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Lecture Note on STRUCTURAL DESIGN - II (STEEL DESIGN)

(5^{th} Semester)

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GENERAL CONSIDERATION OF STEEL AS STRUCTURAL MATERIAL

CHAPTER -1

INTRODUCTION :-

A structure is an assemblage of a group of elements / members capable of withstanding external loads transmitting them safely to the foundation .Infrastructure development of the country mainly consists of structures like buildings, bridges, aerodromes etc.

Depending upon the orientation of structures and their structural use ,the members are subjected to axial forces ,bending or torsion or a combination there of and are according named based upon their natures of stresses i.e. tension ,compression or flexural members etc.

Design of a building structure focuses on two aspects namely

1- Functional design

2- Structural design

COMMON STEEL STRUCTURES:-

Steel has been extensively used as a building material in various types of structure .steel structure can be broadly subdivided into two groups.

1- Framed structures- which combination of beams, columns, ties, and trusses etc.

2- Sheet structure – which are largely made up of plates or sheets, such as tanks, bins, chimneys, roof covering, wall claddings etc.

ADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL: -

1- Smaller weight to strength ratio – It has smaller weight to strength ratio resulting in light weight ,structures for covering large spans.

2- Speed of erection – steel structures can be speedily construction due to prefabrication in the workshop or at site out of standard available sections.

3- Addition, alternation and strengthening – Addition and alternation of steel structures can be easily accomplished by welding and hence steel structures can be strengthened at any later time.

4- Easy dismantling and transportation - By using bolted connections ,steel structure can be easily dismantled and conveniently handled. It can be easily transported to other sites being light weight and small volume.

5- Assured quality and high durability - Being manufactured in the factory, desired quality could be assured and if properly maintained, steel structures have long life.

6- High scarp and recyclable value – IT has high scrap value for it can be easily reused after dismantling and also can be economically recycled.

DISADVANTAGES OF STEEL STRUCTURE : -

1- Corrosion susceptibility – Steel structures when exposed to humid atmosphere are liable to corrosion .

2- High maintenance cost- They require regular painting and maintenance.

3- Chemical detoriation - It detoriaties when comes in contact with certain chemicals or gases and also when buried underground without a protective layer.

4- Fire and heat susceptibility – Although not combustion ,it is a good conductor of heat .Drastic reduction of its strength under high temperature takes place when exposed to fire and hence needs fire proof treatment.

5- Costlyand susceptible to theft – It is a costly material and can be easily cut or dismantled and liable to theft.

6- Susceptible to fatigue – When subjected to large number or repeated stress reversals ,fatigue involves a reduction in strength and may finally lead to failure.

SIMPLE CONNECTIONS, RIVETED, BOLTED AND WELDED CONNECTIONS (CHAPTER -2)

INTRODUCTION

In engineering practice it is often required that two sheets or plates are joined together and carry the load in such ways that the joint is loaded. Many times such joints are required to be leak proof so that gas contained inside is not allowed to escape. A riveted joint is easily conceived between two plates overlapping at edges, making holes through thickness of both, passing the stem of rivet through holes and creating the head at the end of the stem on the other side. A number of rivets may pass through the row of holes, which are uniformly distributed along the edges of the plate. With such a joint having been created between two plates, they cannot be pulled apart. If force at each of the free edges is applied for pulling the plate apart the tensile stress in the plate along the row of rivet hole and shearing stress in rivets will create resisting force. Such joints have been used in structures, boilers and ships. The following are the usual applications for connection.

1. Screws ,
2. Pins and bolts,
3. Cotters and Gibs,
4. Rivets,
5. Welds.

Of these screws, pins, bolts, cotters and gibs are used as temporary fastening i.e., the components connected can be separated easily. Rivets and welds are used as permanent fastenings i.e., the components connected are not likely to require separation.

RIVETS

Rivet is a round rod which holds two metal pieces together permanently. Rivets are made from mild steel bars with yield strength ranges from 220 N/mm^2 to 250 N/mm^2. A rivet consists of a head and a body as shown in Fig.1. The body of rivet is termed as shank. The head of rivet is formed by heating the rivet rod and upsetting one end of the rod by running it into the rivet machine. The rivets are manufactured in different lengths to suit different purposes. The size of rivet is expressed by the diameter of the shank.

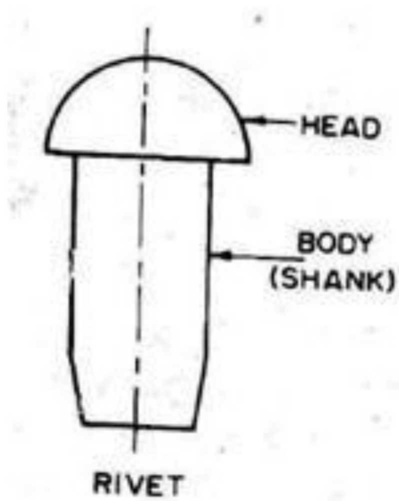


Fig. 1 Parts of Rivet

Holes are drilled in the plates to be connected at the appropriate places. For driving the rivets, they are heated till they become red hot and are then placed in the hole. Keeping the rivets pressed from one side, a number of blows are applied and a head at the other end is formed. When the hot rivet so fitted cools it shrinks and presses the plates together. These rivets are known as hot driven rivets. The hot driven rivets of 16 mm, 18 mm, 20 mm and 22 mm diameter are used for the structural steel works.

Some rivets are driven at atmospheric temperature. These rivets are known as cold driven rivets. The cold driven rivets need larger pressure to form the head and complete the driving. The small size rivets ranging from 12 mm to 22 mm in diameter may be cold driven rivets. The strength of rivet increases in the cold driving. The use of cold driven rivets is limited because of equipment necessary and inconvenience caused in the field.

The diameter of rivet to suit the thickness of plate may be determined from the following formulae:

1. Unwins's formula
2. The French formula
3. The German formula

$d=6.05\ t^{0.5}$

$d=1.5\ t + 4$ $d=(50\ t - 2)^{0.5}$

Where d= nominal diameter of rivet in mm and t= thickness of plate in mm. RIVET HEADS

The various types of rivet heads employed for different works are shown in Fig. 5.2. The proportions of various shapes of rivet heads have been expressed in terms of diameter 'D' of the shank of rivet. The snap head is also termed as round head and button head. The snap heads are used for rivets connecting structural members. Sometimes it becomes necessary to flatten the rivet heads so as to provide sufficient clearance. A rivet head which has the form of a truncated cone is called a countersunk head. When a smooth flat surface is required, it is necessary to have rivets countersunk and chipped.

RIVET HOLES

The rivet holes are made in the plates or structural members by punching or drilling. When the holes are made by punching, the holes are not perfect, but taper. A punch damages the material around the hole. The operation known as reaming is done in the hole made by punching. When the hole are made by drilling, the holes are perfect and provide good alignment for driving the rivets. The diameter of a rivet hole is made larger than the nominal diameter of the rivet by 1.5 mm of rivets less than or equal to 25 mm diameter and by 2 mm for diameter exceeding 25 mm.

DEFINITIONS OF TERMS USED IN RIVETING

Nominal diameter of rivet (d):

The nominal diameter of a rivet means the diameter of the cold shank before driving.

Gross diameter of rivet (D):

The diameter of the hole is slightly greater than the diameter of the rivet shank. As the rivet is heated and driven, the rivet fills the hole fully. The gross or effective diameter of a rivet means the diameter of the hole or closed rivet. Strengths of rivet are based on gross diameter.

Pitch of rivet (p):

The pitch of rivet is the distance between two consecutive rivets measured parallel to the direction of the force in the structural member, lying on the same rivet line. Minimum pitch should not be less than 2.5 times the nominal diameter of the rivet. As a thumb rule pitch equal to 3 times the nominal diameter of the rivet is adopted. Maximum pitch shall not exceed 32 times the thickness of the thinner outside plate or 300 mm whichever is less.

Gauge distance of rivets (g):

The gauge distance is the transverse distance between two consecutive rivets of adjacent chains (parallel adjacent lines of fasteners) and is measured at right angles to the direction of the force in the structural member.

Gross area of rivet:

The gross area of rivet is the cross sectional area of a rivet calculated from the gross diameter of the rivet.

Rivet line:

The rivet line is also known as scrieve line or back line or gauge line. The rivet line is the imaginary line along which rivets are placed. The rolled steel sections have been assigned standard positions of the rivet lines. The standard position of rivet lines for the various sections may be noted from ISI Handbook No.1 for the respective sections. These standard positions of rivet lines are conformed to whenever possible. The departure from standard position of the rivet lines may be done if necessary. The dimensions of rivet lines should be shown irrespective of whether the standard positions have been followed or not.

Staggered pitch:

The staggered pitch is also known as alternate pitch or reeled pitch. The staggered pitch is defined as the distance measured along one rivet line from the centre of a rivet on it to the centre of the adjoining rivet on the adjacent parallel rivet line. One or both the legs of an angle section may have double rivet lines. The staggered pitch occurs between the double rivet lines.

TYPES OF JOINTS

Riveted joints are mainly of two types, namely, Lap joints and Butt joints.

Lap Joint: Two plates are said to be connected by a lap joint when the connected ends of the plates lie in parallel planes. Lap joints may be further classified according to number of rivets used and the arrangement of rivets adopted. Following are the different types of lap joints.

1. Single riveted lap joint (Fig.),

2. Double riveted lap joint:

a. Chain riveted lap joint (Fig.) b. Zig-zag riveted lap joint (Fig.)

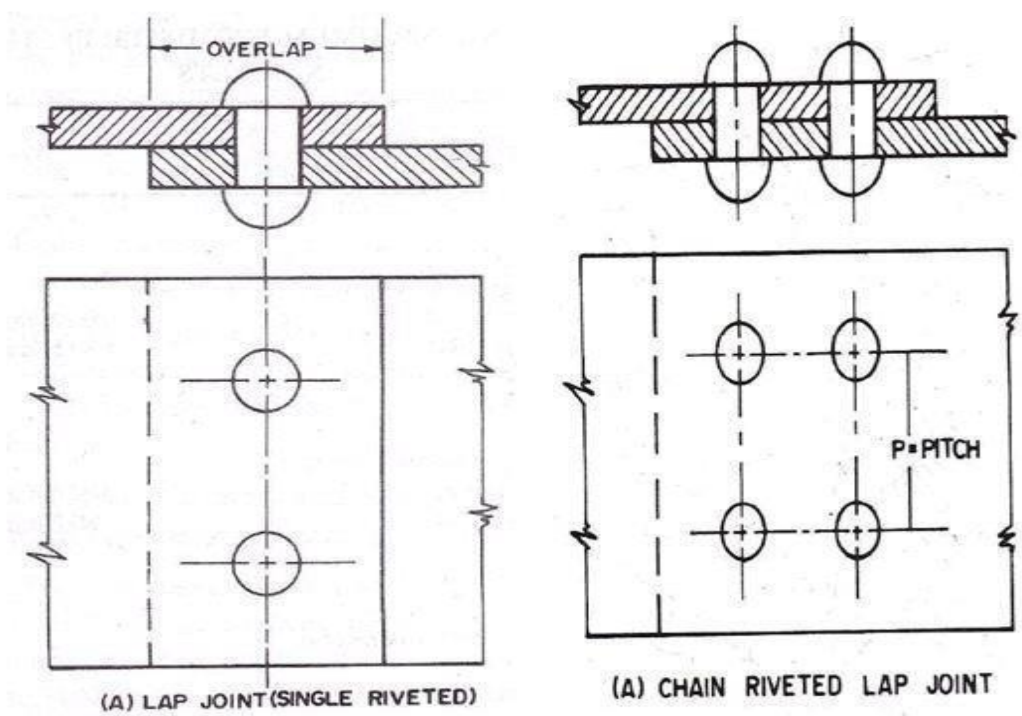


Fig. 2 lap joint

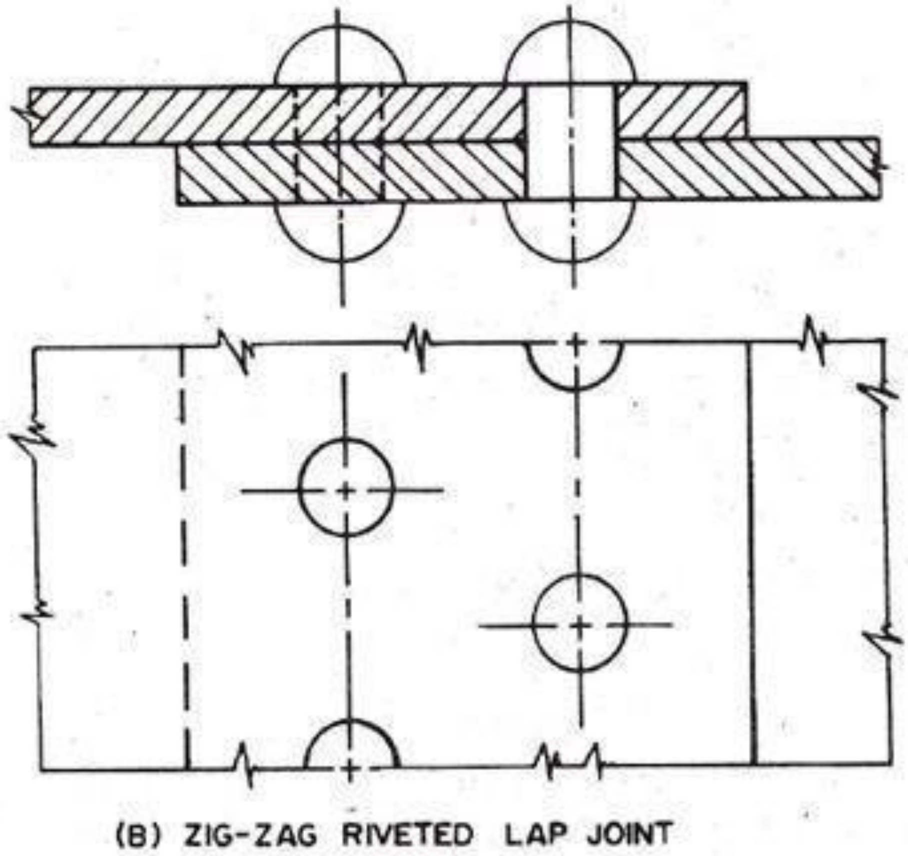
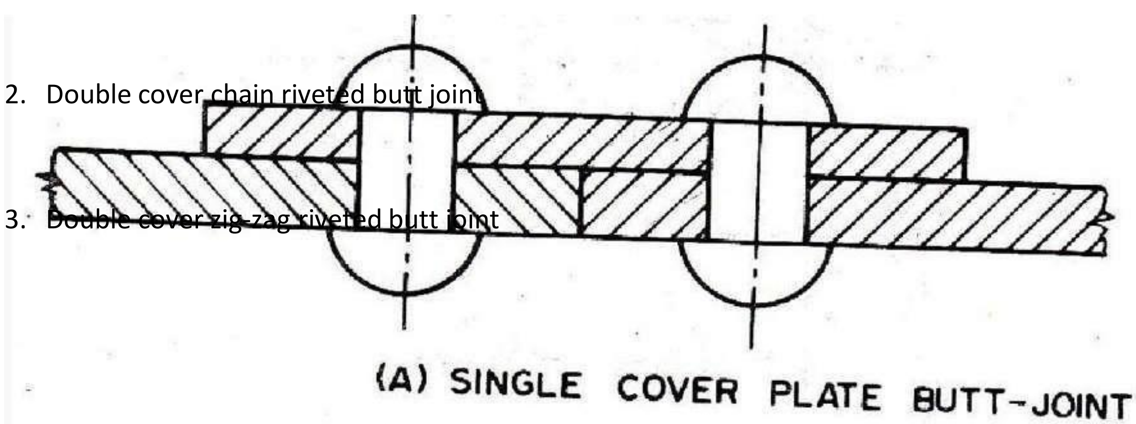


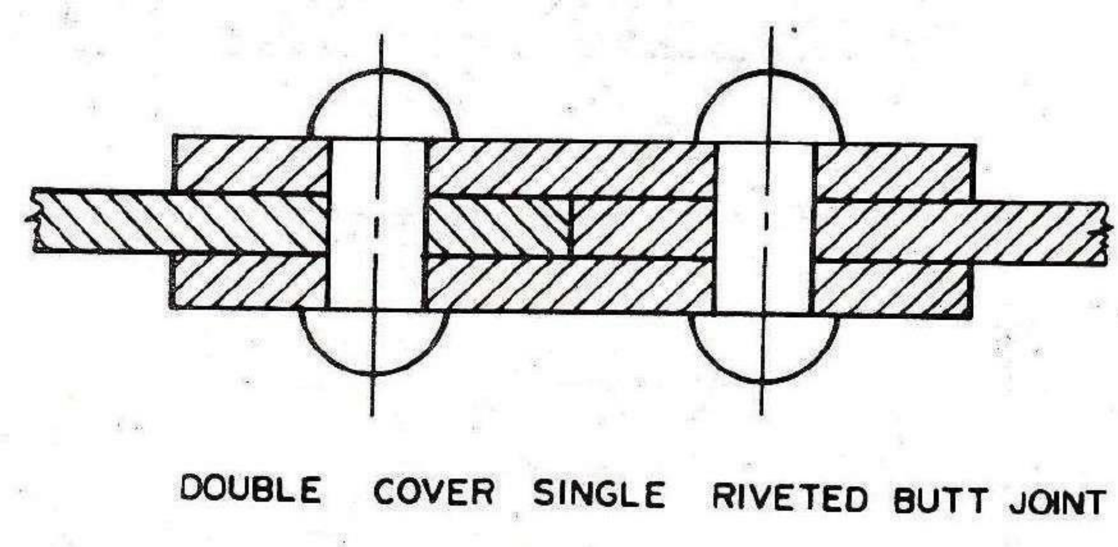
Fig. 3 Zig-Zag Lap Joint

Butt Joint:

In a butt joint the connected ends of the plates lie in the same plane. The abutting ends of the plates are covered by one or two cover plates or strap plates. Butt joints may also be classified into single cover but joint, double cover butt joints. In single cover butt joint, cover plate is provided on one side of main plate. In case of double cover butt joint, cover plates are provided on either side of the main plate. Butt joints are also further classified according tothe number of rivets used and the arrangement of rivets adopted.

1. Double cover single riveted but joint
2. Double cover chain riveted butt joint
3. Double cover zig-zag riveted butt joint





FAILURE

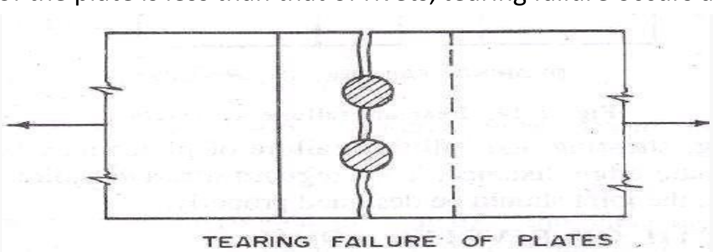
OF A RIVETED JOINT

Failure of a riveted joint may take place in any of the following ways

1. Shear failure of rivets
2. Bearing failure of rivets
3. Tearing failure of plates
4. Shear failure of plates
5. Bearing failure of plates
6. Splitting/cracking failure of plates at the edges

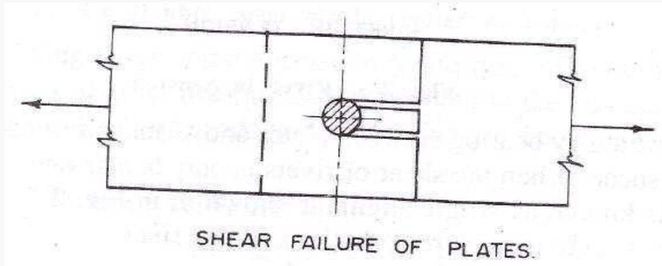
Tearing failure of plates :

When plates riveted together are carrying tensile load, tearing failure of plate may occur. When strength of the plate is less than that of rivets, tearing failure occurs at the net sectional area of plate.



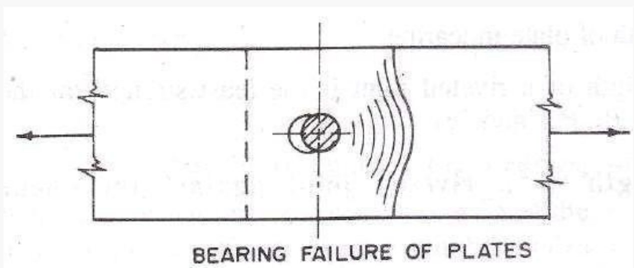
Shear failure of plates:

A plate may fail in shear along two lines as shown in Fig. This may occur when minimum proper edge distance is not provided.



Bearing failure of plates:

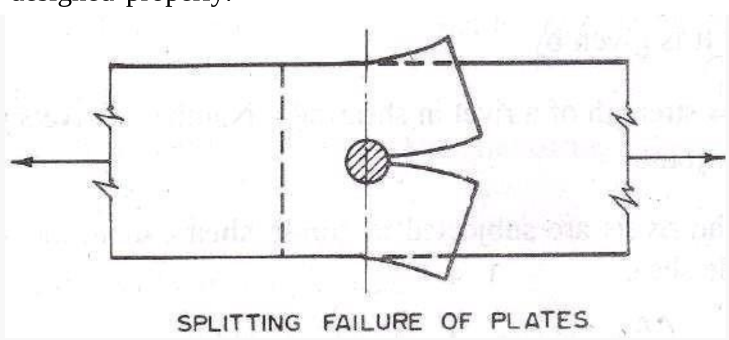
Bearing failure of a plate may occur because of insufficient edge distance in the riveted joint. Crushing of plate against the bearing of rivet take place in such failure.



Splitting/cracking failure of plates at the edges:

This failure occurs because of insufficient edge distance in the riveted joint. Splitting (cracking) of plate as shown in Fig. takes place in such failure.

Shearing, bearing and splitting failure of plates may be avoided by providing adequate proper edge distance. To safeguard a riveted joint against other modes of failure, the joint should be designed properly.



STRENGTH OF RIVETED JOINT

The strength of a riveted joint is determined by computing the following strengths:

1. Strength of a riveted joint against shearing - P_s
2. Strength of a riveted joint against bearing - P_b
3. Strength of plate in tearing -P_t

The strength of a riveted joint is the least strength of the above three strength.

Strength of a riveted joint against shearing of the rivets:

The strength of a riveted joint against the shearing of rivets is equal to the product of strengthof one rivet in shear and the number of rivets on each side of the joint. It is given by

P_s = strength of a rivet in shearing x number of rivets on each side of joint

When the rivets are subjected to single shear, then the strength of one rivet in single shear $= \frac{\pi}{4} D^2 p_s$

Therefore, the strength of a riveted joint against shearing of rivets = $P_s = N \frac{\pi}{4} D^2 p_s$

Where N=Number of rivets on each side of the joint; D=Gross diameter of the rivet; p_s=Maximum permissible shear stress in the rivet(1025 ksc).

When the rivets are subjected to double shear, then the strength of one rivet in double shea = $2 \frac{\pi}{4} D^2 p_s$

Therefore, the strength of a riveted joint against double shearing of rivets,

$$P_s = N[2 \frac{\pi}{4} D^2 p_s]$$

For single shear of rivets, $P_{s1} = n[\frac{\pi}{4} D^2 p_s]$

For double shear of rivets $P_{s2} = n[2 \frac{\pi}{4} D^2 p_s]$

5.8.2 **Strength of riveted joint against the bearing of the rivets:**

The strength of a riveted joint against the bearing of the rivets is equal to the product of strength of one rivet in bearing and the number of rivets on each side of the joint. It is given by,

P_b=Strength of a rivet in bearing x Number of rivets on each side of the

joint In case of lap joint,

the strength of one rivet in bearing = D x t x p_b

Where D= Gross diameter of the rivet; t=thickness of the thinnest plate; p_b= maximum permissible stress in the bearing for the rivet (2360 ksc). In case of butt joint, the total thickness of both cover plates or thickness of main plate whichever is less is considered for determining the strength of a rivet in the bearing.

The strength of a riveted joint against the bearing of

rivets $P_b = N \times D \times t \times p_b$

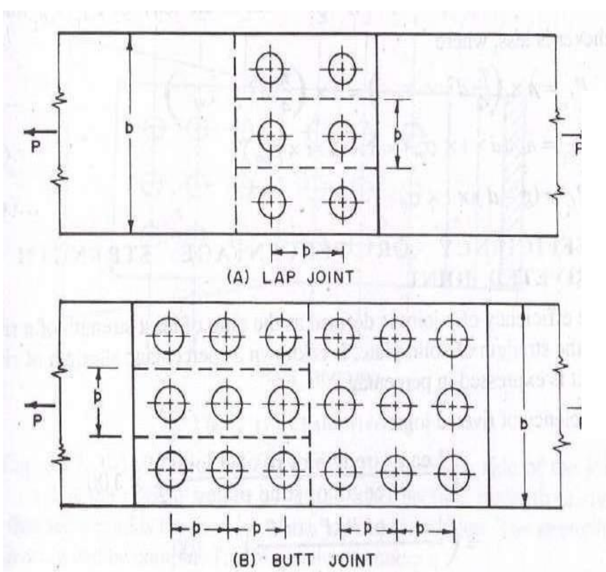
When the strength of riveted joint against the bearing of rivets per gauge widh of the plate is taken into consideration, then, the number of rivets ‘n’ is also adopted per gauge. Therefore,

$P_{b1} = n \times D \times t \times p_b$

Strength of plate in tearing

The strength of plate in tearing depends upon the resisting section of the plate. The strength of plate in tearing is given by

Pt = Resisting section x p_t



1. Strength of riveted joint against shearing $P_s = 6\frac{\pi}{4}D^2 p_s$
2. Strength of riveted joint against bearing $P_b = 6 \times D \times t \times p_b$
3. Strength of riveted joint against tearing $P_t = (b-3D) \times t \times p_t$
4. Strength of riveted joint against shearing per gauge width $P_{s1} = 2\frac{\pi}{4}D^2 p_s$
5. Strength of riveted joint against bearing per gauge width $P_{b1} = 2 \times D \times t \times p_b$ e r ehW
6. Strength of riveted joint against tearing per gauge width $P_{t1} = (p-D) \times t \times p_t$

p_t is the maximum permissible stress in the tearing of plate (1500 ksc). When the strength of plate in tearing per pitch width of the plate is

$P_{t1} = (p-D) \times t \times p_t$

The strength of a riveted joint is the least of P_s, P_b, P_t. The strength of riveted joint per gauge width of plate is the least of P_{s1}, P_{b1}, P_{t1}.

STRENGTH OF LAP AND BUTT JOINT

Strength of butt joint

1. Strength of riveted joint against shearing $P_s = 9 \times 2 \times \frac{\pi}{4} \times D^2 \times p_s$
2. Strength of riveted joint against bearing $P_b = 9 \times D \times t \times p_b$
3. Strength of riveted joint against tearing $P_t = (b-3D) \times t \times p_t$
4. Strength of riveted joint against shearing per gauge width $P_{s1} = 3 \times 2 \times \frac{\pi}{4} \times D^2 \times p_s$
5. Strength of riveted joint against bearing per gauge width $P_{b1} = 3 \times D \times t \times p_b$
6. Strength of riveted joint against tearing per gauge width $P_{t1} = (p-D) \times t \times p_t$

EFFICIENCY OR PERCENTAGE OF STRENGTH OF RIVETED JOINT

The efficiency of a joint is defined as the ratio of least strength of a riveted joint to the strength of solid plate. It is known as percentage strength of riveted joint as it is expressed in percentage.

Efficiency of riveted joint

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

$$\eta = \frac{\text{Least of } P_s, P_b \text{ or } P_t}{P} \times 100$$

Where P is the strength of solid plate = b x t x p_t Efficiency per pitch width

$$= \frac{(p-D) \times t \times p_t}{p \times t \times p_t} \times 100$$

$$= \frac{(p-D)}{p} \times 100$$

RIVET VALUE

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R).

CHAPTER 1&2

QUESTIONS….

Q. What do you mean by action in the limit state method of design ?

Ans: The primary cause for state of deformations in a structure such as dead, live, wind, seismic and temperature loads is commonly known as action in the limit state method of design.

Q. List two important advantages of welding over bolting ?

Ans: Welding is more adaptable than bolting because even circular sections can be easily connected by welding

Welded joints are more rigid and alternations in the connections can be easily made in the design.

Q. How are the structural members graded ?

Ans: 3 types

(1) Steel structures

(2) R.C.C. structure

(3) Timber structure

Q. State any two physical properties of structural steel ?

Ans-(a) Unit mass of steel , … = 7850 $kglm^3$

b) Modulus of elasticity, E= 2.0×10^6 N/mm^2

c) Poisson's ratio, ll=0.3

Q. Explain the special considerations that are to be taken care of in excel design ?

Ans:(i) Minimum thickness : The minimum thickness of the structural steel members are to be specified in view of corrosion, other use a very small amount of conversion may reavelt into reduction of large percentage of effective area.

(ii) Shape and size :- Steel is manufactured in rolling mills and are available in standardshapes and sizes.

(iii) Connection design:- During fabrication and assembling various standard sections in a member and the members them selves in a structure are to be suitably connected bywelling , bolting, riveting and pins.

(iv) Buckling:- As the strength to mass ratio of steel is very high, the permissible load perunit area is much higher steel compared to timber or concrete.

Q. Write down the advantages and disadvantages of steet structures?

* It has high strength per unit mass.
* It has assured quality and high durability.
* Speed of Construction is another important advantage of steel structure.
* Steel structures can be strengthened at any later time, if necessary. It needs just welding additional sections.
* By using bolted corrections, steel structures can be easily dismantled and transported to other sites quickly.
* Meterial is reusable

Disadvantage:-

1. It is susceptible to corrosion.
2. Maintenance cost is high.

DESIGN OF TENSION MEMBER (CH-3)

INTRODUCTION

- structural elements that are subjected to direct axial tensile loads, which tend to elongate the members.

- The strength of these members is influenced by several factors such as the length of connection, size and spacing of fasteners, net area of cross section, type of fabrication, connection eccentricity,and shear lag at the end connection.

- The stress concentration near the holes leads to the yielding of the nearby fibres but the ductility of the steel permits redistribution of over stress in adjoining section till the fibres away from the holes progressively reach yield stress. Therefore at ultimate load it isreasonable to assume uniform stress distribution.

Types of tension members

1. Wires and cables: wires ropes are exclusively used for hoisting purposes and as guy wires in steel stacks and towers. Strands and ropes are formed by helical winding of wires . A strand consists of individual wires wound helically around the central core. These are not recommended in bracing system as they cannot resist compression. The advantages of wire and cable are flexibility and strength.

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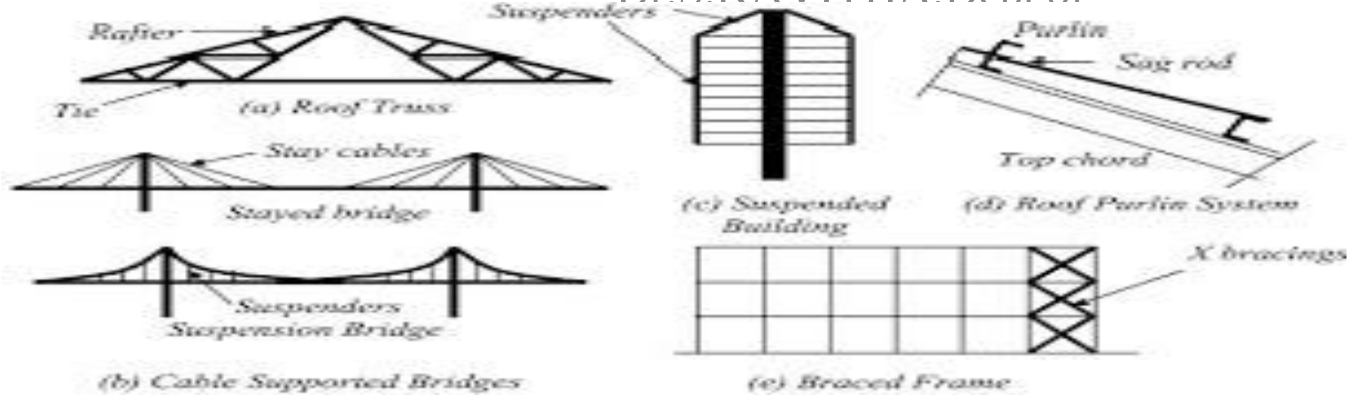


2. Bars and rods: These are simplest forms of tension members.

Bars and rods are often used as tension members in bracing system, as sag rods to support purlins between trusses. Presently these are not favourite of with the designers because large drift they cause during strong winds and disturbing noise induces by thevibrations.

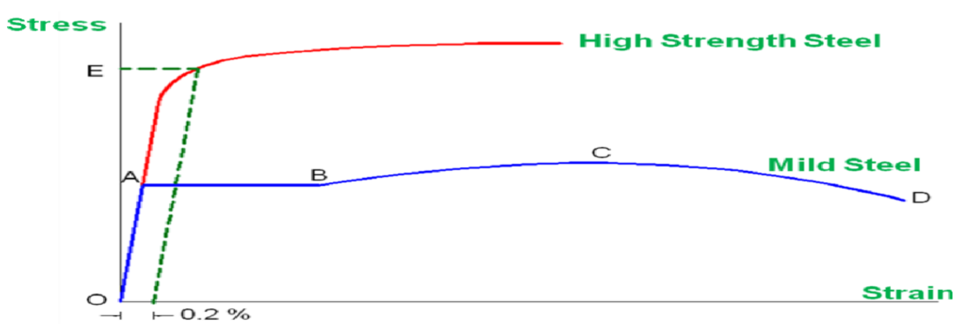
3. Plates and flat bars: These are used often as tension members in transmission towers, foot bridges, etc. These are also used in columns to keep the component members in their correct position.Eg- lacing flats, batten plates, end tie plates etc. Single and built-upstructural shapes: 1. Open sections such as angles, channels and I sections.

- Compound and built-up sections such as double angles and doublechannels with are without additional plates and jointed with some connection system.

- Closed sections such as circular, square, rectangular or hollow.

3. Behaviour of Tension Members

The load-deformation behavior of members subjected to uniform tensile stress is similar to the load-deflection behavior of the corresponding basic material. The typical stress-strain behavior of mild steel under axial tensile load is shown in Fig. 1. The upper yield point is merged with the lower yield point for convenience. The material shows a linear elastic behavior in the initial region (O to A). The material undergoes sufficient yielding in portion A to B. Further deformation leads to an increase in resistance, where the material strain hardens (from B to C). The material reaches its ultimate stress at point C. The stress decreases with increase in further deformation and breaks at D. The high strength steel members do not exhibit the well definedyield point and the yield region (Fig. 1). For such materials, the **0.2 percent proof stress is usually taken as the yield stress (E).**



4.0 Slenderness Ratio

Apart from strength requirement, the tension members have to be checked for minimum stiffness by stipulating the limiting maximum slenderness ratio of the member. This is required to prevent undesirable lateral movement or excessive vibration. The slenderness limits specified in IS: 800-2007 for tension members are given in Table 1.

5. Shear Lag

The tensile force to a tension member is transferred by a gusset plate or by the adjacent member connected to one of the legs either by bolting or welding. This force which is transferred to one leg by the end connection locally gets transferred as tensile stress over the entire cross section by shear. Hence, the distribution of tensile stress on the

section from the first bolt hole to the last bolt hole will not be uniform. Hence, the connected leg will have higher stresses at failure while the stresses in the outstanding leg will be relatively lower. However, at sections far away from the end connection, the stress distribution becomes more uniform. Here the stress transfer mechanism, i.e., the internal transfer of forces from one leg to the other (or flange to web, or from one part to the other), will be by shear and because one part 'lags' behind the other, the phenomenon is referred to as '*shear lag*'.

The shear lag reduces the effectiveness of the component plates of a tension member that are not connected directly to a gusset plate. The efficiency of a tension member can be increased by reducing the area of such components which are not directly connected at the ends. The shear lag effect reduces with increase in the connection length.

1. Modes of Failure

The different modes of failure in tension members are

1. Gross section yielding
2. Net section rupture
3. Block shear failure

The strength of tension members under the different modes are failure, i.e., design strength due to yielding of gross section, T_{dg}, rupture of critical section, T_{dn} and block shear T_{db} are first determined. The design strength of a member under axial tension, T_d, is the lowest of the above three values.

6. Gross section yielding

Steel members (plates, angles, etc.) without bolt holes can sustain loads up to the ultimate load without failure. However, the members will elongate considerably (10 to 15 % of its original length) at this load, and hence make the structure unserviceable. Hence the design strength T_{dg} is limited to the yielding of gross cross section which is given by

$$T_{dg} = f_y \, A_g/_{m0}$$

where

f_y = yield strength of the material in MPa

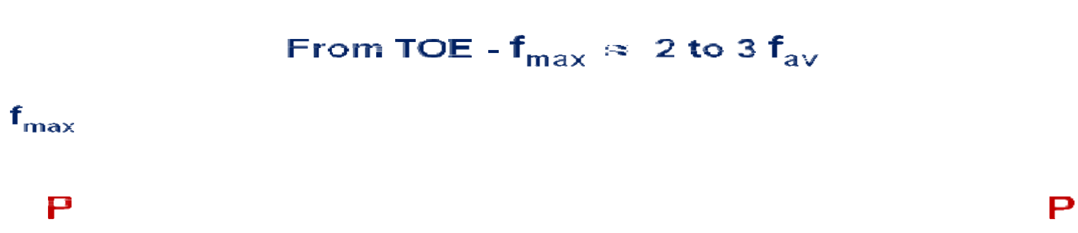
A_g = gross area of cross section in mm^2

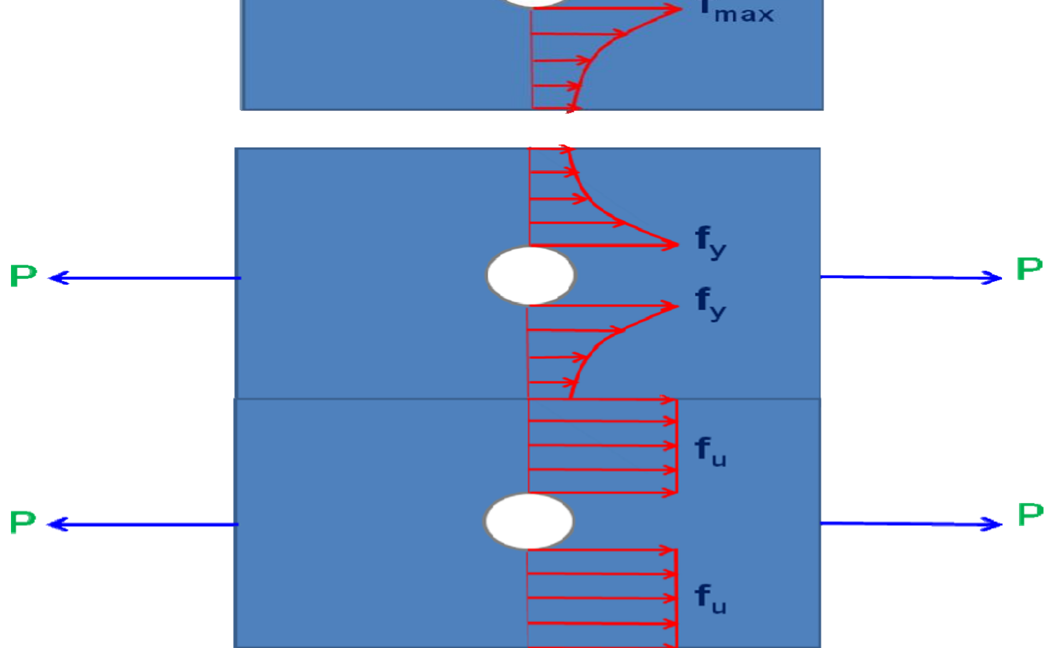
$_{m0}$ = 1.10 = partial safety factor for failure at yielding

6.2 Net section rupture

This occurs when the tension member is connected to the main or other members by bolts. The holes made in members for bolts will reduce the cross section, and hence net area will govern the failure in this case. Holes in members cause stress concentration at service loads. From the theory of elasticity, the tensile stress adjacent to a hole will be about two to three times the average stress on the net area (Fig. 2a). This depends on the ratio of the

diameter of the hole to the width of the plate normal to the direction of the stress.





(a)

(b)

(c)

Fig. 2 Stress-distribution in a plate adjacent to hole due to tensile force.

When the tension member with a hole is loaded statically, the

$$A_n = \left[b - n\, d_h + \sum_i \frac{p_{si}^2}{4 g_i} \right] t$$

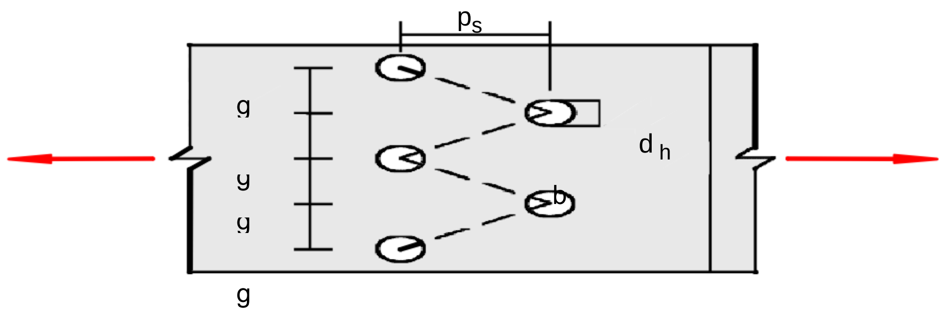


Fig. 3 Plate with bolt holes in tension

where

b, t = width and thickness of the plate, respectively

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case of di punched holes)

A_n = net effective area of the member in mm^2 is given by

g = gauge length between the bolt holes, as shown in Fig. 3

p_s = staggered pitch

length between line of bolt

holes, as shown in Fig. 3n

= number of bolt

holes in the critical section,

and

i = subscript for summation of all the inclined legs

The '0.9' factor included in the design strength equation is based on a statistical evaluation of a large number of test results for net section failure of members.

2. Net section rupture in threaded rods

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

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

where A_n = net root area at the threaded section

3. Net section rupture in single angles

The rupture strength of an angle connected through one leg is affected by she

$$T_{dn} = 0.9 f_u A_{nc} / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

where

$$\beta = 1.4 - 0.076 (w/t) (f_y / f_u) (b_s / L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$$

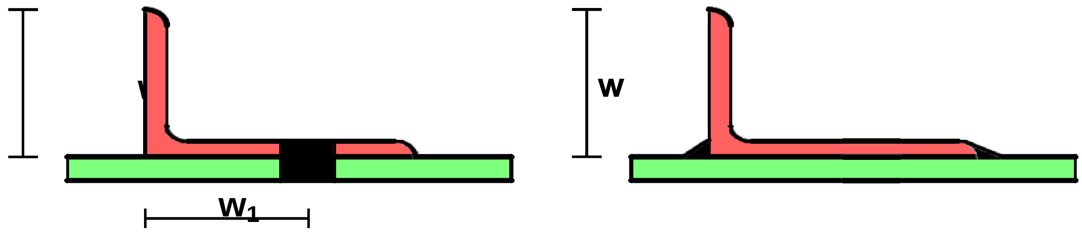
$$\geq 0.7$$

where

w = outstand leg width

b_s = shear lag width, as shown in Fig. 4

L_c = Length of the end connection, i.e., distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction



$b_s = w + w_1 - t$ $b_s = w$

Fig. 4 Angles with single leg connections

For preliminary sizing, the rupture strength of net section may be appro

$T_{dn} = □A_n f_u /_{m1}$

□ = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the

here

length in the end connection or equivalent weld length

A_n = net area of the total cross section

A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg, and

t = thickness of the leg

4 Net section rupture in other sections

The tearing strength, T_{dn}, of the double angles, channels, I sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in 6.2.3, where □is calculated based on the shear lag distance, b_s taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross section.

6.3 Block shear failure

Block shear failure is considered as a potential failure mode at the ends of an axially loaded tension member. In this failure mode, the failure of the member occurs along a path involving tension on one plane and shear on a perpendicular plane along the fasteners. A typical block shear failure of a gusset plate is shown in Fig. 5. Here plane B-C is under tension whereas planes A-B and C-D are in shear.

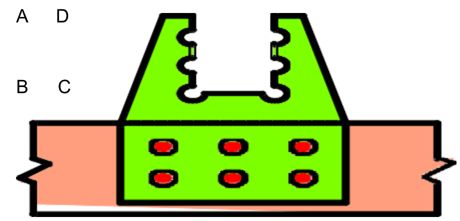


Fig. 5 Block shear failure in gusset plate

Typical block shear failure of angles in a bolted connection is shown in Fig. 6. Here plane 1-2 is in shearand plane 2-3 is in tension.

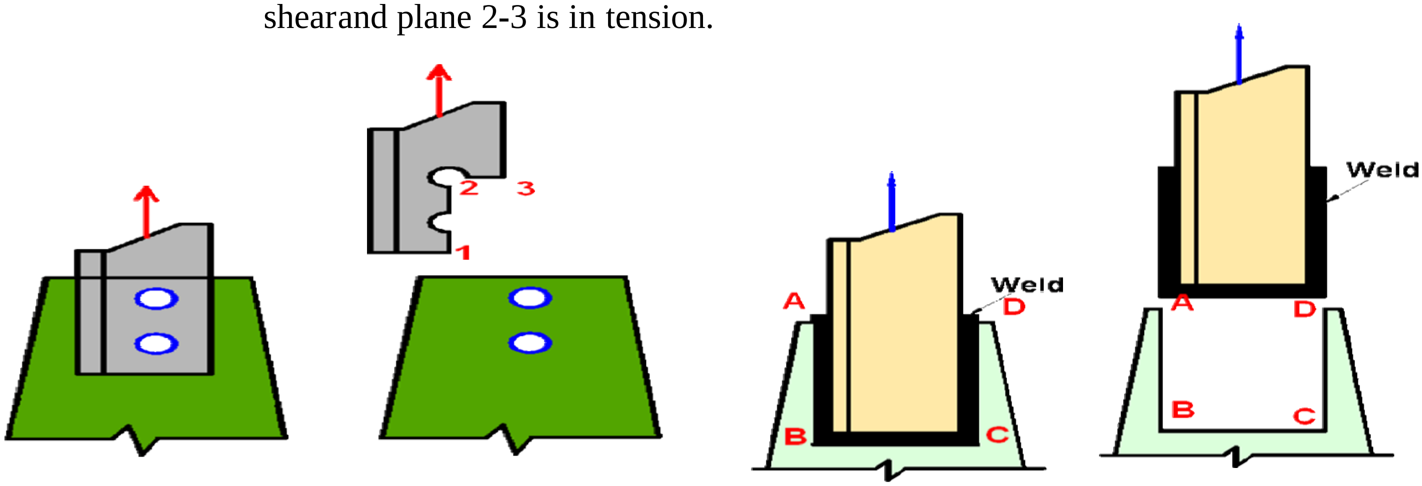


Fig. 6 Block shear failure in angle with boltedconnection

Fig. 7 Block shear failure of gusset plate in weldedconnections

The block shear failure is also seen in welded connections. A typical failure of a gusset in the welded connection is shown in Fig. 7. The planes of failure are chosen around the weld. Here plane B-C is under tension and planes A-B and C- D are in shear.

6.3.1 Design strength due to block shear in bolted connections

The block shear strength, T_{db}, of connection shall be taken as the smaller of

$$T_{db} = \left(A_{vg} f_y/(\sqrt{3}\,\gamma_{m0}) + f_u A_{tn}/\gamma_{m1} \right) \text{ or}$$

$$T_{db} = \left(f_u A_{vn}/(\sqrt{3}\,\gamma_{m1}) + f_y A_{tg}/\gamma_{m0} \right)$$

here

A_{vg}, A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively (1-2 and and 1-2 as shown in Fig. 9)

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt li the line of force (2-3 as shown in Figs. 8 and 9)

f_u, f_y = ultimate and yield stress of the material respectively

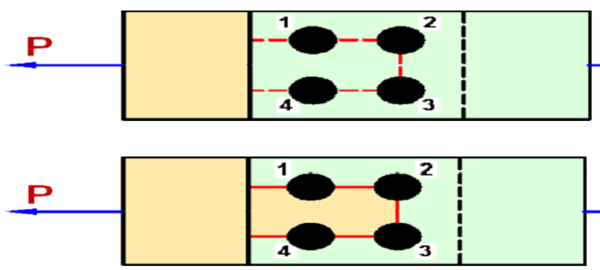


Fig. 8 Block shear failure in plate failure in angle

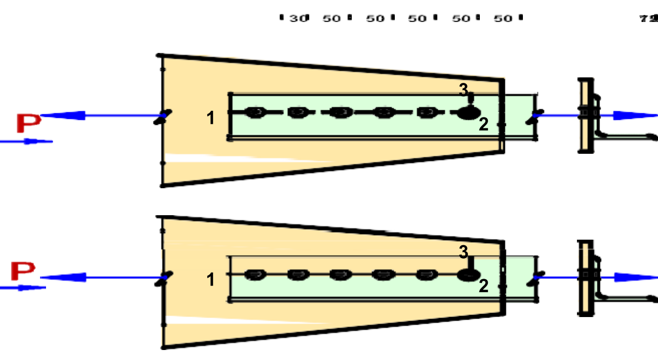
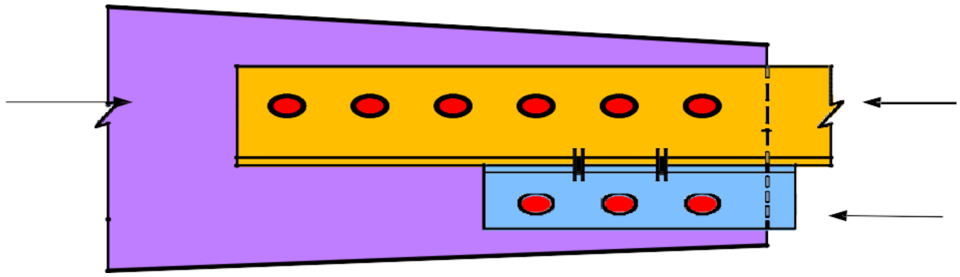


Fig. 9 Block shear

6.3.1 Design strength due to block shear in welded connections

The block shear strength, T_{db}, shall be checked for welded connections by takin

Lug angles are short angles used to connect the gusset and the outstanding leg of the main memberas shown in Fig. 10. The lug angles help to increase the efficiency of the outstanding leg of angles or channels. They are normally provided when the tension member carries a very large load. Higher load results in a larger end connection which can be reduced by providing lug angles. It is ideal to place the lug angle at the beginning of the connection than at any other position.



Numercial Problems

Problem 1

Determine the design tensile strength of the plate 120 mm x 8 mm connected to a 12 mm thick gusset plate with bolt holes as shown in Fig. 11. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa. The diameter of the bolts used is 16 mm.

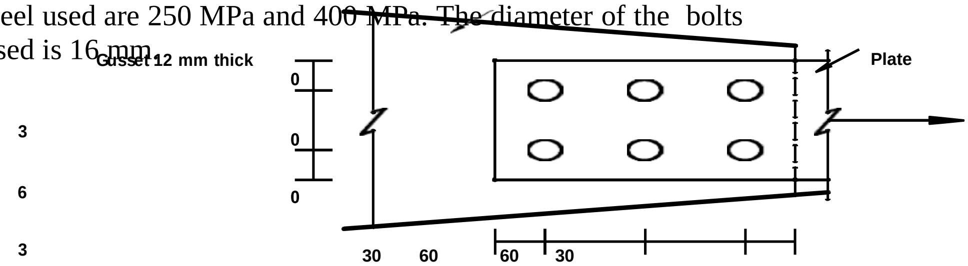


Fig. 11 Details of end connection

Solution

The design tensile strength T_d of the plate is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of plate limited to the yielding of gross cross section A_g is given by

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

Here $f_y = 250$ MPa, $A_g = 120$ x $8 = 960$ mm^2 and $\gamma_{m0} = 1.10$

Hence $T_{dg} = 218.18$ kN

(ii) Net section rupture

The design strength T_{dn} of angle governed by rupture of net cross sectional area, A_n, is given by

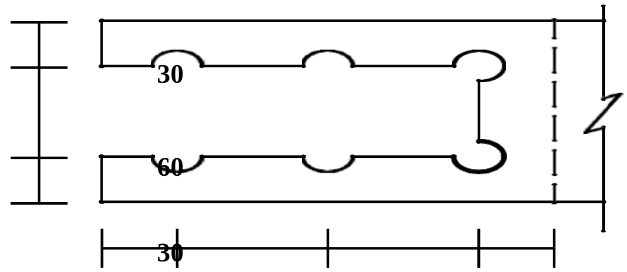
$$T_{dn} = 0.9\, f_u\, A_n / \gamma_{m1}$$

Here f_u = 400 MPa, γ_{m1} = 1.25

Further, diameter of bolt hole = 16 + 2 = 18 mm

Therefore, A_n = (120 – 2 x18) 8 = 672 mm^2. Hence, T_{dn} = 193.54 kN

(iii) Block shear failure



30 60 60 30

Fig. 12 Failure of plate in block shear

The design strength T_{dg} of connection shall be taken as smaller of

$T_{db1} = (A_{vg} f_y /(\sqrt{3}\, \gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1})$, OR

$T_{db2} = (0.9 A_{vn} f_u /(\sqrt{3}\, \gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$ Here,

A_{vg} = (150 x 8) 2 =2

A_{vn} = [(150 – 2.5 x 18) x 8] 2 = 1680 mm^2,

A_{tg} = (60 x 8) = 480 mm^2,

A_{tn} = (60 – 1.0 x 18) x 8 = 336 mm^2

Therefore, T_{db1} = 411.69 kN and T_{db2} = 388.44 kN Hence T_{db} = 388.44 kN

Design tensile strength T_d

The tensile design strength T_d is the least of T_{dg}, T_{dn} and T_{db}

Hence, $T_d = T_{dn}$ = 193.54 kN

Problem 2

A single unequal angle 100 x 75 x 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 numbers of 20 mm diameter bolts to transfer tension as shown in Fig. 13. Determine the design tensile strength of the angle if the gusset is connected to the 100 mm leg. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa. The diameter of the bolts used is 20 mm.

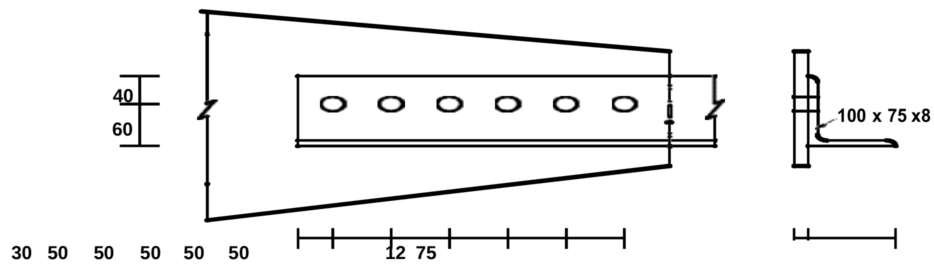


Fig. 13 Details of end connection

Solution

The design tensile strength T_d of the angle is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of angle limited to the yielding of gross cross section A_g is given by

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

Here $f_y = 250$ MPa, $A_g = (100 + 75 - 8)\,8 = 1336\ \text{mm}^2$, $\gamma_{m0} = 1.10$

Hence $T_{dg} = 303.64$ kN

(ii) Net section rupture

The design strength T_{dn} of angle governed by rupture of net cross sectional area is given by

$$T_{dn} = 0.9\, f_u A_{nc} / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u \gamma_{m0} / f_y \gamma_{m1})$$

Here $f_u = 400$ MPa, $f_y = 250$ MPa, $\gamma_{m1} = 1.25$, and $\gamma_{m0} = 1.10$

$w = 75$ mm, $t = 8$ mm, $b_s = (75 + 60 - 8) = 127$ mm, $L_c = 250$ mm Further, diameter of bolt hole = 20 + 2 = 22 mm.

A_{nc} = (100 – 8/2 – 22) 8 = 592 mm^2, A_{go}= (75 – 8/2) 8 = 568 mm^2

Hence,

? = 1.17.

Since 0.7

□

□

□

1.41,

□ = 1.17

Hence

,

T

d

n

=

3

2

1

.

5

3

k

N

(iii) Block shear failure

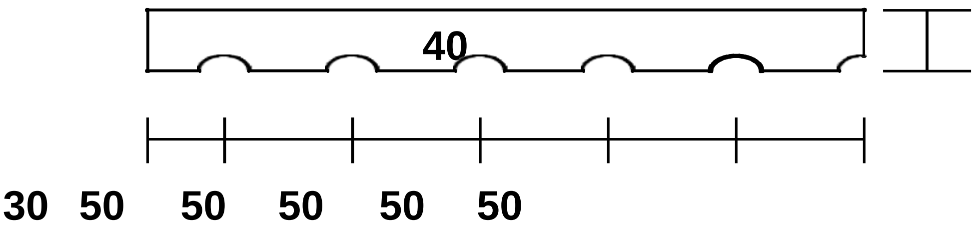


Fig. 14 Failure of plate in block shear

The design strength T_{dg} of connection shall be taken as smaller of

$T_{db1} = (A_{vg} f_y /(\sqrt{3}\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1})$, OR

$T_{db2} = (0.9 A_{vn} f_u /(\sqrt{3}\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$ Here, A_{vg}

$= 280 \times 8 = 2240\ mm^2$,

A_{vn} = (280 – 5.5 x 22) x 8 = 1272 mm^2,

A_{tg} = 40 x 8 = 320 mm^2,

A_{tn} = (40 – 0.5 x 22) 8 = 232 mm^2

Therefore, T_{db1} = 360.74 kN and T_{db2} = 284.23 kN Hence T_{db} = 284.23 kN

Design tensile strength T_d

The tensile design strength T_d is the least of T_{dg}, T_{dn} and T_{db}

Hence, T_d = T_{db} = 284.23 KN

QUESTIONS.....

1(A)DEFINE A TENSION MEMBER ?
(B)TYPES OF TENSION MEMBER ?
(C) WHAT IS TIE IN TENSION MEMBER ?
(D) WHAT IS LUG ANGLE ?
(E)WHAT IS MEANT BY SINGLE SECTION MEMBER ?
(F)WHAT ARE THE OBJECTIVES OF YHE LUG ANGELS ?

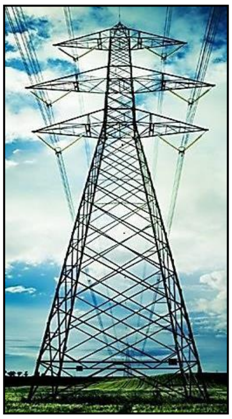
2(A) HOW THE TENSION MEMBERS ARE CLASSIFIED ?
(b)WHAT ARE THE OBJECTIVES OF TENSION MEMBER ?
(C)MENTION THE DESIGN PROCEDURE OF TENSION MEMBER ?

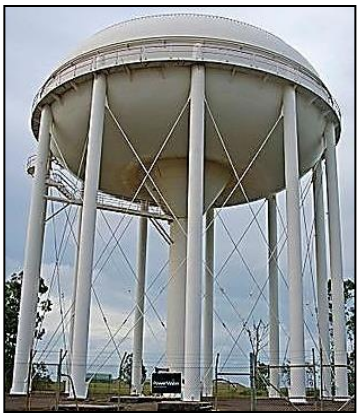
COMPRESSION MEMBER (CH-4)

INTRODUCTION :

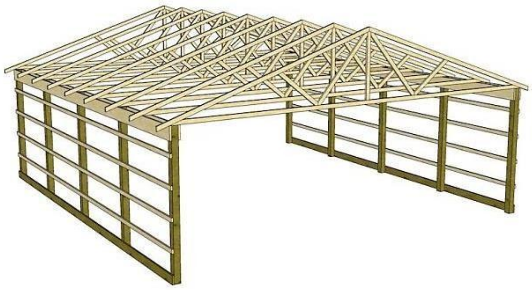
There are several types of compression members, the column being the best known. Among theother types are the top chords of trusses and various bracing members. In addition, many othermembers have compression in some of their parts. These include the compression flanges ofrolled beams and built-up beam sections, and members that are subjected simultaneously tobending and compressive loads. Columns are usually thought of as being straight verticalmembers whose lengths are considerably greater than their thicknesses. Compression member: isa structural member which carries pure axial compression loads like compression members in: Generally the used in:

1- Column Supports & Towers



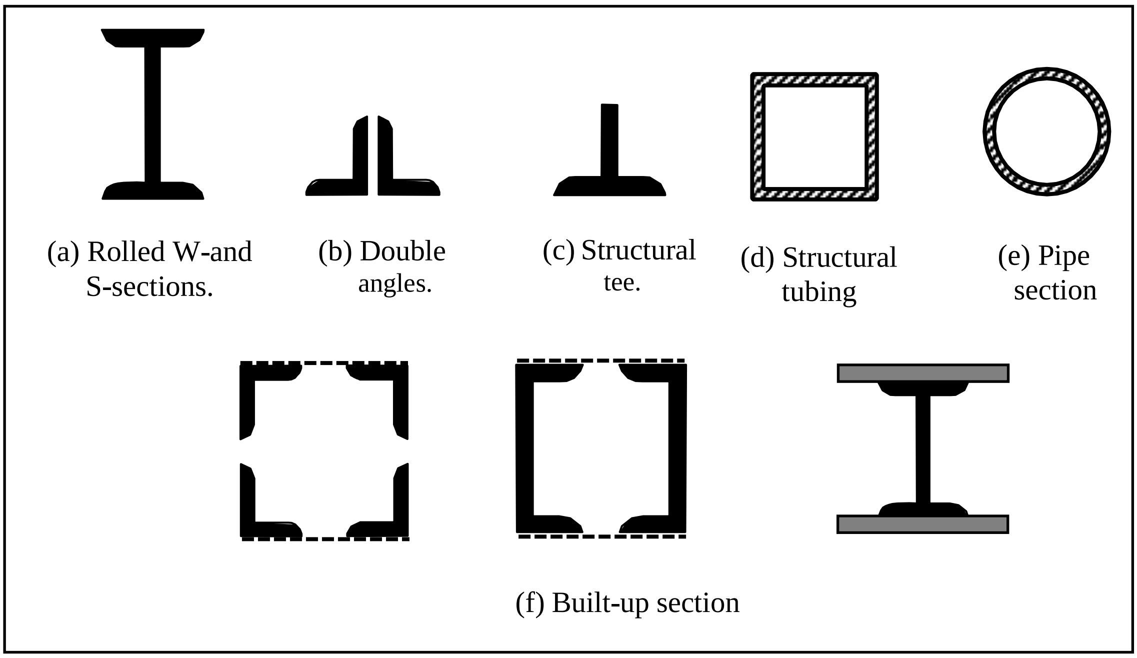


2-Trusses & Bridges

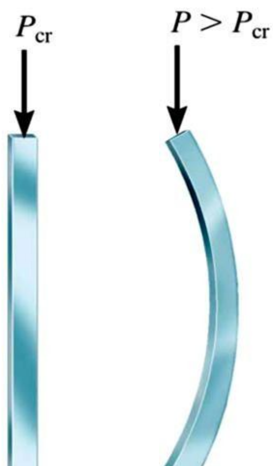



3-Columns in building frames

Steel shapes, which are used as compression members, are shown in the figure below.



The stress in the column cross-section is given by:



$$f = P/A$$

Where,
f is compressive stress which is assumed to be uniform over the entire cross- section,
P is the magnitude of load,
A is the cross-sectional area normal to the load.
If the applied load increased slowly, it will ultimately reach a value $\mathbf{P_{cr}}$ that will cause buckling of the column, $\mathbf{P_{cr}}$ is called the critical buckling load of the column, i.e. **P** >

P_{cr} lead to buckling.

4.2 Elastic Flexural Buckling of a Pin-Ended Column

The deflection at distance **z** is denoted by **u** Moment equilibrium about **A** in the buckling state gives:

M – P.u = 0.0 .... M = P.u

$$M = EI\Phi = -EI\,(d^2u/d^2z) = P.u$$

EI (d²u/d²z) + P.u = 0.0d²u/d²z + (P/EI). u = 0.0

$$d^2u/d^2z + \alpha^2 .\, u = 0.0 \quad \ldots\ldots\ \alpha^2 = P/EI$$

$$u = A \sin \alpha z + B \cos \alpha z$$

u = 0.0 @ z = 0.0................................... B=0.0

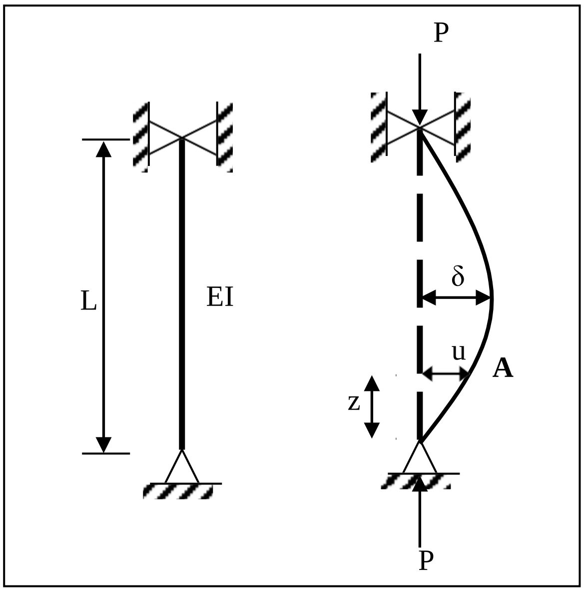
$$u = 0.0 \quad @\ z = L\ldots\ldots\ A \neq 0.0$$

then sin αL = 0.0 αL = nπ ...α = nπ/L

Where n = 1, 2, 3 ....

$$\text{Then } P = P_{crn} = (n\pi/L)^2\, EI$$

Thus, the Euler load of a pin – ended column is: $P_E = P_{cr1} = (\pi/L)^2\, EI$

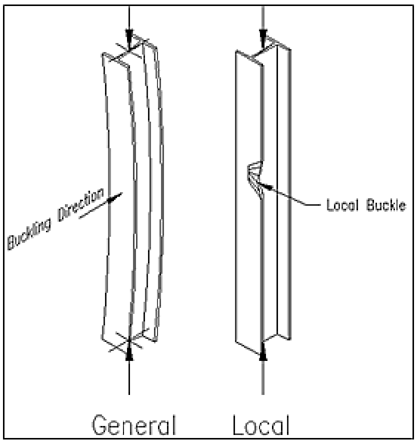


4.1 Buckling Basics

There are two main modes of buckling failure that may be experienced by
steel members: Overall (or general) buckling and local buckling. The Swiss mathematician Leonhard Euler developed an equation that predicts the critical buckling load P_{cr}, for a straight pinned end column. The equation is:

$$P_{cr} = \pi^2 EI/L^2$$

Where, I = moment of inertia about axis of buckling.This equation to be valid:



- The member must be elastic
- Its ends must be free to rotate but translate laterally

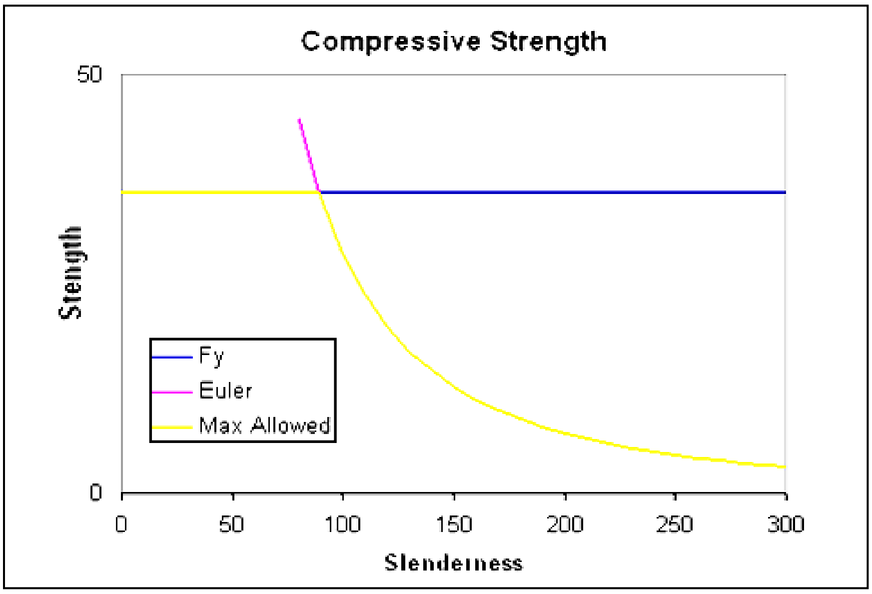
Dividing by the area of the element, we get an equation for the critical buckling stress:

$$\sigma_{cr}= Л^2E/(L/r)^2$$

Where the member cross sectional dependent term **(L/r)** is referred to as the "**slenderness**" of thememember.

$$\sigma_{max} = minimum[Л^2E/(L/r)^2, F_y]$$

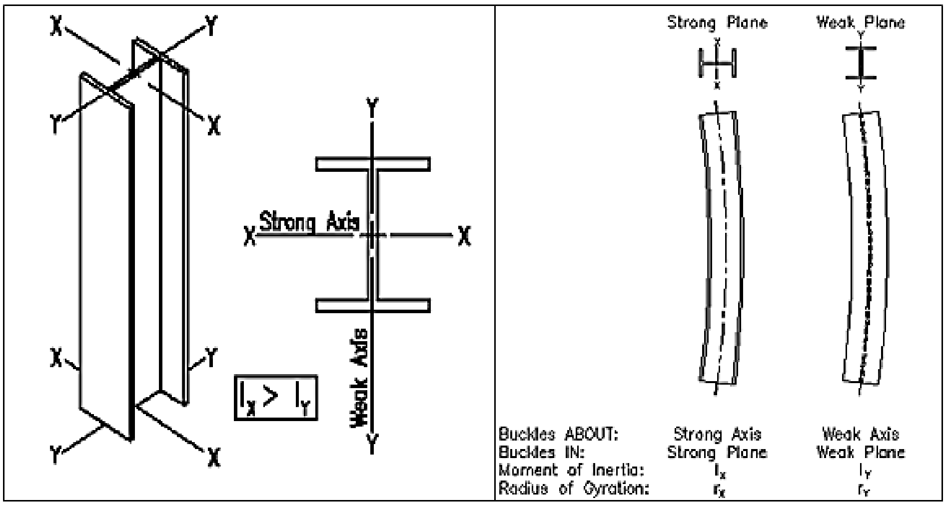
This relationship is graphed in the figure below



Theoretical Maximum Compressive Stress

4.3 General Member Buckling Concepts

The figure below illustrates the principle axes of a typical wide flange compression member. Other members shapes can be similarly drawn. With the exception of circular (pipe) sections, all the available shapes have a readily identifiable set of principle axes. Buckling is a two dimensional (planar) event. In other words it happens **IN** a **PLANE** that is perpendicular to the **AXIS**



3.4.1 Effective Length Coefficients and End Support Conditions

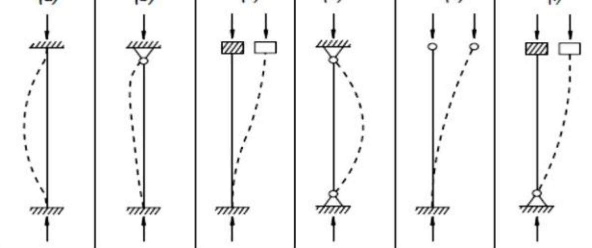
Theoretically, end supports are either pinned or fixed. In reality they can be designed to be pinned or rigid and may actually fall somewhere in between truly pinned or fixed. The support conditions will have an impact on the effective length, $\mathbf{L_e}$. Effective length, $\mathbf{L_e}$, of a compression member is the distance between where inflection points (Inflection point is a location of zero moment) are on a compression member. Effective length can be expressed as:

$$L_e = KL \quad \ldots\ldots \quad P_e = \pi^2 EI/(KL)^2$$

Where K is an effective length coefficient, **L** is the actual length of the compression member in the plane of buckling & $\mathbf{P_e}$ is elastic flexural buckling load in column. Different end conditions give different lengths for equivalent half-sine wave as shown in the figure below.

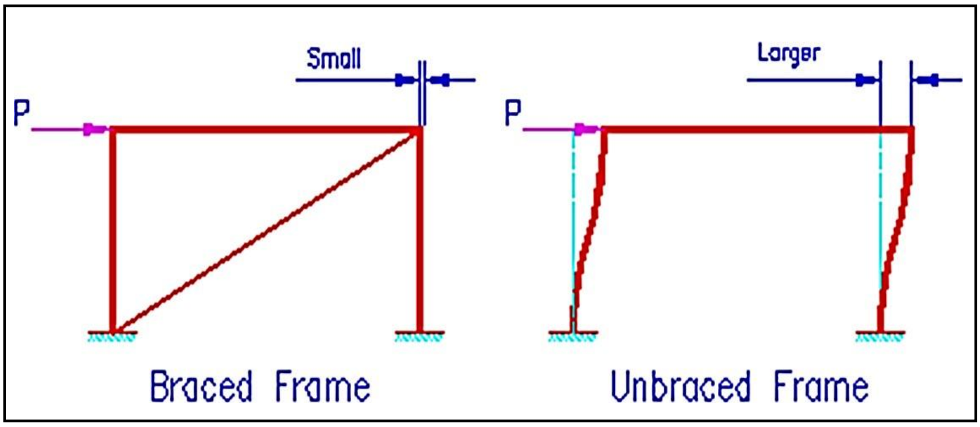
- The theoretical values of effective length coefficients assume that joints are completely fixed against rotation or totally free to rotate. Reality is usually somewhere in between. This affects the value of K.

Table C-2.
Effective Length Factors (*K*) for Columns

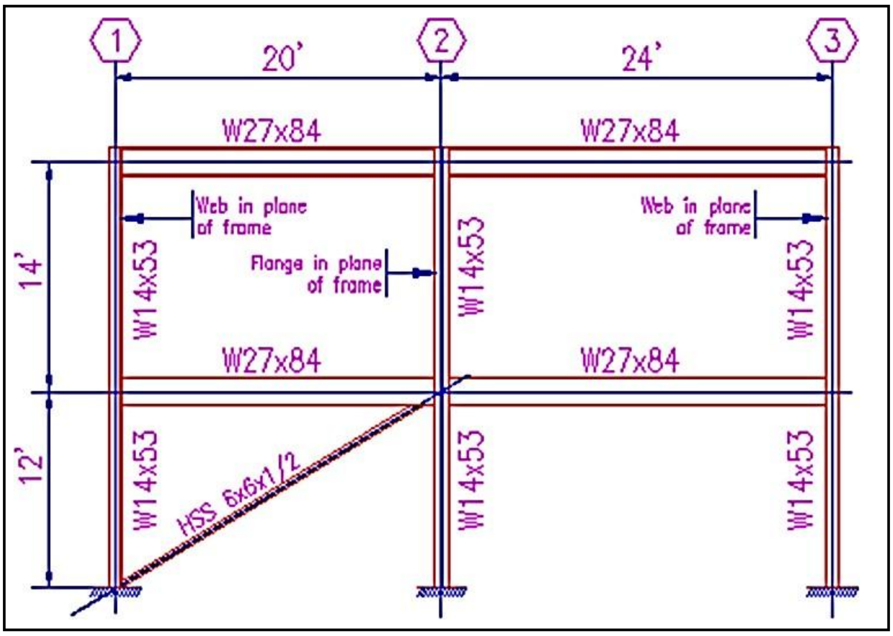
Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free				

- Table C-C2.2 is presented in LRFDM p. 240 to predicted the both theoritical and recommended design value of K of isolated column and its depended on support condition.

- In the building, the cases with no joint translation are considered to be "braced frames" since some kind of bracing between the two levels is necessary to prevent lateral movement under shearing loads. The other cases are referred to as being "unbraced frames".



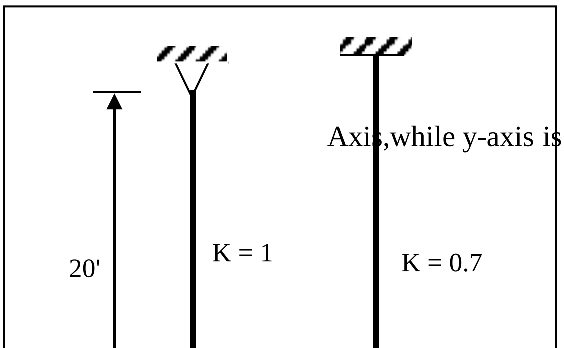
- The figure below shows that different lengths of the same column can have different effective length coefficients in the same plane of buckling. Consider everything in the plane of buckling. Upper portion of this frame is **UNBRACED**. Lower portion of this frame is **BRACED**.



Example 3-1: Determine the buckling strength of **W12*50** column. Its length **20'**, the minor (weak) axis of buckling pinned at both ends, while major (strong) axis of buckling pinned at one end and fixed at the other end. **E = 29 ksi**.

Solution:

Note: for W-section x-axis is the strong Axis,while y-axis is the weak one.



P 1-39
I_x = 394 in^4, I_y = 56.3 in^4

The buckling Euler strength

$P_e = Л^2EI/(KL^2)$

The value of K of isolated column for different end condition can be predicted from table C-2.2 in LRFDM p. 240.

$P_{e\,(x\text{-}x)}$ = $Л^2$(29000)*394/(0.8*20*12)^2
= 3035.8 kips

$P_{e\,(y\text{-}y)}$ = $Л^2$(29000)*56.3/(1*20*12)^2
= 279.8 kips

(a) A W10 × 22 is used as a 15-ft long pin-connected column. Using the Euler expression, determine the column's critical or buckling load. Assume that the steel has a proportional limit of 36 ksi.

(b) Repeat part (a) if the length is changed to 8 ft.

Solution

(a) Using a 15-ft long W10 × 22 ($A = 6.49\ in^2$, $r_x = 4.27$ in, $r_y = 1.33$ in)
Minimum $r = r_y = 1.33$ in

$$\frac{L}{r} = \frac{(12\text{ in/ft})(15\text{ ft})}{1.33\text{ in}} = 135.34$$

$$\text{Elastic or buckling stress } F_e = \frac{(\pi^2)(29 \times 10^3\text{ ksi})}{(135.34)^2}$$

$$= 15.63\text{ ksi} < \text{the proportional limit of 36 ksi}$$

OK column is in elastic range

Elastic or buckling load = (15.63 ksi)(6.49 in^2) = 101.4 k

(b) Using an 8-ft long W10 × 22,

$$\frac{L}{r} = \frac{(12\text{ in/ft})(8\text{ ft})}{1.33\text{ in}} = 72.18$$

$$\text{Elastic or buckling stress } F_e = \frac{(\pi^2)(29 \times 10^3\text{ ksi})}{(72.18)^2} = 54.94\text{ ksi} > 36\text{ ksi}$$

∴ column is in inelastic range and Euler equation is not applicable.

Example 3-2

3.4.2 Long Columns

The Euler formula predicts very well the strength of long columns where the axial buckling stress remains below the proportional limit. Such columns will buckle *elastically*.

3.4.3 Short Columns

For very short columns, the failure stress will equal the yield stress and no buckling will occur.

3.4.4 Intermediate Columns

For intermediate columns, some of the fibers will reach the yield stress and some will not. The members will fail by both yielding and buckling, and their behavior is said to be *inelastic*. Most columns fall into this range. (For the Euler formula to be applicable to such columns, it would haveto be modified according to the reduced modulus concept or the tangent modulus concept to account for the presence of residual stresses.)

3.5 Column Formulas

The AISC Specification provides one equation (the Euler equation) for long columns with elastic buckling and an empirical parabolic equation for short and intermediate columns. With these equations, a flexural buckling stress, F_{cr}, is determined for a compression member. Once this stress is computed for a particular member, it is multiplied by the cross-sectional area of the member to obtain its nominal strength P_n.

P_n is the nominal compressive strength of the member is computed by the following equation :

$$nP_n = F_{cr} A_g$$

$P_d = \emptyset_c P_n = \emptyset_c F_{cr} A_g$ = LRFD compression strength **($\emptyset_c$ = 0.9)**

- F_{cr} is the critical flexural buckling stress.
- A_g is the gross cross sectional area of the member.

The criteria for selecting which formula to use is based on either the slenderness ratio for the member or the relationship between the Euler buckling stress and the yield stress of the material. The selection can be stated as:

- If $KL/r \leq 4.71 * \sqrt{\frac{E}{F_y}}$ then $F_{cr} = [0.658^{Fy/Fe}] F_y$
- If $KL/r > 4.71 * \sqrt{\frac{E}{F_y}}$ then $F_{cr} = 0.877 F_e$

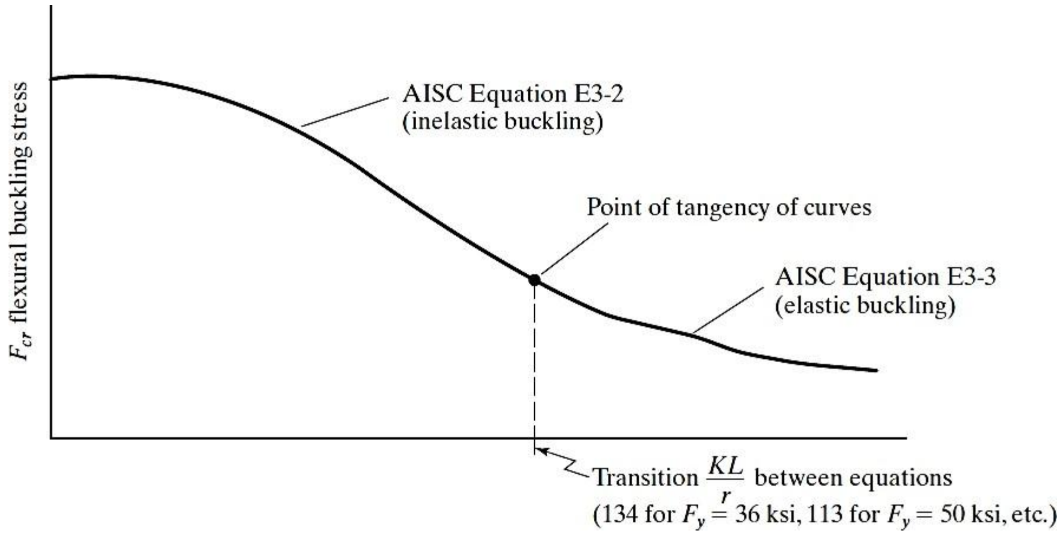
In these expressions, $\mathbf{F_e}$ is the elastic critical buckling stress—that is, the Euler stress—calculatedwith the effective length of the column **KL**.

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$\left(r \right)$

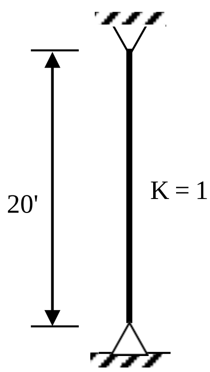
Note: The **AISC** Manual provides computed values of critical stresses **$\phi_c F_{cr}$** in their **Table 4-22 PP (4-318)**. The values are given for practical **KL/r** values (**0** to **200**) and for steels with **Fy = 36, 42, 46,** and **50 ksi.**

These equations are represented graphically in the figure below



Example 3-3:

Determine the design strength of W14*74 column. Its length 20', it's pinned at both ends. E = 29 ksi.
Solution:
Ag = 21.8 in^2, r_x = 6.04 in^4, r_y = 2.48 in^4 , f_y = 36 ksi



$$\frac{K_y L}{r_y} = \frac{1*20*12}{2.48} = 96.77 \text{ ......} (control)$$

$$\frac{K_x L}{r_x} = \frac{1*20*12}{6.04} = 39.73$$

$$\max . \frac{KL}{r} = 96.77 < 200 \text{.........} ok$$

$$\sqrt{\frac{50}{29000}}$$

$$4.71\sqrt{\frac{F_y}{E}} = 4.71 \qquad = 113$$

$$\frac{KL}{r} = 96.77 < 113 \text{.....use AISC Equation E3-2. p33}$$

$F_e = Л^2E/(KL/r)^2=$

30.56 ksi F_{cr} =

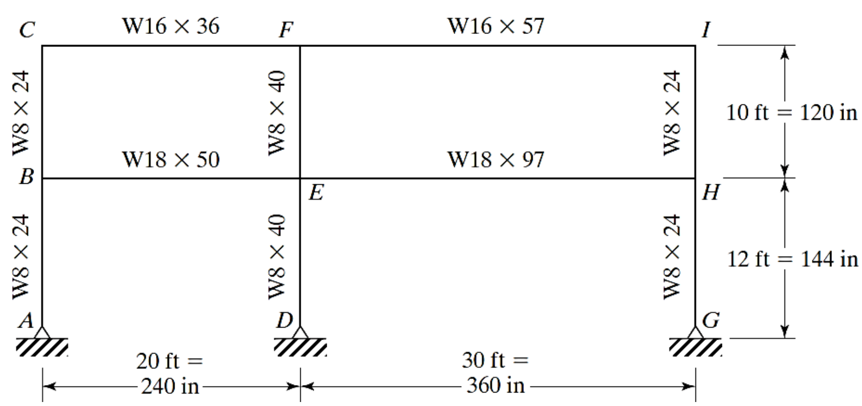
$[0.658^{Fy/Fe}]\ F_y$=

25.21 ksi

$P_d = \phi_c\ F_{cr}\ A_g = 0.9\ (25.21)(21.8)= 495$ kips

Example 3-4:

Determine the effective length factor for each of the columns of the frame shown in the figure, if the frame is not braced against sidesway.



Solution.

Stiffness factors: *E* is assumed to be 29,000 ksi for all members and is therefore neglected in the equation to calculate *G*.

	Member	Shape	I	L	I/L
Columns	AB	W8 × 24	82.7	144	0.574
	BC	W8 × 24	82.7	120	0.689
	DE	W8 × 40	146	144	1.014
	EF	W8 × 40	146	120	1.217
	GH	W8 × 24	82.7	144	0.574
	HI	W8 × 24	82.7	120	0.689
Girders	BE	W18 × 50	800	240	3.333
	CF	W16 × 36	448	240	1.867
	EH	W18 × 97	1750	360	4.861
	FI	W16 × 57	758	360	2.106

Column	G_A	G_B	K
AB	10.0	0.379	1.76
BC	0.379	0.369	1.12
DE	10.0	0.272	1.74
EF	0.272	0.306	1.10
GH	10.0	0.260	1.73
HI	0.260	0.327	1.10

Joint	$\Sigma(I_c/L_c)/\Sigma(I_g/L_g)$	G
A	Pinned Column, $G = 10$	10.0
B	$\dfrac{0.574 + 0.689}{3.333}$	0.379
C	$\dfrac{0.689}{1.867}$	0.369
D	Pinned Column, $G = 10$	10.0
E	$\dfrac{1.014 + 1.217}{(3.333 + 4.861)}$	0.272
F	$\dfrac{1.217}{(1.867 + 2.106)}$	0.306
G	Pinned Column, $G = 10$	10.0
H	$\dfrac{0.574 + 0.689}{4.861}$	0.260
I	$\dfrac{0.689}{2.106}$	0.327

QUESTIONS….

1(A) WHAT IS COLUMN ?
(B) WHAT IS STRUT ?
(C) DEFINE COMPRESSION MEMBER ?
(D)WHAT IS SLENDERNESS RATIO ?
(E) WHAT IS LONG COLUMN ?
(f) WHAT IS SHORT COLUMN ?
(G) WHAT IS INTERMIDEATE COLUMN ?
(F) WHAT ARE THE END CONDTIONS OF COLUMN ?

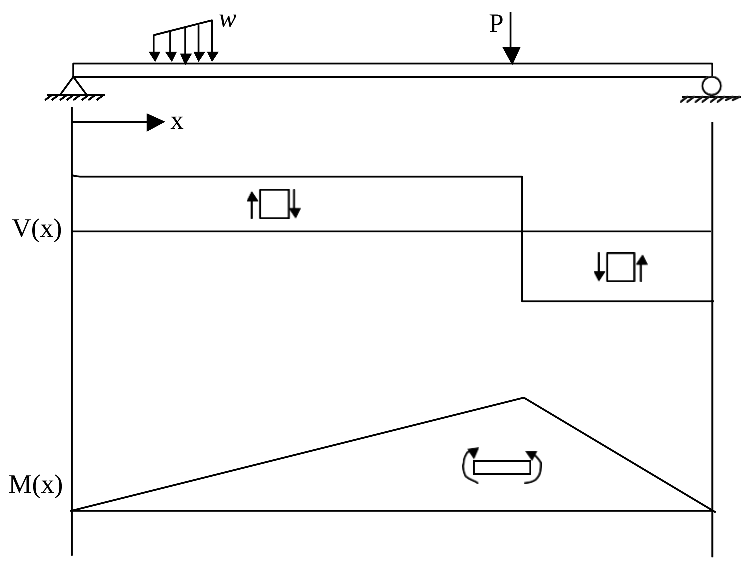
2(A) DESCRIBE THE DESIGN PROCEDURE OF COMPRESSION MEMBER ?
(B) CLASSIFICATION OF COMPRESSION MEMBER ?
(C) WRITE DOWN THE OBJECTIVES OF COLUMN ?

DESIGN OF STEEL BEAM (CH-5)

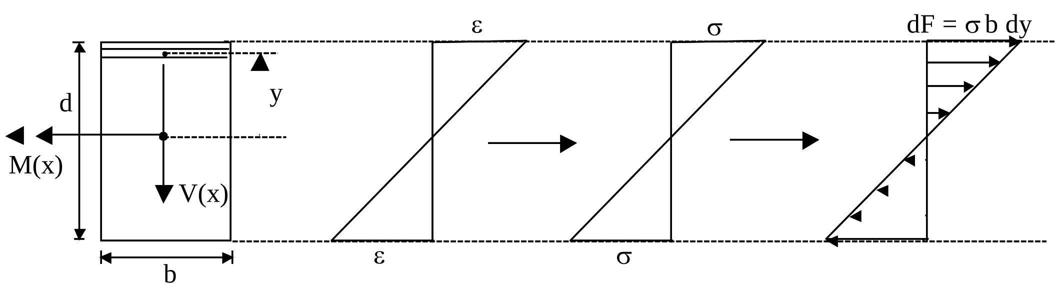
INTRODUCTION:

- A beam is a structural member that is subjected primarily to transverse loads and negligible axial loads.

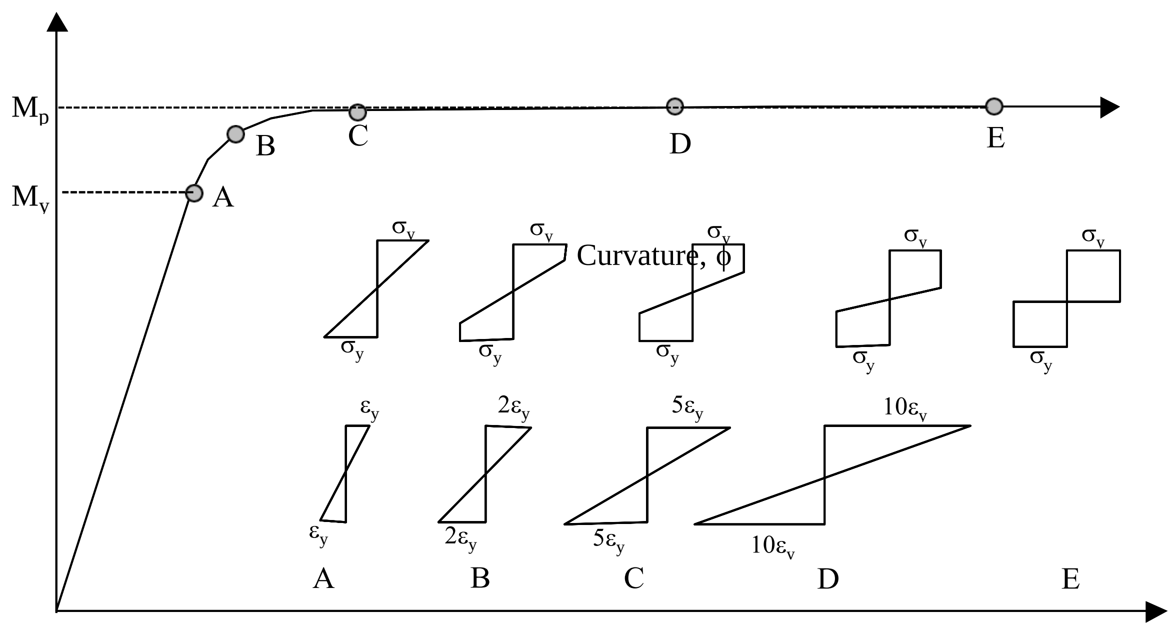
The transverse loads cause internal shear forces and bending moments in the beams as shownin Figure



- These internal shear forces and bending moments cause longitudinal axial stresses and shear stresses in the cross-section as shown in the Figure 2 below.



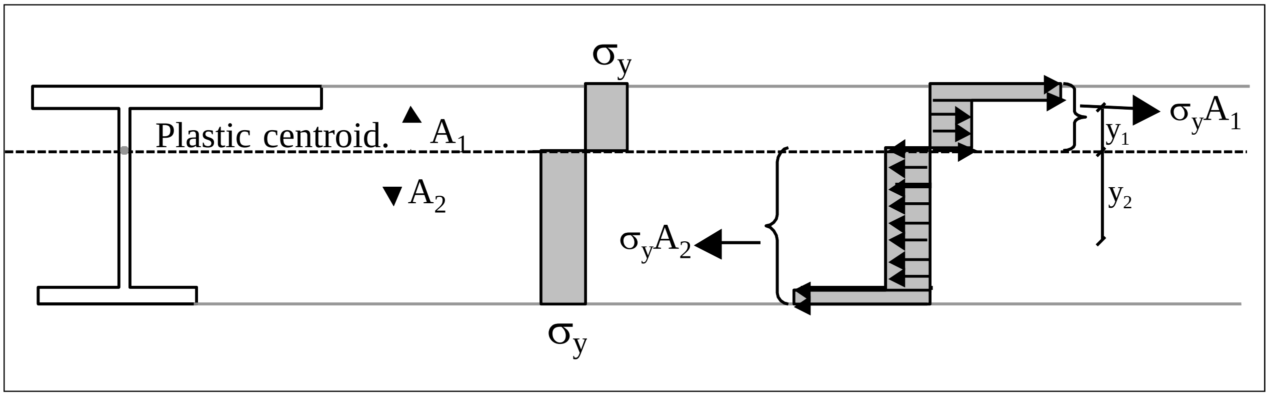
- If the steel stress-strain curve is approximated as a bilinear elasto-plastic curve with yield stress equal to σ_y, then the section Moment - Curvature (M-ϕ) response for monotonically increasing moment is given by Figure 4.



A: Extreme fiber reaches ε_y B: Extreme fiber reaches $2\varepsilon_y$ C: Extreme fiber reaches $5\varepsilon_y$
D: Extreme fiber reaches $10\varepsilon_y$ E: Extreme fiber reaches infinite strain

- In Figure 4, M_y is the moment corresponding to first yield and M_p is the plastic moment capacity of the cross-section.
  - The ratio of M_p to M_y is called as the shape factor *f* for the section.
  - For a rectangular section, *f* is equal to 1.5. For a wide-flange section, *f* is equal to 1.1.
- Calculation of M_p: Cross-section subjected to either $+\sigma_y$ or $-\sigma_y$ at the plastic limit. See Figure

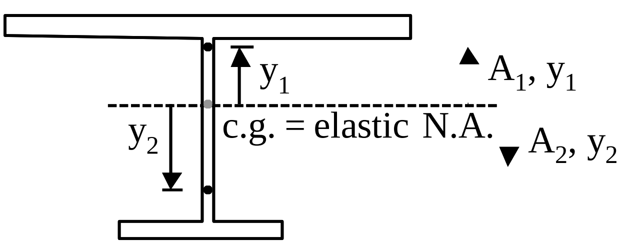
5 below.



The plastic centroid for a general cross-section corresponds to the axis about which the totalarea is equally divided, i.e., $A_1 = A_2 = A/2$

- The plastic centroid is not the same as the elastic centroid or center of gravity (c.g.) of thecross-section.

As shown below, the c.g. is defined as the axis about which $A_1y_1 = A_2y_2$.



About the c.g. $A_1y_1 = A_2y_2$

- For a cross-section with at-least one axis of symmetry, the neutral axis corresponds to the centroidal axis in the elastic range. However, at M_p, the neutral axis will correspond to the plastic centroidal axis.

- **For a doubly symmetric cross-section, the elastic and the plastic centroid lie at the samepoint.**

$M_p = \sigma_y$ x A/2 x (y_1+y_2)

- As shown in Figure 5, y_1 and y_2 are the distance from the plastic centroid to the centroid ofarea A_1 and A_2, respectively.
- A/2 x (y_1+y_2) is called **Z,** the plastic section modulus of the cross-section. Values for Z are tabulated for various cross-sections in the properties section of the LRFD manual.
- $\phi M_p = 0.90 Z F_y$ - See Spec. F1.1

  where,

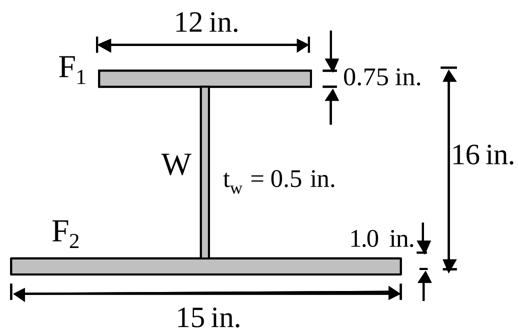
  M_p = plastic moment, which must be $\leq 1.5 M_y$ for homogenous cross-sections

  M_y = moment corresponding to onset of yielding at the extreme fiber from an elastic stressdistribution = F_y S for homogenous cross-sections and = F_{yf} S for hybrid sections.

  Z = plastic section modulus from the Properties section of the AISC manual.

  S = elastic section modulus, also from the Properties section of the AISC manual.

Example 2.1 Determine the elastic section modulus, S, plastic section modulus, Z, yield moment, M_y, and the plastic moment M_p, of the cross-section shown below. What is the design moment for the beam cross-section. Assume 50 ksi steel.



- A_g = 12 x 0.75 + (16 - 0.75 - 1.0) x 0.5 + 15 x 1.0 = 31.125 in^2

  A_{f1} = 12 x 0.75 = 9 in^2

  A_{f2} = 15 x 1.0 = 15.0 in^2

  A_w = 0.5 x (16 - 0.75 - 1.0) = 7.125 in^2

- distance of elastic centroid from bottom = $\bar{y}$

  $$\bar{y} = \frac{9 \times (16 - 0.75/2) + 7.125 \times 8.125 + 15 \times 0.5}{31.125} = 6.619 \text{ in.}$$

  $I_x = 12 \times 0.75^3/12 + 9.0 \times 9.006^2 + 0.5 \times 14.25^3/12 + 7.125 \times 1.506^2 + 15.0 \times 1^3/12 + 15 \times$

  $6.119^2 = 1430 \text{ in}^4$

  $S_x = I_x$ / (16-6.619) = 152.43 in^3

  $M_{y\text{-}x} = F_y\, S_x$ = 7621.8 kip-in. = 635.15 kip-ft.

- distance of plastic centroid from bottom = $\bar{y}_p$

  $$\therefore \quad 15.0 \times 1.0 + 0.5 \times (\bar{y}_p - 1.0) = \frac{31.125}{2} = 15.5625$$

  $$\therefore \quad \bar{y}_p = 2.125 \text{ in.}$$

y_1=centroid of top half-area about plastic centroid = $\dfrac{9\times 13.5+6.5625\times 6.5625}{15.5625}=10.5746$ in.

y_2=centroid of bottom half-area about plas. cent. = $\dfrac{0.5625\times 0.5625+15.0\times 1.625}{15.5625}=1.5866$ in.

Z_x = A/2 x (y_1 + y_2) = 15.5625 x (10.5746 + 1.5866) = 189.26 in^3

$M_{p\text{-}x}$ = Z_x F_y = 189.26 x 50 = 9462.93 kip-in. = 788.58 kip-ft.

- Design strength according to AISC Spec. F1.1= $\phi_b M_p$= 0.9 x 788.58 = 709.72 kip-ft.
- Check = $M_p \leq 1.5\ M_y$

  Therefore, 788.58 kip-ft. < 1.5 x 635.15 = 949.725 kip-ft. - OK!

2.2 Flexural Deflection of Beams – Serviceability

- Steel beams are designed for the factored design loads. The moment capacity, i.e., the factored moment strength ($\phi_b M_n$) should be greater than the moment (M_u) caused by the factored loads.
- A *serviceable* structure is one that performs satisfactorily, not causing discomfort or perceptions of unsafety for the occupants or users of the structure.
  - For a beam, being serviceable usually means that the deformations, primarily the vertical slag, or deflection, must be limited.
  - The maximum deflection of the designed beam is checked at the service-level loads. The deflection due to service-level loads must be less than the specified values.
- The AISC Specification gives little guidance other than a statement in Chapter L, "*Serviceability Design Considerations*," that deflections should be checked. Appropriate limits for deflection can be found from the governing building code for the region.
- The following values of deflection are typical maximum allowable total (service dead load plus service live load) deflections.
  - Plastered floor construction – L/360

– Unplastered floor construction – L/240

– Unplastered roof construction – L/180

- In the following examples, we will assume that local buckling and lateral-torsional buckling are not controlling limit states, i.e, the beam section is compact and laterally supported along the length.

Example 2.2 Design a simply supported beam subjected to uniformly distributed dead load of 450 lbs/ft. and a uniformly distributed live load of 550 lbs/ft. The dead load doesnot include the self-weight of the beam.

- **Step I.** Calculate the factored design loads (without self-weight).

  $w_U = 1.2\ w_D + 1.6\ w_L = 1.42$ kips / ft.

  $M_U = w_u\ L^2 / 8 = 1.42 \times 30^2 / 8 = 159.75$ kip-ft.

- **Step II.** Select the lightest section from the AISC Manual design tables.

  From page_______________of the AISC manual, select **W16 x 26** made from 50 ksi steel with

  $\phi_b M_p = 166.0$ kip-ft.

- **Step III.** Add self-weight of designed section and check design

  $w_{sw} = 26$ lbs/ft

  Therefore, $w_D = 476$ lbs/ft = 0.476 lbs/ft.

  $w_u = 1.2 \times 0.476 + 1.6 \times 0.55 = 1.4512$ kips/ft.

  Therefore, $M_u = 1.4512 \times 30^2 / 8 = 163.26$ kip-ft. $< \phi_b M_p$ of W16 x 26.

OK!

- **Step IV.** Check deflection at service loads.

$w = 0.45 + 0.026 + 0.55$ kips/ft. = 1.026 kips/ft.

$\Delta = 5\ w\ L^4 / (384\ E\ I_x) = 5 \times (1.026/12) \times (30 \times 12)^4 / (384 \times 29000 \times 301)$

$\Delta = 2.142$ in. > L/360 - for plastered floor construction

- **Step V.** Redesign with service-load deflection as design criteria

  $L/360 = 1.0$ in. $> 5\ w\ L^4/(384\ E\ I_x)$

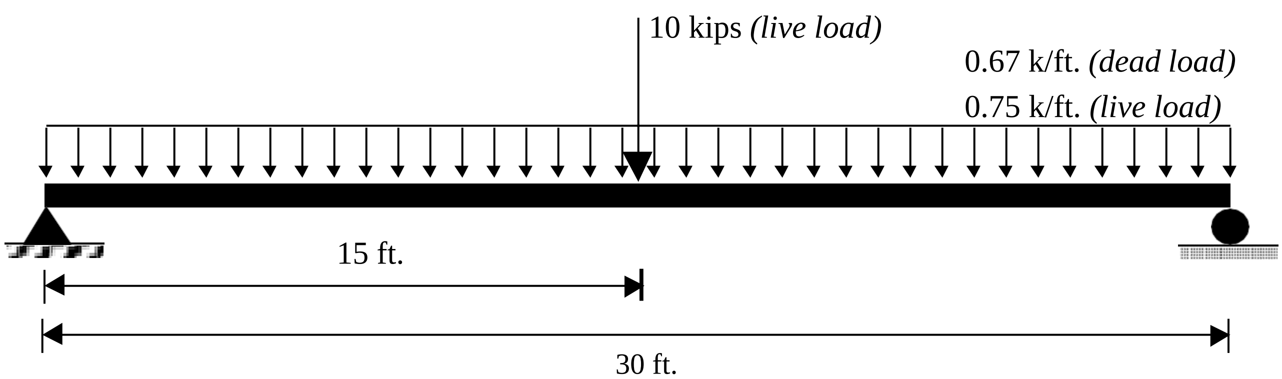
  Therefore, $I_x > 644.8\ in^4$

Select the section from the *moment of inertia* selection tables in the AISC manual. See page – select **W21 x 44.**

W21 x 44 with $\mathbf{I_x} = 843\ in^4$ and $\phi_b M_p = 358$ kip-ft. (50 ksi steel). Deflection at service load = Δ = <u>0.765 in.</u> < L/360 - **<u>OK!</u>**

Note that the serviceability design criteria controlled the design and the section

Example 2.3 Design the beam shown below. The unfactored dead and live loads are shown in the Figure.



- **Step I.** Calculate the factored design loads (without self-weight).

  $w_u = 1.2\ w_D + 1.6\ w_L = 1.2 \times 0.67 + 1.6 \times 0.75 = 2.004$ kips / ft.

  $P_u = 1.2\ P_D + 1.6\ P_L = 1.2 \times 0 + 1.6 \times 10 = 16.0$ kips

$M_u = w_U L^2 / 8 + P_U L / 4 = 225.45 + 120 = 345.45$ kip-ft.

- **Step II.** Select the lightest section from the AISC Manual design tables.

From page_______________of the AISC manual, select **W21 x 44** made from 50 ksi steel with

$\phi_b M_p = 358.0$ kip-ft.

Self-weight = $w_{sw} = 44$ lb/ft.

- **Step III.** Add self-weight of designed section and check design

$w_D = 0.67 + 0.044 = 0.714$ kips/ft

$w_u = 1.2 \times 0.714 + 1.6 \times 0.75 = 2.0568$ kips/ft.

Therefore, $M_u = 2.0568 \times 30^2 / 8 + 120 = 351.39$ kip-ft. $< \phi_b M_p$ of W21 x 44.

<u>**OK!**</u>

- **Step IV.** Check deflection at service loads.

Service loads

- Distributed load = w = 0.714 + 0.75 = 1.464 kips/ft.
- Concentrated load = P = D + L = 0 + 10 kips = 10 kips

Deflection due to uniform distributed load = $\Delta_d = 5 \, w \, L^4 / (384 \, EI)$

Deflection due to concentrated load = $\Delta_c = P \, L^3 / (48 \, EI)$

Therefore, service-load deflection = $\Delta = \Delta_d + \Delta_c$

$\Delta = 5 \times 1.464 \times 360^4 / (384 \times 29000 \times 12 \times 843) + 10 \times 360^3 / (48 \times 29000 \times 843)$

$\Delta = 1.0914 + 0.3976 = 1.49$ in.

Assuming unplastered floor construction, $\Delta_{max} = L/240 = 360/240 = 1.5$ in.

Therefore, $\Delta < \Delta_{max}$ <u>**- OK!**</u>

2.3 Local buckling of beam section – Compact and Non-compact

- M_p, the plastic moment capacity for the steel shape, is calculated by assuming a plastic stress distribution (+ or - σ_y) over the cross-section.
- The development of a plastic stress distribution over the cross-section can be hindered by two different length effects:

  (1) *Local buckling* of the individual plates (flanges and webs) of the cross-section before they develop the compressive yield stress σ_y.

  (2) *Lateral-torsional buckling* of the unsupported length of the beam / member before the cross-section develops the plastic moment M_p.

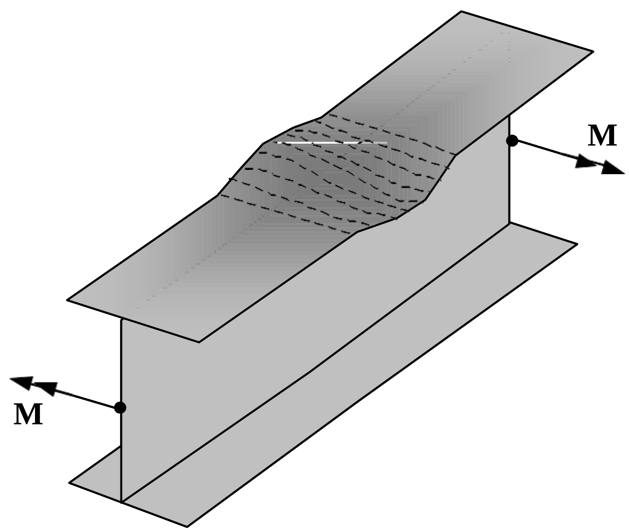


Figure 7. Local buckling of flange due to compressive stress (σ)

- The analytical equations for local buckling of steel plates with various edge conditions and the results from experimental investigations have been used to develop limiting slenderness ratios for the individual plate elements of the cross-sections.
- See Spec. B5 (page 16.1 – 12), Table B5.1 (16.1-13) and Page 16.1-183 of the AISC-manual
- Steel sections are classified as compact, non-compact, or slender depending upon the

slenderness (λ) ratio of the individual plates of the cross-section.

Compact section if all elements of cross-section have $\lambda \leq \lambda_p$

- *Non-compact sections* if any one element of the cross-section has $\lambda_p \leq \lambda \leq \lambda_r$
- *Slender section* if any element of the cross-section has $\lambda_r \leq \lambda$

- It is important to note that:
  - If $\lambda \leq \lambda_p$, then the individual plate element can develop and sustain σ_y for large values of ε before local buckling occurs.
  - If $\lambda_p \leq \lambda \leq \lambda_r$, then the individual plate element can develop σ_y but cannot sustain it before local buckling occurs.
  - If $\lambda_r \leq \lambda$, then elastic local buckling of the individual plate element occurs.

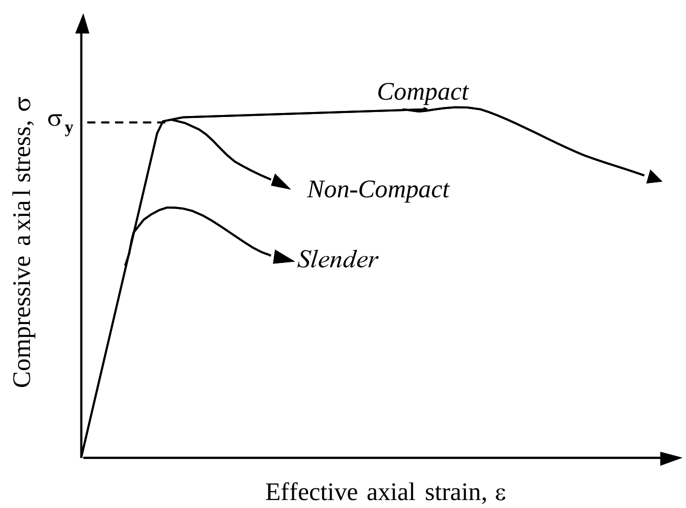


Figure 8. Stress-strain response of plates subjected to axial compression and local buckling.

- Thus, slender sections cannot develop M_p due to elastic local buckling. Non-compact sections can develop M_y but not M_p before local buckling occurs. Only compact sections can develop the plastic moment M_p.
- All rolled wide-flange shapes are **<u>compact</u>** with the following exceptions, which are non-compact.

Section	**Plate element**	λ	λ_p	λ_r
Wide-flange	Flange	$b_f/2t_f$	$0.38 \quad E/F_y$	$0.38 \quad E/F_L$
	Web	h/t_w	$3.76 \quad E/F_y$	$5.70 \quad E/F_y$
Channel	Flange	b_f/t_f	$0.38 \quad E/F_y$	$0.38 \quad E/F_L$
	Web	h/t_w	$3.76 \quad E/F_y$	$5.70 \quad E/F_y$
Square or Rect. Box	Flange	(b-3t)/t	$1.12 \quad E/F_y$	$1.40 \quad E/F_y$
	Web	(b-3t)/t	$3.76 \quad E/F_y$	$5.70 \quad E/F_y$

2.4 Lateral-Torsional Buckling

- The laterally unsupported length of a beam-member can undergo lateral-torsional buckling due to the applied flexural loading (bending moment).

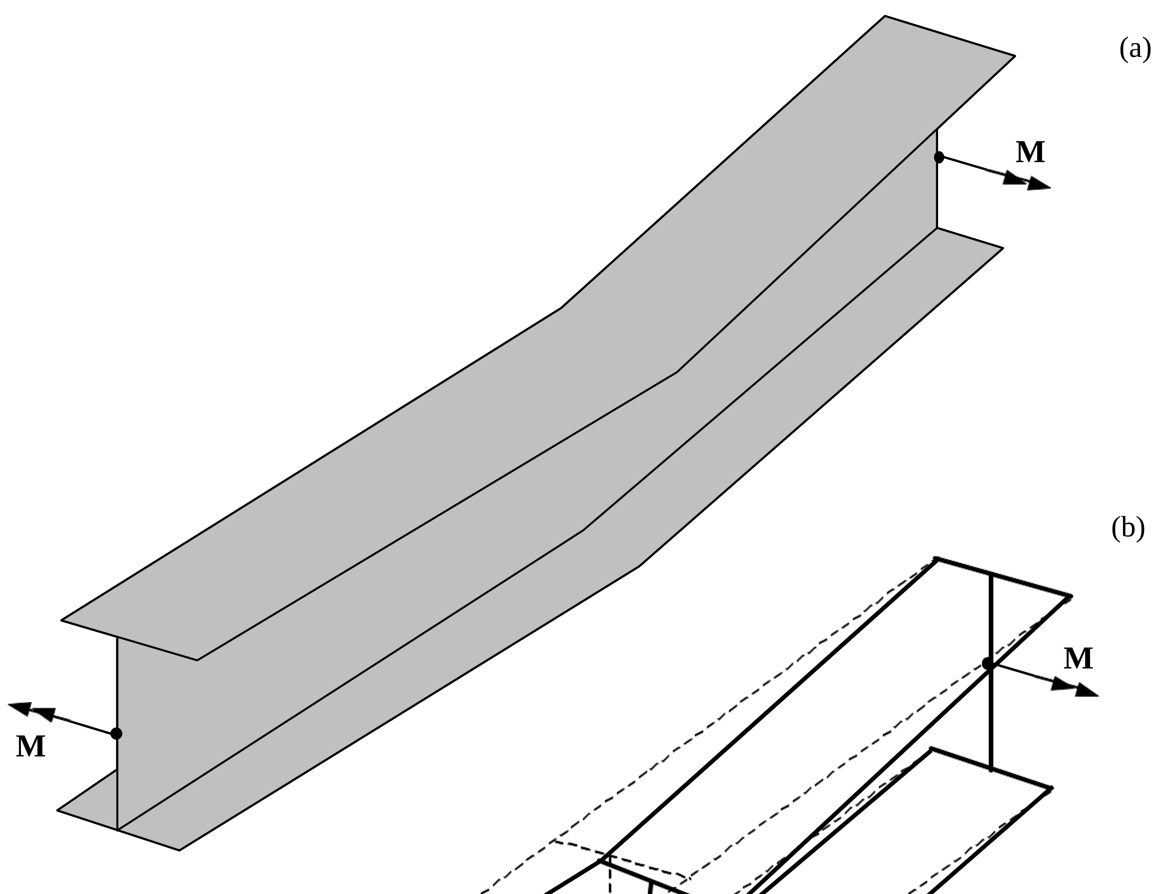


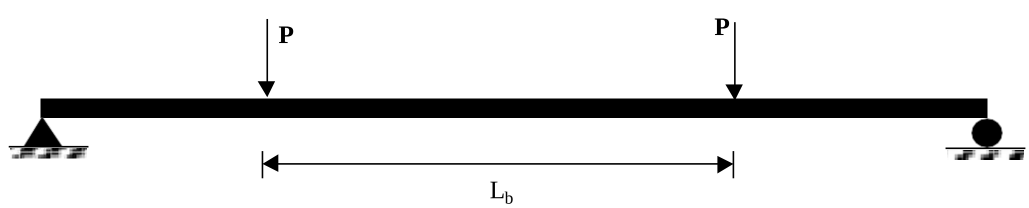
Figure 9. Lateral-torsional buckling of a wide-flange beam subjected to constant moment.

- Lateral-torsional buckling is fundamentally similar to the flexural buckling or flexural-torsional buckling of a column subjected to axial loading.
  - The similarity is that it is <u>also</u> a bifurcation-buckling type phenomenon.
  - The differences are that lateral-torsional buckling is caused by flexural loading (M), and the buckling deformations are coupled in the lateral and torsional directions.

- <u>There is one very important difference.</u> For a column, the axial load causing buckling remains constant along the length. But, for a beam, usually the lateral-torsional buckling causing bending moment M(x) varies along the unbraced length.
  - The worst situation is for beams subjected to <u>uniform bending moment</u> along the unbraced length. Why?

2.4.1 Lateral-torsional buckling – Uniform bending moment

- Consider a beam that is simply-supported at the ends and subjected to four-point loading as shown below. The beam center-span is subjected to <u>uniform bending moment M</u>. Assume that lateral supports are provided at the load points.



- Laterally unsupported length = L_b.
- If the laterally unbraced length L_b is less than or equal to a <u>plastic length L_p</u> then lateral torsional buckling is not a problem and the beam will develop its plastic strength M_p.
- $L_p = 1.76\ r_y \times \sqrt{E/F_y}$ - for I members & channels (See Pg. **16.1**-33)

- If L_b is greater than L_p then lateral torsional buckling will occur and the moment capacity of the beam will be reduced below the plastic strength M_p as shown in Figure 10 below.

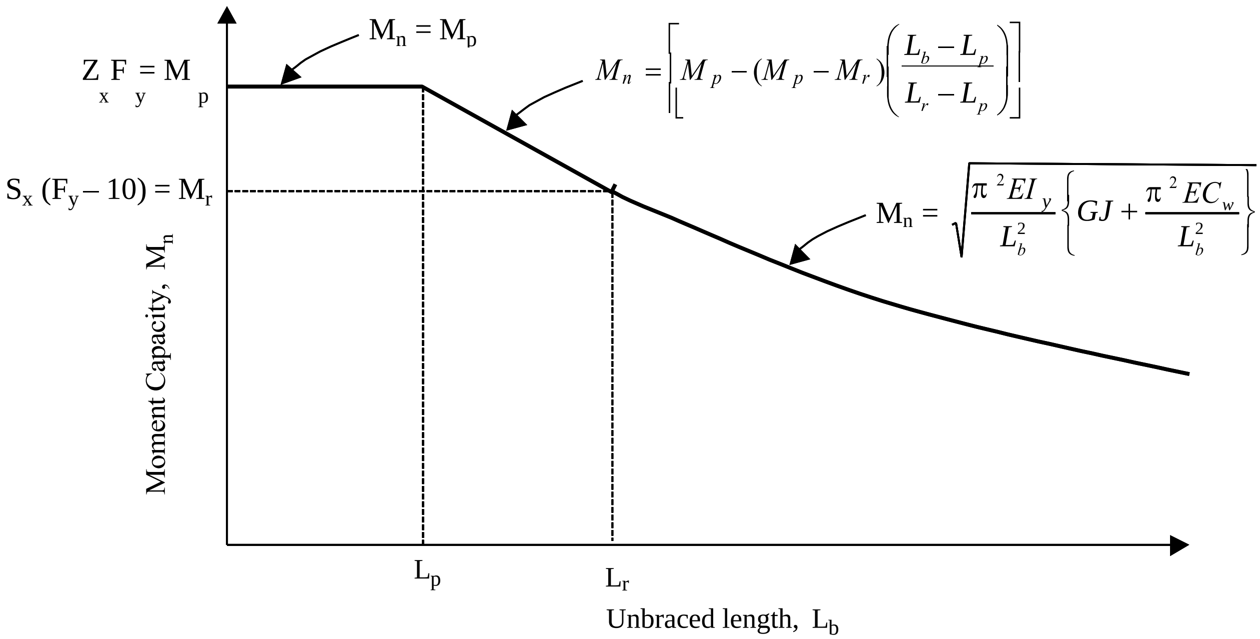


Figure 10. Moment capacity (M_n) versus unsupported length (L_b).

- As shown in Figure 10 above, the lateral-torsional buckling moment ($M_n = M_{cr}$) is a function of the laterally unbraced length L_b and can be calculated using the equation:

$$M_n = M_{cr} = \frac{\pi}{L_b}\sqrt{E \times I_y \times G \times J + \left(\frac{\pi \times E}{L_b}\right)^2 \times I_y \times C_w}$$

where, M_n = moment capacity

L_b = laterally unsupported length.

M_{cr} = critical lateral-torsional buckling moment.E =

29000 ksi; G = 11,200 ksi

I_y = moment of inertia about minor or y-axis (in^4)

J = torsional constant (in^4) from the AISC manual pages______________________.

C_w = warping constant (in^6) from the AISC manual pages______________________.

- This equation is valid for **ELASTIC** lateral torsional buckling only (like the Euler equation). That is it will work only as long as the cross-section is elastic and no portion of the cross-section has yielded.
- As soon as any portion of the cross-section reaches the yield stress F_y, the elastic lateral torsional buckling equation cannot be used.
  - L_r is the unbraced length that corresponds to a lateral-torsional buckling moment $M_r = S_x (F_y - 10)$.
  - M_r will cause yielding of the cross-section due to residual stresses.
- When the unbraced length is less than L_r, then the elastic lateral torsional buckling equation cannot be used.
- When the unbraced length (L_b) is less than L_r but more than the plastic length L_p, then the lateral-torsional buckling M_n is given by the equation below:
  - If $L_p \le L_b \le L_r$, then $M_n = \left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p} \right) \right]$
  - This is linear interpolation between (L_p, M_p) and (L_r, M_r)

2.4.2 Moment Capacity of beams subjected to non-uniform bending moments

- As mentioned previously, the case with uniform bending moment is worst for lateral torsional buckling.
- For cases with non-uniform bending moment, the lateral torsional buckling moment **is greater** than that for the case with uniform moment.
- The AISC specification says that:

- The lateral torsional buckling moment for non-uniform bending moment case

  = $\mathbf{C_b}$ **x** lateral torsional buckling moment for uniform moment case.

- $\mathbf{C_b}$ is always greater than 1.0 for non-uniform bending moment.

  - $\mathbf{C_b}$ is equal to 1.0 for uniform bending moment.
  - Sometimes, if you cannot calculate or figure out $\mathbf{C_b}$, then it can be conservatively assumed as 1.0.

- $$C_b = \frac{12.5\ M_{max}}{2.5\,M_{max} + 3\ M_A + 4\ M_B + 3\ M_c}$$

  where, $\mathbf{M_{max}}$ = magnitude of maximum bending moment in $\mathbf{L_b}$ $\mathbf{M_A}$ =

  magnitude of bending moment at quarter point of $\mathbf{L_b}$$\mathbf{M_B}$ =

  magnitude of bending moment at half point of $\mathbf{L_b}$

  $\mathbf{M_C}$ = magnitude of bending moment at three-quarter point of $\mathbf{L_b}$

- The moment capacity M_n for the case of non-uniform bending moment

  - M_n = $\mathbf{C_b}$ x {M_n for the case of uniform bending moment} $\leq \mathbf{M_p}$
  - Important to note that the increased moment capacity for the non-uniform moment case cannot possibly be more than $\mathbf{M_p}$**.**
  - Therefore, if the calculated values is greater than $\mathbf{M_p}$, then you have to reduce it to $\mathbf{M_p}$

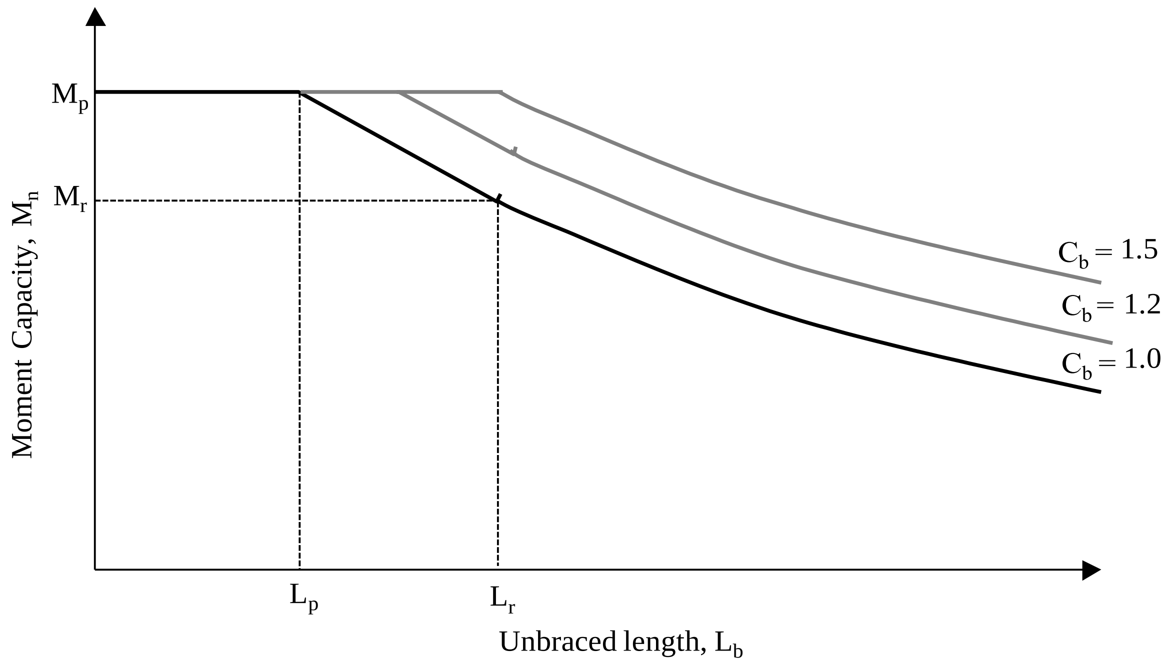
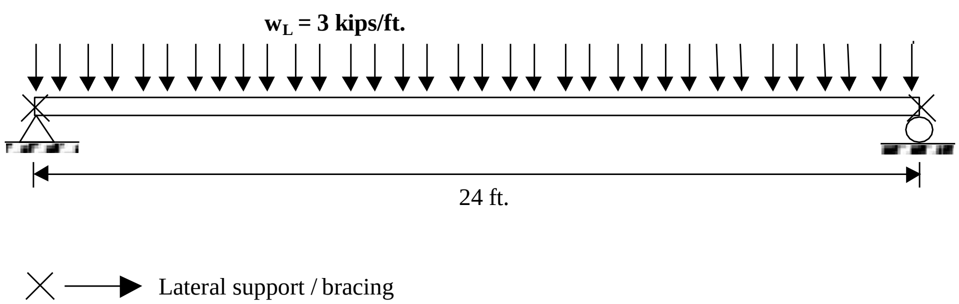


Figure 11. Moment capacity versus L_b for non-uniform moment case.

2.5 Beam Design

Example 2.4

Design the beam shown below. The unfactored uniformly distributed live load is equal to 3 kips/ft. There is no dead load. Lateral support is provided at the end reactions.



Step I. Calculate the factored loads assuming a reasonable self-weight.

Assume self-weight = w_{sw} = 100 lbs/ft.

Dead load = w_D = 0 + 0.1 = 0.1 kips/ft.

Live load = w_L = 3.0 kips/ft.

Ultimate load = w_u = 1.2 w_D + 1.6 w_L = 4.92 kips/ft. Factored ultimate moment = $M_u = w_u L^2/8$ = 354.24 kip-ft.

. Determine unsupported length L_b and C_b There is only one

unsupported span with L_b = 24 ft.

C_b = 1.14 for the parabolic bending moment diagram, See values of C_b shown in Figure.

Step III. Select a wide-flange shape

The moment capacity of the selected section $\phi_b M_n > M_u$ (Note $\phi_b = 0.9$)

$\phi_b M_n$ = moment capacity = C_b x ($\phi_b M_n$ for the case with **uniform moment**) $\leq \phi_b M_p$

- Pages___________________in the AISC-LRFD manual, show the plots of $\phi_b M_n$-L_b for the case of uniform bending moment (C_b=1.0)
- Therefore, in order to select a section, calculate M_u/C_b and use it with L_b to find the first section with a **solid line** as shown in class.
- M_u/C_b = 354.24/1.14 = 310.74 kip-ft.
- Select W16 x 67 (50 ksi steel) with $\phi_b M_n$ =357 kip-ft. for L_b = 24 ft. and C_b =1.0
- For the case with C_b = 1.14,

  $\phi_b M_n$ = 1.14 x 357 = 406.7 kip-ft., **which must be** $\leq \phi_b M_p$ **= 491 kip-ft**.

  OK!

- *Thus, W16 x 67 made from 50 ksi steel with moment capacity equal to 406.7 kip-ft. for an unsupported length of 24 ft. is the designed section.*

Step IV. Check for local buckling.

$\lambda = b_f / 2t_f = 7.7$; Corresponding $\lambda_p = 0.38 (E/Fy)^{0.5} = 9.192$

Therefore, $\lambda < \lambda_p$ - compact flange

$\lambda = h/t_w = 34.4$; Corresponding $\lambda_p = 3.76 (E/F_y)^{0.5} = 90.5$

Therefore, $\lambda < \lambda_p$ - compact web

Compact section. - OK!

This example demonstrates the method for designing beams and accounting for $C_b > 1.0$

QUESTION….

1 (A) WHAT IS BEAM ?

(B)WHAT IS DEFLECTION LIMIT ?

(C) WHAT IS DEFLECTION LIMIT ?

(D)WHAT IS WEB BUCKLING ?

(E)WHAT IS WEB CRIPPLING ?

2 WHAT IS STEEL BEAM AND WRITE ITS CLASSIFICATION ?

3 HOW TO CALCULATE DEFLECTION LIMIT ?

4 DESIGN PROCEDURE OF STEEL BEAM ?

DESIGN OF TUBULAR STRUCUTRE (CH-6)

INTRODUCTION

A tubular structural system is **used in high-rise buildings to resist lateral loads like wind and seismic forces**. These lateral load resisting systems let the building behave like a hollow cylindrical tube cantilevered perpendicular to the ground. Thus the structure exhibits a tubular behavior against the lateral loads.

ADVANTAGES OF TUBULAR SECTIONS

Steel tubing and steel pipes are **greatly resistant to compression against lengthways, and also offer better resistance to bending forces than flat steel sheet**. On another side, a solid tube section has more weight which makes the surface load increases. It also increases the cost of construction and project.

PERMISSIBLE STRESSES (CLAUSE B-2, IS456:2000)

The working stress method is based upon the concept of permissible stresses. Permissible stresses are obtained by dividing the ultimate strength of concrete or yield strength of steel (0.2% proof stress) by appropriate factors of safety. The factors of safety used in working stress method are:

(i) For concrete

(a) in bending compression – 3.0

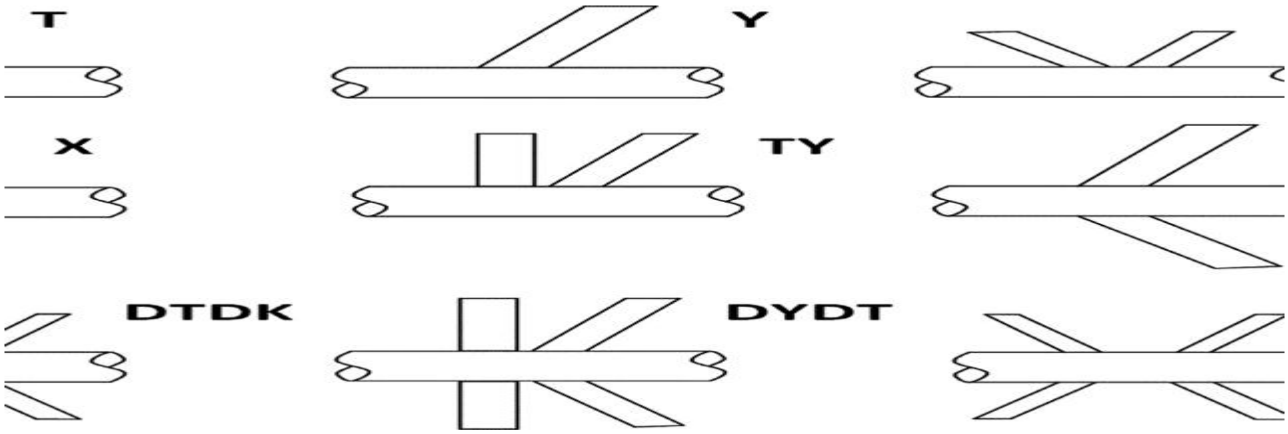
(b) in direct compression – 4.0

(ii) For steel – 1.78

There are greater chances of variation of strength of concrete due to improper compaction, inadequate curing and variation in the properties of concrete. The chances of variation in the properties of steel are less as it is fabricated in factories where good workmanship and better quality control is possible. So, lesser value of factor of safety is used for steel as compared to concrete.

TUBULAR JOINTS

A connection between two or more tubular sections is refereed as tubular joint. For a tubular joint consisting of two pipes of different diameters, the larger diameter pipe is called the chord and the smaller one is known as the brace,



ADVANTAGES OF TUBULAR JOINTS

Steel tubing and steel pipes are greatly resistant to compression against lengthways, and also offer better resistance to bending forces than flat steel sheet. On another side, a solid tube section has more weight which makes the surface load increases. It also increases the cost of construction and project.

Disadvantages of Structural Steel Structures

- Steel is an alloy of iron. This makes it susceptible to corrosion. ...
- There are extensive fireproofing costs involved, as steel is not fireproof. In high temperatures, steel loses its properties.
- Buckling is an issue with steel structures.

QUESTIONS

1. What is tubular sections ?
2. Define round tubular sections ?
3. Define permissible stresses ?
4. What is tubular compression ?
5. What is tublar tension ?
6. What are the advantages of advantages and dis advamntages of tubular sections ?

DESIGN OF MASONARY STRUCTURE(CH-7)

INTRODUCTION

Masonry consists of **building structures from single units that are laid and bound together with mortar**. Brick, stone and concrete blocks are the most common materials used in masonry construction. Masonry is a popular construction technique around the world, due to its many advantages.

EXAMPLES

masonry, the art and craft of building and fabricating in stone, clay, brick, or concrete block. Construction of poured concrete, reinforced or unreinforced, is often also considered masonry.

There are four types of structures;

- Frame: made of separate members (usually thin pieces) put together.
- Shell: encloses or contains its contents.
- Solid (mass): made almost entirely of matter.
- liquid (fluid): braking fluid making the brakes.

The features of masonry include **design flexibility, various textures, structural strength, mold resistance, durability, simple maintenance needs and competitive cost**.

Thanks to these properties, masonry has been one of the most used construction methods throughout history and in modern times.

SYMBOLES USED IN MASONRY DESIGN

t = Actual wall thickness (mm)

t_1= Actual wall thickness one leaf of cavity wall (mm)

t_2= Actual wall thickness second leaf of cavity wall (mm)

t_{ef} = Effective wall thickness (mm)

A'_s = Area of Compression reinforcement

b = width or effective width of section

b_w = average width of web.

α =the bending moment coefficient.

μ = the orthogonal ratio;

γ_f = Partial safety factor for load

γ_m = Partial safety factor for material strength

σ_d = design compressive stresses

E_n = Characteristic Earth Load

G_k = Characteristic dead Load

Q_k = Characteristic imposed Load

W_k = Characteristic Wind Load

f_b = Normalised compressive strength of masonry units

f_k = Characteristic strength of masonry

f_m = Compressive strength of mortar

f_d = design compressive strength of masonry

f_{cu} = Characteristic strength of concrete

f_y = Characteristic strength of reinforcement

W_k =is the characteristic wind load per unit area.

Characteristic strength of masonry

When test data for obtaining the characteristic strength f_k is not available, then the approximate characteristic strength of plain masonry made with **general purpose** mortar may be calculated using the equation

$f_k = K * (f_b^{0.70})*(f_m^{0.30})$ [MPa]

when f_m less than 20 MPa or 2.f_b, whichever is the smaller and f_b is less than 75 MPa

The value of constant K depends on the classification of masonry units into groups. Following is a basic list providing a extract of table in the relevant standard listing K values against mortar type and masonry group. The list below simply provides the values of K related to group 1 and for general purpose mortar.

Clay bricks 1 K = 0,55
Calcium Silicate bricks K =0,55
Aggregate concrete blocks K = 0,55
Autoclaved Aerated concrete K= 0,55
Manufactured stone K= 0,45
Dimensioned natural stone K= 0,45

Note: The equations above provide an edited extract of the range of groups and relationships listed in the relevant code BS EN 1996. For other grades of mortar the indices in the equation are different.

LOADBearing walls have a point load, such as the bottom of a support column, where the weight of the load transfers to the support structure. Walls with a uniform load distribute the weight evenly along the structure. Non-load-bearing walls are sometimes called "partition walls" or "curtain walls".

Permissible stress design is a design philosophy used by mechanical engineers and civil engineers.

The civil designer ensures that the stresses developed in a structure due to service loads do not exceed the elastic limit. This limit is usually determined by ensuring that stresses remain within the limits through the use of factors of safety.

In structural engineering, the permissible stress design approach has generally been replaced internationally by limit state design (also known as ultimate stress design, or in USA, Load and Resistance Factor Design, LRFD) as far as structural engineering is considered, except for some isolated cases.

In USA structural engineering construction, allowable stress design (ASD) has not yet been completely superseded by limit state design except in the case of Suspension bridges, which changed from allowable stress design to limit state design in the 1960s. Wood, steel, and other materials are still frequently designed using allowable stress design, although LRFD is probably more commonly taught in the USA university system.

In mechanical engineering design such as design of pressure equipment, the method uses the actual loads predicted to be experienced in practice to calculate stress and deflection. Such loads may include pressure thrusts and the weight of materials. The predicted stresses and deflections are compared with allowable values that have a "factor" against various failure mechanisms such as leakage, yield, ultimate load prior to plastic failure, buckling, brittle fracture, fatigue, and vibration/harmonic effects. However, the predicted stresses almost always assumes the material is linear elastic. The "factor" is sometimes called a factor of safety, although this is technically incorrect because the factor includes allowance for matters such as local stresses and manufacturing imperfections that are not specifically calculated; exceeding the allowable values is not considered to be good practice (i.e is not "safe").

The **slenderness ratio** of a masonry wall is defined as **the effective height divided by the effective thickness or its effective length divided by the effective thickness, whichever is less**.

The **effective length** of a masonry wall stiffened by buttresses on both ends and continuing beyond these buttresses at both ends is **0.8L.**

$\lambda = 27$

For Load-bearing walls, the maximum permissible slenderness ratio (λ) is: $\boldsymbol{\lambda = 27}$. For non-Load bearing walls, the maximum permissible slenderness ratio (λ) is: $\lambda = 30$.

HEIGHT OF WALL

In a residential property, one can build a boundary wall of **a minimum of 1.5m height to a maximum of 2.4m height** as per the guidelines of NBC. The thickness of the wall should be between 230mm to 400mm.

THICKNESS OF WALL

Typically, the wall thickness will be in the range **0.5 mm to 4 mm**. In specific cases, wall thicknesses that are either smaller or bigger also occur. A basic design guideline is to keep wall thicknesses as thin and as uniform as possible.

QUESTIONS

1. **Define mesonry structure ?**
2. **What is load bearing and non load bearing walls ?**
3. **What is permissible stresses ?**
4. **What is slenderness ratio ?**
5. **What are the effective length of masonry wall ?**

6.Design procedure of masonry wall ?

THANK YOU