CE8601-DESIGN OF STEEL STRUCTURAL ELEMENTS

UNIT-1

INTRODUCTION AND ALLOWABLE STRESS DESIGN

CONTENTS

TECHNICAL TERMS:-

- **RIVET:** A piece of round steel forged in place to connect two or more than two steel members together is known as a rivet.The body of is termed as shank
- **NOMINAL AND GROSS DIAMETER OF RIVET: Nominal Diameter:** The nominal diameter of rivet is the diameter of the cold rivet measured before driving.
- **Gross Diameter:** The gross diameter of the rivet is the diameter of the rivet measured after driving and the diameter of the rivet hole is adopted as the gross diameter of a rivet.
- **RIVET VALUE:** The strength of the rivet in shearing and bearing is computed and the lesser is called the Rivet Value (R) or (RV).
- **RIVETING:** A hole slightly greater than a nominal dia is drilled in the plates to be connected.The rivet is inserted and the head is formed and other end is the complete form process is called the riveting.
- **HOT DRIVEN:**If the rivets used in structural steel work are steel work are heated and driven, these rivets are known as hot driven rivets
- **COLD DRIVEN**:If the rivets are driven at atmospheric temperature, they are known as cold rivets
- **PITCH:** The pitch of rivets is the distance between two adjacent rivets measured parallel to the direction of the force.
- **DIAGONAL PITCH:** The distance between centers of any two adjacent rivets in the diagonal direction is called diagonal pitch.
- **STAGGERED PITCH:** The distance between any two consecutive rivet in a zigzagRiveting measured parallel to the direction of stress in the member is called staggered pitch
- **RIVET LINE OR GAUGE LINE OR BACK LINE:**A row of rivet to the direction of force is called a gauge line.
- **LAP:** It is the distance normal to the joint between edges of the overlapping plates in a lap joint or between the joint and the end of cover plates in a butt joint.
- **GAUGE:** A normal distance between the two adjacent gauge lines is called a gauge.
- **EDGE DISTANCE:** It is the distance between the edge of the member or cover plate and the center of the nearest rivet holes.
- **LAP JOINT:** When one member is placed above the other and the two are connected by means of rivets the joint is known as lap joint.

    - Single row
    - Staggered or Zig – Zag riveted
    - Chair riveting

- **BUTT JOINT:**When plates are placed end to end and flushed with each other and are joined by means of cover plates, the joint is known as butt joint. The efficiency of a joint is defined as the ratio of least strength of a riveted joint to the strength of solid plate.

    - Single row, staggered or chain riveting
    - Single riveted, single cover butt joint
    - Double riveted, double cover butt join

PROPERTIES OF STEEL

The important mechanical properties are

1. Ultimate tensile strength
2. Yield stress or proof stress
3. Ductility
4. Weldability
5. Toughness
6. Corrosion resistance
7. Machinability

STRUCTURAL STEEL SECTIONS

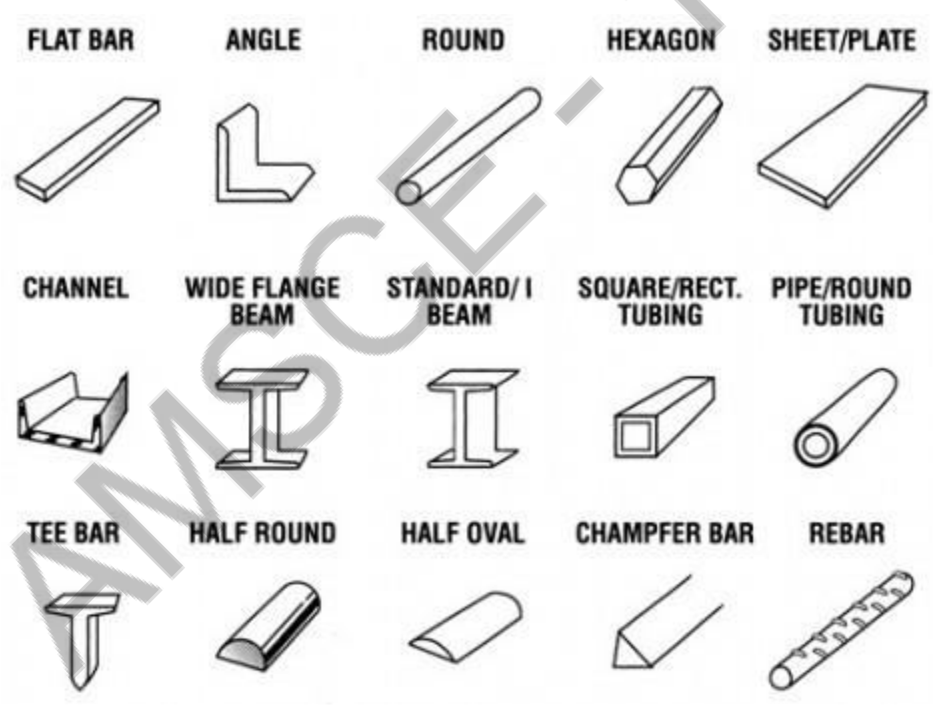


Fig 1.1 Structural steel sections

LIMIT STATE DESIGN CONCEPTS

In the working stress or allowable stress method of design, the emphasis is on limiting a particular stress in a component to a fraction of the specified strength of the material of the component. The magnitude of the factor for a structural action depends upon the degree of safety required. Further, elastic behaviour of the material is assumed. The main objection to the permissible stress method is that the stress safety factor relating the permissible stress to the strength of the material is not usually the same as the ratio of the strength to the design load. Thus it does not give the degree of safety based on collapse load.

In the limit state method, both collapse condition and serviceability condition are considered. In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur on it throughout its life. Designs should ensure that the structure does not become unfit for the use for which it is required. The state atwhich the unfitness occurs is called a limit state. Special features of limit state design method are:

- It is possible to take into account a number of limit states depending upon the particular instance.
- This method is more general in comparison to the working stress method. In this method, different safety factors can be applied to different limit states, which is more rational than applying one common factor (load factor) as in the plastic design method.
- This concept of design is appropriate for the design of structures since any new knowledge of the structural behaviour, loading and materials can be readily incorporated.

The limit state design method is essentially based on the concept of probability. Its basic feature is to consider the possibility and probability of the collapse load. In this respect, it is necessary to consider the possibility of reduced strength and increased load.

The object of design is to keep an acceptable level the probability of any limit state not being exceeded. This is achieved by taking account of the variation in strength and properties of materials to be used and the variations in the loads to be supported by the structure, by using the characteristic values of the strength of materials as well as the loads to be applied. The deviations from the characteristic values in the actual structures are allowed by using their design values. The characteristic values should be based on statistical evidence where necessary data are available; where such data are not available they should be based on an appraisal of experience.

<u>LOADS ON STRUCTURES</u>

DEAD LOAD

Dead load on the roof trusses in single storey industrial buildings consists of dead load of claddings and dead load of purlins, self weight of the trusses in addition to the weight of bracings etc. Further, additional special dead loads such as truss supported hoist dead loads; special ducting and ventilator weight etc. could contribute to roof truss dead loads. As the clear span length (column free span length) increases, the self weight of the moment resisting gable frames increases drastically. In such cases roof trusses are more economical. Dead loads of floor slabs can be considerably reduced by adopting composite slabs with profiled steel sheets

LIVE LOAD

The live load on roof trusses consist of the gravitational load due to erection and servicing as well as dust load etc. and the intensity is taken as per IS:875-1975. Additional special live loads such as snow loads in very cold climates, crane live loads in trusses supporting monorails may have to be considered.

WIND LOAD

Wind load on the roof trusses, unless the roof slope is too high, would be usually uplift force perpendicular to the roof, due to suction effect of the wind blowing over the roof. Hence the wind load on roof truss usually acts opposite to the gravity load, and its magnitude can be larger than gravity loads, causing reversal of forces in truss members.

EARTHQUAKE LOAD

Since earthquake load on a building depends on the mass of the building, earthquake loads usually do not govern the design of light industrial steel buildings. Wind loads usually govern. However, in the case of industrial buildings with a large mass located at the roof or upper floors, the earthquake load may govern the design. These loads are calculated as per IS: 1893-2002.

<u>METAL JOINING METHODS</u>

- Riveted connections
- Bolded connections
- Welded connections

<u>RIVETTED CONNECTIONS:</u>

Types of Riveted Connections are

1. lap joint
2. butt joint
3. single line rivet
4. double line rivet
5. chain riveting
6. zigzag riveting
7. single cover butt joint
8. double cover butt joint

TYPES OF RIVET HEADS

1. Snap head
2. Pan Head
3. Pan Head with tapered head
4. Round counter sunk head
5. Counter head

CLASSIFICATION BASED ON METHOD OF DRIVING:

1. power- driven shop rivets
2. power- driven field rivets
3. cold driven rivets
4. hand driven rivets

TYPES OF FAILURE ON RIVETS:

1. Shear failure of rivet
2. Shear failure of plates.
3. Tension or shearing failure of plates.
4. Splitting of plates

5. Bearing failures of plates.
6. Bearing failures of rivet

- **Shear failure of plates:**
  - Internal pressure of over driven (short length more than the grip) rivet plates at a lesser edge distance than specified cause
- **Tension or fearing failure of plates:**
  - Tensile stress in net cross section > working tensile stress.
- Rivet stronger than the plates.
- **Splitting of plates:**
  - Lesser edge distance than reqd split or shear out.
- **Bearing failure of plates:**
  - Plates crushed- bearing stress in plates > working bearing stress
- **Bearing failure of rivets:**
  - Rivets crushed around the half circumference. Plate may strong in bearing and heaviest stresses plate press the rivet.

BOLTED CONNECTION

A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive nut. Bolts are used for joining together pieces of metals by inserting them through hole in the metal and tightening the nut at the threaded ends.

Classification of bolts

i. Unfinished bolts/black

ii. Finished bolts/turned

iii. High strength friction grip bolt (HSFG)

1. These bolts are made from mild steel rods with square or hexagonal head
2. Shank is left unfinished that is rough as rolled
3. 16,20,24,30&36mm these bolts are designated as M16,M20,M24,M30,M36

4. Holes are larger than nominal dia, as shanks of black bolts are unfinished, bolts may not establish contact with structural member at entire zone of contact surface.
5. Joints remain quick lose resulting into large deflections
6. Yield strength of commonly used black bolts in 240 N/mm^2 ultimate strength 400 N/mm^2
7. Bolts are used for height structures under static loads such as trusses

Finished bolts

i. these bolts are made from mild steel but they are formed from hexagonal rods which are finished by turning to a circular shape.

ii. Actual dimension of these bolts as kept1.2mm to 1.3mm >than the nominal dia.

iii. Hole>1.5mm. Tolerance available for fitting is quite small it needs special methods to align bolt hole. Before bolting

iv. These bolts much better bearing contact between the bolts and holes.

V. these bolts are used in special jobs like connecting machine parts subjected to dynamic loading.

Advantage of bolted connection

1. Material joint is noiseless
2. Do not need skilled labour
3. Needs less labour
4. Connection can be made quickly
5. Structure can be put to use immediately
6. Accommodates minor discrepancies in dimensions
7. Alteration of any can be done easily

8. Working area required in the field is less

Disadvantage

1. Tensile strength is reduced considerably due to stress concentration and reduction of area at roots of the threads
2. Rigidity of joint is reduced due to loose fill resulting into excessive deflections
3. Due to vibration nuts are likely to loosen, endangering the safety of the structures.

WELDING:

a. Welding may be process of connection or joining metallic member by application of heat. Temperature may be around 1450^{o}C.
b. When two structural members are joined by means of welds then the connection is called welded connection.

METHOD OF WELDING:

There are several methods of welding namely,

a. Fusion welding
b. Arc welding
    1. Metal arc welding
    2. Carbon arc welding
c. Gas welding
    1. Oxy acetylene welding
    2. Oxy hydro carbon welding
d. Pressure welding
e. Forge welding
f. Resistance welding
g. Pressure welding
h. Thermalite welding
i. Union welding.

ADVANTAGES:

i. It is less laborious than riveted connection.
ii. It does not require additional; gusset plates, cover plate, angles etc, like riveted connection.
iii. Thin &thick, both types of section can be effectively connected.
iv. Welded joints are 100% efficient.
v. Welding can be done in all types of structures & in all the intricate situations where revered connections are not possible to be made.

DISADVANTAGE:

vi. It requires costly equipment.
vii. It is possible that plate may be jointed splayed.
viii. Welding induces additional internal stresses in the vicinity of the joint.
ix. They affect eyes badly due to bright flash light.

TYPES OF WELDED JOINTS:

c. Welded joints may be classified into following categories:

i. Butt weld	Fillet weld
ii. Plug & slot weld	Spot Weld
iii. Seam weld	Pipe weld

STRENGTH OF BUTT WELD FOR DIRECT OR AXIAL TENSILE FORCE:

The strength of the butt weld for direct or axial tensile force is as follow:

When full penetration of butt weld is assured.

(F)t = L *t * Pt

When full penetration of butt weld is not ensured.

(F)t + L* (5/8) * t *Pt

Where Ft=tensile strength of butt weld

L= thickness of thinner plate if jointed plates are of different thickness.

Pt=permissible tensile stress for weld

STRENGTH OF FILLET WELDS:

Effective area of fillet weld is the product of effective length and effective throat thickness. This is when multiplied by allowable shear stress of the weld gives strength of the fillet weld, strength of fillet weld = Ps * (t)e* Le.

Where Ps=allowable shear stress for fillet weld which is taken as 1100kg/cm²

(t)e = effective thickness of throat of weld.

Le= effective length of the fillet weld.

TYPES OF WELDED BEAM CONNECTIONS:

The welded beam connections are of four types:

a. Direct welded connections.
b. Welded framed connections.
c. Welded seat connections.
d. Moment resistant welded connections

MOMENT RESISTANT CONNECTIONS:

When the ends of beam are subjected to moments and shears, moment's resistant welded connections are used. The end moments of beam are resisted by top plate and bottom flange. When the moments of beam, say, on the left side of connection are clockwise, the top plate carries tensile force and the bottom flange of beam carries compressive force. The forces are equal in magnitude and opposite in direction.

WELDING AND WELDED CONNECTIONS

Welding is the process of joining two pieces of metal by creating a strong metallurgical bond between them by heating 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 interlocking. It is one of the oldest and reliable methods of joining.

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 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 and enable the connection of complicated shapes. Welded structures are 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.

FUNDAMENTALS OF WELDING

A welded joint is obtained when two clean surfaces are brought into contact with each other and either pressure or heat, or both are applied to obtain a bond. The tendency of atoms to bond is the fundamental basis of welding. The inter-diffusion between the materials that are joined is the underlying principle in all welding processes. The diffusion may take place in the liquid, solid or mixed state. In welding the metallic materials are joined by the formation of metallic bonds and a perfect connection is formed. In practice however, it is very difficult to achieve a perfect joint; for, real surfaces are never smooth. When welding, contact is established only at a few points in the surface, joins irregular surfaces where atomic bonding occurs. Therefore the strength attained will be only a fraction of the full strength. Also, the irregular surface may not be very clean, being contaminated with adsorbed moisture, oxide film, grease layer etc. In the welding of such surfaces, the contaminants have to be removed for the bonding of the surface atoms to take place. This can be accomplished by applying either heat or pressure. In practical welding, both heat and pressure are applied to get a good joint.

As pointed out earlier, any welding process needs some form of energy, often heat, to connect the two materials. The relative amount of heat and pressure required to join two materials may vary considerably between two extreme cases in which either heat or pressure alone is applied. When heat alone is applied to make the joint, pressure is used merely to keep the joining members together. Examples of such a process are Gas Tungsten Arc Welding (GTAW), Shielded Metal Arc Welding (SMAW), Submerged Arc Welding (SAW) etc. On the other hand pressure alone is used to make the bonding by plastic deformation, examples being cold welding, roll welding, ultrasonic welding etc. There are other welding methods where both pressure and heat are employed, such as resistance welding, friction welding etc. A flame, an arc or resistance to an electric current, produces the required heat. Electric arc is by far themost popular source of heat used in commercial welding practice.

WELDING PROCESS

In general, gas and arc welding are employed; but, almost all structural welding is arc welding.

In gas welding a mixture of oxygen and some suitable gas is burned at the tip of a torch held in the welder's hand or by an automatic machine. Acetylene is the gas used in structural welding and the process is called oxyacetylene welding. The flame produced can be used both for cutting and welding of metals. Gas welding is a simple and inexpensive process. But, the process is slow compared to other means of welding. It is generally used for repair and maintenance work.

The most common welding processes, especially for structural steel, use electric energy as the heat source produced by the electric arc.IS:816 in this process, the base metal and the welding rod are heated to the fusion temperature by an electric arc. The arc is a continuous spark formed when a large current at a low voltage is discharged between the electrode and the base metal through a thermally ionised gaseous column, called plasma. The resistance of the air or gas between the electrode and the objects being welded changes the electric energy into heat. A temperature of 33000 C to 55000 C is produced in the arc. The welding rod is connected to one terminal of the current source and the object to be welded to the other. In arc welding, fusion takes place by the flow of material from the welding rod across the arc without pressure being applied.The Shielded Metal Arc Welding process is explained in the following paragraph.

In Shielded Metal Arc Welding or SMAW, heating is done by means of electric arc between a coated electrode and the material being joined. In case bare wire electrode (without coating) is employed, the molten metal gets exposed to atmosphere and combines chemically with oxygen and nitrogen forming defective welds. The electrode coating on the welding rod forms a gaseous shield that helps to exclude oxygen and stabilise the arc.

The coated electrode also deposits a slag in the molten metal, which because of its lesser density compared to the base metal, floats on the surface of the molten metal pool, shields it from atmosphere, and slows cooling. After cooling, the slag can be easily removed by hammering and wire brushing. The coating on the electrode thus: shields the arc from atmosphere; coats the molten metal pool against oxidation; stabilises the arc; shapes the molten metal by surface tension and provides alloying element to weld metal.

The type of welding electrode used would decide the weld properties such as strength, ductility and corrosion resistance. The type to be used for a particular job depends upon the type of metal being welded, the amount of material to be added and the position of the work. The two general classes of electrodes are lightly coated and heavily coated. The heavily coated electrodes are normally used in structural welding. The resulting welds are stronger, more corrosion resistant and more ductile compared to welds produced by lightly coated electrodes. Usually the SMAW process is either automatic or semi-automatic.

The term weldability is defined as the ability to obtain economic welds, which are good, crack-free and would meet all the requirements. Of great importance are the chemistry and the structure of the base metal and the weld metal. The effects of heating and cooling associated with fusion welding are experienced by the weld metal and the Heat Affected Zone (HAZ) of the base metal. The cracks in HAZ are mainly caused by high carbon content, hydrogen enbrittlement and rate of cooling. For most steels, weld cracks become a problem as the thickness of the plates increases.

ADVANTAGES OF WELDING:

- As no holes are required for welding structural members are more effectively taking loads.
- Overall weight of the structural steel required is reduced by use of welded joints.
- Overall weight of the structural steel required is reduced by use of welded joints.
- Welded joints are often economic as less labour and materials are required for the joints.
- Welded connection looks better than usually bulky riveted joints.
- Speed of fabrication and erection helps compress production schedules.
- Any shape of the joints can be made with easy welding process required less working place than riveting process.
- Completely rigid joint can be provided with welding.
- No noise is produced in the welding process as in the riveting process.
- Welding process reduced the material and cost of the labour.
- Welding process made shorten the production side.
- Welding is the only process that produced the one piece construction.

DISADVANTAGES OF WELDING:

- »Skilled labour and electricity are required.
- »Internal stress and warping are produced due to uneven heating and cooling.
- »Welding joints are more brittle and there force. Fatigue strength is less than the number jointed.
- »Defects like internal air pockets, slag, inclusion and incomplete penetration are difficult to detect.

TYPES OF WELD:-

I. Based on welding

II Based on position

III Based on joint

BASED ON WELDING:

→Fillet welding

→Butt welding

→Plug welding

→Slot welding

→Spot welding

BASED ON POSITION:

→Flat weld

→ Horizontal

→Vertical

→ Over

BASED ON JOINT:

→Butt joint

→Lap joint

→Tee joint

→Corner joint

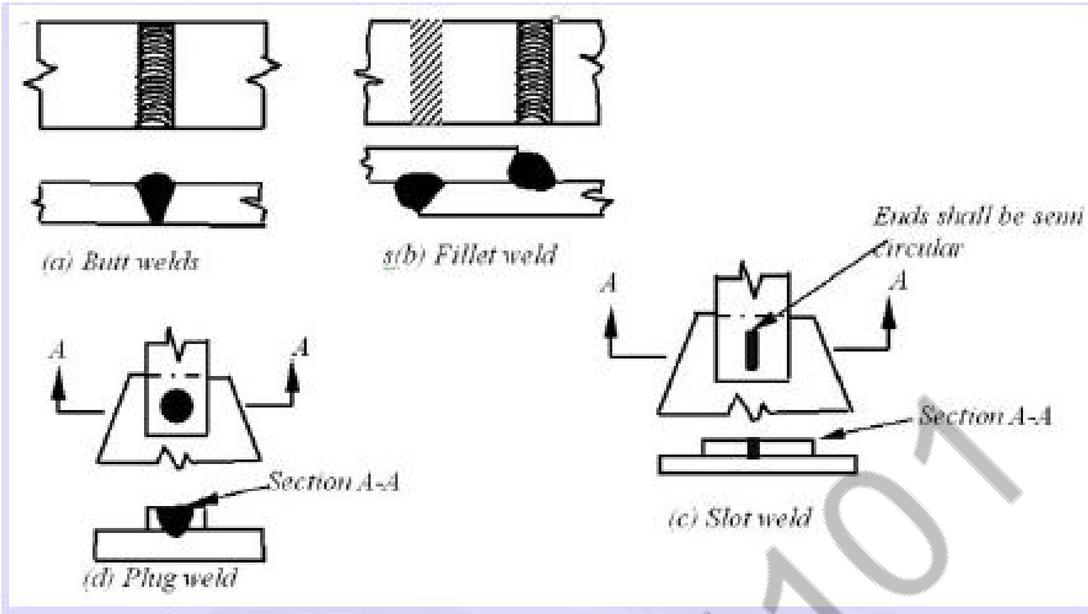


Fig 1.2 Type of weld

BUTT WELD

Butt weld is also called as groove weld. The faces of two members are placed with each other and connected by fillet metals. Butt weld is used to join structural members carrying direct compression or tension.

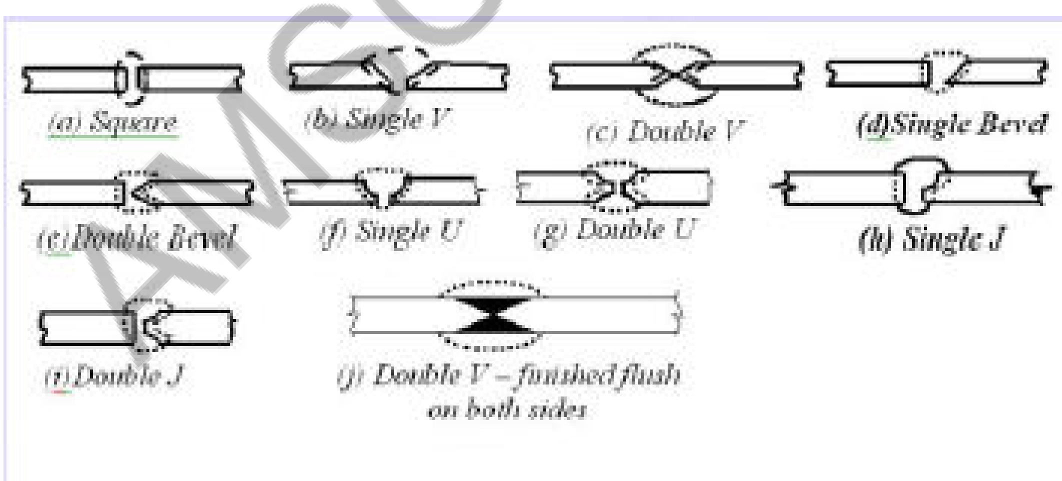


Fig 1.3 Type of butt welds

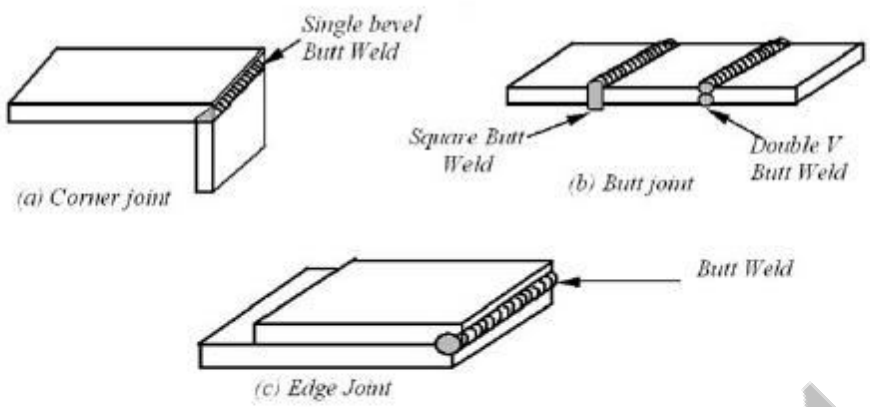


Fig 1.4 Typical connection with butt weld

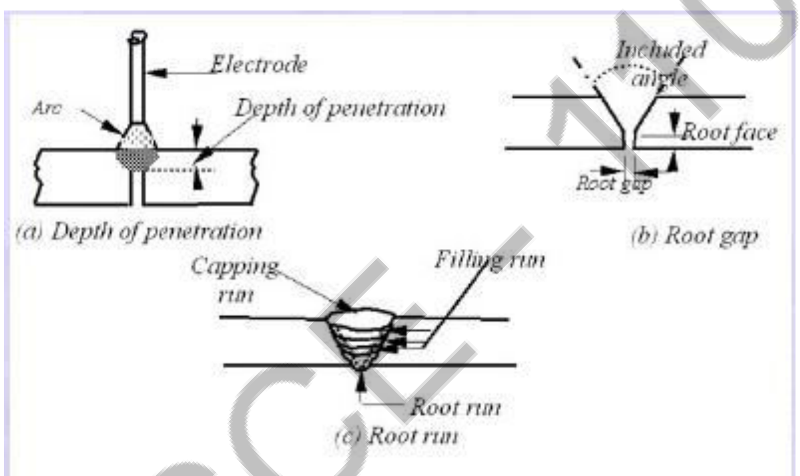


Fig 1.5 Butt weld details

FILLET WELD

A fillet weld is a weld of approximately triangular cross section joining two surfaces approximately at right angles to each other in lap joint.

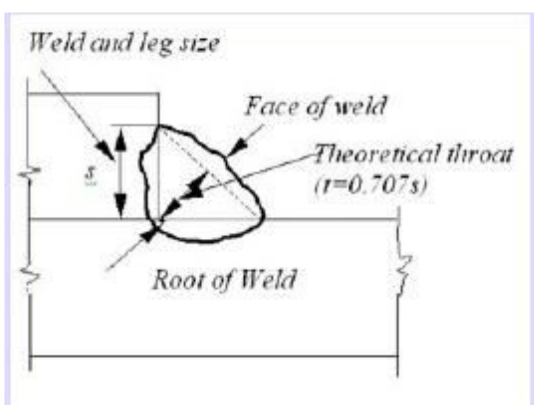


Fig 1.6 Typical fillet weld

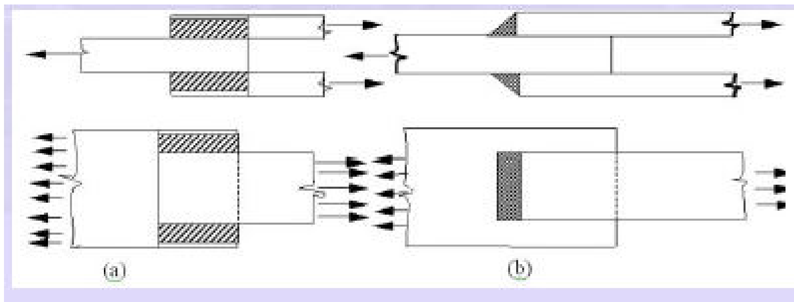


Fig 1.7 Fillet (a) side welds and (b) end welds

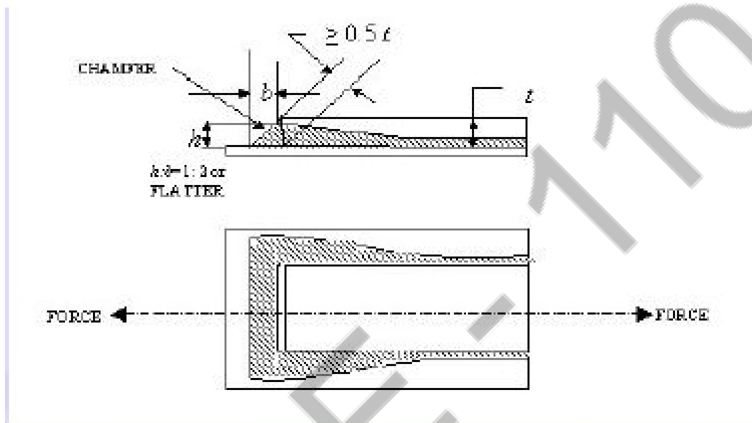


Fig 1.8 End fillet weld normal to direction of force

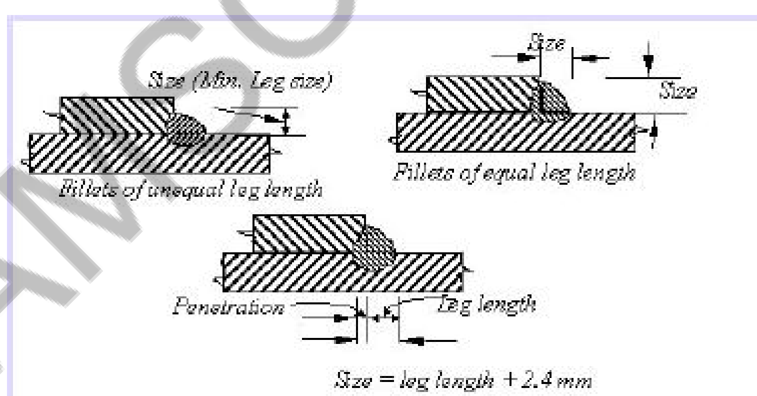


Fig 1.9 Sizes of fillet welds

Table 1.1 Minimum size of first run or of a single run fillet weld

Thickness of thicker part (mm)	Minimum size (mm)
t ≤ 10	3
10 < t ≤ 20	5
20 < t ≤ 32	6
32 < t ≤ 50	8 (First run)10 (Minimum size of fillet)

CE8601-DESIGN OF STEEL STRUCTURAL ELEMENTS

UNIT-II

CONNECTIONS IN STEEL STRUCTURAL ELEMENTS

EQUATION FOR CALCULATING THE EFFECTIVE THROAT THICKNESS OF A WELD

Effective throat thickness, t=0.7x size of weld.

UNWIN'S FORMULA

$d=6.04\sqrt{t}$

where,

t= thickness of plate in mm

d= diameter of rivet in mm.

PROBLEMS

DESIGN OF BOLTED, RIVETED AND WELDED JOINTS FOR AXIALLY LOADED, ECCENTRIC LOADED MEMBERS AND EFFICIENCY OF JOINTS

Problem 1: Calculate the value of rivet in a lap joint used to connect two plates 12mm tk in following cases (a) power driven rivets (b) hand driven rivets.

Solution:-

Unwin's formula.

$\emptyset = 6.01\sqrt{t} = 6.01\sqrt{12} = 20.819 \cong 20$mm.

Gross diameter d= 20+1.5 = 21.5mm

(a) Power driven rivets.

Shear τ_{uf}=100 N/mm^2

Bearing σ_{pf}=300 N/mm^2

Strength of rivet in single shear = $\pi/4$*d^2* τ_{uf}

= $\pi/4$*21.5^2*100

= 36305 N (or) 36.305 KN

Strength of rivet in bearing = dt* σ_{pf} = 21.5*12*300

=77400 N (or) 77.4 KN

Value of rivet Rv=36.305 KN

(b) Hand driven rivet

Shear τ_{uf}=80 N/mm^2

Bearing σ_{pf}=250 N/mm^2

Strength of rivet in shear = $\pi/4$*d^2* τ_{uf}

=$\pi/4$*21.5^2*80

=29.044 KN

Strength of rivet in bearing = dt* σ_{pf} = 21.5*12*250

= 64.5 KN

Rivet value R_v=29.044 KN

Problem 2: Two plates 12mm thick are joined by double riveted double cover butt joint as shown. Use 20mm dia rivets, design pitch of rivets σ_{at}=150Mpa.also find efficiency of joints.

Solution:-

Gross dia = 20+1.5= 21.5 mm

For power driven shop rivets

Shear τ_{uf}=100 N/mm^2

Bearing σ_{pf}=300 N/mm^2

Strength of rivets in bearing =300/1000*21.5*12

=77.4 KN

Strength of rivet in double shear= 2*100/1000 *21.5^2* $\pi/4$

= 72.6 KN

R_v= 72.6 KN

For efficiency of joint per pitch length

Strength of plate per pitch = 2*rivet value

σ_{at} (p-d)t = 2*72.6*1000 N

150 (p-21.5)*12=2*72.6*1000 N

P=102.16 mm ≅ 100 mm

Min per pitch = 2.5*d = 2.5*21.5 = 53.7 mm

Adopt pitch = 100 mm

Efficiency of joint = (150*(100-21.5)*12 /(150*100*12)) * 100

= 78.5%

Riveted Eccentric connection:

Problem 3: Design a riveted bracket connection transfer the load of 150KN as shown in fig load acting on an eccentric of 300mm from face of column.

Soln:

Rivet along line A-A

Shear load= 150KN

Tension due to BM

=150*300=45000KN

use 2 rows of 22mm dia rivet

= 2+1.5=23.5mm

Let rivet value be tension of rivet

$R=\pi/4*23.5^2*100$

=43373.6 KN

To find no of bolt:

$n = \sqrt{(6m/mpr)}$

$=\sqrt{(6*45000*10^3)/(2*65*43373)}$

=7 nos

Depth of bracket plate:

Min edge =40mm

Depth of plate +6*65+2*40=470mm

H=470-40=430mm

h/7=430/7=61.42mm

$\sum r=42[(65+40-61.42)+(65+40-61.42)+(195+40-61.42)+(260+40-61.42)+(325+40-61.42)+(390+40-61.42)]$

=2472.96mm

$\sum r^2=2[(43.58^2)+(108.58^2)+(172.58^2)+(238.58^2)+(303.58^2)+(368.58^2)]$

$=657502.6mm^2$

Tensile stress in criteria rivet:

$M' = M/(1+(2h/21)(\sum y_1/\sum y_1^2)$

$= (45000*10^3)/(1+(2*430/21)(2472.96/657502.6)$

$=38993.865*10^3$ Nmm

$\sigma_{tf}=M'y_1/A\sum Y_1$

$=(38993.865*10^3*368.58)/((\pi/4*(23.5)^2)*657.502)$

$=50.397N/m^2$

Shear stress

$\tau_{vf}=150*10$^3/($\pi/4*23.5^2*7*2$)

$=24.70N/m^2$

$24.7/100+50.39/100=0.7507 \leq 1.4$

Which is safe

Rivet along B-B

e=225

M=150*225=33750

$N=\sqrt{(\frac{6m}{spr})}$

Assume $\emptyset = 22mm\ gross\ dia = 23.5$

Value of rivet $=2*\pi/4*23.5^2*100=86747.227N$

Bearing=23.5*10*300=70500N

R=70500N

- Assume s=65mm

$N=\sqrt{6*33750*10^3}/(65*1*70500$

$=6.64\sim7$

F1=150/7=21.42Kn

F2=mr/$\in r^2$

=33750*195/2(65^2+$(2*65)^2$+$(3*65)^2$

=55.63KN

R=$\sqrt{(F_1^2 + F_2^2)}$

$$\sqrt{(21.42^2 + 55.63^2)}$$

=59.61Kn$< 70.5KN$

Which is safe

Rivet along line c-c

Assume 22mm∅ e=225-2*70=85

M=150*85=12750KN.mm

F_1=150/5=30KN

F_2=Mr/ϵr^2=12750*140/2(70^2+140^2)

=36.42

Resultant force=$f_1 + f_2$

=30+36.42=66.42KN$<$ 70.5KN

Problem 4: Design a single bolted double cover butt joint to connect the boiler plate of thick 12mm for maximum efficiency use M16 blot of grade 4.6, find the efficiency of the joint.

GIVEN DATA:

d = 16mm d_o= 18mm

f_{ub} = 400 n/mm^2

f_u = 410 n/mm^2

t =12mm

soln:

Nominal strength of bolt in shear:

v_{nsb}= $f_{ub}/\sqrt{3}[n_nA_{nb} + n_nA_{sb}]\tau$

n_n = 1 n_s = 1

$v_{nsb}=400/\sqrt{3}[1*0.78*\frac{\pi}{2}*16^2+1*\pi/4*16^2]$

= 82651 N

Design strength of shear = 82651/1.25

=66121 N

Design strength of plate per pitch length:

T_{dn} = 0.9 A_nF_u/γ_{ml}

$A_n=[b-nd_o]t$

= [p-18]*12

T_{dn}=0.9*[p-18]*12*410/1.25

66121=3542.4[p-18]

P = 36.67mm

Minimum pitch =2.5*d

= 2.5*16

= 40mm

Edge distance = 1.5*d_o

=1.5*18

=27mm

Provide e = 30mm

Check for bearing:

$V_{dpb} = 2.5k_b dtf_u/\gamma_{mb}$

To find K_b

1. $K_b = e/3d_o = 30/3*18 = 0.555$
2. $K_b = p/3d_o - 0.25 = 40/3*18 - 0.25 = 0.4907$
3. $F_{ub}/f_u = 400/410 = 0.9756$
4. 1

$V_{dpb} = 2.5k_b dtf_u/\gamma_{mb}$

=2.5*0.49*16*12*410/1.25

=77255 N

Cross check:

$T_{dn} = 0.9\ A_n F_u/\gamma_{ml}$

T_{dn}=0.9*[40-18]*12*410/1.25

=77932 N

Design strength of plate per pitch:

$=F_y/M_g*A_g$

=250*40*12/1.1

=109091 N

Efficiency of the joint =66121/109091*100

= 66.61%

Problem 5: A bracket is bolted to the flange of the column shown in fig use 8mm thickness plate and use M20 bolt and grade 4.6 design the onnection.

Given data:

Flange tk of ISMB300@588N/m = 10.6mm

Hence thickness= 8mm<10.6mm

For M20 bolt of grade 4.6

d=20mm d_o=22mm

f_{ub} = 400N/mm^2

for rolled section f_u = 410n/mm^2

Bolt in single section:

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

$v_{nsb} = (f_{ub}/\sqrt{3})[n_n A_{nb} + n_n A_{sb}]$

$v_{nsb} = (400/\sqrt{3}[1*0.78*\frac{\pi}{2}*20^2])/1.25$

= 45272 N

Strength of the bolt in bearing :

Minimum pitch =2.5*d

= 2.5*16

= 40mm

Edge distance = $1.5*d_o$

$=1.5*18$

$=27mm$

Provide e = 30mm

$V_{dpb} = 2.5k_bdtf_{u/}\gamma_{mb}$

To find K_b

$K_b = e/3d_o = 35/3*18 = 0.53035$

$K_b = p/3d_o - 0.25 = 50/3*18-0.25 = 0.5075$

$F_{ub}/f_u = 400/410 = 0.9756$

$V_{dpb} = 2.5k_bdtf_{u/}\gamma_{mb}$

$=2.5*0.5075*120*8*410/1.25$

$=66584$ N

Hence the designstrength of bolt = 45272 N

To find the no of bolt:

$n = \sqrt{(6m/2v_p)}$

$=\sqrt{(6*300*10^3*350/(2*45272*50))}$

$= 11.29$

Provide 12 nos

$F_1 = per/\sum r^2$

$r=\sqrt{(70^2+275^2)}$

$=283.77mm$

$\sum r^2=4[(70^2+25^2)+(70^2+75^2)+(70^2+125^2)+(70^2+175^2)+(70^2+225^2)+(70^2+275^2)]$

$\sum r^2 =832600mm^2$

$F_2=(300*10^3*350*283.77)/832600$

$F_2=35786.5N$

Direct shear;

$F_1=p/n$

$=300*10^3/[2*12]$

$=12500N$

Resultant force:

$F=\sqrt{(F_1^2+F_2^2+2F_1F_2\cos\theta)}$

$\text{Cos}\theta=70/283.77$

$=0.2466$

$F = \sqrt{(12500^2+35786.5^2+2*12500*3578605*0.2466)}$

$=40713.86<45272$

Hence the design is safe provide 24 nos of M20 bolts

Problem 6:Design a suitable bracketed connection of ISHF75 section attached to the flange of ISHB300@588N/m to carry a vertical factor load of 600KN at an eccentricity of 300NM use M20 bolt of grade 4.6

Given data;

Flange tk of ISHB300@588N/m = 10.6mm

Hence thickness= 8mm<10.6mm

For M20 bolt of grade 4.6

d=24mm d_o=26mm

f_{ub} = 400N/mm^2

for rolled section f_u = 410n/mm^2

Bolt in single section:

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

$v_{nsb} = (f_{ub}/\sqrt{3})[n_n A_{nb} + n_n A_{sb}]$

$v_{nsb} = (400/\sqrt{3}[1*0.78*\frac{\pi}{2}*24^2])/1.25$

= 45272 N

Strength of the bolt in bearing:

Minimum pitch =2.5*d

= 2.5*24

= 60mm

Edge distance = 1.5*d_o

=1.5*26

=39mm

Provide e = 50mm

$V_{dpb} = 2.5 k_b d t f_{u}/\gamma_{mb}$

To find K_b

$K_b = e/3d_o = 50/3*26 = 0.641$

$K_b = p/3d_o - 0.25 = 70/3*26 - 0.25 = 0.647$

$F_{ub}/f_u = 400/410 = 0.9756$

$V_{dpb} = 2.5k_b dtf_u/\gamma_{mb}$

$= 2.5*0.5075*120*8*410/1.25$

$= 113533$ N

Design tension capacity

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

$T_{nb} = 0.9f_{ub}a_n/\gamma_{mb}$

$= (0.9*400*0.78*\pi/4*24^2)/1.25$

$T_{db} = 98703$ N

To find no of bolt:

V= 65192 N

$n = \sqrt{(6m/2v_p)}$

$= \sqrt{(6*600*10^3*300/(2*65192*70))}$

$= 10.87$

$= 11$ nos

Provide 11 bolt

H=10*70+50

=750mm

h/7=750/7

$=107.14$

Tensile force in extreambolt.

$T_b=M'y_1/\sum Y_1^2$

$M' = M/(1+(2h/21)(\sum y_1/\sum y_1^2)$

$\sum Y=2*3278.6$

$\sum y^2=2*1479172$

Total moment restricted by bolt in tension:

$M' = M/(1+(2h/21)(\sum y_1/\sum y_1^2)$

$=(600*10^3*300)/(1+(2*750/21)(2*3278.6/2*1479172)$

$=155397181.7$ Nmm

$T_b=M'y_1/\sum Y_1^2$

$=155397181.7*642.86/(2*1479172)$

$=33768$ N

Direct shear force:

$V_{sb}=600*1000/22$

$=27273N$

$[V_{sb}/V_{db}]^2+[T_b/T_{db}]^2<=1.0$

$[27273/65192]^2+[33768/98703]^2<=1$

$0.2921<=1$

Hence the bolt are safe provided bolt are shown in fig

Problem 7: A 16 mm thick angle section is jointed to a 10mm thick gusset plate. The angle is supporting a load of 55KN. Find out the number of 16mm diameter power of rivets

Solution:

Nominal diameter of rivet Ø = 16mm

Grass diameter of rivet d = 16+1.5

=17.5

Using power driven rivets

Strength of rivet in single shear = $\pi/4$ d^2 v_f

$= \pi/4 * 17.5^2 * 100*10^{-3}$

= 24.052KN

For calculating the strength of the rivet in bearing, the minimum thickness of the section to be jointed is considered

t=6mm

Strength of rivet in bearing = $dt\sigma_{pf}$

$= 17.5 * 6 * 300 * 10^{-3}$

=31.5KN

The strength of the rivet, R_v= 24.052KN

Number of rivets n= P/R_v= 55/24.052=2.28 ≈ 3 Nos

Provide 3 rivet to connect angle with gusset plate

Problem 8: A single riveted double cover butt joint is used to connect two plates 16mm thick with chain riveting. The rivets used are power driven 20mm in diameter at a pitch of 60mm. Find out the safe load per pitch length and efficiency of the joint.

Solution:

σ_{vf}= 100N/mm^2

σ_{pf}= 300 N/mm^2

σ_{at}= 0.6fy=150 Mpa (for fy = 250Mpa)

Nominal $\emptyset$ of the rivet, $\emptyset$ =20mm

Gross $\emptyset$ of rivet , d=20+1.5= 21.5

Pitch, s = 60mm

Strength of rivet in double shear = $2* \pi/4 \ d^2\sigma_{vf}$

$= 2* \pi/4 \ *21.5^2*100*10^{-3}$

=72.61KN

Thickness of cover plate > 5/8* t

= 5/8*16 = 10mm

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

Strength of the rivet in bearing = $dt\sigma_{pf}$

$= 21.5 *16*300*10^{-3}$

=103.200KN

Strength of the plate in tearing = $(s-d)t\sigma_{at}$

$=(60-21.5)*16*150*10^{-3}$

Strength of the joint per pitch length in shear is the minimum

Hence, strength of joint per pitch length =dt σ_{at}

$= 60*16*150*10^{-3}$

=144KN

Efficiency of joint = (strength of joint / strength of solid plate) *100

= (72.61/144) * 100

= 50.42 %

Problem 9: Fig shows a joint in lower chord of a roof rivets desing the riveted connection by use land driven riveted as shown in fig and use gusset plate of 12mm thick

Solution:

σ_{at}= 150Mpa σ_{ps}= 250 Mpaσ_{vf}= 80Mpa

Using gusset plate 12mm thick

Diameter of rivet $6.01\sqrt{10}$ = 19.01mm ≈ 20mm

Gross diameter = 20 + 1.5 =21.5mm

Member OB

Set of rivet in bearing on 8 mm thick angle = 250 * 21.5 * 8

Set of rivet in single shear = 80/ 1000 * $\pi/4$ * 21.5^2 =29 KN

R_v= 29KN

No. of rivet req= 56/29 = 1.9 ≈ 2

Member OC

Set of rivet in bearing 10mm to angle = 250*21.5*16 = 53.75KN

Set of rivet single shear = 80*$\pi/4$*21.5^2= 29KN

R_v=29KN

No of rivets =112/29=3.86≈ 4 rivet

Member AP

Set of rivet in bearing 12mm thick angle = 250 *21.5*12

=64.5KN

Set of rivet in double shear = 2 * 80 *$\pi/4$*21.5^2

=58KN

No of rivet = (284-187)/58 =1.6≈ 2rivet

For all rivet = 3D = 3*21.5 = 64.5 ≈ 65mm

Edge distance = 2D = 45mm

Due to moment

F_m= Mr/∈ r^2

Axial load

F_a= w/b

Direction

It is perpendicular to the line joint the rivet under consideration and example of the rivet group

Resultant force $F_r = \sqrt{(f_a^2+f_m^2+2*f_a*f_m\cos\theta)}$

θ is< between f_a&f_m

Problem 10: A bracket transmits a load of 80KN at an ecc of 30 cm to column through10 rivets of 22mm diameter arrayed in two vertical rows 10 cm apart. The pitch of the rivets is 80 the load lies in the plane of the rivets find max stress in the rivets

Solution:

$F_a = w/b$

$= 80/10 = 8KN$

Force due to moment $F_m = Mr/\in r^2$

$= 80*30*\sqrt{(16^2+5^2)}/(2*5^2+4(5^2+8^2)+4(5^2+16^2))$

Angle between I_m&F_a $\theta = \tan^{-1} 16/5$

Res for $F_r = \sqrt{(f_a^2+f_m^2+2*f_a*f_m\cos\theta)}$

$= \sqrt{(8^2+26.29^2+2*8*26.29\cos 72.646}$

$= 29.675$ KN

Max stress $= F_r/(\pi/4 * d^2)$

$= 29.675 * 10^3/\pi/4 * 23.5^2)$

$= 68.4$Mpa

Problem 11: 18mm thick plate is welded to 16mm plate by 200mm long effective bolt weld . Determine the strength of the joint

i) **Double v butt weld**
ii) **Single v butt weld**

Assume fe=410 N/mm^2 and shop weld fy=250 N/mm.^{2}

Solution:

Design strength:

Double v butt weld

Tk=16mm , effective length = 200mm, γ_{mw} = 1.25, fe=410 N/mm^2

Effective area= effective length X throat thickness

Design strength = $\frac{Lwt*\frac{fu}{\sqrt{3}}}{\gamma mw}$

= (200*16*410/$\sqrt{3}$)/ 1.25

= 605987N

=605.98KN

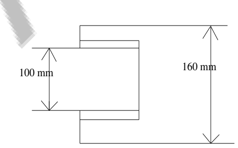
Single v butt weld joint
Due to incomplete penetration
Thickness= 5/8*tk
=(5/8)*16=10mm

Design strength=$\frac{Lwt*\frac{fu}{\sqrt{3}}}{\gamma mw}$
= (200*10*410/$\sqrt{3}$)/1.25

= 378742 N

=378.74 KN.

Problem 12: Design a suitable longitudinal fillet weld to connect the plates as shown. To transmit a pull equal to pull the strength of small plate. Given plate 12mm thick and grade of the plates fe=410. And welding to be made in workshop.



Minimum size =5mm

Max sixe = 12-15=10.5mm

Use s=10 mm fillet weld

Fy=410 N/mm, γ_{mw} = 1.25 ,tk=12mm, breadth of plate = 100 mm

∴ Full design strength of smaller plates = Agfy/ γ_{mo}

Fy=250 Mpa, γ_{mo}= 1.1

St of smaller plates = 12*100*250/1.1

= 272727 N

Let effective length of welds be Lw

Assume normal weld, throat thickness t= 0.7*20=7mm.

∴ design strength of weld = $\frac{Lwt*\frac{fu}{\sqrt{3}}}{\gamma mw}$

= Lw*7*410/√3*1/1.25

Equating it to strength of plate we get

Lw*7*410/√3*1/1.25=272727

Lw=205.7mm

Provide eff. length of 105mm.

Problem 13: Determine the max load that can be resisted by the bracket by fillet weld of size 6mm, if it is shop welding.

Solution:

Size of weld =6mm

∴ Throat thickness =0.7*6 = 4.2mm

Consider the area of the weld which has channel shape and has width 4.2mm throughout.

Total area of weld = 320*4.2+140*4.2*2

= 600*4.2 mm^2

Due to symmetry, centroidal x-x axis is at the mid height of vertical weld. Let centroidal y-yaxis be at a distance $\bar{x}$ from the vertical weld.then

$\bar{x}$= (140*4.2*70*2)/(600*4.2)= 32.67mm

$I_{xx}=(4.2*320^3/12)+(140*4.2*160^2*2)$

$=41574400mm^4$

$I_{yy} = (320*4.2*(32.67^2))+2((140^3*4.2/12)+(4.2*140*(70-32.67)^2)$

$= 4994080mm^4$

∴ $I_{zz}=I_{xx}+I_{yy}=46568480mm^4$.

$R_{max}=\sqrt{160^2+(140-32.64)^2}$

= 192.66mm

Tanθ= 160/(140-32.67)^2

∴θ=56.15^o

Eccentricity of load from the center of gravity of weld

e=240+140-32.67=347.33mm

letpe be in KN.

Direct shear stress q1=P*1000/total load

= P*1000/(600*4.2)

=0.3968P N/mm^2

Shear stress at extreme edge due to torsion moment

$Q_2=P*e*r_{max}/I_{zz}$

= P*1000*347.33*192.66/46568480

= 1.4370P N/mm^2

∴ resultant stress= $\sqrt{(q_1^2+q_2^2+2q_1q_2cos\theta)}$

= p$\sqrt{0.3968^2}+1.4370^2+2*0.3968*1.4370*cos56.15$

=1.69046P

Weld can resist a stress of = (f_u/√3)*(1/1.25)=410/(√3*1.25)=189.32

Equating max stress to resisting stress we get

2.69046P=189.32

∴P=111996 N = 111.996 KN.

Problem 14: An I section bracket is connected to a column by welds. Determine the load which can be safely carried the size of the web weld is 5mm while the size of flange weld is 10mm. assume field welds

Solution

Assuming normal fillet welds

Throat thickness of flange weld=0.7*10=7mm

Throat thickness of web weld=0.7*5=3.5mm

∴ total throat area of weld= 165*7*2+250*3.5*2=4060mm^2

I_{xx}=165*7*200^2*2+(3.5*250^3*2)=101514583mm^4

Bending moment to be transferred is=P*250 KN-mm (if P is factored load in KN)

= P*250*1000 N-mm

Consider flange weld which is subjected to max stress.

Q_v=P*1000/4060 N/mm^2 = 0.2463P N/mm^2

Due to bending,

Q_h=(250*1000P/101514583)*200 = 0.49254P N/mm^2

$\therefore$ Q=$\sqrt{(q_v^2 + q_h^2)}$

= p $\sqrt{(0.2463^2 + 0.49254^2)}$ =0.30326P

Equating it to design stress (f_u/√3)*(1/1.5) = 410/ (√3*1.25) we get 0.30326P

$\therefore$ P=520.4 KN

$\therefore$ Working load that can be permitted is = P/1.5=520.4/1.5= 347 KN

1.7 HSFG bolts

1. These bolts made form high strength steel rods
2. Surface of the hank is kept unfinished as in the case of black bolts.
3. These bolts are tightened to a proof load using calibrated wrenches., hence they grip the member tightly.
4. If the joint is subjected to shearing load it is primarily resisted by frictional force between the members and washers.
5. The shank of the bolts is not subjected to any shearing. The results into no-slippage in the joint

Advantage of HSFG bolts

1. Joint are rigid(no slip takes places in the joint)

2. As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.
3. High static strength due to high frictional resistance
4. High fatigue strength since nuts are prevented from loosening and stress concentration avoided due to friction grip
5. Smaller number of bolts results into smaller sizes of gusset plates.

Disadvantage of HSFG bolts

1. Material cost is high
2. The special attention is to be given to workmanship

QUESTION BANK:

Part-A (2 Marks)

1 Define riveting?
2 Define Pitch of rivets?
3 Define Gauge of rivets?
4 What are different types of connections?
5 List out type of riveted joints?
6 What are the advantages of HFSG bolts over other bolts?
7 Write the formula to check the bolted connection subjected to combined shear and tension?
8 Write the disadvantage of bolted connection?
9 Define edge distance?
10 List type of bolts?
11 List out the type of welds?
12 Write some disadvantages of welding?
13 Write any four disadvantages of welded connections?
14 Describe the effective length of the fillet weld?
15 Write about the design stress in fillet welds?
16 Write about the effective throat thickness of butt welds?
17 Write about the intermittent fillet welds?
18 Define lack of fusion?
19 Define throat thickness of welding?
20 Define fillet welding?

Part-B (16 Marks)

1 Design a bracket connection to transfer a load of 150KN as shown. The load is acting at an eccentricity of 300mm from the face of the column. Design riveted connection at section AA.

2 Design the riveted connection by using hand driven rivets and gusset plate of 12mm thick for roof truss as shown in fig?

3 Find the efficiency of lap joint 20mm tk 180mm wide at a pitch & gauge of 60mm and edge distance of 30mm with chain double bolting. Use M20 bolts of grade 4.6 and Fe 410(E250) plates are used.

4 Design a suitable bolted bracket connection of a ISHT-75 section attached to the flange of a ISHB 300 at 588 N/m to carry a vertical factored load of 600kN at an eccentricity of 300mm.use M24 bolts of grade 4.6.

5 A) Explain the failures of riveted joints? (7)
B) Calculate the value of a rivet in a lap joint used to connect two plates 12mm thick in the following cases a) Power driven rivets and b) Hand driven rivets. (8)

6 Determine the maximum load that can be resisted by the bracket shown in fig. by fillet weld of size 6 mm, if it is shop welding ?

7 A) Write the advantages of welding? (8)
B) Write the defects of weld with neat sketch? (7)

8 An 18mm thick Plate is jointed to a 16 mm plate by 200mm long (effective) butt weld. Determine the strength of joint if a double V butt weld is used. Assume the Fe410 grade plates and shop welds are used.

9 Design a suitable longitudinal fillet weld to connect the plates as shown in fig to transmit a pull equal to the full strength of small plate. Given plates are 12mm thick, Grade of plates Fe410 and welding to be made in workshop.

10 A tie member of roof truss consists of 2ISA 100 x 75 x8mm.The angles are connected to either sides of a 10mm gusset plates and member is subjected to working pull of 300kN, Design welded connection. Assume connection in workshop.

CE8601-DESIGN OF STEEL STRUCTURAL ELEMENTS

UNIT-3

TENSION MEMBERS

CONTENTS

TECHNICAL TERMS:-

- **Tension Member-**A tension member is a structural member subjected to tensile force in the direction parallel to its longitudinal axis.
- **Tie Member-** A tension member is also called as a tie member or simply a tie.
- **Built - up section-** A built - up section may be made of two channels placed back to back with a gusset plate in between them. Such sections are used for medium loads in a truss.
- **Heavy Built - up Sections-** The heavy built - up sections are used in the bridge truss girders are made up of angles and plates. Such members can resist compression if the reversal of stresses.
- **Net Sectional area-** The net sectional area of a tension member is the gross sectional area of the member less than the maximum deduction for holes.
- **Tension Member Splice-**Tension splices are provided on both the sides of member jointed, in the form of cover plates, so as to form a butt joint.
- **Lug Angle-** A lug angle is a short length of an angle section, which is attached to the main tension member at the connecting end.

TYPES OF TENSION MEMBERS

1. Rolled sections
    - Angle section
    - T- Section
    - Channel Section
    - Rectangular section
    - Circular section
2. Compound sections
    - Double angle section
    - Compound channel section
    - Compound angle section
3. Built-up sections
4. Threaded round bar
5. Flat bar
6. Round strand rope
7. Locked coil rope

NET AREA

Chain riveting

A_{net} =t(b-nd)

Zigzag riveting

A_{net} =t[(b-nd)+ms^2/4g]

Where,
t= thikness of plate
d=diameter of the hole
n= no of rivets in the section considered
m =no of zigzag or inclined lines.

NET EFFECTIVE AREA

The net cross sectional area on a section is reduced to account for this non-uniform stress distribution resulting from eccentricity. The reduced net sectional area of such a section is known as net effective area.

NET EFFECTIVE AREA OF A SINGLE ANGLE USED AS TENSION MEMBER

In case of single angle connected through one leg the

Net sectional area =A1+A2k

Where, A1=effective cross sectional area of the connected leg

A2=gross cross sectional area of the un connected leg

K=3A1/(3A1+A2)

SPECIFICATION FOR THE DESIGN OF LUG ANGLE

•Lug angle connected a channel shaped member shall, as far as possible, be disposed symmetrically with respect to the section.

•In no case shall fewer than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

NET EFFECTIVE AREA OF A DOUBLE ANGLE USED AS TENSION MEMBER

In case of double angle connected through one leg,

Net sectional area =A1+A2k

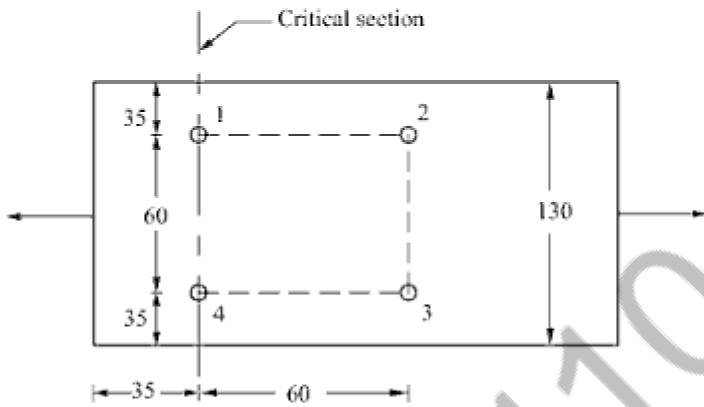
Where, A1=effective cross sectional area of the connected leg

A2=gross cross sectional area of the un connected leg

K=5A1/(5A1+A2)

DESIGN OF CONNECTIONS IN TENSION MEMBERS

Problem 1: Determine the design tensile strength of the plate 200mm×12mm with the holes for 16mm diameter bolts as shown in fig. Steel used is of Fe 415 grade quality.



Solution:

Strength of the plate is the least of

(a) Yielding of gross section
(b) Rupture of critical section
(c) The block shear strength

(a) From consideration of yielding
$T_{dg} = A_g f_y / \gamma_{mo}$

Now, $A_g = 130 \times 12 = 1560mm^2$, $f_y = 250N/mm^2$, $\gamma_{mo} = 1.1$

$T_{dg} = 1560 \times 250 / 1.1$

$= 354.545kN$

(b) From consideration of Rupture along the critical section
Critical section is having two holes
Diameter of holes = 16+2 = 18mm
$A_n = (130 - 2\times18) \times 12 = 1128mm^2$
Strength of member From consideration of rupture
$T_{dn} = 0.9 A_n f_u / \gamma_{ml}$
$= 0.9 \times 1128 \times 410 / 1.25$
$= 332.986kN$

(c) Block shear strength

$A_{vg} = (35 + 60) \times 12 = 1140mm^2$

$A_{tg} = 60 \times 12 = 720mm^2$

$A_{vn} = (35+60 - 1\times22) \times 12 = 856\ mm^2$

$A_{tn} = (60 - 22) \times 12 = 456\ mm^2$

The block shear strength is the least of the following two:

(1) $T_{db} = (A_{vg}f_y/\gamma_{mo}\sqrt{3}) + (0.9A_{tn}f_u/\gamma_{ml})$

$= (1140 \times 250 / \sqrt{3}\times1.1) + (0.9 \times 446 \times 410 / 1.25)$

$= 284.198kN$

(2) $T_{db} = (0.9A_{vn}f_u/\gamma_{ml}\sqrt{3}) + (A_{tg}f_y/\gamma_{mo})$

$= (0.9 \times 856 \times 410 / 1.25\times\sqrt{3}) + (720 \times 250 / 1.1)$

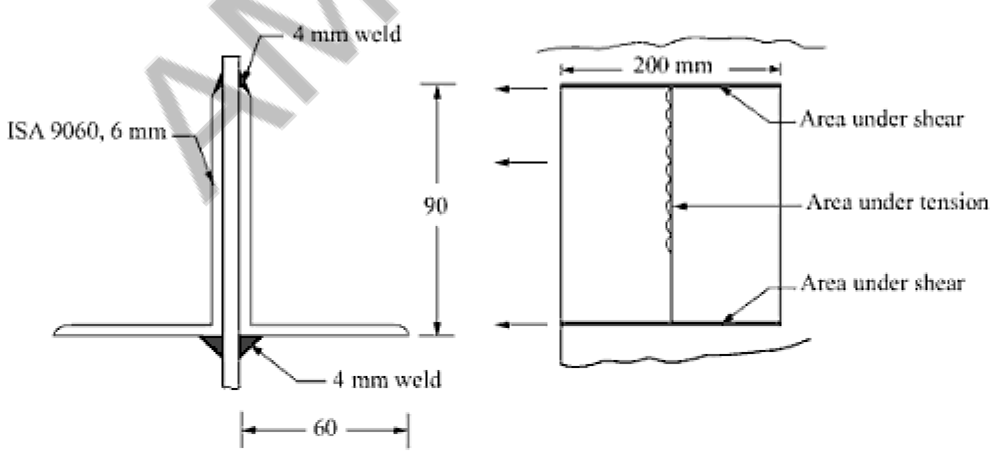
$= 312.936kN$

$T_{db} = 284.198kN$

The least strength is inblock shear

Strength of the plate = 284.198kN

Problem 2: Determine the tensile strength of a roof of truss member 2 ISA9060, 6mm connected to the gusset plate of 8mm plate by 4mm weld as shown in fig. The effective length of weld is 200mm.



Solution:

Gross area of angles, $A_g = 2 \times (90+60 - 6) \times 6 = 1728\ mm^2$

Area of the connected leg, $A_{nc} = 2 \times (90 - 6/2) \times 6 = 1044\ mm^2$

Area of the connected leg, $A_{go} = 2 \times (60 - 6/2) \times 6 = 684\ mm^2$

(i) Strength governed by yielding

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

$= 1728 \times 250 / 1.1$

$= 392.727kN$

(ii) Strength of plate in rupture at critical section

$T_{dn} = (0.9A_{nc}f_u/\gamma_{ml}) + (\beta A_{go}f_y/\gamma_{mo})$

$\beta = 1.4 - 0.076 \times w/t \times f_y/f_u \times b_s/L_w$

$w = 90mm$, $t = 6mm$, $f_y = 250N/mm^2$, $f_u = 410N/mm^2$, $b_s = w = 90mm$, $L_w = 200mm$

$\beta = 1.4 - 0.076 \times 90/6 \times 250/410 \times 90/200$

$= 1.087$

$T_{dn} = (0.9 \times 1044 \times 410 / 1.25) + (1.087 \times 684 \times 250 / 1.1)$

$= 477.168kN$

(iii) Strength governed by block shear:

Area under shear, $A_{vg} = A_{vn} = 2 \times 200 \times 6 = 3200\ mm^2$

Area under tension at failure, $A_{tg} = A_{tn} = 90 \times 8 = 720\ mm^2$

The block shear strength is the least of the following two:

(1) $T_{db} = (A_{vg}f_y/\gamma_{mo}\sqrt{3}) + (0.9A_{tn}f_u/\gamma_{ml})$

$= (3200 \times 250 / \sqrt{3} \times 1.1) + (0.9 \times 720 \times 410 / 1.25)$

$= 632.435kN$

(2) $T_{db} = (0.9A_{vn}f_u/\gamma_{ml}\sqrt{3}) + (A_{tg}f_y/\gamma_{mo})$

$= (0.9 \times 3200 \times 410 / 1.25 \times \sqrt{3}) + (720 \times 250 / 1.1)$

$= 709.025kN$

$T_{db} = 632.435kN$

Strength of the tension member = 392.727kN

Problem 3: Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225kN. The member is subjected to the possible reversal of stress due to the action of wind. The length of the member is 3m.Use 20mm shop bolts of grade 4.6 for the connection.

Solution:

From consideration of yieldstrength, gross area of angle required

$= 225\times1000\times1.1/ 250$

$= 990\ mm^2$

Try IAS 10075, 8mm which has gross area $A_g = 1336\ mm^2$

Number of bolts required: $d = 20mm$, $d_o = 22mm$

Use gusset plate of thickness 10mm.

Strength of one bolt in single shear $= 400 \times \pi \times 20^2 \times 0.78 / \sqrt{3} \times 1.25 \times 4 = 45272N$

Adopting edge distance e = 40mm; p = 60mm

K_b is smaller of 40/3×22, 60/3×22–0.25, 400/410, 1

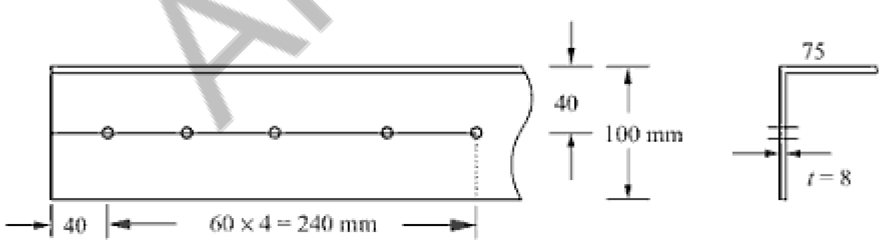
$K_b = 0.606$

$V_{dpb} = 2.5\times0.606\times20\times8\times400/1.25 = 77568N$

Bolt value = 45272N

Number of bolts required = 225×1000/45272 = 5

Provide the bolts as shown in fig



Checking the design:

(a) Strength governed by yielding

$= A_g f_y / \gamma_{mo}$

$= 1336 \times 250 / 1.1$

$= 303636N > 225000N$ O.K

(b) Strength of plate in rupture

Area of connected leg, $A_{nc} = (100 - 22 - 8/2) \times 8 = 592\ mm^2$

Area of connected leg, $A_{go} = (75 - 8/2) \times 8 = 568\ mm^2$

$\beta = 1.4 - 0.076 \times 75/8 \times 250/410 \times (75+40-8) / (40+4\times60-4\times22)$

$= 1.094$

$T_{dn} = (0.9 A_{nc} f_u / \gamma_{ml}) + (\beta A_{go} f_y / \gamma_{mo})$

$= (0.9 \times 592 \times 410 / 1.25) + (1.094 \times 568 \times 250 / 1.1)$

$= 316006N > 225000N$ O.K

(c) Block shear strength

$A_{vg} = (40 + 60\times2) \times 8 = 2240\ mm^2$

$A_{tg} = (100-40) \times 8 = 480 mm^2$

$A_{vn} = (40+60 \times4 - 4.5\times22) \times 8 = 1448\ mm^2$

$A_{tn} = (100 - 40 \times0.5 \times22) \times 8 = 392\ mm^2$

The block shear strength is the least of the following two:

(1) $T_{db} = (A_{vg} f_y / \gamma_{mo} \sqrt{3}) + (0.9 A_{tn} f_u / \gamma_{ml})$

$= (2240 \times 250 / \sqrt{3}\times1.1) + (0.9 \times 392 \times 410 / 1.25)$

$= 409642N$

(2) $T_{db} = (0.9 A_{vn} f_u / \gamma_{ml} \sqrt{3}) + (A_{tg} f_y / \gamma_{mo})$

$= (0.9 \times 1448 \times 410 / 1.25\sqrt{3}) + (480 \times 250 / 1.1)$

$= 355879N$

$T_{db} = 355879N > 225000N$ O.K

Hence the design is safe.

Check for maximum l/r

r –radius of gyration = 12.7 (from steel table)

$l/r = 3000/12.7 = 236 < 350$

Hence O.K

DESIGN OF TENSION SPLICES

Problem 1: Design a splice to connect a 300× 20mm plate with a 300× 10mm plate. The design load is 500kN.Use 20mm black bolts, fabricated in the shop.

Solution:

Let double cover butt joint with 6mm cover plates be used.

Strength of bolts:

$d = 20mm$, $d_o = 22mm$, $\beta_{pk} = 1 - 0.0125 \times 10 = 0.875$

Strength indouble shear = $\beta_{pk}(\pi d^2 + 0.78\pi d^2)\ f_u / (4 \times \sqrt{3} \times 1.25)$

$= 0.875 \times 1.78 \times 20^2 \times \pi \times 410 / (4 \times 1.25 \times \sqrt{3})$

$= 92660N$

Strength in bearing:

Adopting edge distance e = 40mm; p = 60mm

K_b is smaller of 40/3×22, 60/3×22–0.25, 400/410, 1

$K_b = 0.606$

Strength in bearing against 10mm plate

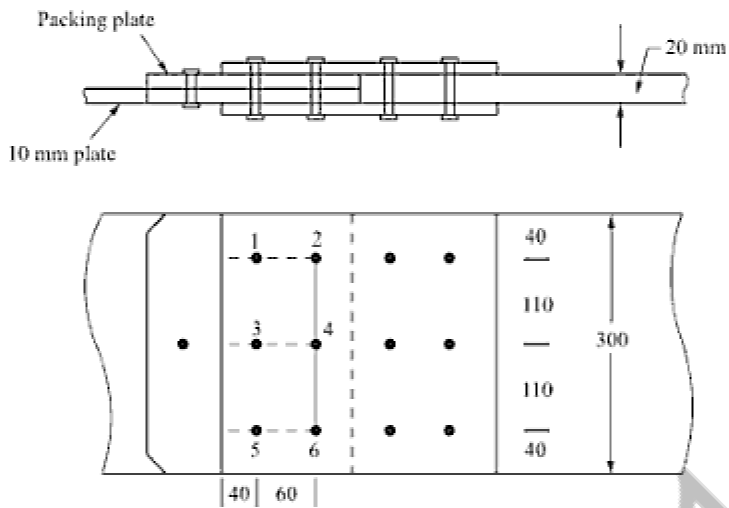
$= 2.5K_b \times d \times t \times f_u/\gamma_{mb}$

$= 2.5 \times 0.606 \times 20 \times 10 \times 400/1.25 = 105897N$

Bolt value = 92660N

Number of bolts required = 225×1000/92660 = 6

Provide 6 bolts on each side of the joint as shown in fig



Checking the design:

(a) Strength governed by yielding

$= A_g f_y/\gamma_{mo}$

$= 300\times10 \times 250 / 1.1$

$= 681818N > 500000N$ O.K

(b) Strength of plate in rupture

$A_n = (300 - 3\times22) \times 10 = 2340mm^2$

$T_{dn} = 0.9A_nf_u/\gamma_{ml}$

$= 0.9 \times 2340 \times 410 / 1.25$

$= 690768N > 500000N$ O.K

(c) Block shear strength

$A_{vg} = 3(40 + 60) \times 10 = 3000\ mm^2$

$A_{tg}= (300–40–40) \times 10 = 2200mm^2$

$A_{vn} = 3(40+60 \times4 - 1.5\times22) \times 10 = 2010\ mm^2$

$A_{tn} = (220 - 2 \times22) \times 10 = 1760\ mm^2$

The block shear strength is the least of the following two:

(1) $T_{db} = (A_{vg}f_y/\gamma_{mo}\sqrt{3}) + (0.9A_{tn}f_u/\gamma_{ml})$

$= (3000 \times 250 / \sqrt{3}\times1.1) + (0.9 \times 1760 \times 410 / 1.25)$

$= 913200N$

(2) $T_{db} = (0.9A_{vn}f_u/\gamma_{ml}\sqrt{3}) + (A_{tg}f_y/\gamma_{mo})$

$= (0.9 \times 2010 \times 410 / 1.25\sqrt{3}) + (2200 \times 250 / 1.1)$

$= 842572N$

$T_{db} = 842572N > 500000N$ O.K

Hence the design is safe.

Provide an extra bolt in the cover plate on packing material.

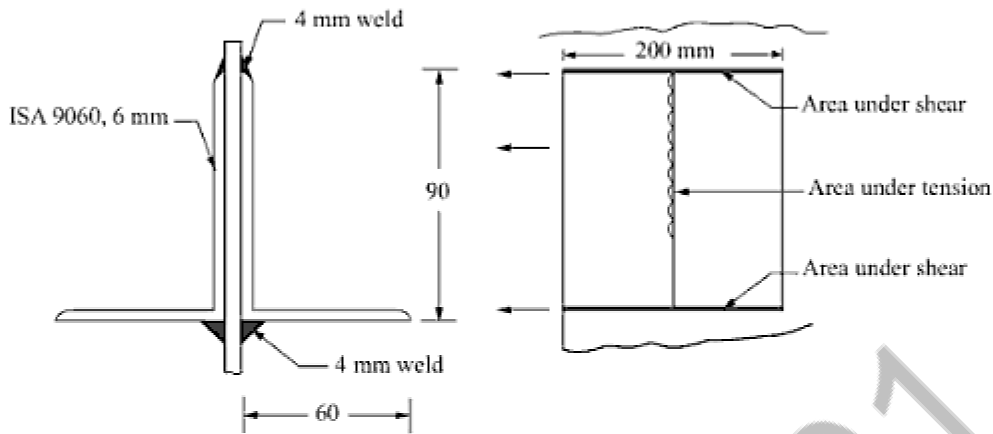
QUESTION BANK

Part A-(2Marks)

1. List some of the tension members used in buildings and bridges.
2. What are the factors that influence the strength of tension members?
3. List the type of cross section that can be used as tension members and their use in typical structures.
4. How can the rods, which are used as tension members, be pre-tensioned?
5. Why are rods, which are used as tension members, required to be pre-tensioned?
6. Write a short note on the use of ropes and strands in bridges?
7. What is Tension Member?
8. What is Tie Member?
9. What do you meant by Built - up section?
10. Define Net Sectional area?

Part B- (16 Marks)

1. Design a splice to connect a 400× 30mm plate with a 350×10mm plate. The design load is 550kN.Use 20mm black bolts, fabricated in the shop.
2. Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 500kN. The member is subjected to the possible reversal of stress due to the action of wind. The length of the member is 5m.Use 20mm shop bolts of grade 4.6 for the connection.
3. Determine the tensile strength of a roof of truss member 2 ISA9060, 6mm connected to the gusset plate of 8mm plate by 4mm weld as shown in fig. The effective length of weld is 200mm.



4. Determine the design tensile strength of the plate 200mm×12mm with the holes for 16mm diameter bolts. Steel used is of Fe 415 grade quality.
5. Determine the tension capacity of 150x90x8mm angles in Fe410 steel assuming
   a) Connection through the longer leg by two rows of M20 bolts and
   b) Connection through the shorter leg by single row of M24 bolts.

DESIGN OF STEEL STRUCTURAL ELEMENTS

UNIT-4

COMPRESSION MEMBERS

CONTENTS

TECHNICAL TERMS:-

- **Compression Member-** A compression member is primarily designed to carry axial compression
- **Column (Stanchion)** – Any structural member loaded axially in compression is called a column. The term stanchion is used for columns.
- **Struts-** A strut is a structural member subjected to compression in a direction parallel to its longitudinal axis.
- **Radius of Gyration-** The radius of gyration of a section is a geometrical property of the section and is denoted by (r).
- **Slenderness ratio-** Slenderness ratio is the ratio between the effective lengths to the appropriate radius of gyration and is denoted by (A.)
- **Lacings-**Lacings are nothing but lateral system that is needed to connect the elemental sections, so that they act as a composite section rather than acting individually.
- **Batten plates -** Batten plates normally consist of flats or plates connecting the components of the built - up columns in two parallel planes.
- **Tie plate-**The end batten is usually known as a tie plate.

TYPES OF COMPRESSION MEMBERS

Column, top chords of trusses, diagonals and bracing members are all examples of compression members. Columns are usually thought of as straight compression members whose lengths are considerably greater than their cross-sectional dimensions.

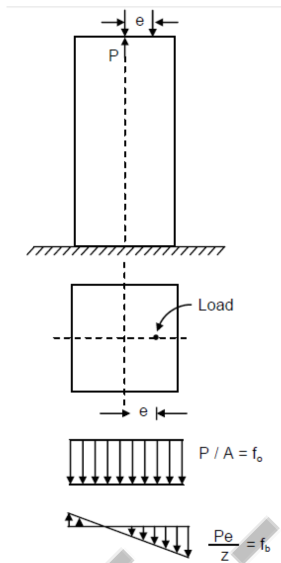
An initially straight strut or column, compressed by gradually increasing equal and opposite axial forces at the ends is considered first. Columns and struts are termed **"long"** or **"*short*"** depending on their proneness to buckling. If the strut is **"short"**, the applied forces will cause a compressive 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 yields or **"squashes"**. However, if the strut is **"long"**, similar axial shortening is observed only at the initial stages of incremental loading. Thereafter, as the applied forces are increased in magnitude, the strut becomes **"unstable"** and develops a deformation in a direction normal to the loading axis and its axis is no longer straight. The strut is said to have "buckled".

Structural steel has high yield strength and ultimate strength compared with other construction materials. Hence compression members made of steel tend to be slender compared with reinforced concrete or prestressed concrete compression members. Buckling is of particular interest while employing slender steel members. Members fabricated from steel plating or sheeting and subjected to compressive stresses also experience local buckling of the plate elements. This chapter introduces buckling in the context of axially compressed struts and identifies the factors governing the buckling behaviour. Both global and local buckling is instability phenomena and should be avoided by an adequate margin of safety.

Traditionally, the design of compression members was based on Euler analysis of ideal columns which gives an upper band to the buckling load. However, practical columns are far from ideal and buckle at much lower loads. The first significant step in the design procedures for such columns was the use of Perry Robertson's curves. Modern codes advocate the use of multiple-column curves for design. Although these design procedures are more accurate in predicting the buckling load of practical columns, Euler's theory helps in the understanding of the behaviour of slender columns and is reviewed in the following sections.

THEORY OF COLUMNS

In order to study the effect of eccentric load more closely, let us consider a short axial member, loaded with load *P*, placed at a distance '*e*' from the centroidal vertical axis through the centroid of the section, as shown in Figure



Along the vertical axis, introduce two equal and opposite forces, each equal to load *P*. Their introduction obviously makes to difference to the loading of the member, as they cancel out each other. However, if the upward force *P* is considered along with at a distance *e* from each other, from a clockwise couple of magnitude $P \times e$, the effect of which is to produce bending stress in the member. The remaining central downward force *P*, produces a direct compressive stress f_0, of magnitude *P*/*A* as usual. Hence, we can conclude that an eccentric load produces direct compressive stress as well as the bending stress.

The bending couple $P \times e$ will cause longitudinal tensile and compressive stresses. The fibre stress due to bending f_0, at any distance '*y*' from the neutral axis is given by,

$$f_b = \frac{M}{I_{xx}} \times y = \frac{P \times e \times y}{I_{xx}} \qquad \text{(tensile or compressive)}$$

Hence, the total stress at any section is given by

$$f = f_0 \pm f_b = \frac{P}{A} \pm \frac{P \times e \times y}{I_{xx}}$$

$$f = \frac{P}{A} \pm \frac{M}{Z_{xx}}$$

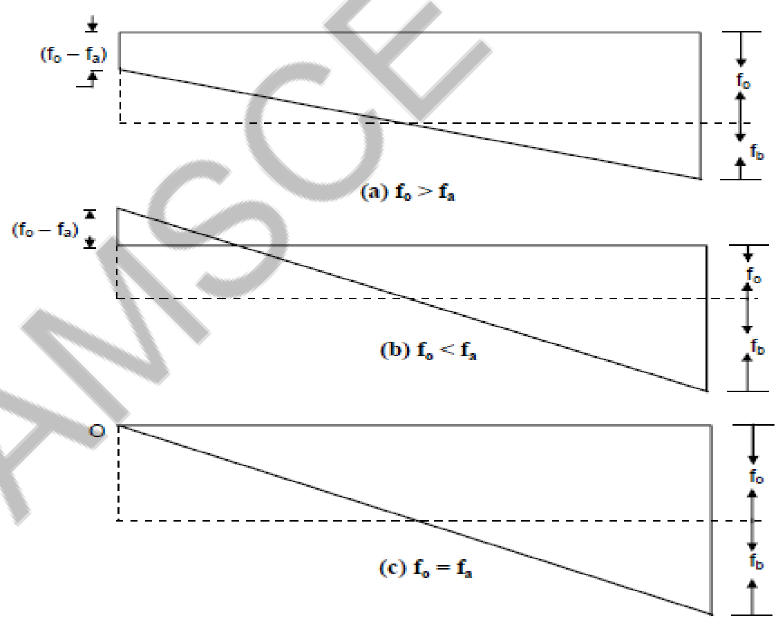
[where $P \times e = M$ and $\frac{I_{xx}}{y} = Z_{xx}$ (the section modulus)

The extreme fibre stresses are given by,

$$f_{\max} = f_0 + f_b = \frac{P}{A} + \frac{M}{Z_{xx}}$$

and $$f_{\min} = f_0 - f_b = \frac{P}{A} - \frac{M}{Z_{xx}}$$

If f_0 is greater than f_b, the stress throughout the section will be of the same sign. If however, f_0 is less than f_b, the stress will change sign, being partly tensile and partly compressive across the section. Thus, there can be three possible stress distributions as shown in Figures



BASIS OF CURRENT CODAL PROVISION FOR COMPRESSION MEMBER DESIGN

Design compressive strength:
The design compressive strength P_d, of a memberis given by:

$$P < P_d$$

where

$$P_d = A_c f_{cd}$$

The design compressive stress, f_{cd}, of axially loaded compression members shall be calculated using the following equation:

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

SLENDERNESS RATIO

Slenderness ratio is the ratio between the effective lengths to the appropriate radius of gyration and is denoted by (λ.)

DESIGN OF SINGLE SECTION AND COMPOUND SECTION COMPRESSION MEMBERS

Problem 1: Design a column support a factored axial load of 1100kN. The column has an effective length of 6m with respect to the z-axis and 4m with respect to the y-axis. Use 410 grade steel.

Solution:
Assume the design compressive stress as f_{cd}= 0.6f_y= 150MPa.

Required area = 1100 x 10^3/150= 7333mm^2

Try ISHB 300 @ 588 N/m. The relevant properties of the section are as follows

A= 7480mm^2; r_z= 130mm; r_y= 54.1mm; h/b = 300/250 = 1.2, t_f= 10.6mm;

L/r_z= 6000/130 = 46.15 < 180. Hence OK.

L/r_y = 4000/54.1 = 73.94< 180. Hence ok.

L/r_y= 73.94 and f_y= 250MPa

F_{cd}= 159.7 N/mm^2

Therefore, strength of column = 159.7 x 7480/1000 = 1194.5kN> 1100kN.

Hence, The chosen section is safe.

Problem 2: The strut of a space frame member having a length of 3m has to carry a factored load of 230kN. Assuming f_y= 220N/mm^2, design a compound section to carry the load. Assume that the ends are simply supported.

Solution:

Assume medium class pipe of outside dia 114.3mm with thickness 4.5mm, weight = 121 N/m, r= 38.9mm and area= 1550mm^2.

i) Check for limiting width to thickness ratio

D/t = 114.3/4.5 = 25.4 < 42 e^2

42 x 0.938^2 = 36.95

Hence, the tube section is a plastic section.

ii) Resistance to flexural buckling.

KL/r = 1 x 3000/ 38.9 = 77.12

f_y= 220 N/mm^2 and KL/r = 77.12

f_{cd}= 157.45 N/mm^2

Design strength = A x f_{cd}= 1550 x 157.45 / 1000 = 244kN> 230kN.

Hence safe.

3.6 DESIGN OF LACING AND BATTENING

Problem 3: Design a laced column 10m long to carry a factored load of 1100kN. The column is restrained in position but not in direction at both ends. Provide single lacing system and determine the spacing of lacing.

Assume the design compressive stress as f_{cd}= 0.6f_y= 150MPa.

Required area = 1100 x 10^3/150= 7333mm^2

Try ISHB 300 @ 588 N/m. The relevant properties of the section are as follows

A= 7480mm^2; r_z= 130mm; r_y= 54.1mm; h/b = 300/250 = 1.2, t_f= 10.6mm;

L/r_z= 6000/130 = 46.15 < 180. Hence OK.

L/r_y = 4000/54.1 = 73.94< 180. Hence ok.

L/r_y= 73.94 and f_y= 250MPa

F_{cd}= 159.7 N/mm^2

Therefore, strength of column = 159.7 x 7480/1000 = 1194.5kN> 1100kN.

Hence, the chosen section is safe.

Spacing of lacing channels:

$2I_z = 2(I_y + A(S/2 + C_y)^2)$

$2 \times 6420 \times 10^4 = 2(313x104 + 4630\ (S/2 + 23.5)^2)$

S= 182.70mm.

Keep the lacing channels at a spacing of 183mm.

Problem 4: Design a batten column system 10m long to carry a factored load of 1100kN. The column is restrained in position but not in direction at both ends. Provide single batten system and determine the spacing of batten.

Assume the design compressive stress as $f_{cd} = 0.6f_y = 150$MPa.

Required area = $1100 \times 10^3/150 = 7333\text{mm}^2$

Try ISHB 300 @ 588 N/m. The relevant properties of the section are as follows

$A = 7480\text{mm}^2$; $r_z = 130$mm; $r_y = 54.1$mm; h/b = 300/250 = 1.2, $t_f = 10.6$mm;

$L/r_z = 6000/130 = 46.15 < 180$. Hence OK.

$L/r_y = 4000/54.1 = 73.94 < 180$. Hence ok.

$L/r_y = 73.94$ and $f_y = 250$MPa

$F_{cd} = 159.7\ \text{N/mm}^2$

Therefore, strength of column = 159.7 x 7480/1000 = 1194.5kN> 1100kN.

Hence, the chosen section is safe.

Spacing of battens:

(L_o/r_y) should be less than 0.7 times the slenderness of the built-up column.

i.e., $L_o/r_y < 0.7\ (L/r)$

$L_0 < 0.7\ r_y\ (L/r)$

0.7 x 26 x 93.21 = 1696.4mm

Also, $L_o/r_y < 50$

$L_o < 50 \times 26 = 1300$mm.

Hence provide battens at a spacing of 1250mm.

DESIGN OF COLUMN BASE AND GUSSETED BASE

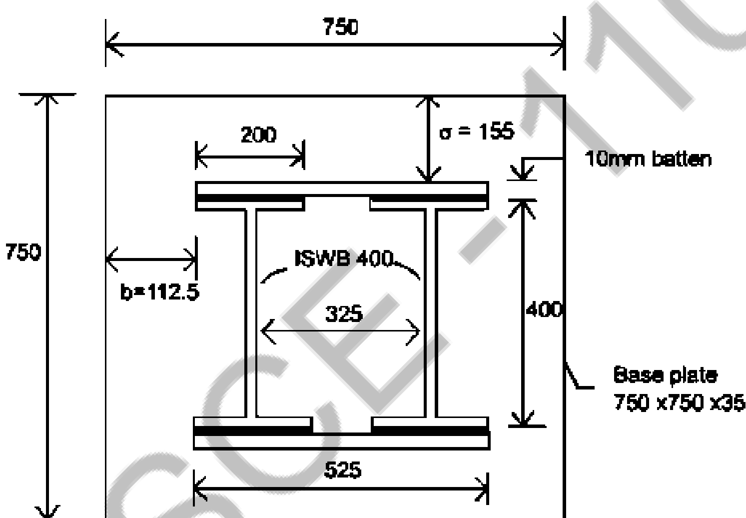
Problem 5: A built-up steel column comprising two ISWB 400 RSJ sections with their webs spaced at 325mm and connected by 10 mm thick battens transmits an axial load of 2000 kN. SBC of boil at site=300kN/m2. The safe permissible stress on the concrete bed= 4N/mm2. Design a suitable foundation for the column adopting a column base, gusseted base, and sketch the details of the foundation.

1. Data

Axial load on column = 2000kN

Permissible compressive stress on concrete = 4N/mm2

Column built up of two ISWB 400 RSJ sections connected by 10 mm thick battens.



2. Column base

Area of base plate = (2000 / 4000) = 0.5 m2

using a square base plate,

Side length of base plate =$5^{1/2}$ = 0.706 m

Adopt a base plate of size 750mm x 750mm.

Referring to figure 8.10, the projection of the base plate from the edge of the column is obtained as

a = greater projection

= 0.5(750 - 420) = 165 mm

b = smaller projection

= 0.5(750 - 525) = 112.5 mm

Intensity of pressure on base plate

= 3.56 N/mm2

Permissible bearing stress in base plate sbs = 185N/mm2

The thickness of the base plate is obtained from the relation

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

Aiming the thickness of base plate

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

$$= \sqrt{\frac{3 \text{ x } 3.56}{185}\left(165^2 - \frac{112.5^2}{4}\right)} = 37\text{mm}$$

Adopt a base plate of size 750mm x 750mm x 40mm.

3. Cleat angle

For connecting the column section to the base plate, adopt ISA 100 x 100 x 10 mm angles with four 22mm diameter rivets on flange side and ISA 75 x 75 x 8 mm with three 22mm diameter rivets in the webs.

4. Size of base plate

Area of base plate = (2000 / 4000) = 0.5 m2

Adopt ISA 150 x 100 x 12 mm gusset angles on the flange side with

100mm leg horizontal, gusset plate 12mm thick, 10 mm batten, and cover plates.

Minimum length required allowing 30mm projection on either side in the direction

parallel to the webs

= (400 + 20 + 24 + 200 + 60) = 704mm

Length of base plate parallel to the flanges = 750mm.

Adopt a base plate of size 750mm x 750mm

5. Thickness of base plate

Intensity of pressure below the plate

w = (2000 x 10^3 / 750 x 750) = 3.55 Nmm^2

cantilever projection of plate from the face of the gusset angle = 141 mm.

Bending moment

$$M = \left(\frac{wL^2}{2}\right) = \left(\frac{3.55 \text{ x } 141^2}{2}\right) = 35288 \text{ N / mm}$$

If t = thickness of plate required,

$$M = \left(\frac{\sigma_{bs} bt^2}{6}\right)$$

$$t = \sqrt{\frac{6M}{\sigma_{bs} b}} = \sqrt{\frac{6 \text{ x } 35288}{185 \text{ x } 1}} = 33.8 \text{ mm}$$

In LSD, No allowable being stress,

In WSD σb allow =0.75 fy

Thickness of base plate = (t - thickness of angle leg) = (33.8 - 12) = 21.8mm

Adopt 750 x 750 x 22 mm base plate.

6. Connections

Outstand on each side = (750 - 400)/2 = 175 mm

Load on each connection = (175 x 750 x 3.55)/1000 = 466 kN

Using 22mm diameter rivets,

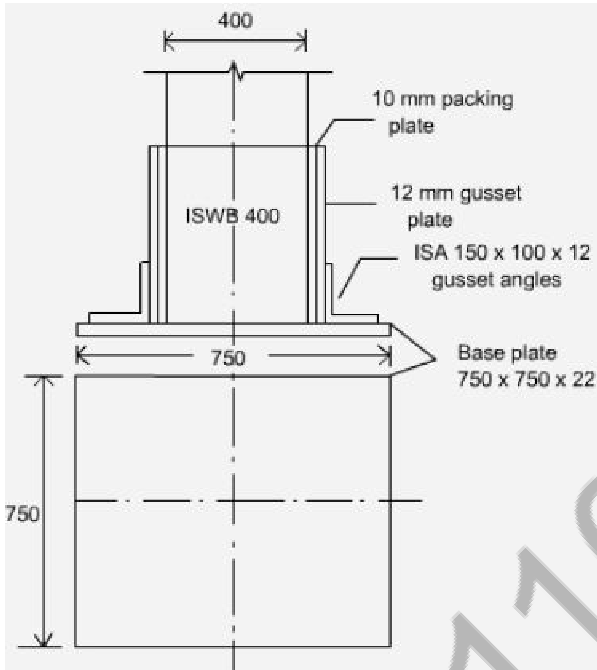
Rivet value in single shear = $\left(\frac{\pi \text{ x } 23.5^2 \text{ x } 100}{4 \text{ x } 1000}\right) = 43.4 \text{ kN}$

Rivet value in bearing = $\left(\frac{23.5 \text{ x} 12 \text{ x} 300}{1000}\right) = 84.6 \text{kN}$

Therefore least value of rivet = 43.4kN

Number of rivets = (466/43.4) = 11

Adopt 16 rivets connecting gusset angles with plate and the same number of rivets to connect the gusset plate with the column. The arrangement of rivets and the details of the gusseted base are shown in Fig.



Details of base plate

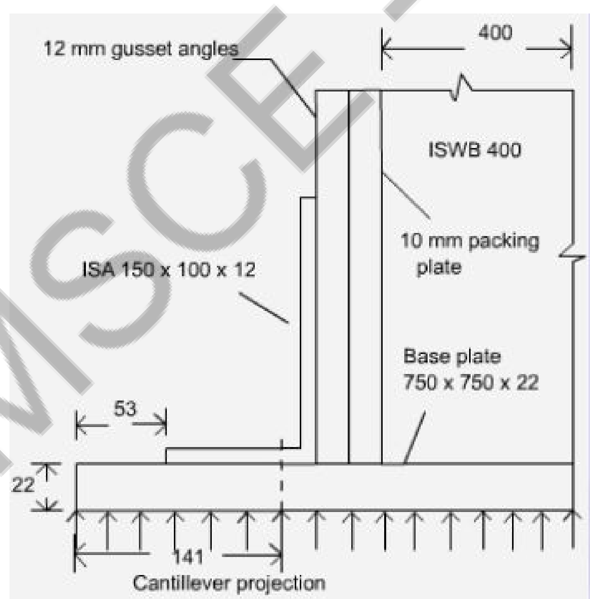


Fig 3.1 Cantilever projection
Gusset and base plate details

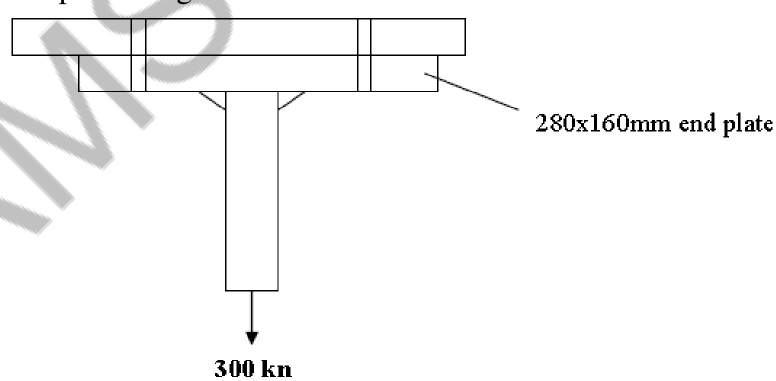
QUESTION BANK

Part A-(2Marks)

1. What are the different types of steel structural members?
2. State different loads and load combination for the design of structures
3. Sketch the General forms of tension and compression members.
4. State the different methods of design of steel structures..
5. Define Compression Member.
6. Define Struts
7. Define Radius of Gyration.
8. Define Slenderness ratio.
9. Define Lacings.
10. Define Batten plates.

Part B- (16 Marks)

1. Two plates of same width 65mm and different thickness18mm and16mmare to be connected by a lap joint to resist a tensile load of 75 kn usingM16 bolts of grade 4.6 and plates of grade 410. Design the lap joint.
2. A hanger joint is shown in fig below carry a factored load of 300kn using an end plate of size 280mm X160mm. Design the hanger joint using M24 HSFG bolts provided as shown. Take end plates of grade Fe410.



3. A double angle discontinuous strut 150mmx75mmx12mm long leg back to back is connected to either side by gusset plate of 12mm thick with 2 bolts. The length of the strut between the intersections is 3.3m. Determine the safe load carrying capacity of the section.

4. Calculate the safe compressive load of a bridge compression member of two channels ISMC300, 35.8 kg/m placed toe to toe. The effective length of member is 6m. the widths over the back of the channel is 320mm and the section is properly connected to bracings.
5. An ISMB 300 @ 577 N/m column carries a factored axial load of 900 kn . Design the base plate under it assuming the grade of steel used as Fe410 & the grade of concrete to be provided below the base plate as M25.
6. Design a gusseted base on a concrete pedestal for a column ISHB 400 @ 759N/m with two flange plates 400x20mm carrying a factored load of 4000 kn. The column is to be supported on concrete pedestal to be built with M20 concrete.
7. Design a single angle discontinuous strut to carry an axial compression of 100 kN. The length of strut between center to center of intersection is 2.0 m. Take f y =250 mpa.
8. Determine percentage change of strength of 2 ISA 100 X 75 X 8 mm. If the angles are attached to same side and opposite side of gusset plate.
9. Design an economical built up column to carry an axial load of 1200 KN using two channels back to back. The unsupported length of column is 5.4 m. both ends are held in position and only one end is restrained against rotation. Also design a suitable lacing system. Take fy =260 M Pa.
10. Design a gusseted base for a columnISHB450@872 N/m with one plate 300 x 10 mm carries an axial load of 2200 KN. The column is supported on concrete pedestal with bearing capacity of 4 Mpa.
11. A column section ISHB 300@0.63 KN/m with cover plate 350 x16 mmon either side is carrying an axial load of 3000 Kn. Design a gusseted base. The allowable bearing pressure of concrete is 4 Mpaand base plate is 185 Mpa.SBC of soil is 190 KN/m^2Assume fy=260 Mpa.
12. Design an economical built up column to carry an axial load of 1200 KN using two channels back to back. The unsupported length of column is 5.4 m. both ends are held in position and only one end is restrained against rotation. Also design a suitable lacing system. Take fy =260 M Pa.
13. Design a gusseted base for an ISHB 450 carrying an axial load of 1200 KN. the allowable bearing stress in concrete is 4 Mpa. The safe bearing capacity of soil is 160 $KN/m^{2.}$

DESIGN OF STEEL STRUCTURAL ELEMENTS

UNIT-5

FLEXURAL MEMBERS

CONTENTS

TECHNICAL TERMS

- **Beam**– A member subjected to predominant by bending.
- **Accompanying Load–** Live (Imposed) of lower magnitude, which may act together with a leading imposed load.
- **Action Effect or Load Effect–** The internal force, axial, shear, bending or twisting moment, due to external actions such as temperature loads.
- **Action–** The primary cause for stress or deformations in a structure such as dead, live, wind, seismic or temperature loads.
- **Actual Length–** The length between centre to centre of intersection points with supporting members or the cantilever length in the case of a free standing member.
- **Bearing Type Connection–** A connection effected using snug-tight bolts, where the load is transferred by bearing of bolts against plate inside the bolt hole.
- **Braced Member–** A member in which the relative transverse displacement at brace locations is effectively prevented.
- **Buckling load**– The load at which an element, a member or a structure as a whole, either collapses in service or buckles in a load test and develops excessive lateral deformation out of plane or instability.
- **Buckling Strength or Resistance–** Force or Moment, upto which a member can withstand without buckling.
- **Built-up Section–** A member fabricated by interconnecting more than one element to form a compound section acting as single member.
- **Characteristic Load (Action)–** The value of specified load (action), above which not more than a specified percentage (usually 5%) of corresponding stresses of samples tested are expected to occur.

- **Characteristic Yield/Ultimate Stress–** The minimum value of stress below which not more than a specified percentage (usually 5%) of corresponding stresses of samples tested are expected to occur.

- **Column–** A member in upright (vertical) position which supports a roof or floor system and predominantly subjected to compression.

- **Compact Section**– A cross-section, which can develop plastic moment, but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure

- **Deflection–** It is the displacement, transverse to the axis of the member.

- **Design Life–** Time Period for which a structure or a structural element is required to perform its function without damage.

- **Design Load/Factored Load–** A load value obtained by multiplying the characteristic load with a load factor, which is generally greater than one.

- **Ductility –** It is the property of the material or a structure indicating the extent to which it can deform beyond the limit of yield deformation before failure or fracture. The ratio of ultimate to yield deformation is also termed as ductility.

- **Durability–** It is the ability of a material to resist deterioration over long periods of time.

- **Earthquake Loads**– The inertia forces produced in a structure due to the ground movement due to earthquake

- **Edge Distance**–Distance from the centre of a fastener hole to the nearest edge of an element measured in the direction perpendicular to the direction of load transfer.

- **Effective Lateral Restraint**–Restraint, which produces sufficient resistance, to prevent deformation in the lateral direction.

- **Effective Length–** Actual length of a member between points of effective restraint or effective restraint and free end, multiplied by a factor to take account of the end conditions in buckling strength calculations.

- **Elastic Critical Moment–** The elastic moment which initiates lateral-torsional buckling of a laterally unsupported beam.

- **Elastic Design–**Design, which assumes elastic behaviour of materials throughout the service load range.

- **Elastic Limit–** It is the stress below which the material regains its original size and shape when the load is removed. In steel design, it is taken as the yield stress.

- **End Distance–**Distance from the centre of a fastener hole to the edge of an element parallel to the direction of load transfer.

- **Factor of Safety–** The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

- **Fatigue–** Damage caused by repeated fluctuations of stress, leading to progressive and consequent cracking of a structural element.

- **Flexural Stiffness–**Stiffness of a member against rotation as evaluated by the value of bending deformation moment required to cause a unit rotation while all other degrees of freedom of the joints of the member except the rotated one are assumed to be restrained.

- **Load–** An externally applied force or action. (see also Action)

- **Main Member–** A structural member, which is primarily responsible for carrying and distributing the applied load or action

INTRODUCTION

One of the frequently used structural members is a beam whose main function isto transfer load principally by means of flexural or bending action. In a structuralframework, it forms the main horizontal member spanning between adjacent columns oras a secondary member transmitting floor loading to the main beams. Normally onlybending effects are predominant in a beam except in special cases such as cranegirders, where effects of torsion in addition to bending have to be specificallyconsidered.

The type of responses of a beam subjected to simple uniaxial bending is shownin Table 4.1. The response in a particular case depends upon the proportions of thebeam, the form of the applied loading and the type of support provided. In addition tosatisfying various strength limits as given in the Table, the beam should also not deflecttoo much under the working loads i.e. it has to satisfy the serviceability limit state also.

Recently, IS: 800, the structural steel code has been revised and the limit statemethod of design has been adopted in tune with other international codes of practicesuch as BS, EURO, and AISC.

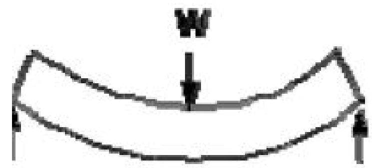
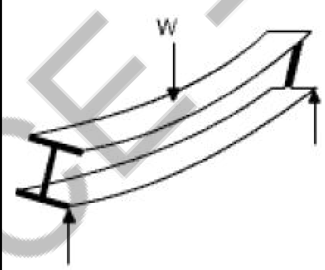
Limit state design of beams

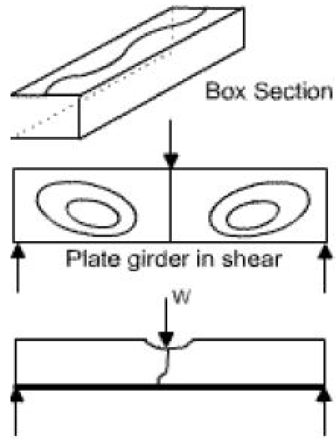
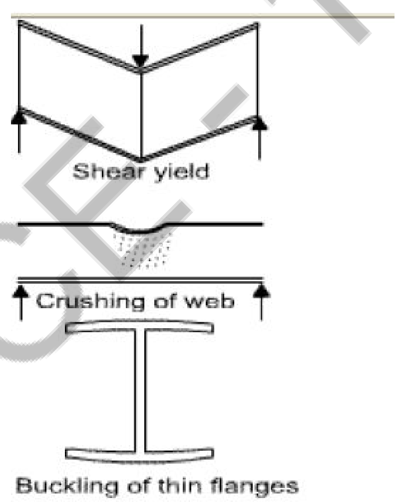
In the working stress or allowable stress method of design, the emphasis is on limiting a particular stress in a component to a fraction of the specified strength of the material of the component. The magnitude of the factor for a structural action depends upon the degree of safety required. Further, elastic behaviour of the material is assumed. The main objection to the permissible stress method is that the stress safety factor relating the permissible stress to the strength of the material is not usually the same as the ratio of the strength to the design load. Thus it does not give the degree of safety based on collapse load.

In the limit state method, both collapse condition and serviceability condition are considered. In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur on it throughout its life. Designs should ensure that the structure does not become unfit for the use for which it is required. The state at which the unfitness occurs is called a limit state. Special features of limit state design method are:

- It is possible to take into account a number of limit states depending upon the particular instance
- This method is more general in comparison to the working stress method. In this method, different safety factors can be applied to different limit states, which is more rational than applying one common factor (load factor) as in the plastic design method.
- This concept of design is appropriate for the design of structures since any new knowledge of the structural behaviour, loading and materials can be readily incorporated.
- The limit state design method is essentially based on the concept of probability. Its basic feature is to consider the possibility and probability of the collapse load. In this respect, it is necessary to consider the possibility of reduced strength and increased load

Main failure modes of hot-rolled beams

Category	Mode		Comments
1	Excessive bending triggering collapse		This is the basic failure mode provided (1) the beam is prevented from buckling laterally,(2) the component elements are at least compact, so that they do not buckle locally. Such "stocky" beams will collapse by plastic hinge formation.
2	Lateral torsional buckling of long beams which are not suitably braced in the lateral direction.(i.e. "un restrained" beams)		Failure occurs by a combination of lateral deflection and twist. The proportions of the beam, support conditions and the way the load is applied are all factors, which affect failure by lateral torsional buckling.

3	Failure by local buckling of a flange in compression or web due to shear or web under compression due to concentrated loads		Unlikely for hot rolled sections, which are generally stocky. Fabricated box sections may require flange stiffening to prevent premature collapse. Load bearing stiffeners are sometimes needed under point loads to resist web buckling.
4	Local failure by 1) shear yield of web (2) local crushing of web (3) buckling of thin flanges		Shear yield can only occur in very short spans and suitable web stiffeners will have to be designed. Local crushing is possible when concentrated loads act on unstiffened thin webs. Suitable stiffeners can be designed. This is a problem only when very wide flanges are employed. Welding of additional flange plates will reduce the plate b / t ratio and thus flange buckling failure can be avoided.

The object of design is to keep an acceptable level the probability of any limit state not being exceeded. This is achieved by taking account of the variation in strength and properties of materials to be used and the variations in the loads to be supported by the structure, by using the characteristic values of the strength of materials as well as the loads to be applied. The deviations from the characteristic values in the actual structures are allowed by using their design values. The characteristic values should be based on statistical evidence where necessary data are available; where such data are not available they should be based on an appraisal of experience. The design values are derived from the characteristic values through the use of partial safety factors, one for material strengths and the other for loads and load effects.

Behaviour of steel beams

Laterally stable steel beams can fail only by (a) Flexure (b) Shear or (c) Bearing,assuming the local buckling of slender components does not occur. These threeconditions are the criteria for limit state design of steel beams. Steel beams would alsobecome unserviceable due to excessive deflection and this is classified as a limit stateof serviceability.

The factored design moment, M at any section, in a beam due to external actions

shall satisfy

$M \leq Md$

Where Md = design bending strength of the section

Design strength in bending

The behaviour of members subjected to bending demonstrated in Fig 4.1

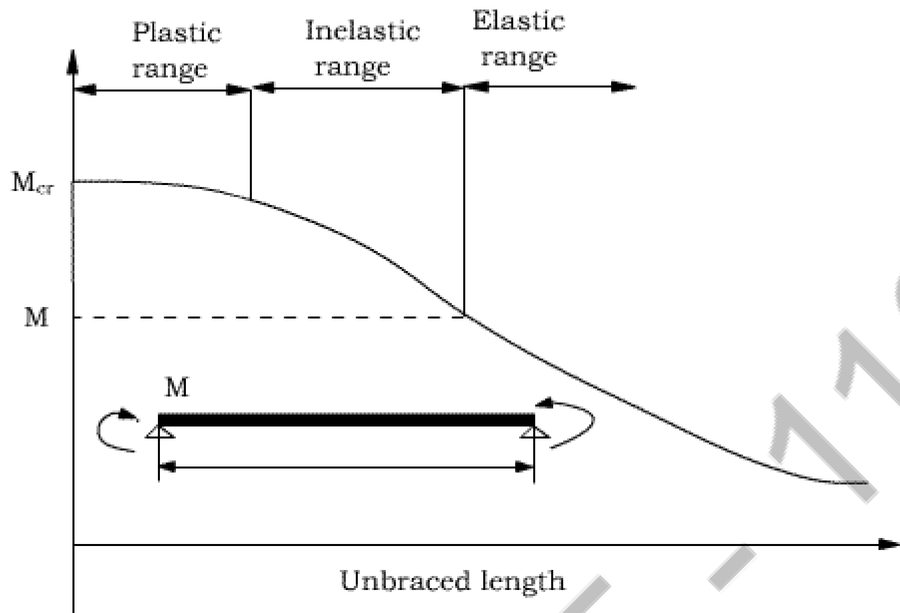


Fig 4.1 Beam buckling behaviour

This behaviour can be classified under two parts:

• When the beam is adequately supported against lateral buckling, the beamfailure occurs by yielding of the material at the point of maximum moment. The beam isthus capable of reaching its plastic moment capacity under the applied loads. Thus thedesign strength is governed by yield stress and the beam is classified as laterallysupported beam.

• Beams have much greater strength and stiffness while bending about themajor axis. Unless they are braced against lateral deflection and twisting, they arevulnerable to failure by lateral torsional buckling prior to the attainment of their full inplane plastic moment capacity. Such beams are classified as laterally supported beam.

Beams which fail by flexual yielding

Type1: Those which are laterally supported

The design bending strength of beams, adequately supported against buckling(laterally supported beams) is governed by yielding. The bending strength of a laterallybraced compact section is the plastic moment *Mp*. If the shape has a large shape factor(ratio of plastic moment to the moment corresponding to the onset of yielding at theextreme fiber), significant inelastic deformation may occur at service load, if the sectionis permitted to reach *Mp*at factored load. The limit of 1.5*My* at factored load will controlthe amount of inelastic deformation for sections with shape factors greater than 1.5.This provision is not intended to limit the plastic moment of a hybrid section with a webyield stress lower than the flange yield stress. Yielding in the web does not result insignificant inelastic deformations.

Type2: Those which are laterally shift

Lateral-torsional buckling cannot occur, if the moment of inertia about thebending axis is equal to or less than the moment of inertia out of plane. Thus, forshapes bent about the minor axis and shapes with *Iz*= *Iy*such as square or circularshapes, the limit state of lateral-torsional buckling is not applicable and yielding controlsprovided the section is compact.

4.3.1.1 Laterally supported beam

When the lateral support to the compression flange is adequate, the lateralbuckling of the beam is prevented and the section flexural strength of the beam can bedeveloped. The strength of I-sections depends upon the width to thickness ratio of thecompression flange. When the width to thickness ratio is sufficiently small, the beamcan be fully plastified and reach the plastic moment, such section are classified ascompact sections. However provided the section can also sustain the moment during theadditional plastic hinge rotation till the failure mechanism is formed. Such sections arereferred to as plastic sections. When the compression flange width to thickness ratio islarger, the compression flange may buckle locally before the complete plastification ofthe section occurs and the plastic moment is reached. Such sections are referred to asnon-compact sections. When the width to thickness ratio of the compression flange issufficiently large, local buckling of compression flange may occur even before extremefibre yields. Such sections are referred to as slender sections.

The flexural behaviour of such beams is presented in Fig. 4.2. The sectionclassified as slender cannot attain the first yield moment, because of a premature localbuckling of the web or flange. The next curve represents the beam classified as 'semicompact'in which, extreme fibre stress in the beam attains yield stress but the beammay fail by local buckling before further plastic redistribution of stress can take placetowards the neutral axis of the beam. The factored design moment is calculated as perSection **8.2** of the code.

The curve shown as 'compact beam' in which the entire section, bothcompression and tension portion of the beam, attains yield stress. Because of thisplastic redistribution of stress, the member o attains its plastic moment capacity (*Mp*) butfails by local buckling before developing plastic mechanism by sufficient plastic hingerotation. The moment capacity of such a section can be calculated by provisions givenin Section **8.2.1.2**. This provision is for the moment capacity with low shear load.

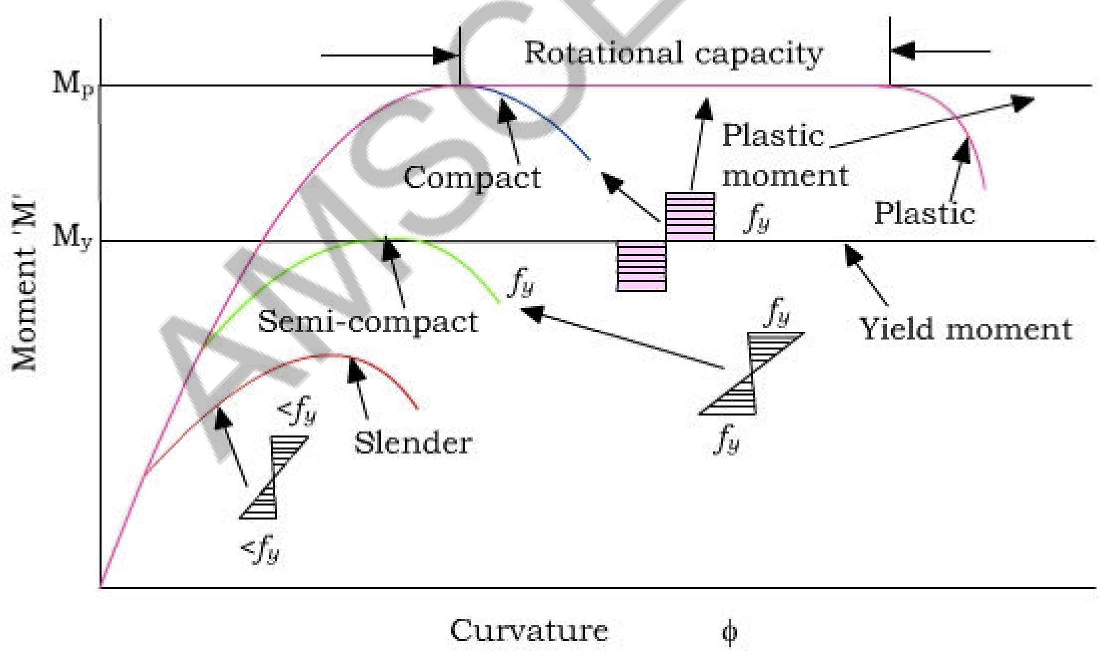


Fig 4.2 Flexural member performance using section classification

Low shear load is referred to the factored design shear force that does notexceed 0.6*Vd,* where *Vd*is the design shear strength of cross section as explained in**8.2.1.2** of the code.

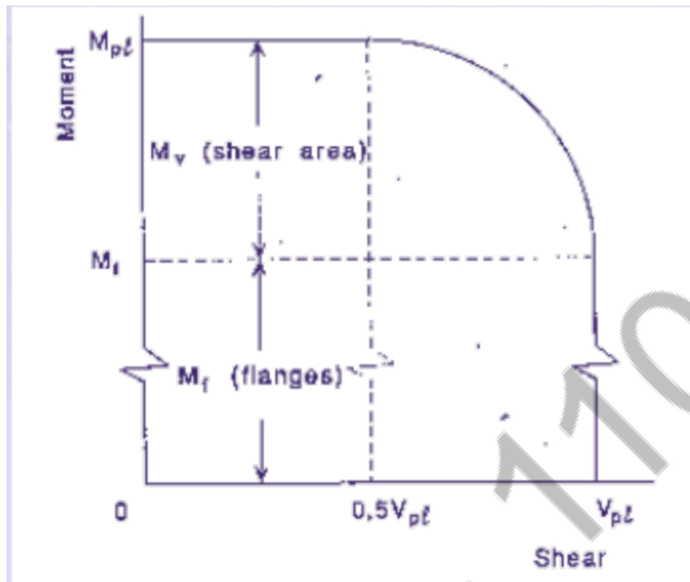


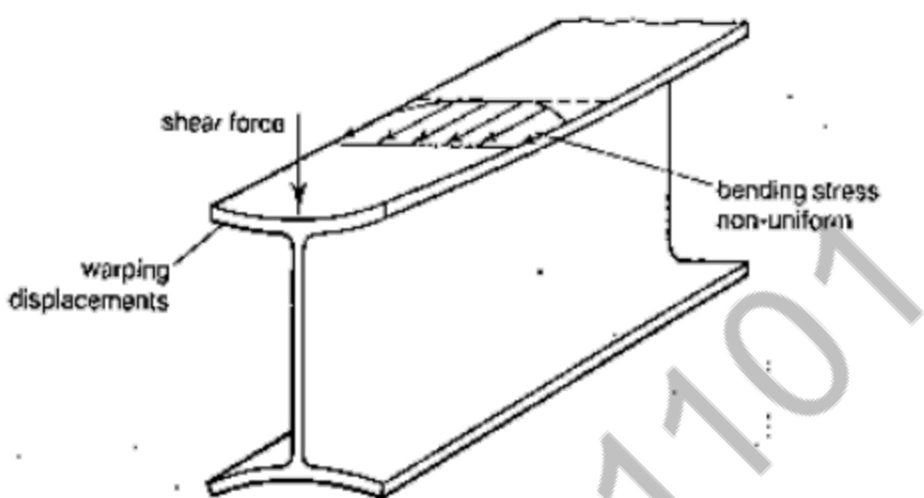
Fig.4.3 Interaction of high shear and bending moment

Holes in the tension zone

The fastener holes in the tension flange need not be allowed for provided that forthe tension flange the condition as given in **8.2.1.4** of the code is satisfied. Thepresence of holes in the tension flange of a beam due to connections may lead toreduction in the bending capacity of the beam.

Shear lag effects

The simple theory of bending is based on the assumption that plane sectionsremain plane after bending. But, the presence of shear strains causes the section towarp. Its effect in the flanges is to modify the bending stresses obtained by the simpletheory, producing higher stresses near the junction of a web and lower stresses atpoints away from it (Fig. 4.4). This effect is called 'shear lag'. This effect is minimal inrolled sections, which have narrow and thick flanges and more pronounced in plategirders or box sections having wide thin flanges when they are subjected to high shearforces, especially in the vicinity of concentrated loads. The provision with regard to shear lag effects is given in **8.2.1.5.**



Shear Lag effects

Laterally unsupported beams

Under increasing transverse loads, a beam should attain its full plastic momentcapacity. This type of behaviour in a laterally supported beam has been covered inSection **8.2.1**. Two important assumptions have been made therein to achieve the idealbeam behaviour.

They are:

• The compression flange of the beam is restrained from moving laterally; and

• Any form of local buckling is prevented

A beam experiencing bending about major axis and its compression flange notrestrained against buckling may not attain its material capacity. If the laterallyunrestrained length of the compression flange of the beam is relatively long then aphenomenon known as lateral buckling or lateral torsional buckling of the beam maytake place and the beam would fail well before it can attain its full moment capacity.This phenomenon has close similarity with the Euler buckling of columns triggeringcollapse before attaining its squash load (full compressive yield load).

Lateral-torsional buckling of beams

Lateral-torsional buckling is a limit-state of structural usefulness where thedeformation of a beam changes from predominantly in-plane deflection to a combinationof lateral deflection and twisting while the load capacity remains first constant, beforedropping off due to large deflections. The analytical aspects of determining the lateral torsionalbuckling strength are quite complex, and close form solutions exist only for thesimplest cases.

The various factors affecting the lateral-torsional buckling strength are:

• Distance between lateral supports to the compression flange.

• Restraints at the ends and at intermediate support locations (boundaryconditions).

• Type and position of the loads.

• Moment gradient along the length.

• Type of cross-section.

• Non-prismatic nature of the member.

• Material properties.

• Magnitude and distribution of residual stresses.

• Initial imperfections of geometry and loading.

They are discussed here briefly:

The distance between lateral braces has considerable influence on the lateraltorsional buckling of the beams.

The restraints such as warping restraint, twisting restraint, and lateral deflectionrestraint tend to increase the load carrying capacity.

If concentrated loads are present in between lateral restraints, they affect theload carrying capacity. If this concentrated load applications point is above shear centreof the cross-section, then it has a destabilizing effect. On the other hand, if it is belowshear centre, then it has stabilizing effect.

For a beam with a particular maximum moment-if the variation of this moment isnon-uniform along the length (Fig. 4.5) the load carrying capacity is more than the beamwith same maximum moment uniform along its length.

If the section is symmetric only about the weak axis (bending plane), its loadcarrying capacity is less than doubly symmetric sections. For doubly symmetricsections, the torque-component due to compressive stresses exactly balances that dueto the tensile stresses. However, in a mono-symmetric beam there is an imbalance andthe resistant torque causes a change in the effective torsional stiffeners, because theshear centre and centroid are not in one horizontal plane. This is known as "WagnerEffect".

If the beam is non-prismatic within the lateral supports and has reduced width offlange at lesser moment section the lateral buckling strength decreases.

The effect of residual stresses is to reduce the lateral buckling capacity. If thecompression flange is wider than tension flange lateral buckling strength increases, andif the tension flange is wider than compression flange, lateral buckling strengthdecreases. The residual stresses and hence its effect is more in welded beams ascompared to that of rolled beams.

The initial imperfections in geometry tend to reduce the load carrying capacity.

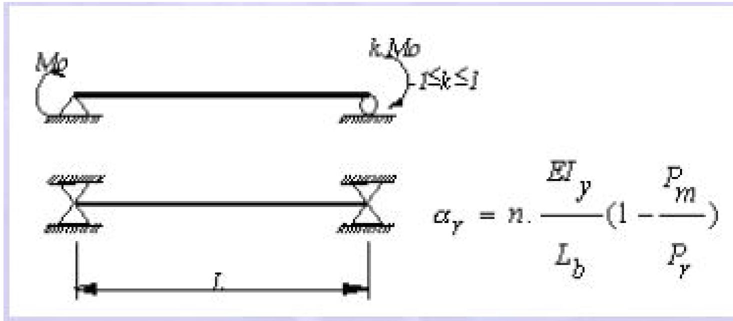


Fig 4.5 Beam subjected to Non-uniform moment

The design buckling (Bending) resistance moment of laterally unsupportedbeams are calculated as per Section **8.2.2** of the code.

If the non-dimensional slenderness is$\lambda LT \leq$ 0.4,no allowance for lateral-torsional bucklingis necessary. Appendix **F** of the code gives the method of calculating *Mcr,* the elasticlateral torsional buckling moment for difficult beam sections, considering loading and asupport condition as well as for non-prismatic members.

Effective length of compression flanges

The lateral restraints provided by the simply supported condition assumption inthe basic case, is the lowest and therefore the *Mcr*is also the lowest. It is possible, byother restraint conditions, to obtain higher values of *Mcr*, for the same structural section,which would result in better utilisation of the section and thus, saving in weight ofmaterial. As lateral buckling involves three kinds of deformation, namely, lateralbending, twisting and warping, it is feasible to think of various types of end conditions.But the supports should either completely prevent or offer no resistance to each type ofdeformation. Solutions for partial restraint conditions are complicated. The effect ofvarious types of support conditions is taken into account by way of a parameter calledeffective length.

For the beam with simply supported end conditions and no intermediate lateralrestraint the effective length is equal to the actual length between the supports, when agreater amount of lateral and torsional restraints is provided at support. When theeffective length is less than the actual length and alternatively the length becomes morewhen there is less restraint. The effective length factor would indirectly account for theincreased lateral and torsional rigidities by the restraints.

Shear

Let us take the case of an 'I' beam subjected to the maximum shear force (at thesupport of a simply supported beam). The external shear '*V*' varies along thelongitudinal axis 'x' of the beam with bending moment as $V = dM/dx$. While the beam isin the elastic stage, the internal shear stresses τ , which resist the external shear, V, canbe written as,

$\tau = VQ/lt$

where

V = shear force at the section

I = moment of inertia of the entire cross section about the neutral axis

Q = moment about neutral axis of the area that is beyond the fibre at which τ is

calculated and 't' is the thickness of the portion at which τ is calculated.

The above Equation is plotted in Fig. 4.6, which represents shear stresses in theelastic range. It is seen from the figure that the web carries a significant proportion ofshear force and the shear stress distribution over the web area is nearly uniform.Hence, for the purpose of design, we can assume without much error that the averageshear stress as

$\tau = V/twdw$

where

tw= thickness of the web

dw= depth of the web

The nominal shear yielding strength of webs is based on the Von Mises yieldcriterion, which states that for an un-reinforced web of a beam, whose width tothickness ratio is comparatively small (so that web-buckling failure is avoided), theshear strength may be taken as

$\tau = fy/\sqrt{3} = 0.58fy$

where

fy= yield stress.

The shear capacity of rolled beams *Vc*can be calculated as

$Vc \approx 0.6fy\ twdw$

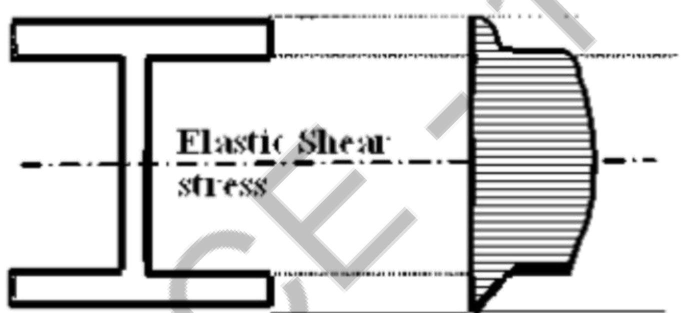


Fig 4.6 Elastic shear stresses

When the shear capacity of the beam is exceeded, the 'shear failure' occurs byexcessive shear yielding of the gross area of the webs as shown in Fig 4.7. Shearyielding is very rare in rolled steel beams.

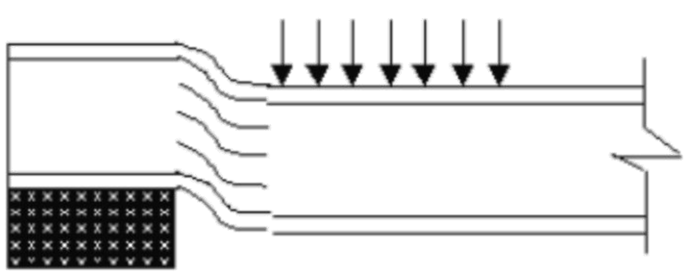


Fig 4.7 Shear yielding near support

The factored design shear force V in the beam should be less than the designshear strength of web. The shear area of different sections and different axes ofbending are given in Section **8.4.1**.**1**

Resistance to shear buckling

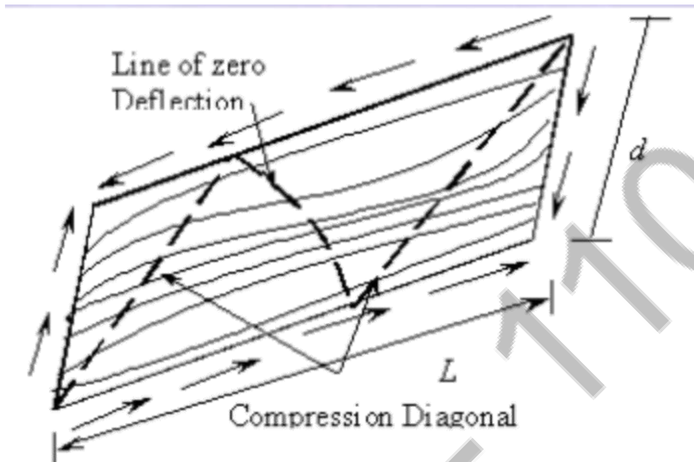
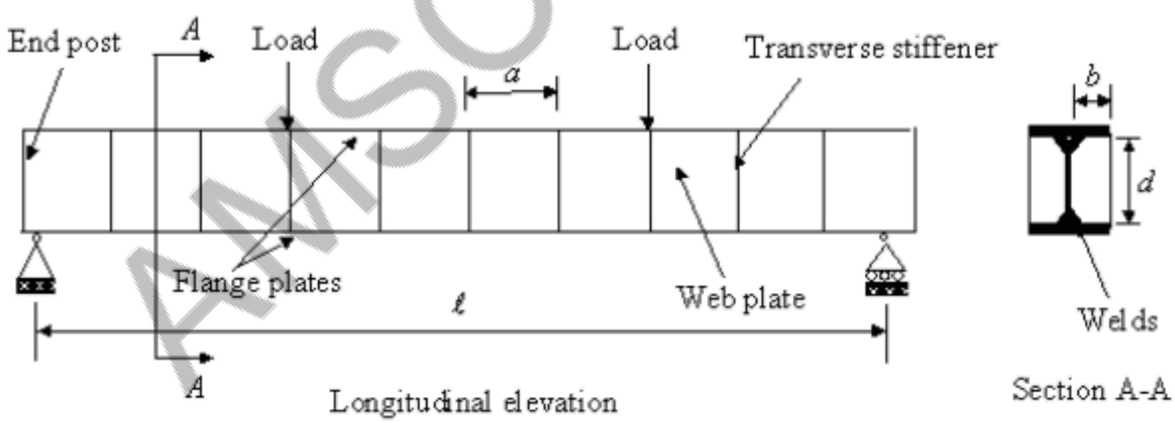


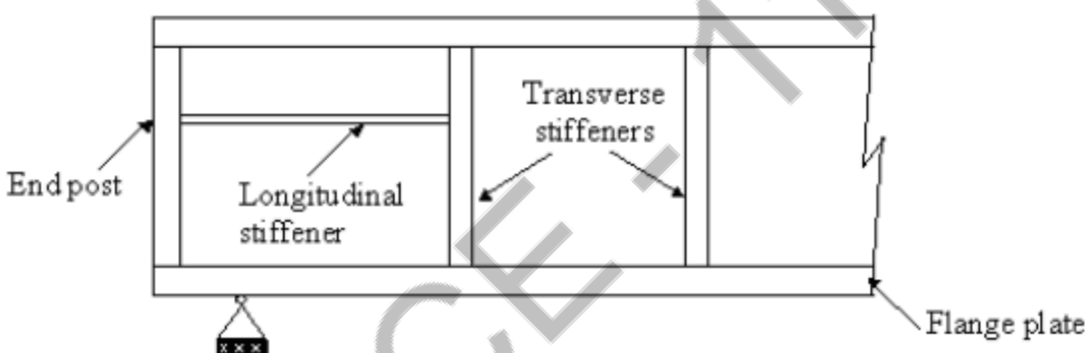
Fig 4.8 Buckling of a girder web in shear



A typical plate girder

The girder webs will normally be subjected to some combination of shear andbending stresses. The most severe condition in terms of web buckling is normally thepure shear case. It follows that it is those regions adjacent to supports or the vicinity ofpoint loads, which generally control the design. Shear buckling occurs largely as a resultof the compressive stresses acting diagonally within the web, as shown in Fig.4.8withthe number of waves tending to increasewith an increase in the panel aspect ratio c */ d.*

When d */tw*≤ 67ε where ε = (250 / *fy*)0.5 the web plate will not buckle because theshear stress τ is less than critical buckling stress '*τcr'*. The design in such cases issimilar to the rolled beams here. Consider plate girders having thin webs with *d/tw*> 67ε.In the design of these webs, shear buckling should be considered. In a general way, wemay have an un-stiffened web, a web stiffened by transverse stiffeners (Fig. 4.9) and aweb stiffened by both transverse and longitudinal stiffeners (Fig. 4.10)



End panel strengthened by longitudinal stiffener

Shear buckling design methods

The webs, designed either with or without stiffeners and governed by bucklingmay be evaluated by using two methods

1. Simple Post Critical Method

2. Tension Field Method

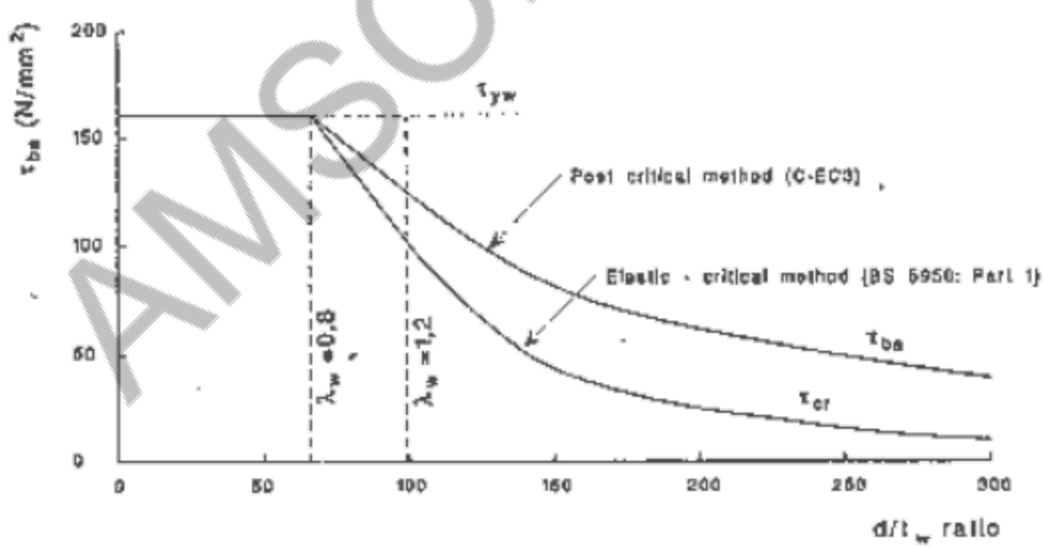
Simple post critical method

It is a simplified version of a method for calculating the post-buckled memberstress. The web possesses considerable post-buckling strength reserve and is shown inFig. 4.11.

When a web plate is subjected to shear, we can visualize the structuralbehaviour by considering the effect of complementary shear stresses generatingdiagonal tension and diagonal compression. Consider an element E in equilibriuminside a square web plate with shear stress *q*. The requirements of equilibrium result inthe generation of complementary shear stresses as shown in Fig.. This result in theelement being subjected to principal compression along the direction AC and tensionalong the direction BD. As the applied loading is incrementally enhanced, withcorresponding increases in *q,* very soon, the plate will buckle along the direction ofcompression diagonal AC.

The plate will lose its capacity to any further increase in compressive stress. Thecorresponding shear stress in the plate is the "critical shear stress" τ_{cr}. The value of τ_{cr}can be determined from classical stability theory, if the boundary conditions of the plateare known. As the true boundary conditions of the plate girder web are difficult toestablish due to restraints offered by flanges and stiffeners we may conservativelyassume them to be simply supported. The critical shear stress in such a case is givenby

$\tau_{cr} = k_r \pi^2 E/12(1= \mu^2)(d / t_w)^2$

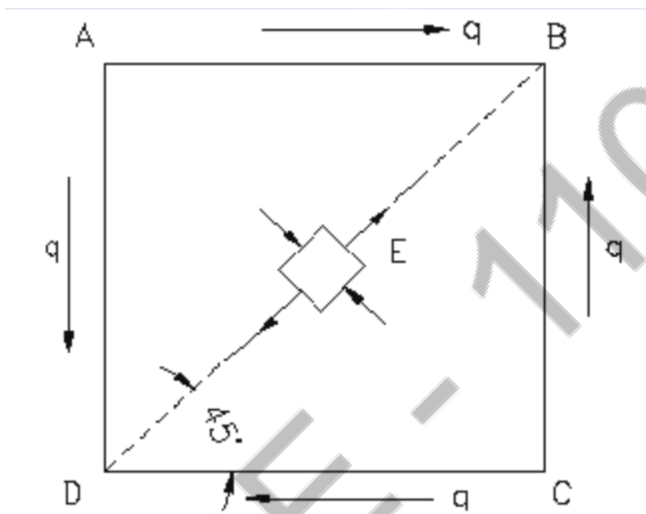


Post buckling reserve strength of web

kv= 5.35 When the transverse stiffeners is provided at the support,

kv=4.0 +5.35(c/d)2 for c/d < 1.0 i.e., for webs with closely spaced transverse stiffeners.

kv=5.35 +4(c/d)2 for c/d ≥1.0 for wide panels.



Unbuckled shear panel

When the value of (d/t) is sufficiently low (d/t < 85) τcr increases above the valueof yield shear stress, and the web will yield under shear before buckling.Based on this theory, the code gives the following values for $\tau cr$for webs, which are nottoo slender (Section 8.4.2.2a). The values depend on the slenderness parameter λwasdefined in the code.

Tension field methods

Design of plate girders with intermediate stiffeners, as indicated in Fig. 4.10, canbe done by limiting their shear capacity to shear buckling strength. However, thisapproach is uneconomical, as it does not account for the mobilisation of the additionalshear capacity as indicated earlier. The shear resistance is improved in the followingways:

i. Increasing in buckling resistance due to reduced c/dratio;

ii. The web develops tension field action and this resists considerably larger

stress than the elastic critical strength of web in shear

Figure 4.13 shows the diagonal tension fields anchored between top and bottomflanges and against transverse stiffeners on either side of the panel with the stiffenersacting as struts and the tension field acting as ties. The plate girder behaves similar toan N-truss Fig.4.14.

The nominal shear strength for webs with intermediate stiffeners can becalculated by this method according to the design provision given in code.

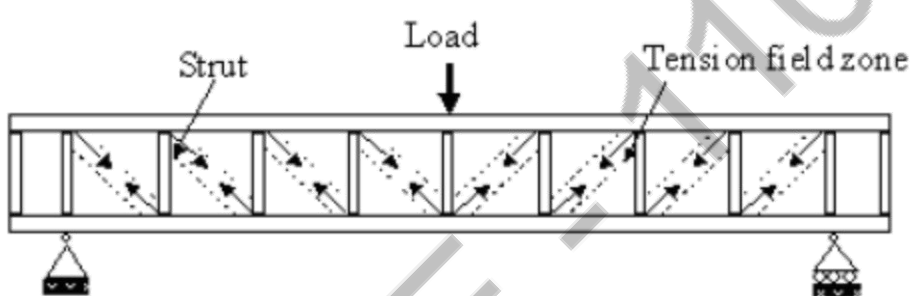
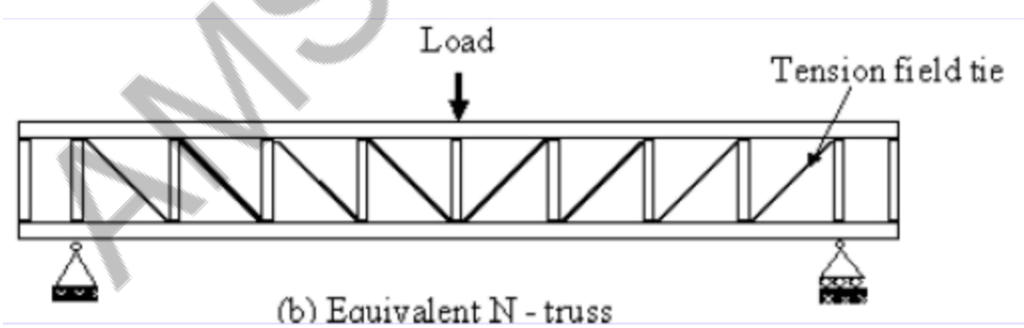


Fig 4.13 Tension field in individual sub-panel



Tension field action and the equivalent N-truss

Stiffened web panels

For tension field action to develop in the end panels, adequate anchorage should beprovided all around the end panel. The anchor force *Hq*required to anchor the tensionfield force is

H q =1.5 V dp(1- Vcr/V dp) 1/ 2

The end panel, when designed for tension field will impose additional loads onend post; hence, it will become stout (Fig 8.2 of the code). For a simple design, it maybe assumed that the capacity of the end panel is restricted to *Vcr*, so that no tensionfield develops in it (Fig 8.1 of the code). In this case, end panel acts as a beamspanning between the flanges to resist shear and moment caused by *Hq*and producedby tension field of penultimate panel.

This approach is conservative, as it does not utilise the post-buckling strength ofend panel especially where the shear is maximum. This will result in *c/d*value of theend panel spacing to be less than of other panels. The end stiffeners should bedesigned for compressive forces due to bearing and the moment *Mtf,* due to tension fieldin the penultimate panel in order to be economical the end panel also may be designedusing tension field action. In this case, the bearing stiffeners and end post are designedfor a combination due to bearing and a moment equal to 2/3 caused due to tension inthe flange *Mtf,* instead of one stout stiffener we can use a double stiffener as shown inFig. 8.3 of the code. Here, the end post is designed for horizontal shear and moment*Mtf*.

Design of beams and plate girders with solid webs.

The high bending moment and shear forces caused by carrying heavy loads overlong spans may exceed the capacity of rolled beam sections. Plate girders can be usedin such cases and their proportions can be designed to achieve high strength/weightratio. In a plate girder, it can be assumed that the flanges resist the bending momentand the web provides resistance to the shear force. For economic design, low flangesize and deep webs are provided. This results in webs for which shear failure mode is aconsequence of buckling rather than yielding**.**

Minimum web thickness − In general, we may have unstiffened web, a webstiffened by transverse stiffeners, (Fig 4.9) or web stiffened by both transverse andlongitudinal stiffeners (Fig 4.10).

By choosing a minimum web thickness *tw*, the self-weight is reduced. However,the webs are vulnerable to buckling and hence, stiffened if necessary. The webthickness based on serviceability requirement is recommended in Section **8.6.1.1** of thecode.

Compression flange buckling requirement

Generally, the thickness of flange plate is not varied along the span of plategirders. For non-composite plate girder the width of flange plates is chosen to be about0.3 times the depth of the section as a thumb rule. It is also necessary to choose thebreadth to thickness ratio of the flange such that the section classification is generallylimited to plastic or compact section only. This is to avoid local buckling before reachingthe yield stress. In order to avoid buckling of the compression flange into the web, theweb thickness shall be based on recommendation given in Section **8.6.1.2** of the code.

Flanges

For a plate girder subjected to external loading the minimum bending momentoccurs at one section usually, e.g. when the plate girder is simply supported at the endsand subjected to the uniformly distributed load, then, maximum bending moment occursat the centre. Since the values of bending moment decreases towards the end, theflange area designed to resist the maximum bending moment is not required to othersections. Therefore the flange plate may be curtailed at a distance from the centre ofspan greater than the distance where the plate is no longer required as the bendingmoment decreases towards the ends.

Usually, two flange angles at the top and two flange angles at the bottom areprovided. These angles extend from one end to the other end of the girder. For a goodproportioning, the flange angles must provide an area at least one-third of the totalflange area.

Generally, horizontal flange plates are provided to and connected to theoutstanding legs of the flange angles. The flange plates provide an additional width tothe flange and thus, reduced the tendency of the compression flange to buckle. Theseplates also contribute considerable moment of inertia for the section of the girder it giveseconomy as regards the material and cost. At least one flange plate should be run forthe entire length of the girder

Flange splices

A joint in the flange element provided to increase the length of flange plates isknown as flange splice. The flange plate should be avoided as far as possible.Generally, the flange plates can be obtained for full length of the plate girder. In spite ofthe availability of full length of flange plates, sometimes, it becomes necessary to makeflange splices. Flange joints should not be located at the points of main bendingmoment. The design provisions for flange splices are given in detail in Section **8.6.3.2** ofthe code.

Connection of flange to web

Rivets/bolts or weld connecting the flanges angles and the web will be subjectedto horizontal shear and sometimes vertical loads which may be applied directly to theflanges for the different cases such as depending upon the directly applied load to theeither web or flange and with or without consideration of resistance of the web.

Splices in the web

A joint in the web plate provided to increase its length is known as web splices.The plates are manufactured upto a limited length. When the maximum manufacturedlength of the plate is insufficient for full length of the plate girder web splice becomesessential, when the length of plate girder is too long to handle conveniently duringtransportation and erection. Generally, web splices are not used in buildings; they aremainly used in bridges.

Splices in the web of the plate girder are designed to resist the shear andmoment at the spliced section. The splice plates are provided on each side of the web.Groove-welded splice in plate girders develop the full strength of the smaller splicedsection. Other types of splices in cross section of plate girders shall develop thestrength required by the forces at the point of the splice.

Stiffener design

General

These are members provided to protect the web against buckling. The thin butdeep web plate is liable to vertical as well as diagonal buckling. The web may bestiffened with vertical as well as longitudinal stiffeners

Stiffeners may be classified as

a) Intermediate transverse web stiffeners

b) Load carrying stiffeners

c) Bearing stiffeners

d) Torsion stiffeners

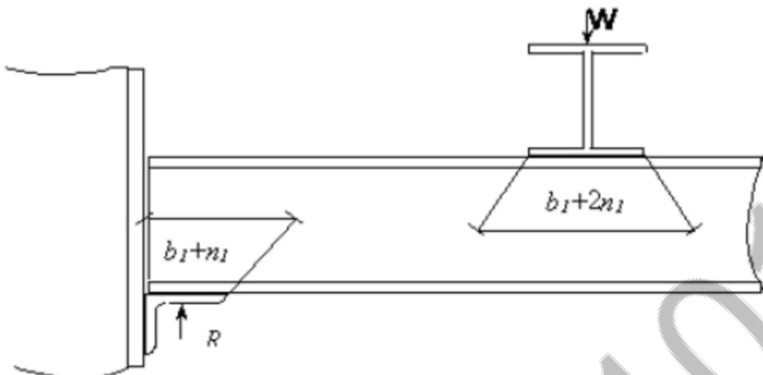
e) Diagonal stiffeners and

f) Tension stiffeners

The functions of the different types of stiffeners are explained in the Section **8.7.1.1** ofthe code.

Stiff bearing length

The application of heavy concentrated loads to a girder will produce a region ofvery high stresses in the part of the web directly under the load. One possible effect ofthis is to cause outwards buckling of this region as if it were a vertical strut with its endsrestrained by the beam's flanges. This situation also exists at the supports where the'load' is now the reaction and the problem is effectively turned upside down. It is usualto interpose a plate between the point load and the beam flange, whereas in the case ofreactions acting through a flange, this normally implies the presence of a seating cleat.



Dispersion of concentrated loads and reactions

In both cases, therefore, the load is actually spread out over a finite area by thetime it passes into the web as shown in Fig.4.15. It is controlled largely by thedimensions of the plate used to transfer the load, which is itself termed "the stiff lengthof bearing"

Outstand of web stiffeners and eccentricity of the stiffeners is explained as perSection 8.7.1.2 to 8.7.1.5of the code.

Design of Intermediate transverse web stiffeners

Intermediate transverse stiffeners are provided to prevent out of plane buckling ofweb at the location of the stiffeners due to the combined effect of bending moment andshear force.

Intermediate transverse stiffeners must be proportioned so as to satisfy twoconditions

1) They must be sufficiently stiff not to deform appreciably as the web tends tobuckle.

2) They must be sufficiently strong to withstand the shear transmitted by theweb.

Since it is quite common to use the same stiffeners for more than one task (forexample, the stiffeners provided to increase shear buckling capacity can also becarrying heavy point loads), the above conditions must also, in such cases, include theeffect of additional direct loading.

The condition (1) is covered by Section **8.7.2.4** of the code.

The strength requirement is checked by ensuring that the stiffeners acting as astrut is capable of withstanding *Fq.* The buckling resistance *Fq*of the stiffeners acting asstrut (with a cruciform section as described earlier) should not be less than thedifference between the shear actually present adjacent to the stiffeners *V,* and the shearcapacity of the (unstiffened) web *Vcr*together with any coexisting reaction or moment.Since the portion of the web immediately adjacent to the stiffeners tends to act with it,this "strut" is assumed to consist also of a length of web of 20 *t* on either side of thestiffeners centre line giving an effective section in the shape of a cruciform. Full detailsof this strength check are given in Section **8.7.2.5** of the code. If tension field action isbeing utilized, then the stiffeners bounding the end panel must also be capable ofaccepting the additional forces associated with anchoring the tension field.

Load carrying stiffeners

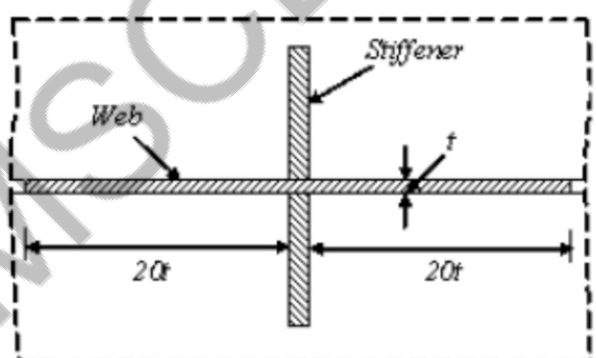


Fig.4.16 Cruciform section of the load carrying stiffener

Whenever there is a risk of the buckling resistance of the web being exceeded,especially owing to concentrated loads, load-carrying stiffeners are provided. Thedesign of load-carrying stiffeners is essentially the same as the design of verticalstiffeners. The load is again assumed to be resisted by strut comprising of the actualstiffeners plus a length of web of 20*t* on either side, giving an effective cruciform section.Providing the loaded flange is laterally restrained, the effective length of the 'strut' maybe taken as 0.7*L*. Although no separate stiffener check is necessary, a load-bearingstiffener must be of sufficient size that if the full load were to be applied to them actingindependently, i.e., on a cross-section consisting of just the stiffeners, as per Fig 4.16.Then the stress induced should not exceed the design strength by more than 25%. Thebearing stress in the stiffeners is checked using the area of that portion of the stiffenersin contact with the flange through which compressive force is transmitted.

Bearing stiffeners

Bearing stiffeners are required whenever concentrated loads, which could causevertical buckling of web of the girder, are applied to either flange. Such situations occuron the bottom flange at the reactions and on the top flange at the point of concentratedloads. Figure 4.17 shows bearing stiffeners consisting of plates welded to the web. Theymust fit tightly against the loaded flange. There must be sufficient area of contactbetween the stiffeners and the flange to deliver; the load without exceeding thepermissible bearing on either the flange material or the stiffeners must be adequateagainst buckling and the connection to the web must be sufficient to transmit the load asper Section **8.7.4** of the code.

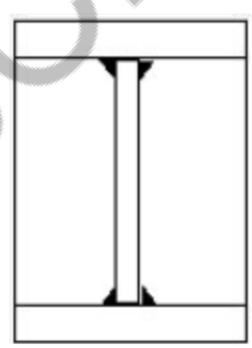


Fig. 4.17 Bearing stiffeners

The bearing stress on the contact area between stiffeners and flanges isanalogous to the compressive stress at the junction of web and flange of rolled beamssubjected to concentrated load.

Since buckling of bearing stiffeners is analogous to buckling of web at point ofconcentrated load, the required moment of inertia is not easy to evaluate. The buckledstiffeners may take any forms depending on the manner in which the flanges arerestrained. In most cases, the compression flange of the girder will be supportedlaterally at points of concentrated load by bracing or by beams framing into it, sobuckling will approximate the form of an end-fixed column. Even if the flanges are freeto rotate, the stiffeners need not be considered as end-hinged columns, because theload concentrated on one end of the stiffeners is resisted by forces distributed along itsconnection to the web instead of by a force concentrated at the opposite end as incolumns.

The connection to the web is merely a matter of providing sufficient welding totransmit the calculated load on the stiffeners as per the Section **8.7.6** of the code.

Design of different types of stiffeners is given in the code from Sections 8.7.4 to8.7.9.

Connection of web of load carrying and bearing

The web connection of load carrying stiffeners to resist the external load andreactions through flange shall be designed as per the design criteria given in the code.

Horizontal stiffeners

Horizontal stiffeners are generally not provided individually. They are used inaddition to vertical stiffeners, which are provided close to the support to increase thebearing resistance and to improve the shear capacity.

The location and placing of horizontal stiffeners on the web are based on thelocation of neutral axis of the girder. Thickness of the web *tw*and second moment ofarea of the stiffeners *Is*are as per the conditions and design provisions given in Section**8.7.13** of the code.

Bending in a non-principal plane

When deflections are constrained to a non-principal plane by the presence oflateral restraints, the principal axes bending moments are calculated due to the restraintforces as well as the applied forcs by any rational method. The combined effect isverified using the provisions in Section 9. Similarly, when the deflections areunconstrained due to loads acting in a non-principal plane, the principal axes bendingmoments are arrived by any rational method. The combined effects have to satisfy therequirements of Section 9.

BEAM COLUMNS

Introduction

The Indian steel code is now in the process of revision as specification-baseddesign gives way to performance-based design. An expert committee mainly comprisingeminent academics from IIT Madras, Anna University Chennai, SERC Madras andINSDAG Kolkata was constituted to revise IS: 800 in LSM version. The Limit StateMethod (referred to as LSM below) is recognized, as one of the most rational methodstoward realization of performance-based design, but to date there are no steel-intensivebuildings in India that have been designed using LSM. We considered that, becausebuilding collapse is caused by excessive deformation, the ultimate state should beevaluated from the deformation criteria. The proposed design procedure evaluates theultimate limit state on the basis of the deformation capacity of structural members.

The magnification factors, used to confirm suitable flexural mechanisms, severelyaffect the overall probability of failure, and should be determined so that the overallprobability of failure does not exceed specific allowable limits.

In practice, the structural members are generally subjected to various types ofcombination of stress resultants. Depending upon external actions over the members instructural framing system, the combined forces may be broadly categorized as i)Combined Shear and Bending, ii) Combined Axial Tension and Bending and iii)Combined Axial Compression and Bending.

Normally, the design of an individual member in a frame is done, by separating itfrom the frame and dealing with it as an isolated substructure. The end conditions of themember should then comply with its deformation conditions, in the spatial frame, in aconservative way, e.g. by assuming a nominally pinned end condition, and the internalaction effects, at the ends of the members, should be considered by applying equivalentexternal end moments and end forces. Before proceeding for any analysis, classificationof these members shall have to be satisfied in accordance with clause no. 3.7 and allrelated sub-clauses under section 3 of IS: 800 – LSM version.

For all practical purposes, we can equate the third case with the case of Beamcolumns.Beam-columns are defined as members subject to combined bending andcompression. In principle, all members in moment resistant framed structures (wherejoints are considered as rigid) are actually beam-columns, with the particular cases ofbeams (F = 0) and columns (M = 0) simply being the two extremes. Depending upon theexact way in which the applied loading is transferred into the member, the form ofsupport provided and the member's cross-sectional shape, different forms of responsewill be possible.

The simplest of these involves bending applied about one principal axis only, withthe member responding by bending solely in the plane of the applied moment.

Recently, IS: 800, the Indian Standard Code of Practice for General Constructionin Steel is in the process of revision and an entirely new concept of limit state method ofdesign has been adopted in line with other international codes of practice such as BS,EURO, and AISC. Additional Sections and features have been included to make thecode a state-of-the-art one and for efficient & effective use of structural steel. Attempthas been made in the revised code to throw some light into the provisions for memberssubjected to forces, which are combined in nature.

Concept of limit state design of beam columns

Steel structures are important in a variety of land-based applications, includingindustrial (such as factory sheds, box girder cranes, process plants, power andchemical plants etc.), infrastructural (Lattice girder bridges, box girder bridges, flyovers,institutional buildings, shopping mall etc.) and residential sector. The basic strengthmembers in steel structures include support members (such as rolled steel sections,hollow circular tubes, square and rectangular hollow sections, built-up sections, plategirders etc.), plates, stiffened panels/grillages and box girders. During their lifetime, thestructures constructed using these members are subjected to various types of loadingwhich is for the most part operational, but may in some cases be extreme or evenaccidental.

Steel-plated structures are likely to be subjected to various types of loads anddeformations arising from service requirements that may range from the routine to theextreme or accidental. The mission of the structural designer is to design a structurethat can withstand such demands throughout its expected lifetime.

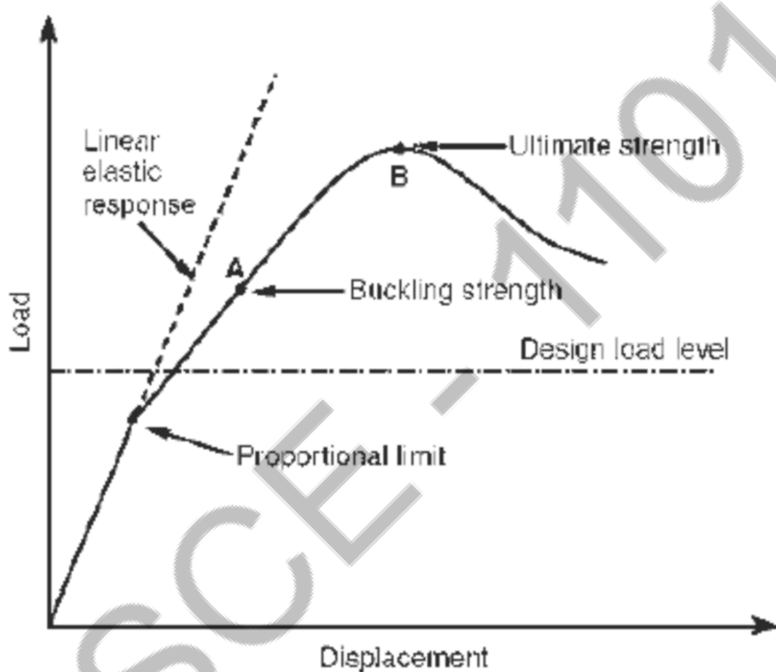
The structural design criteria used for the *Serviceability Limit State Design*(hereafter termed as **SLS**) design of steel-plated structures are normally based on thelimits of deflections or vibration for normal use. In reality, excessive deformation of astructure may also be indicative of excessive vibration or noise, and so, certaininterrelationships may exist among the design criteria being defined and usedseparately for convenience.

The SLS criteria are normally defined by the operator of a structure, or byestablished practice, the primary aim being efficient and economical in-serviceperformance without excessive routine maintenance or down-time. The acceptablelimits necessarily depend on the type, mission and arrangement of structures. Further,in defining such limits, other disciplines such as machinery designers must also beconsulted.

The structural design criteria to prevent the *Ultimate Limit State Design* (hereaftertermed as ULS) are based on plastic collapse or ultimate strength. The simplified ULSdesign of many types of structures has in the past tended to rely on estimates of thebuckling strength of components, usually from their elastic buckling strength adjusted bya simple plasticity correction. This is represented by point A in Figure 4.18 In such adesign scheme based on strength at point A, the structural designer does not usedetailed information on the post-buckling behavior of component members and theirinteractions. The true ultimate strength represented by point B in Figure 4.18 may behigher although one can never be sure of this since the actual ultimate strength is notbeing directly evaluated.

In any event, as long as the strength level associated with point B remainsunknown (as it is with traditional allowable stress design or linear elastic designmethods), it is difficult to determine the real safety margin. Hence, more recently, thedesign of structures such as offshore platforms and land-based structures such as steelbridges has tended to be based on the ultimate strength.

The safety margin of structures can be evaluated by a comparison of ultimatestrength with the extreme applied loads (load effects) as depicted in Figure 4.18. Toobtain a safe and economic structure, the ultimate load-carrying capacity as well as thedesign load must be assessed accurately. The structural designer may even desire toestimate the ultimate strength not only for the intact structure, but also for structureswith existing or premised damage, in order to assess their damage tolerance andsurvivability.



Structural design considerations based on the ultimate limit state

In the structural design process, “analysis” usually means the determination ofthe stress resultants, which the individual structural members must be capable to resist.“Design” can mean the development of the structural layout, or arrangement ofmembers, but it usually means the selection of sizes of members to resist the imposedforces and bending moments. Three methods of analysis are available, i.e. elasticanalysis, plastic analysis and advanced analysis. Limit state design is a design methodin which the performance of a structure is checked against various limiting conditions atappropriate load levels. The limiting conditions to be checked in structural steel designare ultimate limit state and serviceability limit state.Limit state theory includes principlesfrom the elastic and plastic theories and incorporates other relevant factors to give asrealistic a basis for design as possible.

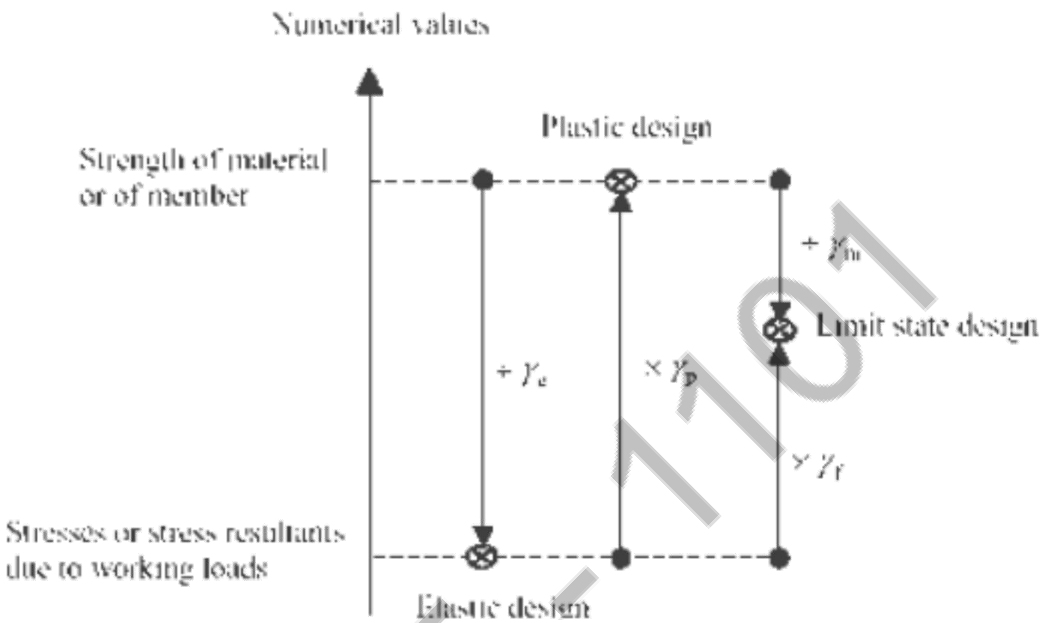


Fig. Level for different design methods at which calculationsare conducted (Commentary on BS5950 1 2000)

Ultimate Limit State Design of Steel Structures reviews and describes bothfundamentals and practical design procedures in this field. Designs should ensure thatthe structure does not become unfit / unserviceable for the use for which it is intendedto. The state at which the unfitness occurs is called a limit state.

Figure 4.19 shows how limit-state design employs separate factors γf, whichreflects the combination of variability of loading γl, material strength γm and structuralperformance γp. In the elastic design approach, the design stress is achieved by scalingdown the strength of material or member using a factor of safety γe as indicated inFigure 4.19, while the plastic design compares actual structural member stresses withthe effects of factored-up loading by using a load factor of γp.

Special features of limit state design method are:

• Serviceability and the ultimate limit state design of steel structural systems andtheir components.

• Due importance has been provided to all probable and possible design conditionsthat could cause failure or make the structure unfit for its intended use.

• The basis for design is entirely dependent on actual behaviour of materials instructures and the performance of real structures, established by tests and long-termobservations

• The main intention is to adopt probability theory and related statistical methods inthe design.

• It is possible to take into account a number of limit states depending upon theparticular instance

• This method is more general in comparison to the working stress method. In thismethod, different safety factors can be applied to different limit states, which is morerational and practical than applying one common factor (load factor) as in the plasticdesign method.

• This concept of design is appropriate for the design of structures since anydevelopment in the knowledge base for the structural behaviour, loading and materialscan be readily implemented.

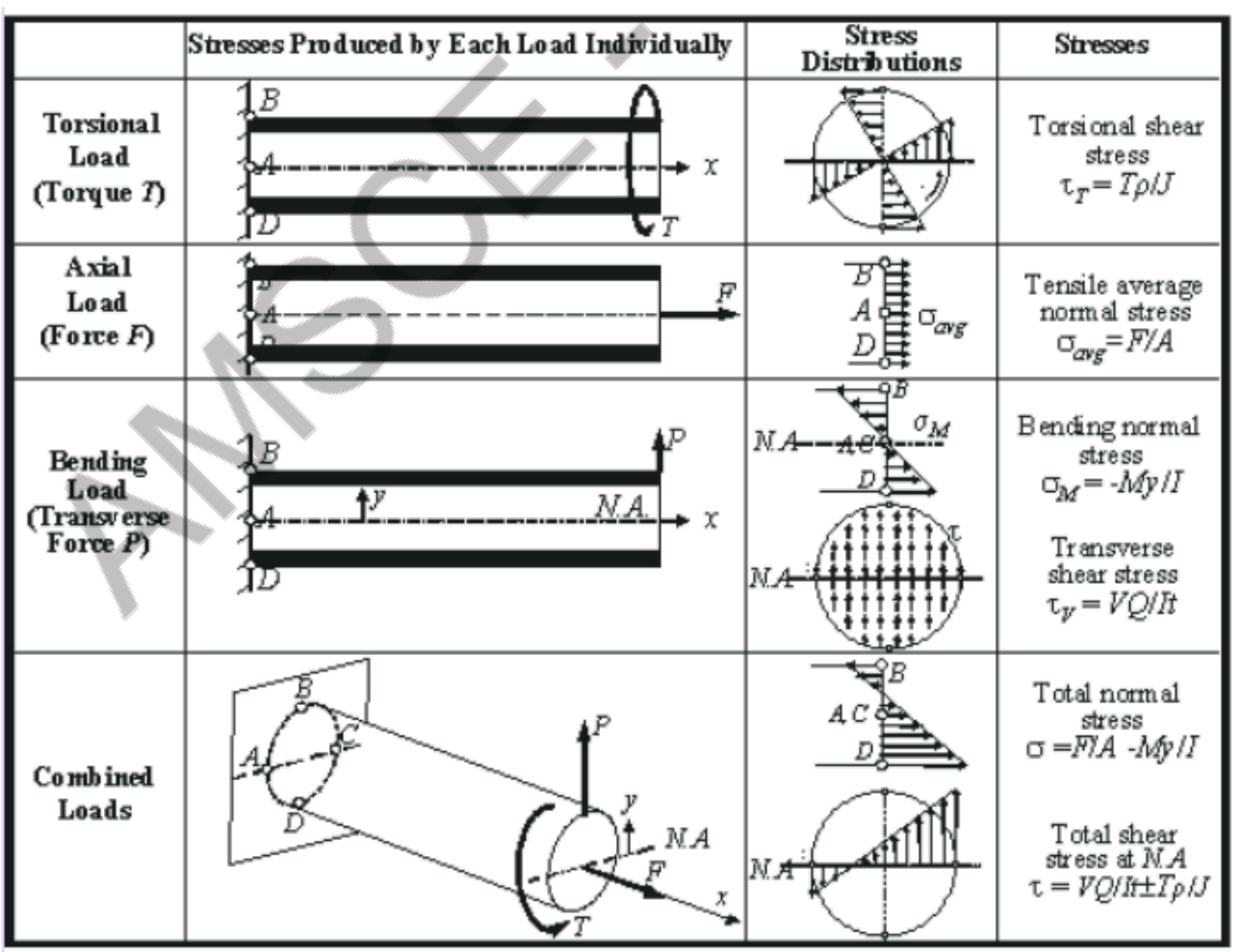
Design of members subjected to combined forces

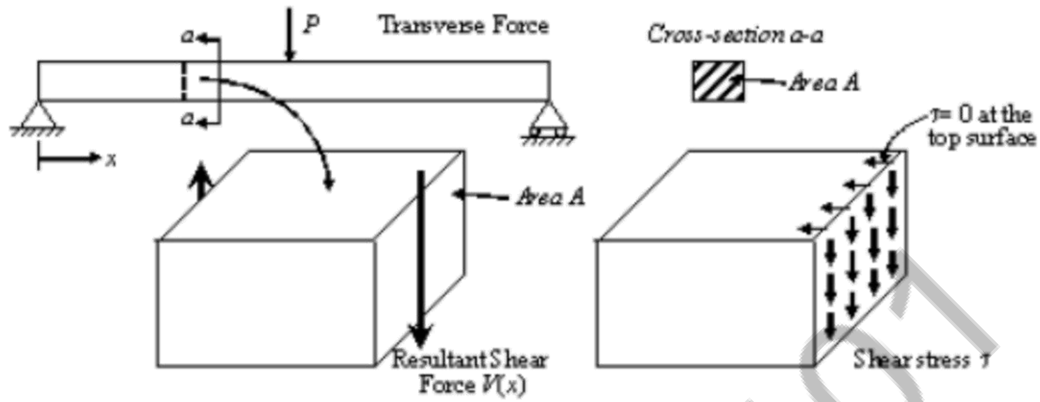
General

In the previous chapters of Draft IS: 800 – LSM version, we have stipulated thecodal provisions for determining the stress distribution in a member subjected todifferent types of stress resultants such as axial tensile force (Section 6), axialcompressive force (Section 7) and bending moment along with transverse shear force(Section 8). Most often, the cross section of a member is subjected to several of theseloadings *simultaneously*. As we shall see presently, we may combine the knowledgethat we have acquired in the previous sections. As long as the relationship betweenstress and the loads is *linear* and the geometry of the member would *not undergosignificant change* when the loads are applied, the principle of superposition can beapplied. Here, as shown in Table 4.2, one typical case of combination due to tensileforce *F*, torque *T* and transverse load *P* has been diagrammatically discussed.

In addition to the pure bending case, beams are often subjected to transverseloads which generate both bending moments M(x) and shear forces V(x) along thebeam. The bending moments cause bending normal stresses s to arise through thedepth of the beam, and the shear forces cause transverse shear-stress distributionthrough the beam cross section as shown in 4.20.

Superposition of individual loads (a case study for solid circular shaft)





Beam with transverse shear force showing the transverse shear stressdeveloped by it

General procedure for combined loading

• Identify the relevant equations for the problem and use the equations as a checklist for the quantities that must be calculated.

• Calculate the relevant geometric properties (A, Iyy ,Izz , J) of the cross-sectioncontaining the points where stresses have to be found.

• At points where shear stress due to bending is to be found, draw a lineperpendicular to the center-line through the point and calculate the first moments of thearea (Qy , Qz) between free surface and the drawn line. Record the s-direction from thefree surface towards the point where stress is being calculated.

• Make an imaginary cut through the cross-section and draw the free body diagram.On the free body diagram draw the internal forces and moments as per our signconventions if subscripts are to be used in determining the direction of stresscomponents. Using equilibrium equations to calculate the internal forces and moments.

• Using the equations identified, calculate the individual stress components due toeach loading. Draw the torsional shear stress $\tau x\theta$and bending shear stress τxs on astress cube using subscripts or by inspection. By examining the shear stresses in x, y, zcoordinate system obtainτxy and τxz with proper sign.

• Superpose the stress components to obtain the total stress components at apoint.

• Show the calculated stresses on a stress cube.

• Interpret the stresses shown on the stress cube in the x, y, z coordinate systembefore processing these stresses for the purpose of stress or strain transformation.

Design of member subjected to combined shear and bending:

In general, it has been observed that for structures, which are subjected tocombined shear and bending the occurrence of high shear force is seldom. Here, highshear force has been designated as that shear force, which is more than 50 percent ofthe shear strength of the section. For structures where the factored value of appliedshear force is less than or equal to 50 percent of the shear strength of the section noreduction in moment capacity of the section is required (refer clause 8.4 of section 8 ofdraft IS: 800 – LSM version) i.e. the moment capacity may be taken as Md (refer clause8.2 of section 8 and clause 9.2.1 of section 9 of draft IS: 800 – LSM version). If thefactored value of actual shear force is more than 50 percent of the shear strength of thesection, the section shall be checked and moment capacity, Mdx, shall be reduceddepending upon classification of section (refer clause 9.2.2 of section 9 of draft IS: 800– LSM version). This is done to take care of the increased resultant vector stressgenerated due to vector addition of stress due to high shear and bending moment atthat particular section. The corresponding bending moment capacity is reduced byincorporating a factor ‘β’ ( )$2\beta = 2V/Ve-1$, which in turn depends upon the ratio of actualvalue of high shear and shear strength of that particular section. In no case, themaximum value of moment capacity shall exceed$1.2Zefy / \gamma m0$, where, Ze is elasticsection modulus of the whole section $\gamma m0$, is the partial safety factor against yield stressand buckling and fy is the characteristic yield stress (250 N/mm2). For semi-compactsections, this reduction in moment capacity is not required to be exercised as thesection will be predominantly governed by the elastic moment capacity i.e. Zefy / γm0 ,where the terms Ze , γm0 and fy are as defined previously.

Design of member subjected to combined bending and axialforce:

General

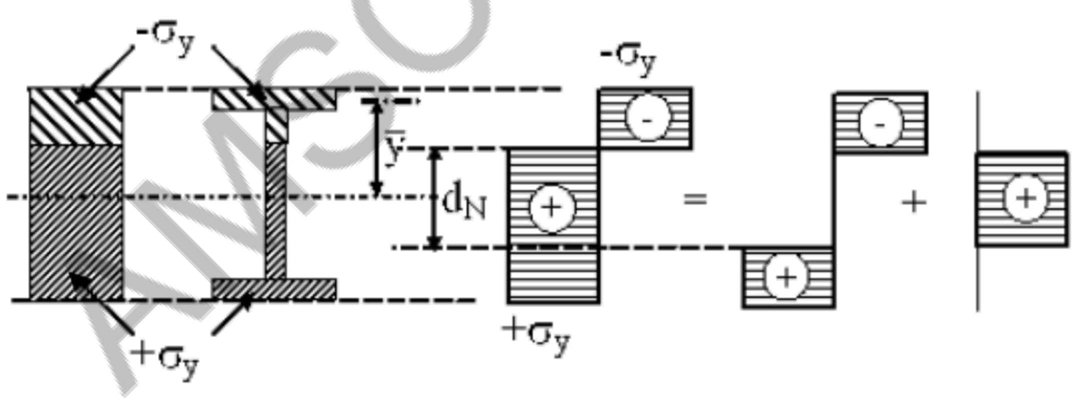
As with combined bending moment and axial force in the elastic range, the planeof zero strain moves from the centroid so that the ultimate stress distribution appears asfollows:

• To compute the modified plastic moment, Mp', the stress blocks are divided intothree components.

• The outer pair is equal and opposite stress blocks which provide the moment.

• The inner block acts in one direction, the area being balanced around the originalneutral axis, defined by *dN*. In the cases illustrated the inner block has a constant width,*b* or *tw*, so that

$$M'_p = 2A_p \bar{y}\,\sigma_y;\ \ N = bd_N\,\sigma_y\,(\text{rectangle});\ N = t_w\,d_N\sigma_y\,(\text{I-section})$$



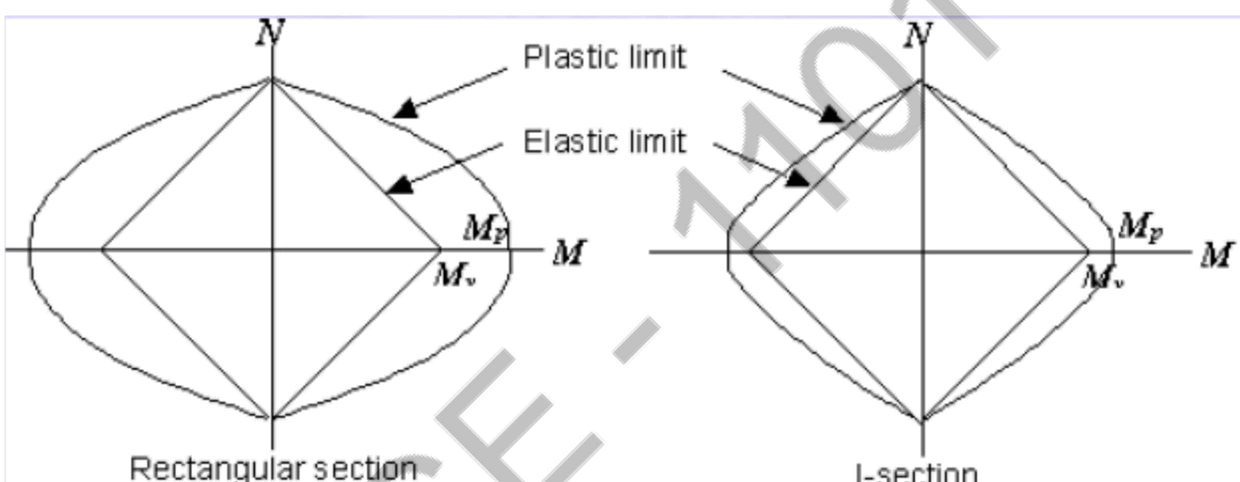
Stress block diagram for combined bending and axial force with reducedmoment carrying capacity for I-section

We can draw interaction diagrams relating N and M for initial yield and ultimatecapacity as follows:

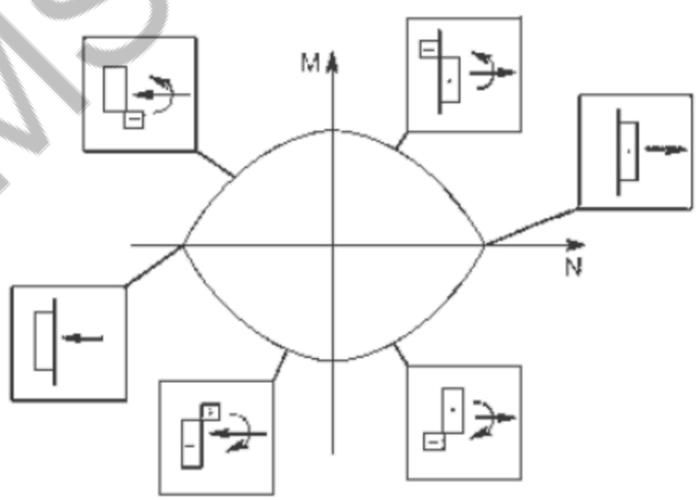
When the web and the flange of an I-beam have different yield strengths it ispossible to calculate the full plastic moment or the combined axial force and bendingmoment capacity taking into account the different yield strengths.

$$M_p = 2\sum_i A_i \,\overline{y}_i\, \sigma_{yi}$$

for all i segments with different yield stresses.

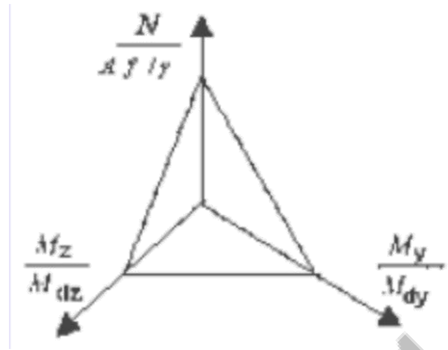


Interaction diagram of bending moment and axial tension or compression



Plastic limit envelope with stress distributions for combined bending (M)and axial force (N)

Members subjected to combined bending & axial forces



Interaction surface of the ultimate strength under combined biaxialbending moment and axial tensile force

Any member subjected to bending moment and normal tension force should bechecked for lateral torsional buckling and capacity to withstand the combined effects ofaxial load and moment at the points of greatest bending and axial loads. Figure 4.21illustrates the type of three-dimensional interaction surface that controls the ultimatestrength of steel members under combined biaxial bending and axial force. Each axisrepresents a single load component of normal force N, bending about the y and z axesof the section (*My* or *Mz*) and each plane corresponds to the interaction of twocomponents.

Local capacity check (Section 9 Draft IS: 800 – LSM version)

For *Plastic and Compact sections*, the design of members subjected to combinedaxial load and bending moment shall satisfy the following interaction relationship:

$$\left(\frac{M_y}{M_{ndy}}\right)^{u1} + \left(\frac{M_z}{M_{ndz}}\right)^{u2} \leq 1.0$$

Where

My, Mz = factored applied moments about the minor and major axis of the crosssection, respectively

Mndy, Mndz = design reduced flexural strength under combined axial force and therespective uniaxial moment acting alone, **(Cl. 9.3.1.2 of Draft IS: 800 – LSM version)**

N = factored applied axial force (Tension *T*, or Compression *F*)

Nd = design strength in tension (*Td*) as obtained from **(section 6 Draft IS: 800– LSM version)** or in compression and Nd= Agfy / γm0

Ag = gross area of the cross section

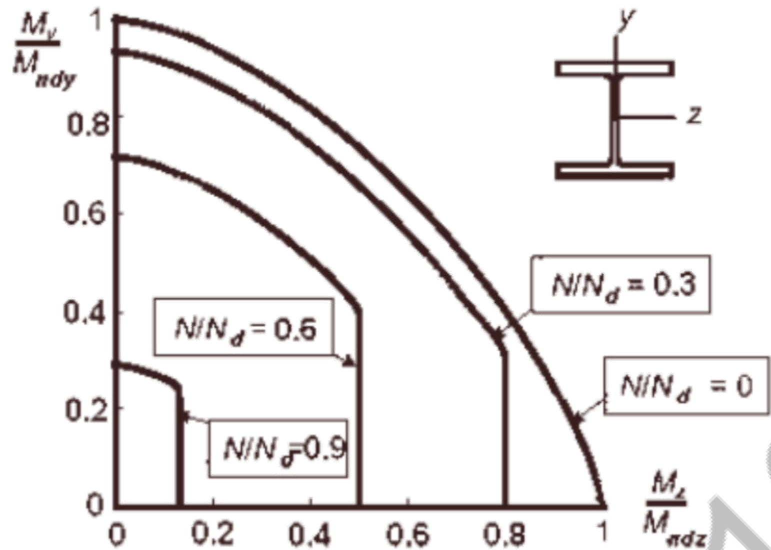
n = N / Nd

α*1*, α*2* = constants as given in Table 4.3of new IS: 800 and shown below:

Constants α1 and α2 *(Section 9.3.1.1of Draft IS: 800 – LSM version)*

Section	α_1	α_2
I and Channel	$5n \geq 1$	2
Circular tubes	2	2
Rectangular tubes	$1.66/(1-1.13n^2) \leq 6$	$1.66/(1-1.13n^2) \leq 6$
Solid rectangle	$1.73+1.8n^3$	$1.73+1.8n^3$

A typical interaction diagram for I beam-column segment with combined biaxialbending and compressive force is as shown in figure 4.25



Interaction curves for I beam-column segment in bi-axial bending andcompression

The above interaction formulae, $\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$ is a function of α*1* and α*2* and the values of α*1* and α*2* are in turn functions of the ratio $n = N / Nd$(refer Table 4.3 above), where, N and $Nd$are factored applied axial force and design strength in tension or compression as defined earlier. It can be observed from the table 4.3 that for I-section or channel section, in case, the ratio $n = N / Nd$is equal to 0.2, the value of α*1* becomes 1 and for $n = N / Nd > 0.2$, the value of α*1* is more than 1. As the value of α*1* increase above 1, the value of the component $\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1}$ gets reduced since the ratio is always less than 1. The values of Mndy and Mndz are also proportionately reduced to accommodate the value of axial tension or compression depending upon type of sections. The ratio $n = N / Nd$is also directly related in reducing the bending strength, Mndy and Mndz

The code stipulates **(as per clause 9.3.1.2 of Draft IS: 800 – LSM version)** thatfor plastic and compact sections without bolts holes, the following approximations maybe used while calculating / deriving the values of design reduced flexural strengthMndz and Mndy under combined axial force and the respective uniaxial moment actingalone i.e. Mndz and Mndyacting alone. The values of reduced flexural strength of thesection, (eitherMndz or Mndy) is directly related to the geometry of a particular section.We will now discuss how these values are changing depending upon geometry of aparticular section:

i) Plates (Section 9 of Draft IS: 800 – LSM version)

For rolled steel plates irrespective of their thickness the value of reduced flexuralstrength can be derived from the equation:–Mnd= Md(1− n2). Here, the equation for reduced flexural strength is again a function of the ratio n = N/ Nd , where, *N* and *Nd*arefactored applied axial force and design strength in tension or compression as definedearlier. For smaller values of the ratio *n*, the reduction in flexural strength is notsignificant since the reduction in flexural strength is directly proportional to square of theratio *n*. It is obvious from the equation that as the ratio tends towards the value 1, theamount of reduction in flexural strength increases and for extreme case, when the ratiois equal to 1, the value of reduced flexural strength is zero i.e. for this particular case noflexural or bending strength is available within the plate section. This situation alsosatisfies the Condition, $\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$

ii) Welded I or H sections (Section 9 of Draft IS: 800 – LSM version)

For welded I or H sections, the reduced flexural strength about the major axiscan be derived from the equation: $-M_{ndz} = M_{dz}(1-n)/(1-0.5a) \leq M_{dz}$ and about theminor axis: $-M_{ndy} = M_{dy}\left[1-\left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy}$ where, n = N/ Nd anda = (A − 2bt) /A/ ≤0.5.

Here the reduction in flexural strength for major axis is linearly and directlyproportional to the ratio *n* and inversely proportional to the factor *a*, which is a reductionfactor for cross sectional area ratio. It is pertinent to note that for a particular sectionalarea *A*, as the width and/or thickness of the flange of I or H section increases, the factor*a* reduces which in turn increases the value ofMndz .

For minor axis, the reduction in flexural strength is non-linearly proportional toboth the factors n and a, but as the value of the factor n increases considering otherfactor remaining unchanged, the value of Mndy decreases, conversely as the value ofthe factor increases a, the value of Mndy increases. For a particular case, when thenumerical value of factor n is equal to 1 and the numerical value of the factor is a 0.5,the numerical value of Mndy becomes zero. It can be observed that the factor a beingthe area ratio, it takes into account the effect of flange width and flange thickness. Asthe value of b or tfincreases, the value of the factor a reduces which in turn reducesfurther the value of design reduced flexural strengthMndy .

iii) Standard I or H sections (Section 9 of Draft IS: 800 – LSM version)

For standard I or H sections, the reduced flexural strength about the major axiscan be derived from the equation: –Mndz= 1.11Mdz (1− n)≤Mdz and about the minoraxis :–for n ≤ 0.2 , Mndz= Mndy and for, n ≤ 0.2 where, Mndy= 1.56Mdy (1− n)(n + 0.6)

Unlike welded I or H sections, Here we do not find the factor a, but reduction inflexural strength for major axis is linearly and directly proportional to the ratio n. It ispertinent to note that for all cases, as the factor increases n, further reduction inreduced flexural strength of the member takes place. For a particular case, when thefactor n becomes 1, the value of Mndz reduces to zero.

For minor axis, no reduction in flexural strength takes place till the ration = N/ Nd is restricted to 0.2. When the value of n is more than 0.2, the reduction inflexural strength for minor axis is linearly and directly proportional to the ratio n. For avalue of n = 1, the value of Mndy reduces to zero.

iv) Rectangular Hollow sections and Welded Box sections (Section 9of Draft IS: 800 – LSM version)

When the section is symmetric about both axis and without bolt holes, thereduced flexural strength about the major axis can be derived from the equation: –

Mndz= Mdz(1− n) / (1− 0.5aw) ≤ Mdz

and about the minor axis:–

$M_{ndy} = M_{dy}(1 - n) / (1 - 0.5a_f) \leq M_{dy}$.

As indicated in above equations, for rectangular hollow sections and welded boxsections, the reduction in flexural strength for both the axes, takes place in line with thatof Mndz for welded I or H sections as described earlier in ***ii)*** above. The only variation is,the factor *a* is replaced either by *a*w for Mndz or by *af*forMndy .

v) Circular Hollow Tubes without Bolt Holes (Section 9 of Draft IS:800 – LSM version)

The reduced flexural strength about both the axes can be derived from the equation :–(1.7)Mnd= 1.04 Md 1− n ≤Md . For smaller values of the ratio *n*, the reduction in flexuralstrength is not significant since the reduction in flexural strength is directly proportionalto the power of 1.7 for the ratio *n*. When the ratio *n* is equal to 1, the value of reducedflexural strength is zero i.e. for this particular case no flexural or bending strength isavailable for the circular section.

Usually, the points of greatest bending and axial loads are either at the middle or endsof members under consideration. Hence, the member can also be checked,conservatively, as follows:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where *N*, is the factored applied axial load in member under consideration,( ) Nd Ag fy / γm0 is the strength in tension as obtained from section **6,** *Mz*and *My* are theapplied moment about the major and minor axes at critical region, *Mdz*and *Mdy*are themoment capacity about the major and minor axes in the absence of axial load i.e. whenacting alone and Ag is the gross area of cross-section. This shows that in point of time,the summation of ratios of various components of axial forces and bending moments(including bi-axial bending moments) will cross the limiting value of 1.

For *Semi-compact sections*, when there is no high shear force (as per 9.2.1 ofDraft IS: 800 – LSM version) semi-compact section design is satisfactory undercombined axial force and bending, if the maximum longitudinal stress under combinedaxial force and bending fx ,satisfies the following criteria.

fx≤ fy / γm0

For cross section without holes, the above criteria reduces to

$$\frac{N}{N_d}+\frac{M_y}{M_{dy}}+\frac{M_z}{M_{dz}}\leq 1.0$$

Where Nd, Mdy, M dz are as defined earlier

Overall member strength check (section 9 of Draft IS : 800 -LSM version)

Members subjected to combined axial force and bending moment shall bechecked for overall buckling failure considering the entire span of the member. Thisessentially takes care of lateral torsional buckling.

a) For Bending moment and Axial Tension, the member should be checked forlateral torsional buckling to satisfy overall stability of the member under reducedeffective moment Meff due to tension and bending. The reduced effective momentMeff , can be calculated as per the equationMeff= [M − ψTZec / A] ≤ Md but in no caseshall exceed the bending strength due to lateral torsional buckling Md(**as per 8.22 ofDraft IS :800 - LSM version**). Here M,T are factored applied moment and tensionrespectively, A is the area of cross section, Zec elastic section modulus of the sectionwith respect to extreme compression fibre and the factor is equal to 0.8 when tensionand bending moments are varying independently or otherwise equal to 1. For extremecase, when the factor ψTZec / A is equal to M,Meff reduces to zero.

b) For Bending moment and Axial Compression, when the member is subjectedto combined axial compression and biaxial bending, the section should be checked tosatisfy the generalized interaction relationship as per the equation $\frac{P}{P_d} + \frac{K_y M_y}{M_{dy}} + \frac{K_z M_z}{M_{dz}} \le 1.0$ Here Ky ,Kz are the moment amplification factor about minor

and major axis respectively $\left(K_z = 1 - \frac{\mu_y P}{P_{dz}} \right)$ and $K_y = 1 - \frac{\mu_y P}{P_{dy}}$ where the factor μz andμy are dependent on equivalent uniform moment factor, b obtained from Table 4.3 ofDraft IS : 800 - LSM version, according to the shape of the bending moment diagrambetween lateral bracing points in the appropriate plane of bending and non-dimensionalslenderness ratio, l), P is the applied factored axial compression, My ,Mz are theapplied factor bending moments about minor and major axis of the member,respectively and Pd ,Mdy ,Mdz are the design strength under axial compression , bendingabout minor and major axis respectively , as governed by overall buckling criteria. Thedesign compression strength, Pd , is the smallest of the minor axis ( ) Pdy and major axis(Pdz) buckling strength as obtained from 7.12 of Draft IS :800 - LSM version and thedesign bending strength (Mdz)about major axis is equal to(Md),where (Md) is thedesign flexural strength about minor axis given by section 8.2.1 of Draft IS : 800 – LSMversion, when lateral torsional buckling is not significant and by section 8.2.2 of DraftIS: 800 - LSM version, where lateral torsional buckling governs. For design BendingStrength about minor axis, Mdy= Md where, Md is the design flexural strength aboutminor axis calculated using plastic section modulus for plastic and compact sectionsand elastic section modulus for semi-compact sections

c) The factors are as defined below.

μz is the larger of μLT and μfz as given below.

μLT = 0.15λyβMLT − 0.15 ≤ 0.90

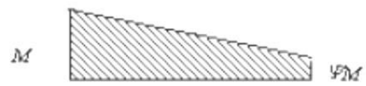
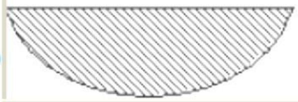
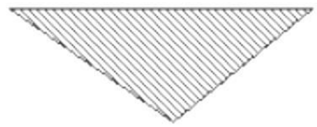
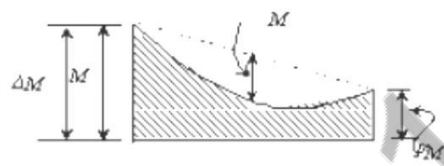
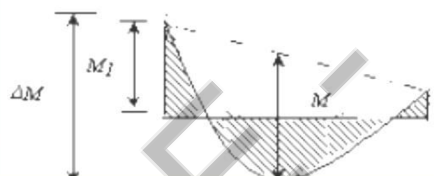
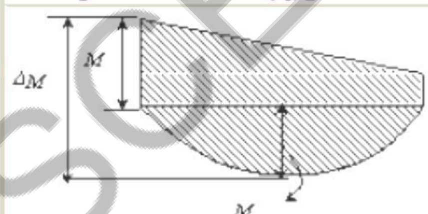
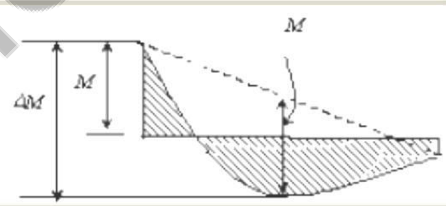
$$\mu_{fz} = \lambda_z \left(2\beta_{Mz} - 4\right) + \left[\frac{Z_Z - Z_{eZ}}{Z_{eZ}}\right] \le 0.90$$

$$\mu_y = \lambda_y \left(2\beta_{My} - 4\right) + \left[\frac{Z_y - Z_{ey}}{Z_{ey}}\right] \le 0.90$$

β_{My} ,β_{Mz} ,β_{MLT} = equivalent uniform moment factor obtained from Table 4.4 of Draft IS:800-LSM version, according to the shape of the bending moment diagram betweenlateral bracing points in the appropriate plane of bending

λ_y ,λ_z = non-dimensional slenderness ratio (7.1.2 of Draft IS 800- LSM version)about the respective axis.

Draft IS: 800-LSM Version, Equivalent uniform moment factor(Section 9.3.2.2.1 of draft IS: 800-LSM version)

Particulars	BMD	b_m
Due to end moments		1.8-0.7y
Moment due to lateral loads		1.3
		1.4
moment due to lateral loads and end moments		1.8-0.7y+M_Q / Δ_M(0.74-0.5) M_Q= $\lvert M_{max}\rvert$ due to lateral load alone Δ_M= $\lvert M_{max}\rvert$ (same curvature) Δ_M= $\lvert M_{max}\rvert$ + $\lvert M_{min}\rvert$ (reverse curvature)
		
		
		

DESIGN OF BEAM COLUMN

Combined action of bending and axial force (tension or compression) occurs in following situations.

- Any member in a portal frame.
- Beam transferring reaction load to column.
- Effect of lateral load on a column due to wind, earthquake
- Effect of eccentric load by crane loading due to bracket connection to column.
- In case of principal rafter, purlins not placed exactly over joint of roof truss.

- Minimum eccentricity of load transferred by beam to column is specified by clause 7.3.3 (p. no. 46)

- Section-9, Member subjected to combined forces.

clause 9.3 for combined axial force and bending moment (p. no. 70) recommends check for section

a) By material failure clause 9.3.1

b) By overall buckling failure clause 9.3.2

Example No : 1 Design a suitable I beam for a simply supported span of 5 m. and carrying a dead load of 20 kN/m and imposed load of 40 kN/m. Take fy = 250 MPa

Design load calculations :

Factored load = γ_{LD} x 20 + γ_{LL} x 40

Using partial safety factors for D.L γ_{LD} = 1.50 and for L.L γ_{LL} = 1.5 (*Cl. 5.3.3 Table 4, Page 29)*

Total factored load = 1.50 x 20 + 1.5 x 40 = 90 kN/m

Factored Bending Moment M = 90 x 5 x 5 / 8

= 281.25 kN.m

Zp required for value of *fy* = 250 MPa and

γ_{mo} = 1.10 (*Table 5, Page 30*)

Zp = (281.25 x 1000 x 1000 x 1.1) / 250 = 1237500 mm^3

= 1237.50cm3

Using shape factor = 1.14, *Ze* = 1237.50/1.14 =1085.52 *cm3*

Options ISWB 400 @ 66.7 *kg/m* or ISLB 450 @ 65.3 *kg/m*

Try ISLB 450

Ze = 1223.8 cm^3> 1085.52

Geometrical Properties : ISLB 450

D = 450 mm , B = 170 mm , tf = 13.4 mm , tw = 8.6 mm , h1 = 384 mm , h2 = 33 mm

Ixx = 27536.1 cm4

As $fy = 250\ MPa$,

Section Classification :

B/2tf = 85 / 13.4 = 6.34 < 9.4ε

h1 / tw = 384/8.6 = 44.65 < 83.9 ε

Section is Classified as *Plastic*

Zp = 1.14 x 1223.8 = 1395.132 cm3

Design Bending Strength: Md

$$M_d = \frac{\beta_b Z_p fy}{\gamma_{mo}} = \frac{1.0x1395.132x1000x250}{1.10} = 317.075 kN.m$$

>281.25 kN.m

βb = 1.0for plastic section (Cl. 8.2.1.2, Page 53)

Check for Serviceability – Deflection

Load factor = γLDand γLL = 1.00 both , (Cl. 5.6.1, Page 31)

Design load = 20 + 40 = 60 kN/m

$$\delta = \frac{5x60x(5000)^4}{384x2x10^5x27536.1x10^4} = 8.866\,mm$$

Limiting deflection = Span/360 *(Table. 5.3, Page 52)*

= 5000/360 = 13.889 mm….*OK*

Hence Use ISLB 450

DESIGN OF BEAM WITH HIGH SHEAR

Example No. 2 Factored Load 100 KN/m.A beam two span of each 5m.

Degree of Redundancy = r = 1

No. of plastic hinges required to transform structure into mechanism = r + 1 = 2

Failure of any span is failure of continuous beam.

Failure mechanism of AB & BC is identical due to symmetry & this is similar to failure mechanism of propped cantilever beam with udl.

$$wp = 11.656\ Mp / l^2$$

$$\therefore Mp = wp.l^2 / 11.656$$

$$= 100 \times 25 / 11.656$$

$$= 214.48\ KNm.$$

As both spans fail simultaneously actual no of plastic hings are three – two hinges each at 0.414 l from A & C & third at B.

$\therefore$ as n = 3 > 2 required

Collapse is over complete

$$Zp = 214.48 \times 10^6 \times 1.10 / 250 \quad mm^3$$

$$= 943.72\ cm^3$$

$Ze = 943.72 / 1.14 = 827.82\ cm^3$

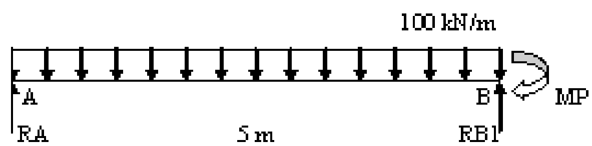
Select ISLB 400 $\quad$ $Zxx = 965.3\ cm^3$

$Md = 1.0 \times 1.14 \times 965.3 \times 250 / 1.10 = 250.1$ KNm

> 214.48

<u>Reaction at A</u>

Considering free body of AB



$Mp = 214.48$ KNm

$Mp + RA \times 5 = 100 \times 5 \times 5/2 \;\therefore RA = 207.1$ KN

$RB1 = 500 – 207.1 = 292.9$ KN

Due to symmetry in loading

Maximum shear is at B = 292.9 KN= V

$Vd = 0.577 \times 400 \times 8 \times 250 / 1.1 = 419.636$ KN

Where 400×8 = D.tw of ISLB 400

As $V/Vd = 292.9 / 419.636 = 0.697 > 0.6$

As per *C1.9.2.2 Page No. 70*

Effect of shear is to be considered for reduction in moment capacity

$Mdv = Md – \beta(Md – Mfd)$

$\beta = (2V/Vd – 1)2 = 0.156$

Mfd = Plastic moment capacity of flanges only

$= 165 \times 12.5 (400 - 12.5) \times 250 / 1.1 = 181.64$ KNm

$\therefore Mdv = 250.1 - 0.156 (250.1 - 181.64)$

$= 239.42$ KNm

As Mdv = 239.42 >Mp = 214.48 ------- Ok

Select ISLB 400 @ 56.9 kg / m

DESIGN OF BEAM COLUMN

Example No: 3A column in a building 4m in height bottom end fixed, top end hinged.reaction load due to beam is 500 kN at an eccentricity of 100 mm from major axis of section.

Design

Column is subjected to axial compression of 5×10^5 N with bending moment of 50×10^6 Nmm. Taking design compressive stress for axial loading as 80 Mpa.

$A_e$reqd = $500 \times 10^3 / 80 = 6250$ mm^2

To account for additional stresses developed due to bending compression.

Try ISHB 300 @ 0.58 kN/m

$A_g = 7485$ sq.mm, $r_{xx} = 129.5$ mm, $r_{yy} = 54.1$ mm

$f_y = 250$ Mpa

Classification of section

$b/t_f = 125 / 10.6 = 11.79 > 10.5$ (limit for compact section)

Flange is semicompact

$h_1/t_w = 249.8 / 7.6 = 32.86 < 84$

Web is plastic

Therefore overall section is semicompact.

a) Section strength as governed by material failure *(clause 9.3.1)*

Axial stress = $N/A_e = 500 \times 10^3 / 7485$

$= 66.80\ N/mm^2$

Bending stress $M_z/Z_e = 50 \times 10^6 / 836.3 \times 10^3$

$= 59.78\ N/mm^2$

As the section is semicompact use clause 9.3.1.3 *(p. no. 71)*

Due to bending moment at top, horizontal shear developed 'V' is 18.75 kN = 18750 N

Shear strength of section $V_d = ((f_y / \sqrt{3}) . h . t_w) / 1.10$

$= 299\ kN$

As $V/V_d = 18750 / 299 \times 10^3 = 0.062 < 0.6$

Reduction in moment capacity need not be done.

As per clause 9.3.1.3 *(p. no. 71)*

Total longitudinal compressive stress

$f_x = 66.80 + 59.78$

$= 126.58 < f_y/\gamma_{mo} = 227.27$…… OK

Alternately

N = 500 kN

$N_d = A_g .f_y / \gamma_{mo} = 7485 \times 250 / 1.1 = 1701.136\ kN$

$M_z = 50 \times 10^6 Nmm = 50\ kNm$

$M_{dz} = Z_e .f_y / \gamma_{mo} = 836.3 \times 10^3 \times 250 / 1.10$

$= 190.068\ kN$

Hence, $(500 / 1701.136) + (50 / 190.068)$

$= 0.557 < 1$ ……. OK

b) Member strength as governed by buckling failure *clause 9.3.2 (p. no. 71)*

In the absence of M_y, equations are reduced to

$$\frac{P}{P_{dy}} + k_{LT} \frac{M_z}{M_{dz}} \le 1$$

$$\frac{P}{P_{dz}} + k_z \frac{C_{mz} M_z}{M_{dz}} \le 1$$

Where, $P = 500 \times 10^3\ N$

$M_z = 50 \times 10^6 Nmm$

$M_{dz}= \beta_b . Z_p .f_{bd}$

$\beta_b= Z_e/ Z_p$ as section is semicompact

Therefore $M_{dz} = Z_e f_{bd}$

$f_{bd} = \chi_{LT} f_y / \gamma_{mo}$

χ_{LT} = bending stress reduction factor to account torsional buckling.

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1$$

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

$\alpha_{LT} = 0.21$ for rolled section

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}}$$

$f_{cr,b}$ depends on following factors

$k_L/ r_{yy}= 0.8 \text{ X } 4000 / 54.1 = 59.15$

$h / t_f = 300/10.6 = 28.30$

Using table 14, (p. no. 57)

$f_{cr,b} = 691.71 \text{ N/mm}^2$

$$\lambda_{LT} = \sqrt{\frac{250}{691.71}} \qquad = 0.060 < 0.4$$

As per *clause 8.2.2 (p. no. 54)*

Resistance to lateral buckling need not be checked and member may be treated as laterally supported.

$M_{dz}=Z_e .f_y / \gamma_{mo} = 190$ kNm

Evaluation of P_{dy} buckling load @ yy axis

Referring table 10 (p. no. 44)

$h/b_f=300/250 = 1.2$

buckling @ yy axis is by class ‘c’

$t_f = 10.6$ mm < 100mm

buckling @ zz axis is by class ‘b’

$l_y / r_y = 3200/54.1 = 59.15$

For $f_y = 250$ and using *Table 9 (c), (p. no. 42)*

F_{cdy} = 169.275 N/mm^2

$P_{dy} = A_g . f_{cdy}$

= 1267.02 kN

Evaluation of P_{dz} buckling @ zz axis

l_z / r_z = 3200 / 129.5 = 24.71

For f_y= 250 and using *Table 9 (b), (p. no. 41)*

f_{cdz} = 220.76 N/mm^2

Therefore $p_{dz} = A_g . f_{cdz}$ = 1652.38 kN

$K_z = 1 + (\lambda_z - 0.2) n_z$

Where,

$$\lambda_z = \sqrt{\frac{f_y}{f_{cr,z}}}$$

l_z / r_z = 24.71, h/t_f = 300 / 10.6 = 28.30

From table 14 (p. no. 57)

$f_{cr,z}$= 4040 N/mm^2

Ratio of actual applied load to axial strength,

n_z = 500 / 1625.38 = 0.30

n_y = 500 / 1267.02 = 0.39

$\lambda_z = \sqrt{250/4040} = 0.246$

$K_z = 1 + (\lambda_z - 0.2)\, n_z = 1.0138 < 1 + 0.8\, n_z$ < 1.24…. OK

ψ = ratio of minimum to maximum BM

ψ = -25 / 50 = -1 / 2

C_{mz} = 0.6 + 0.4 X (ψ) = 0.4

$$K_{LT} = 1 - \frac{0.1 \lambda_{LT} n_y}{C_{mLT} - 0.25}$$

= 0.844

$$\frac{P}{P_{dy}} + K_{LT} \frac{M_z}{M_{dz}} = 0.612$$ < 1 ……. OK

$$\frac{P}{P_{dz}} + K_z \frac{C_{mz} M_z}{M_{dz}} = 0.406$$ < 1 ……. OK

Hence select ISHB 300 @ 0.58 kN/m as a section for eccentrically loaded column.

Example No: 4

An ISMB 400 beam is to be connected to an ISHB 250 @ 537 N/m to transfer a end force of 140 kN. Design double plated welded connection.

Solution:

Factored $V = 140 \times 1.5 = 210$ kN

Using 50 mm wide plates, factored moment on weld connecting plate and web of beam (weld B)

$M = 210 \times 50 =$ kN-mm $= 210 \times 50 \times 10^3$ N-mm.

Thickness of plate should be 1.5 mm more than the web thickness of the beam.

$\therefore$ Thickness of plate $= t_w + 1.5 = 8.9 + 1.5 = 10.4$ mm.

Use 12 mm plates.

Since one weld is shop weld and the other is field weld, design is made for field weld and the same is adopted for shop weld also. For field weld partial safety factor $V_{mw} = 1.5$. Hence

$$\text{Strength of weld} = \frac{f_u}{\sqrt{3}} \times \frac{1}{1.5} = \frac{410}{\sqrt{3}} \times \frac{1}{1.5}$$

$f_{wd} = 157.81$ N/mm^2

Design of Weld B:

$V = 210$ kN-m; $M = 210 \times 50 \times 10^3$ N-mm. Assuming 6 mm as the size of weld, throat thickness of weld $= 0.7 \times 6$. Since there are two rows of welds.

$$d = \sqrt{\frac{6M}{2 \times t \times f_{wd}}} = \sqrt{\frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 157.81}}$$

$$= 218 \text{ mm.}$$

The above depth is required to resist bending alone. Since the weld has to resist shear also, try 15 to 20% additional depth.

Trial depth $h = 1.2 \times 218 = 261.6$ mm

Try $h = 260$ mm

Selected h is between $\frac{1}{2}$ to $\frac{2}{3}$ rd of depth of beam.

$$\therefore \quad \text{Bending stress } q_2 = \frac{M}{2} = \frac{6M}{2th^2}$$

$$\text{i.e.} \quad = \frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 260^2} = 110.95 \text{ N/mm}^2$$

Direct shear stress $q_1 = \frac{p}{2th} = \frac{210\times10^3}{2\times0.7\times6\times260}$
$= 96.15 \text{ N/mm}^2$

$\therefore$ Resultant stress $q = \sqrt{110.95^2 + 96.15^2}$

$= 146.8 \text{ N/mm}^2 < 157.81 \text{ N/mm}^2$

Hence adequate. Provide 6 mm size fillet welds. 260 mm long.

Design of Weld A:

This weld carries only shear.

$$V = 210 \text{ kN}$$

The length of this weld is also kept 260 mm.

Let size of weld be s.

$\therefore$ Throat thickness of weld $t = 0.7\,s$. Since there are two weld lines equating strength of these welds to shear, we get

$$2\,t\,d\,f_{wd} = V$$

$$2 \times 0.7\,s \times 260 \times 157.81 = 210 \times 10^3$$

S=3.65

Provide 5mm welds.

Example No: 5

Design a stiffened seat connection to connect the ISMB 500 transferring a load of 260 kN to an ISHB 300 @ 577 N/m.

Solution:

End reaction $F = 260$ kN

For ISMB 500,

$b_f = 180$ mm $\quad t_f = 17.2$ mm

$t_w = 10.2$ mm $\quad h_2 = t_f + r_1 = 17.2 + 17 = 34.2$

$B = \frac{260\times10^3}{185\times10.2} = 137.8 \text{ mm}$

Bearing length $b = B - \sqrt{3}\,h_2 \nless \frac{1}{2}B$

$= 137.8 - \sqrt{3} \times 34.2 \nless \frac{1}{2} \times 137.8$

$= 78.6$ mm

10 mm clearance is provided between the end of beam and the flange of column.

Minimum width of plate required = 78.6 + 10 = 88.6 mm.

Use 90 mm seat plate. Its thickness should be at least equal to t_f = 17.2 mm. Hence use 18 mm thick plate. Width of this plate is kept equal to the flange width of beam = 180 mm.

Thickness of stiffening plate should be more than t_w (10.2 mm in this case).

Provide 12 mm plate. Let its depth also be equal to 180 mm.

The distance of centre of gravity of reaction from column flange $= 90 - \frac{1}{2} \times 78.6 = 90.7$ mm.

Factored bending moment $M = 1.5 \times 260 \times 50.7 = 19773$ kN-mm

$= 19.733 \times 10^6$ N-mm

Let throat thickness of weld be 't'. Figure 9.7 shows the size of weld.

The distance of c.g. of weld from top

$$y_1 = \frac{2 \times 180 \times t \times 90}{168t + 2 \times 180t} = 61.4 \text{ mm}$$

$$\therefore \quad I_{xx} = 168t \times 61.4^2 + 2 \times \left[\frac{1}{12} \times t \times 180^3 + t \times 180(90 - 61.4)^2\right]$$

$$= 1899819t \text{ mm}^4$$

Equating the resultant stress to it, we get

$$\frac{975.4}{t} = 189.37$$

$$\therefore \quad t = 5.15$$

$$\therefore \quad \text{Size of weld } s = \frac{t}{0.7} = \frac{5.15}{0.7} = 7.35 \text{ mm}$$

Provide 10 mm weld.

Provide clip angle ISA 100100, 6 mm at the top with 6 mm weld.

Size of seat plate = 90 × 180 × 18 mm

Size of stiffener plate = 180 × 90 × 12 mm

Size of shop weld = 10 mm.

Example No: 6

An ISMB 450 is connected to the flange of a column ISHB 300 @ 618 N/m. The end reaction transmitted by the beam is 120 kN. Design an unstiffened seated connection. Use M20 black bolts.

Solution:

Shear force at working Load = 120 kN.

For ISMB 450, $t_w = 9.4$ mm

$$h_2 = t_w + r_1 = 17.4 + 15 = 32.4$$

$$B = \frac{F}{f_p t_w} = \frac{120\times10^3}{185\times9.4} = 69 \text{ mm}$$

$\therefore \quad b = B - \sqrt{3}\,h_2$, subject to a minimum of $\frac{B}{2}$

$= 69 - \sqrt{3}\times32.4$, subject to a minimum of 34.5 mm

$= 12.886$, subject to a minimum of 34.5 mm

$\therefore \quad b = 34.5$ mm

Try ISA 150115, 12 mm.

$r_1 = 11$ mm.

$\therefore$ Critical section x-x is at

$= 12 + 11 = 23$ mm, from the back of the angle.

Centre of gravity of load is at

$$= 10 + \frac{34.5}{2} = 27.25 \text{ mm}$$

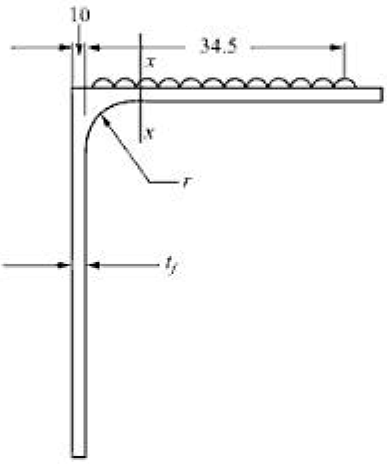
Factored Force $= 1.5 \times 120$ kN.

$\therefore$ Factored Moment $= 1.5 \times 120\,(27.25 - 23)$

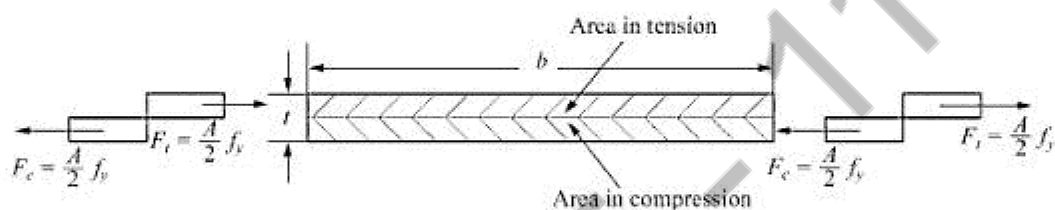
$= 765 \times 10^3$ N-mm

Length of seating angle = Width of flange of beam

= 150 mm



Z_p of rectangular section of width 'b', thickness 't':



$$M_p = \frac{1}{2} b \times t \times f_y \times \frac{t}{2} = \frac{bt^2}{4} f_y$$

$$\therefore \quad Z_p = \frac{bt^2}{4}$$

$$\therefore \quad M_d = \frac{f_y Z_p}{\gamma_{mp}} = \frac{250}{1.25} \times \frac{bt^2}{4} = \frac{250}{1.25} \times \frac{150t^2}{4} = 6250t^2$$

t=11.06 mm

Provide 4 M20 bolts in two rows.

Hence use ISA 150125, 12mm thick angle so that 2 rows bolts can be provided.

Connect tp cleat angle ISA 10075, 10mm with 2 field bolts of M20 in either leg of angle.

QUESTION BANK

PART A

1. What is meant by Web crippling and Web Buckling?
2. Define Effective length.
3. What are the commercial forms of beams in steel structures?
4. Define laterally supported and laterally unsupported beams.
5. Why are rolled I-sections widely used as beam members?
6. What are the reasons for specifying deflection limitations?
7. What is the maximum deflection caused at the mid-span of simply supported beams?
8. What is meant by Plate Girder?What are the components of Plate Girder?
9. What is meant by intermediate and bearing Stiffeners?
10. What is meant by web splices?
11. Mention the modes of failure of beams?

PART B

1. A simply supported beam spanning 5m carries an udl of 3 KN/m including its self weight. Floor construction restraints it against lateral bukling.What size of beam required with f_y=250Mpa?

2. A welded plate girder has i) each top and bottom flanges =435x28mm and ii) web=1250x10mm.Design the vertical and horizontal stiffeners.

3. A simply supported beam in both planes of 6m effective span is subjected to biaxial bending forces i) a vertical concentrated force of 65kN at mid span and ii)a lateral concentrated force of 8kN at mid span. Design the beam using rolled beam sections.

4. A proposed cantilever beam is built in to a concrete wall and free at its end. It supports dead load of 20kN/m and a live load of 10kN/m.The length of the beam is 5m.Select an available section with necessary checks. Assume bearing length of 100mm.

5. Design a simply supported beam of 10m effective span carrying a total factored load of 60kN/m. The depth of beam should not exceed 500mm.The compression flange is laterally supported by floor construction. Assume stiff bearing is 75mm.