THEORY & OBJECTIVE

STEEL STRUCTURES

By
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Contents

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.

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	–

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.

The concept of introducing a factor of safety is to make the structure safe to the account for the following :

1. The analysis methods are based on assumptions and do not give the exact stresses.
2. Structural members may be temporarily overloaded under certain circumstances.
3. The stresses due to fabrication and erection are not considered in the design of ordinary structures.
4. The secondary stresses may be appreciable.
5. Underestimation of the future live loads.
6. Stress concentrations.
7. Unpredictable natural calamities.

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

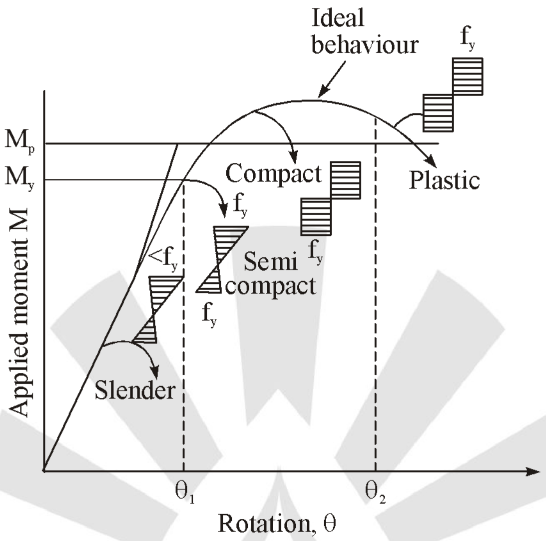
When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling to enable the redistribution of bending moment required before formation of failure mechanism.

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

1.6.1 Limit State of Strength

The limit state of strength are those associated with failures (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure.

1.6.2 Limit State of Serviceability

The limit state of serviceability includes :

- Deformations and deflections
- Vibrations
- Repairable damage due to fatigue
- Corrosion and durability

The major innovation in the limit state method is the introduction of the partial safety factor. Which essentially splits the factor of safety into two factors - one for the material and one for the load.

In accordance with these concepts, 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.

Thus, the design requirements are expressed as follows :

$$f_d \leq S_d$$

where f_d = Value of internal forces and moments caused by factored design loads F_d.

$F_d = \gamma_f \times$ characteristic loads

γ_f = Partial safety factor for load

γ_m = Partial safety factor for material

S_u = Ultimate strength

S_d = Design strength

$$S_u = \gamma_m \times S_d$$

Both the partial factors for load and material are determined on a probabilistic basis of the corresponding quantity. It should be noted that γ_f makes allowance for possible deviation of loads and also the reduced possibility of all loads acting together.

On the otherhand γ_m allows for uncertainties of element behaviour and possible strength reduction due to manufacturing tolerances and imperfections in the materials.

The values of γ_f (Partial safety factors for loads/load factor)

Combination	Limit state of strength				AL	Limit state of Serviceability			
	DL	LL*		WL/EL		DL	LL*		WL/EL
		Leading	Accompanying				Leading	Accompanying	
1.	**2.**	**3.**	**4.**	**5.**	**6.**	**7.**	**8.**	**9.**	**10.**
DL+LL+CL	1.5	1.5	1.05	–	–	1.0	1.0	1.0	–
DL+LL+CL	1.2	1.2	1.05	0.6	–	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2	–	–	–	–	–
DL+WL/EL	1.5(0.9)**	–	–	1.5	–	1.0	–	–	1.0
DL+ER	1.2(0.9)**	1.2	–	–	–	–	–	–	–
DL+LL+AL	1.0	0.35	0.35	–	1.0	–	–	–	–

*When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.

**This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

Abbreviations :

DL = Dead load, AL = Accidental load,

LL = Imposed load (Live loads) ER = Erection load,

WL = Wind load EL = Earthquake load,

CL = Crane load (Vertical/Horizontal)

Values of γ_m (Partial safety factor for materials)

Definition	**Partial Safety Factor**	
Resistance, governed by yielding , γ_{m0}	1.10	
Resistance of member to buckling, γ_{m0}	1.10	
Resistance, governed by ultimate stress , γ_{m1}	1.25	
Resistance of connection :	**Shop Fabrications**	**Field Fabrications**
Bolts-Friction Type γ_{mf}	1.25	1.25
Bolts-Bearing Type γ_{mb}	1.25	1.25
Rivets, γ_{mr}	1.25	1.25
Welds, γ_{mw}	1.25	1.50

After checking the structure for limit state of strength, the structure is then checked for limit state of serviceability.

Collapse / strength limit states are related to the maximum design loads under extreme conditions. The partial load factors are chosen to reflect the probability of extreme conditions, when loads act alone or in combination. Stability shall be ensured for the structure as a whole and for each of its elements. It includes overall frame stability against overturning and sway, uplift or sliding under factored loads.

Serviceability limit states are related to the criteria governing normal use. Hence unfactored loads are used to check the adequacy of the structure. Load factor γ_f of value equal to unity shall be used for all loads leading to serviceability limit states.

1.7 Service Criteria under Limit States of Serviceability

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishing.

Table below gives the limits of deflections for certain structural members systems as recommended by IS 800-2007.

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.

- The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection.
- Roofs, which are very flexible, shall be designed to withstand any additional load that is likely to occur as a result of ponding of water or accumulation of snow.

1.7.1 Vibration

Suitable provisions in the design shall be made for the dynamic effects of live loads impact loads and vibration due to machinery operating loads. In severe cases possibility of resonance, fatigue or unacceptable vibrations shall be investigated.

1.7.2 Durability

Several factors affecting the durability of the buildings, under conditions relevant to their intended life are listed below:

- The environment
- The degree of exposure
- The shape of the member and the structural detailing
- The protective measure
- Ease of maintenance

IS: 800:2007 specifies the requirement of durability and also specifications for different coating system under different exposure conditions.

Five exposure conditions have been specified mild, moderate, severe, very severe and extreme.

"It should be noted that code does not specify any minimum thickness for members. The earlier code 800:1984 specified a minimum thickness of 6 mm for members directly exposed to weather and fully accessible for cleaning and repainting and 8mm for members directly exposed to weather but not accessible for cleaning repainting."

It is assumed that if durability requirements as given in section 15 (IS 800) are followed, it will ensure adequate protection for all thickness.

1.7.3 Fire Resistance

Fire resistance of a steel member is a function of its mass, its geometry, the actions to which it is subjected, its structural support conditions, fire protection measures adopted and the fire to which it is exposed.

1.7.4 Fatigue

Fatigue limit state is important when repeated loading is considered. It is important in case of bridges, crane girders, platform carrying vibrating machines.

As per code, stress changes due to fluctuations in wind loading need not be considered as fatigue. Fatigue failure in normally considered as ultimate limit state but fatigue checks are darned out at working load ($\gamma_f = 1$).

This is because fatigue failure occurs due to large no, of application of loads normally expected to act on the structure (service load).

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.

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2

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

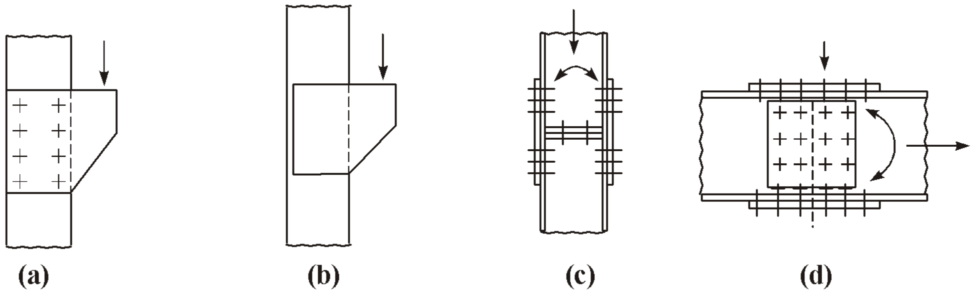
It is desirable to avoid connection failure before member failure due to the following reasons.

- 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

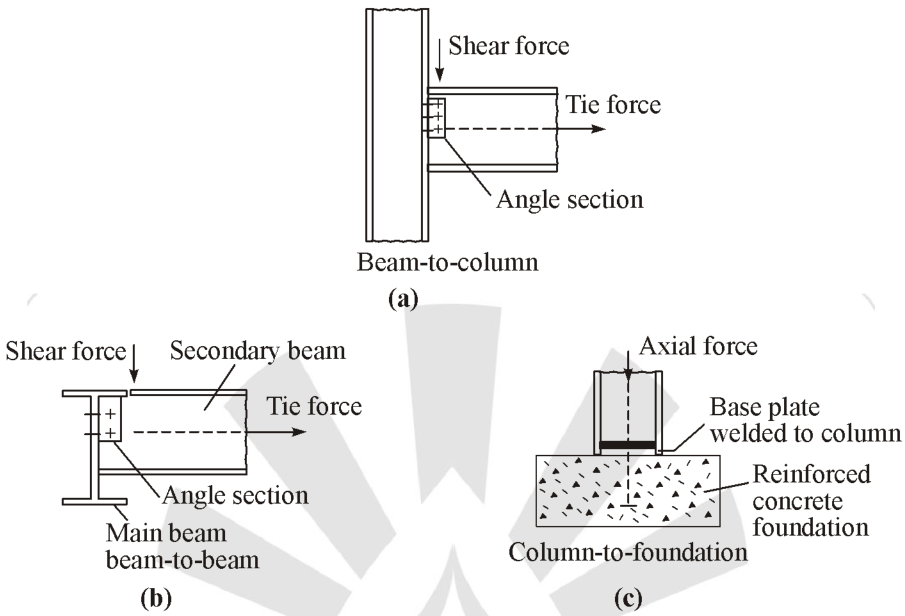
Rigid connections develop the full moment capacity of connecting members and retain the original angle between the members under any joint rotation, that is rotational movement of the joint will be very small on these connections.



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.

- The diameter of the shank is called the Nominal Diameter.
- When the rivets are heated before driving they are called Hot Driven Field Rivets or Hot Driven

$\therefore$ Equivalent stress

$$F_e = \sqrt{f^2 + 3q^2} \le \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

Subject to condition

$$q + f \le \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

- For the purpose of finding the effective depth h required, first depth required for bending only may be found. To take care of shear also, increase this value by about 10% i.e.

$$h' = \sqrt{\frac{6M}{2t\, f_{wd}}}$$

and hence $h = 1.1\, h'$.

Example :

A 6 mm thick angle section is jointed to a 10 mm thick gusset plate. The angle is supporting a load of 55 kN. Find out the number of 16 mm diameter power driven rivets.

Solution :

Nominal diameter of rivet, $\phi = 16$ mm

Gross diameter of rivet, $d = 16 + 1.5 = 17.5$ mm

$$\text{Strength of rivet in single shear} = \frac{\pi}{4}d^2\tau_{vf} = \frac{\pi}{4}(17.5)^2 \times 100 \times 10^{-3} = 24.052 \text{ kN}$$

$t = 6$ mm

$$\text{Strength of rivet in bearing} = dt\,\sigma_{pF}$$
$$= 17.5 \times 6 \times 300$$
$$= 31.50 \text{ kN}$$

The strength of the rivet is the least of the strength in shear and bearing.

Strength of rivet, $R_v = 24.052$ kN

$$\text{Number of rivets, } n = \frac{P}{R_v} = \frac{55}{24.052} = 2.2867 \approx 3$$

Provide three rivets to connect angle with gusset plate.

Example :

A single-riveted double-cover butt joint is used to connect two plates 16 mm thick with chain riveting. The rivets used are power driven 20 mm in diameter at a pitch of 60 mm. Find out the safe load per pitch length and efficiency of the joint.

Solution :

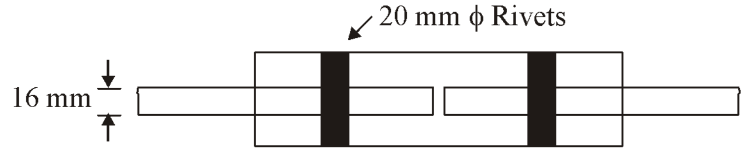
$$\tau_{vf} = 100 \text{ N/mm2}, \sigma_{pf} = 300 \text{ N/mm}^2$$

$$\sigma_{at} = 0.6\, f_y = 150 \text{ MPa (for steel with } f_y = 250 \text{ MPa)}$$

Nominal diameter of rivet, $\phi = 20$ mm

Gross diameter of rivet, d = 20 + 1.5 = 21.5 mm

Pitch, s = 60 mm



Strength of rivet in double shear = $2 \times \frac{\pi}{4} d^2 \tau_{vf}$

$= 2 \times \frac{\pi}{4}(21.5)^2 \times 100 \times 10^{-3}$

= 72.61 kN

Thickness of cover plate = $\frac{5}{8}t$

$= \frac{5}{8} \times 16$ = 10 mm

Let us provide two cover plates each 10 mm thick. The thickness to be considered for calculation purposes will be the thickness of the main plate or the sum of thickness of cover plates, whichever is less.

Strength of rivet in bearing = dt σ_{pf} = 21.5 × 16 × 300 × 10^{-3} = 103.200 kN

Strength of plate in tearing = (s – d)t σ_{at} = (60 – 21.5) ×16×150×10–3

= 92.400 kN

Strength of solid plate per pitch length in shear is the minimum.

Hence, Strength of joint per pitch length = 72.61 kN

Strength of solid plate per pitch length = st σ_{at} = 60 × 16 × 150 × 10^{-3} = 144 kN

$$\text{Efficiency of joint} = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100 = \frac{72.61}{144} \times 100 = 50.42\ \%$$

Example :

Design a riveted joint to connect two plates 14 mm thick. Power driven rivets amy be used for making the connection. Assume F_y = 250 N/mm^2.

Solution :

The joint can be designed as a lap joint or as a butt joint. The design of each type of riveted joint is illustrated by this example. Finally the selection of the type of joint is done on the basis of efficiency.

The diameter of the rivet can be found by using Unwin's formula

$$\phi = 6.01\sqrt{t} = 6.01\sqrt{14} = 22.48\text{mm} \approx 22\text{mm}$$

(As Unwin's formula gives a higher value, provide 22 mm ϕ power driven rivets)

Gross diameter = 22 + 1.5 = 23.5 mm

Lap joint The rivets will be in single shear

$$\text{Strength of rivet in shearing} = \frac{\pi}{4}d^2\tau_{vf} = \frac{\pi}{4}(23.5)^2 \times 100 \times 10^{-3} = 43.373 \text{ kN}$$

$$\text{Strength of rivet in bearing} = dt\,\sigma_{pf} = 23.5 \times 14 \times 300 \times 10^{-3} = 98.700 \text{ kN}$$

$$\text{Strength of rivet/pitch length} = 43.373 \text{ kN}$$

Let s be the pitch of rivet. For the best design, the rivets should be placed at such a way that the strength of rivet per pitch length is equal to the strength of plate in tearing per pitch length

$$\text{Strength of plate in tearing} = (s-d)t\,\sigma_{at} = (s - 23.5) \times 14 \times 0.6\ 0+ 250$$

or $$43.373 \times 10^3 = (s - 23.5) \times 14 \times 250$$

or $$s = 44.15 \text{ mm} \approx 45 \text{ mm}$$

Hence, provide rivets at a pitch of 55 mm.

$$\text{Strength of the plate in tearing} = (55 - 23.5)\times 14 \times 0.6 \times 250 \times 10^{-3} = 66.150 \text{ kN}$$

$$\text{Strength of joint/pitch length} = 43.73 \text{ kN}$$

$$\text{Strength of solid plate/pitch length} = st\,\sigma_{at} = 55 \times 14 \times 0.6 \times 250 \times 10–3$$

$$= 115.500 \text{ kN}$$

$$\text{Efficiency of the joint} = \frac{\text{strength of joint/pitch length}}{\text{strength of solid plate / pitch length}} \times 100$$

$$= \frac{43.373}{115.500} \times 100 = 37.55\ \%$$

Single cover butt joint The rivets will be in single shear.

$$\text{Strength of rivet in shear} = \frac{\pi}{4}d^2\tau_{vf} = \frac{\pi}{4}(23.5)^2 \times 100 \times 10^{-3} = 43.373 \text{ kN}$$

Thickness of cover plate $\not< \frac{5}{8}t$

$$= \frac{5}{8} \times 14 = 8.75 \text{ mm} \approx 10 \text{ mm}$$

The thickness of plate to be considered for bearing of rivets will be = 10 mm.

Strength of rivet in bearing $= dt\,\sigma_{bf} = 23.5 \times 10 \times 300 \times 10^{-3} = 70.500$ kN

Strength of rivet/pitch length = 43.373 kN

Equating the least strength of rivet to the strength of plate per pitch length in

$$43.373 \times 10^3 = (s - 23.5) \times 10 \times 0.6 \times 250$$

$$s = 52.415 \text{ mm}$$

$$\not< (2.5\,\phi = 2.5 \times 22 = 55 \text{ mm})$$

Hence, provide rivets at a pitch of 55 mm.

Strength of the joint/pitch length = 43.73 kN

Strength of solid plate/pitch length = $st\sigma_{at}$

$$= 55 \times 10 \times 0.6 \times 250 \times 10^{-3} = 82.5 \text{ kN}$$

$$\text{Efficiency} = \frac{\text{strength of joint/pitch length}}{\text{strength of solid plate/pitch length}} \times 100$$

$$= \frac{43.373}{82.5} \times 100 = 52.57\ \%$$

Double cover butt joint

The rivets will be in double shear.

$$\text{Strength of rivet in shear} = 2 \times \frac{\pi}{4} d^2 \tau_{vf} = 2 \times \frac{\pi}{4} (23.5)^2 \times 100 \times 10^{-3}$$

$$= 86.747 \text{ kN}$$

The thickness to be considered for the calculation of bearing value and strength in tearing of plate is the main plate thickness or aggregate thickness of cover plate whichever is less.

$\therefore$ $t = 14$ mm

strength of rivet in bearing = $dt\ \sigma_{pf} = 23.5 \times 14 \times 300 \times 10–3 = 98.70$ kN

$\therefore$ Strength of rivet per pitch length = 86.747 kN

Equating the strength of rivet per pitch length to the strength of plate per pitch length in tearing.

$$86.747 \times 10^3 = (s - 23.5) \times 14 \times (0.6 \times 250)$$

or $s = 64.80 \text{ mm} \approx 60 \text{ mm}$

$$\not< 2.5\phi = 2.5 \times 22 = 55 \text{ mm}$$

Provide rivets at a pitch of 60 mm.

Strength of joint in tearing/pitch length = $(s - d)t\ \sigma_{at}$

$$= (60 - 23.5) \times 14 \times 0.6\ 0 \times 250 \times 10^3$$

$$= 76.65 \text{ kN}$$

Strength of solid plate per pitch length = $st\ \sigma_{at}$

$$= 60 \times 14 \times 0.6 \times 250 \times 10^{-3}$$

$$= 126 \text{ kN}$$

$$\text{Efficiency } \eta = \frac{\text{strength of joint per pitch length}}{\text{strength of solid plate per pitch length}} \times 100$$

$$= \frac{76.65}{126} \times 100 = 60.83\%$$

From the above illustration it is clear that the efficiency of a double cover but joint is maximum and therefore it should be preferred.

Example :

A tie member has to transmit a pull of 300 kN. Design a butt joint to connected it with 12 mm thick gusset plate. Also find the efficiency of the joint. Assume steel of yield strength 250 MPa.

Solution :

Nominal diameter of the rivet = $6.01\sqrt{t}$ = 6.01 $\sqrt{12}$ = 20.819 mm ≈ 20 mm

Provide 20 mm ϕ power driven rivets.

Gross diameter of the rivet = 20 + 1.5 = 21.5 mm

Pitch of the rivets = 2.5 ϕ = 2.5 × 20 = 50 ≈ 60 mm

Let us provide a double cover butt joint. The rivets will be in double shear.

$$\text{Shear strength of one rivet} = 2\frac{\pi}{4}d^2\tau_{vf}$$

$$= 2\times\frac{\pi}{4}(21.5)^2\times 100\times 10^{-3} = 72.61\text{kN}$$

Bearing strength of one rivet = dt σ_{pf}

= 21.5 × 12 × 300 × 10^{-3} = 77.4 kN

Strength of one rivet R_v = 72.61 kN

$$\text{Number of rivets required on each side of the butt joint} = \frac{300}{72.61} = 4.13 \approx 5$$

Arrange the rivets as shown in Fig.

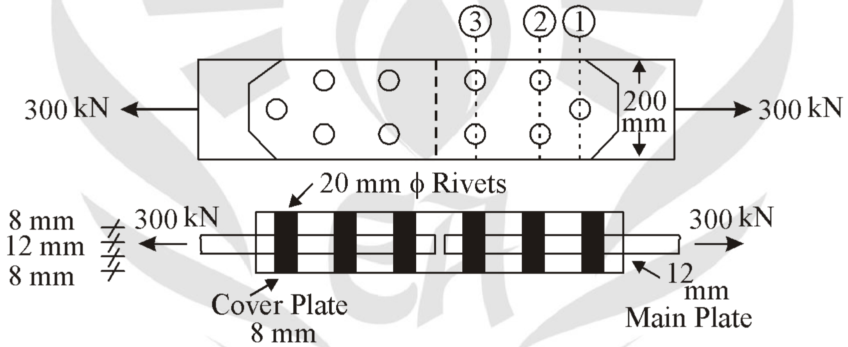


Plate width The width of the tie plate and the thickness of the cover plate can be computed on the basis of the tearing strength of the plate section. The strength of the joint is minimum at section (1) –(1). Therefore the width of the plate required will be maximum at Section (1) – (1). The calculations at Section (2) – (2) and (3) (3) have also been done to illustrate the above fact.

Let x = minimum width of the tie plate required.

Section (1) – (1)

$(x - d)t\sigma_{at}$ = 300 kN

or $(x - 21.5) \times 12 \times 0.6 \times 250 = 300\times10^3$

$(x - 21.5) = 166.66$

or $x = 188.16 \approx 200$ mm

Section (2) – (2)

$(x - 2d)t\sigma_{at} + nR_v = 300\times10^3$

$(x–2 \times 21.5) \times 12 \times 0.6 \times 250 + 1 \times 72.61 \times 10^3 = 300 \times10^3$

or $(x - 43.0) = 126.32$

or $x = 169.32$ mm

Section (3) – (3)

$$(x - 2d)t\sigma_{at} + nR_v = 300 \times 103$$

$$(x - 2 \times 21.5) \times 12 \times 0.6 \times 250 + 3 \times 72.61 \times 10^3 = 300 \times 10^3$$

or $(x - 43.0) = 45.65$

or $x = 88.65$ mm

Hence, the necessary width of the tie plate is 200 mm.

Thickness of cover plate the maximum thickness of the cover plate will be at

Section (3) – (3) being the critical section.

The thickness t_1 of each cover plate so calculated should not be less than 5/8 times the thickness of the tie plate $\left(\frac{5}{8} \times 12 = 7.5\text{mm}\right)$.

Section (3) – (3)

$$(x - 2d) \times 2t_1 \times \sigma_{at} = 300 \times 10^3$$

$$(200 - 43) \times 2t_1 \times 150 = 300 \times 10^3$$

or $t_1 = 6.369 \text{ mm} \approx 8 \text{ mm}$

Efficiency of joint

$$\eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$$

Tearing strength of plate at section (1) – (1)

$$= (200 - 21.5) \times 12 \times 0.6\ 0 + 250 \times 10^{-3}$$

$$= 321.3 \text{ kN}$$

Tearing strength of the two cover plates at section (3) – (3)

$$= (200 - 2 \times 21.5) \times 2 \times 8 \times 0.6 + 250 \times 10^{-3}$$

$$= 376.8 \text{ kN}$$

Hence the tearing strength of the joint = 321.3 kN

The actual safe load of the joint is the minimum of the strength in shear, bearing and tearing.

Strength of joint in shear = $5 \times 72.61 = 363.05$ kN

Strength of joint in bearing = $5 \times 77.4 = 387.00$ kN

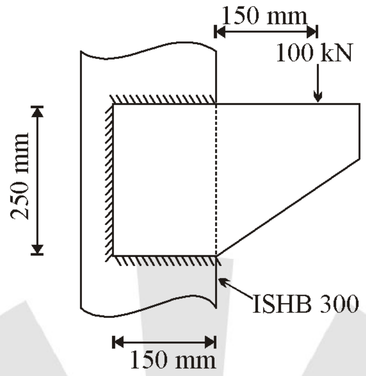
Hence, Strength of the joint = 321.3 kN

Strength of the solid plate = $xt\sigma_{at} = 200 \times 12 \times 0.6 \times 250 \times 10–3 = 360$ kN

$$\text{Efficiency of the joint} = \frac{321.3}{360} \times 100 = 89.25\%$$

Example :

A welded bracket connects a plate to the column flange as shown in figure below. Determine the size of the weld if the allowable stress in the weld is 110 N/mm^2.



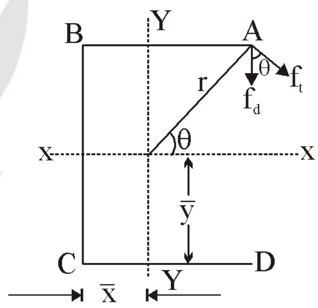
Solution :

Let 't' be the throat thickness and 'S' be the size of the weld.

Shear stress due to direct load = $\dfrac{\text{Direct load}}{\text{Area of weld resisting the direct load}} = \dfrac{100\times10^3}{(2\times150+250)t}$

$$f_d = \frac{181.82}{t}$$

Shear stress due to torsional moment,



$$f_t = \frac{T_Y}{J} = \frac{Per}{J_Y}$$

$$\bar{x} = \frac{2[(150\times t)\times 75]}{2\times150\times t+250t} = 40.91\text{mm}$$

$$\bar{y} = \frac{(150\times t)\times 250+(250\times t)125}{(2\times150\times t)+250t} = 125\text{mm}$$

If θ is the angle between f_d and f_t, then

$$\tan\theta = \frac{150-\bar{x}}{125} = \frac{150-40.91}{125} = 0.873$$

If r is the distance of A from the CG of weld then,

$$r = \sqrt{(125)^2+(150-40.91)^2} = 165.91\text{mm}$$

$$\therefore \qquad \cos\theta = \frac{150-40.91}{165.91} = 0.6575$$

$$e = 150 + (150 - 40.91) = 259.09 \text{ mm}$$

Now, $J = I_{xx} + I_{yy}$

$$I_{xx} = \frac{t\times(250)^3}{12} + 2\left[\frac{150\times t^3}{12} + (150\times t\times 125^2)\right]$$

[t^3 term is negligible as compared to other terms]

$$= \frac{t\times(250)^3}{12} + (2\times150\times t\times125)^2 = 5.99t\times10^6\ mm^4$$

$$I_{YY} = \left[\frac{250t^3}{12} + 250\times t\times(40.91)^2\right] + 2\left[\frac{t\times150^3}{12} + 150\times t\times(75-40.91)^2\right]$$

$$= 250\times t\times(40.91)^2 + 2\left[\frac{t\times150^3}{12} + 150\times t\times(34.09)^2\right]$$

$$= 1.33t\times10^6\ mm^4$$

$\therefore \quad f_t = \dfrac{Per}{I_{xx}+I_{YY}}$

$\Rightarrow \quad f_t = \dfrac{100\times10^3\times259.09\times165.91}{5.99t\times10^6+1.33t\times10^6}$

$\Rightarrow \quad t_t = \dfrac{587.23}{t}$

thus resultant shear stress is given by

$$f_r = \sqrt{(f_d)^2+(f_t)^2+2\times f_d\times f_t\times\cos\theta}$$

$\Rightarrow \quad f_r = \sqrt{\left(\dfrac{181.82}{t}\right)^2+\left(\dfrac{587.23}{t}\right)^2+2\times\dfrac{181.82}{t}\times\dfrac{587.23}{t}\times0.6575}$

$\Rightarrow \quad f_r = \dfrac{719.93}{t}$

For safety of the connection, $f_r \le 0110$

$\Rightarrow \quad \dfrac{719.93}{t} \le 110$

$\Rightarrow \quad t \le 6.54$ mm

$\therefore$ Size of weld, $S = \sqrt{2}\times6.54 = 9.25mm$

Example :

Design a riveted splice for a tie of a steel bridge, 20 cm wide, 20 mm thick carrying an axial tensile force of 50,000 kg. Use 12 mm thick cover plates, 22 mm diameter rivets. Permissible stresses :

Tension in plates = 1500 kg/cm^2

Shear in rivets = 1000 kg/cm^2

Bearing in rivets = 3000 kg/cm^2

Give a neat sketch of the arrangement.

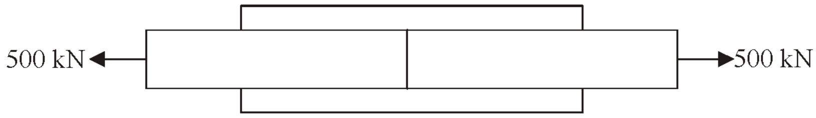
Solution : Taking g = 10 m/s^2 and 1 kg/cm^2 = 0.1 N/mm^2

$\therefore$ Axial tensile force, $P = \dfrac{50,000\times10}{1000} = 500kN$

Nominal diameter of rivets = 22 mm

$\therefore$ Gross diameter of rivets (d)' = 22 + 1.5 = 23.5 mm

Designing the splice as a double cover butt joint as it will give maximum efficiency.



Given that thickness of cover plates = 12 mm

Width of main plate = 20 cm = 200 mm

Thickness of main plate = 20 mm

Assuming the width of cover plate = 200 mm

Strength of rivet in double shear = $\frac{\pi}{4}(d')^2 \times f_s \times 2 = \frac{\pi}{4} \times (23.5)^2 \times \frac{100}{1000} \times 2 = 86.75\,\text{kN}$

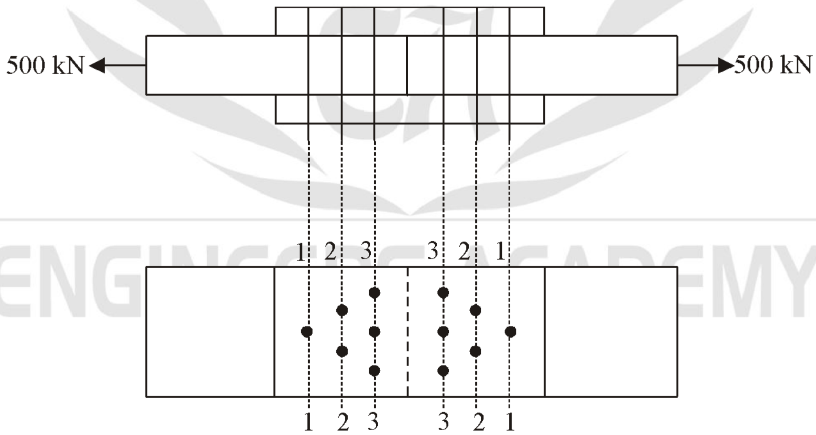
Strength of rivet in bearing = d't

f_b = $23.5 \times 20 \times \frac{300}{1000} = 141\,\text{kN}$

$\therefore$ Rivet value, R_v = 86.75 kN

Number of rivets, $n = \frac{P}{R_v} = \frac{500}{86.75} = 5.76 \approx 6$

The rivets can be arranged in diamond pattern



Checking the strength of cover plate and main plate in tearing

As we know that 3-3 is critical for cover plates.

$\therefore$ Strength of cover plates in tearing at 3 - 3 $\geq$ 500 kN

$\Rightarrow$ $(200 - 3 \times 23.5) \times 24 \times \frac{150}{1000}$ = 466.2 kN < 500, Hence unsafe.

Providing two rivets at 3-3, then

Strength of cover plats in tearing at 3-3

$$= (200 - 2 \times 23.5) \times 24 \times \frac{150}{1000}$$

$$= 550.8 \text{ kN} > 500 \text{ kN (Hence safe)}$$

Thus 2 rivets can be provided at 3-3, Hence safe

Thus arranging rivets in chain pattern, in 3 pairs of two rivets each.

1-1 is critical for main plate,

$\therefore$ Strength of main plate in tearing at 1-1 > 500 ≥ 500 kN

$$\Rightarrow \quad = (200 - 2 \times 23.5) \times 20 \times \frac{150}{1000}$$

$$= 459 \text{ kN} < 500 \text{ kN, Hence unsafe}$$

Thus one rivet can be provided at 1-1

We can provide three rivets at 2-2, thus checking for tearing of main plate at 2-2

$$\Rightarrow \quad = (200 - 3 \times 23.5) \times 20 \times \frac{150}{1000} + R_v > 500$$

$$\Rightarrow \quad = (200 - 3 \times 23.5) \times 20 \times \frac{150}{1000} + 86.75$$

$$= 475.25 < 500, \text{ Hence unsafe}$$

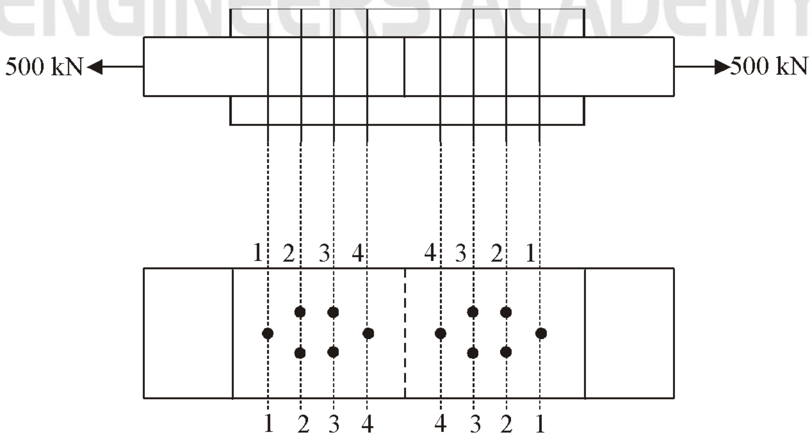
Thus, providing two rivets at 2-2

Strength of main plate in tearing at 2-2

$$= (200 - 2 \times 23.5) \times 20 \times \frac{150}{1000} + 86.75$$

$$= 545.75 > 500, \text{ Hence safe.}$$

Thus at 1-1 at the most one rivet can be provided. two rivets can be provided at 2-2. Two rivets can be provided at 3-3. We have to create 4-4 in order to incorporate the remaining one rivet. Thus the arrangement will be as given below.



Equating the strength of rivet per pitch length to the strength of plate per pitch length in tearing.

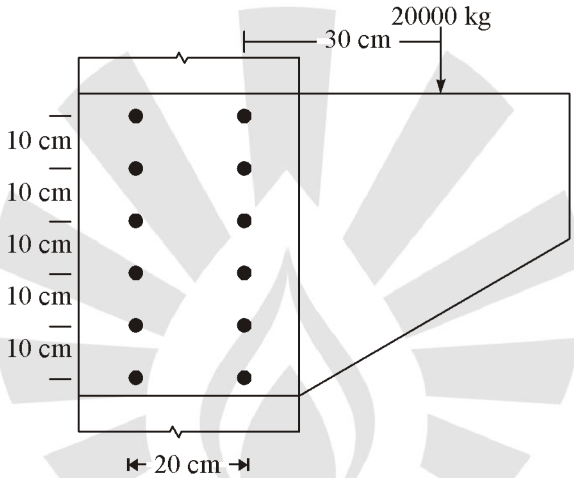
$$R_v = (p - 23.5) \times 20 \times \frac{150}{1000}$$

$$\Rightarrow \quad 86.75 = (p - 23.5) \times 20 \times \frac{150}{1000}$$

$$\Rightarrow \quad p = 52.42 \text{ mm} = 60 \text{ mm} \nless 2.5 \times 22 = 55 \text{ mm}$$

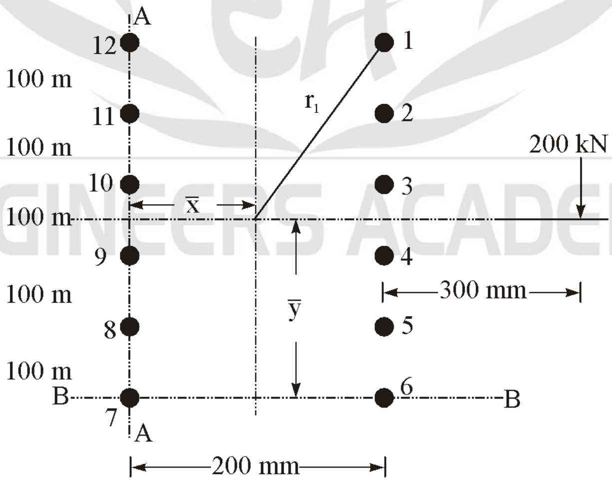
Example :

A load of 20000 kg is carried by a plate bracket riveted to column as shown in figure given below. Calculate the maximum force taken up by any rivet.



Solution :

Assuming the diameter of all the rivets is same. Now locating the centre of gravity of the rivet group. Taking 100 kg = 1 kN



Taking moments about line AA, we get

$$\bar{x} = \frac{6 \times a \times 200}{12a} \qquad [\because \text{ a = area of any rivet}]$$

$$\Rightarrow \quad \bar{x} = 100 \text{ mm}$$

Taking moments about line BB, we get

$$\bar{y} = \frac{2a\times100+2a\times200+2a\times300+2a\times400+2a\times500}{12a}$$

$$\Rightarrow \qquad \bar{y} = 250 \text{ mm}$$

Force in each rivet due to direct shear,

$$F_D = \frac{200}{12} = 16.67 \text{ kN}$$

$$\therefore \qquad F_{D_1} = 16.67 \text{ kN}$$

It the distance of rivet 1 from the CG of rivet group is r_1, then

$$r_1 = \sqrt{(100)^2+(250)^2} = 269.26 \text{ mm}$$

similarly $$r_2 = \sqrt{(100)^2+(150)^2} = 180.28 \text{ mm}$$

and $$r_3 = \sqrt{(100)^2+(50)^2} = 111.80 \text{ mm}$$

As the rivet group is symmetrical about the CG, we have

$$r_1 = r_6 = r_7 = r_{12} = 269.26 \text{ mm}$$
$$r_2 = r_5 = r_8 = r_{11} = 180.28 \text{ mm}$$
$$r_3 = r_4 = r_9 = r_{10} = 111.80 \text{ mm}$$

$$\therefore \qquad \sum_{i=1}^{n} r_i^2 = 4\left[r_1^2+r_2^2+r_3^3\right]$$
$$= 4[(269.26)^2 + (180.28)^2 + (111.80)^2]$$
$$= 470004.26 \text{ mm}^2$$

Now, force in a rivet due to torsional moment,

$$F_T = \frac{(Pe)r_i}{\sum_{i=1}^{n} r_i^2}$$

Here $$P = 200 \text{ kN}$$
$$e = 300 + 100 = 400 \text{ mm}$$

$$\therefore \qquad F_{T_1} = \frac{(Pe)r_1}{\sum_{i=1}^{n} r_i^2} = \frac{200\times400\times269.26}{470004.26} = 45.83 \text{ kN}$$

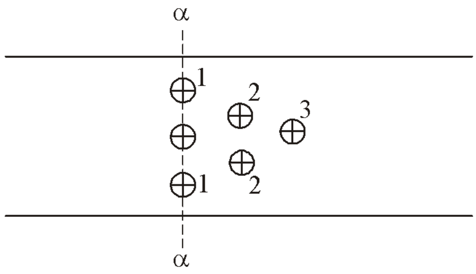
If θ_1 is the angle between F_{D_1} and F_{T_1} then

$$\cos\theta_1 = \frac{100}{269.26} = 0.3714$$

$\therefore$ Resultant force in rivet $$1 = \sqrt{(F_{D_1})^2+(F_{T_1})^2+2\times F_{D_1}\times F_{T_1}\times\cos\theta_1}$$
$$= \sqrt{(16.67)^2+(45.83)^2+2\times16.67\times45.83\times0.3714}$$
$$= 54.27 \text{ kN}$$

(b) Strength of plate in rupture at critical section

Critical section for rupture can be :



(I) a – (1) – (1) – a ;

(II) a – (1) – (2) – (2) – (1) – a

(III) a – (1) – (2) – (3) – (2) – (1) – a

For critical section (I)

$$A_{net} = (160 - 3 \times (16 + 2)) \times 8 = 848 \text{ mm}^2$$

$(d_0 = 16 + 2 = 18 \text{ mm})$

For critical section (II)

$$A_{net} = \left[(160 - 4 \times 18) + 2 \times \frac{40^2}{4 \times 25}\right] 8 = 960 \text{mm}^2$$

For critical section (III)

$$A_{net} = \left(160 - 5 \times 18 + \frac{4 \times 40^2}{4 \times 25}\right) \times 8 = 1072 \text{ mm}^2$$

Hence the most critical section is (I)

Which gives the minimum net area = 848 mm^2

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{\left(\frac{0.9 \times 848 \times 410}{1.25}\right)}{1000} = 250.33 \text{ kN}$$

Hence strength of the plate is $\boxed{250.33 \text{ kN}}$.

❑❑❑

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

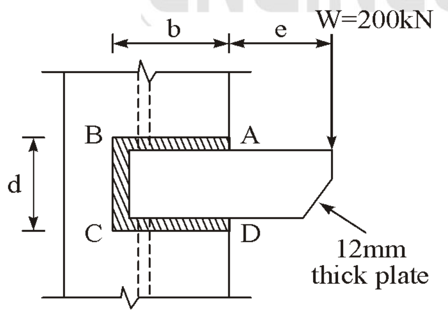
(d) 1.5 mm more than the thickness of the plate

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

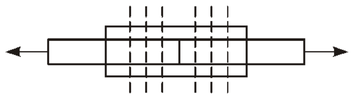
(a) Bending (b) Tension

(c) Compression (d) Torsion

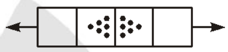
4. A 12mm bracket plate is connected to a column flange as shown in the figure below. The bracket transmits a load of W = 200 kN to the column flange. A 10mm fillet weld is provided along AB, BC and CD. If e = 350 mm, b = 200 mm and d = 600mm, verify if the size of the weld provided is adequate. Allowable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

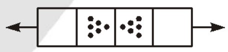


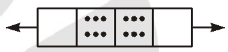
5. Identify the most efficient but joint (with double cover plates) for a plate in tension from the patterns (plan views) shown below, each comprising 6 identical bolts with the same pitch and gauge.

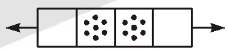


Common elevation

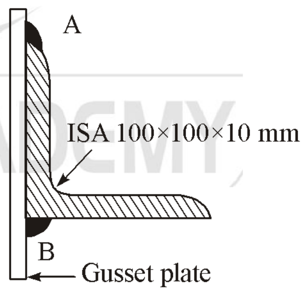
(a) 

(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

34. Area of cross-section of single rolled sections under tension to be considered for the calculation of the actual stress in design is

(a) Gross area

(b) Gross area less the area of rivet holes

(c) Minimum of the net areas

(d) Modified area

35. In pair of steel angles tach-riveted as a tension element, the net area considered for design criterion is equal to the

(a) Gross area of the angles

(b) Full area of one tack welded legs plus a small portion of the outstanding legs

(c) Full area of the tack welded leg

(d) None of the above

36. Maximum deflection allowed in steel ties is

(a) $\frac{L}{750}$ (b) $\frac{L}{480}$

(c) $\frac{L}{350}$ (d) Unlimited

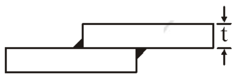
❑❑❑

ANSWERS AND EXPLANATIONS

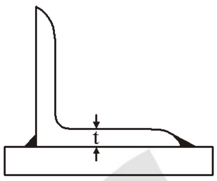
1. ***Ans. (c)***

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

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



$s_{max} = t - 1.5\text{mm}$
For square edge

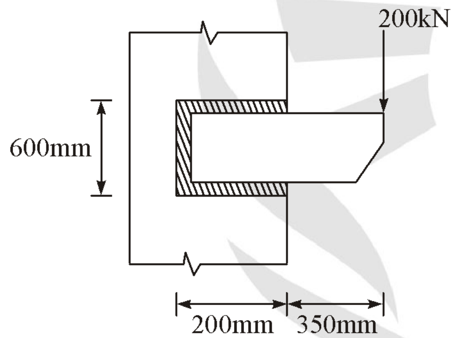


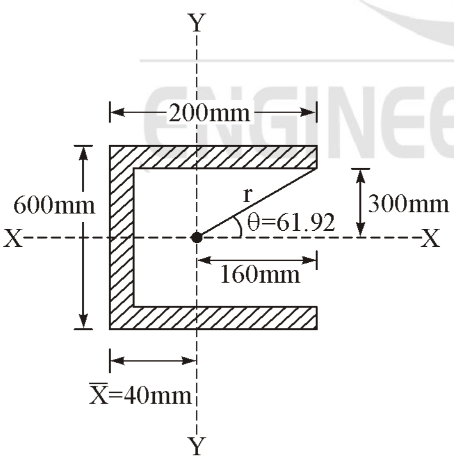
$s_{max} = 3/4 \times t$
For round edge

where, t is thickness of thinner member.

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

4. Throat fillet,





$t = 0.7 \times \text{size} = 0.7 \times 10 = 7$ mm

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

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

$$= 40 \text{ mm}$$

For weld,

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

$$= 378 \times 10^6 \text{ mm}^4$$

For weld, I_y

$$= 2\left[\frac{7 \times 200^3}{12} + 200 \times 7 \times (100 - 40)^2\right]$$

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

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

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

$$= 28.57 \text{ N/mm}^2$$

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

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

$$= 102 \times 10^6 \text{ N.mm}$$

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

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

$$= 85.8 \text{ M/mm}^2$$

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