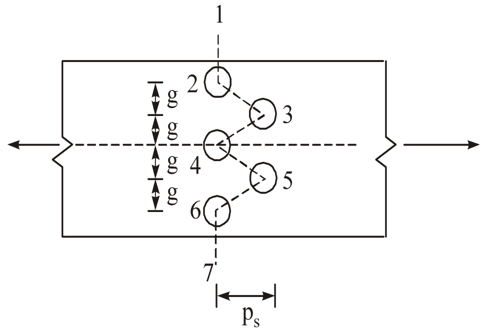
Case-I : Chain bolting



$A_n = (b - n\ d_h)t$

(n = 2 in this case)

Case-II : Zig-Zag bolting



- Rupture can occur along 1–2–3–4–5–6–7

$$A_n = \left[b - nd_h + \sum \frac{P_i^2}{4g_i}\right]t$$

In this case,

$$A_n \text{ in this case} = \left[b - 5d_h + \frac{4p_{si}^2}{4g_i}\right]t$$

where,

d_h = Diameter of Bolt hole

b, t = Width and thickness of plate respectively

n = Number of bolt holes in critical section

p_{si} = Staggered pitch length (parallel to pull)

g_i = Gauge length between bolt holes (perpendicular to pull)

3.4 Strength of Threaded Rods in Rupture

In Limit State Method

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

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

where, A_n = net root area at the threaded section = $0.78 \times \frac{\pi}{4} \times d^2$

d = Nominal diameter of Bolt

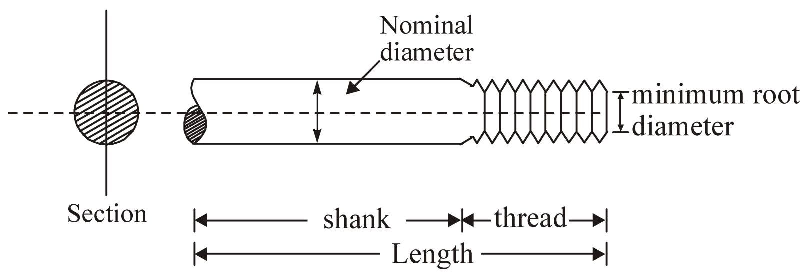


Fig. Threaded Rod