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SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT – I- Introduction– SCI1303

INTRODUCTION

PROPERTIES:

- Physical properties
- Mechanical properties

PHYSICAL PROPERTIES of structural steel irrespective of its grade may be taken as:

a) Unit mass of steel,	p = 7850 kg/m3
b) Modulus of elasticity,	*E* = 2.0 x 10s N/mm2 (MPa)
c) Poisson ratio,	p = 0.3
d) Modulus of rigidity,	G = 0.769 x 10s N/mm2 (MPa)
e) Co-efficient of thermal expansion	cx.= 12 x10^{-6} /°c

MECHANICAL PROPERTIES OF STRUCTURAL STEEL:

The principal mechanical properties of the structural steel which is important in design are the

- Yield stress, *fy*
- The tensile or ultimate stress, *fu*
- The maximum percent elongation on a standard gauge length
- Notch toughness.

STEEL PRODUCTS AND STEEL TABLES:

The long products are normally used in the as-hot-rolled condition. Plates are used in hot rolled condition as well as in the normalized condition to improve their mechanical properties particularly the ductility and the impact toughness. The structural sections produced in India include open sections such as beams, channels, tees and angles, Closed (hollow) sections such as rectangular and circular tubes are available only in smaller sizes.

Solid sections like bars, flats and strips are available. Steel plates are also available in various sizes and thicknesses. These sections are designated in a standard manner with the letters IS indicating that they satisfy the prescriptions of the Indian Standards Specifications (SP 6(1)) followed by the letter indicating the classification and type of section and a number indicating the size of the section. Usually the depth of the section is chosen to indicate its size.

The beam sections are classified as ISLB (light), ISJB (junior), ISMB (medium), ISHB (heavy) and ISWB (wide-flanged) sections. Similarly, Channel sections are designated as ISLC, ISMC etc. and angles are designated as ISA followed by the size of each leg and the thickness. Both equal and unequal angles are available. Sometimes two different sections have the same designation but their weight per unit length is slightly different. In such cases, the weight per unit length is also specified as ISMB 600 @ 48.5 kg/m.

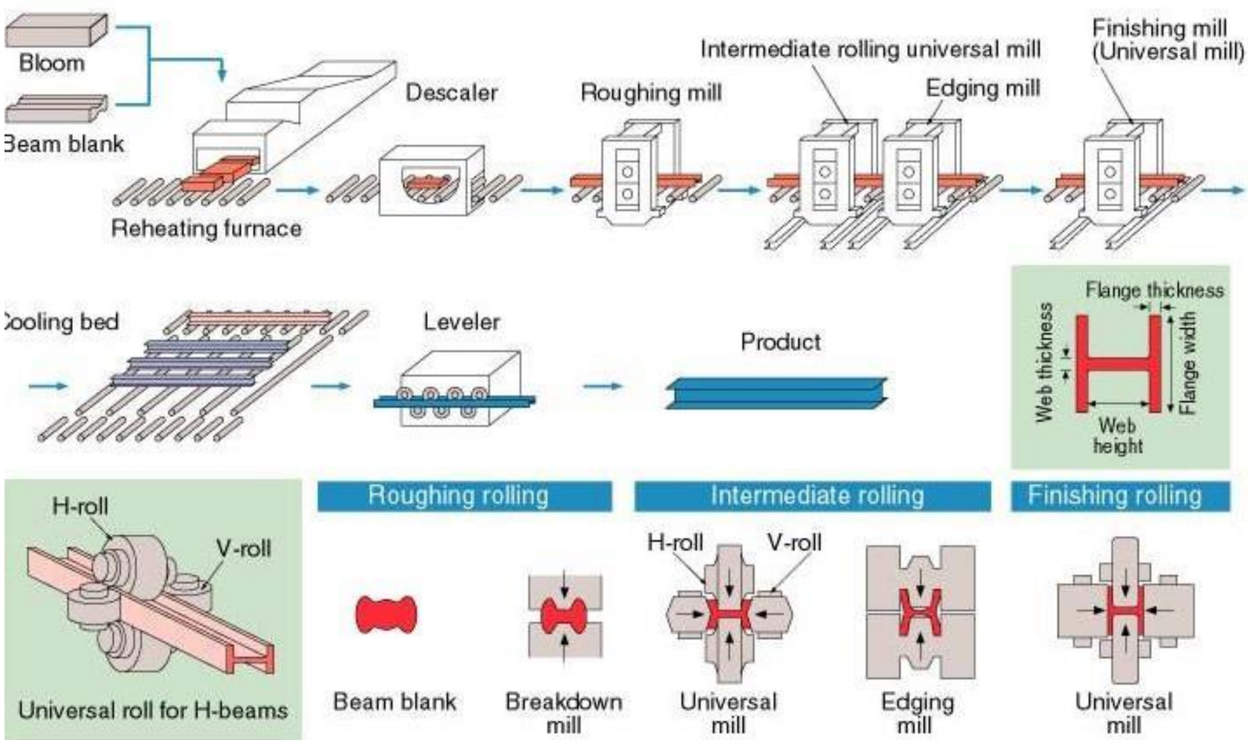

Fig 3.Manufacturing process of h beams

The properties of sections, including the geometric details such as average thickness, area, moment of inertia about various axes and preferred location and diameter of holes for bolts etc are tabulated in the steel tables such as SP6(1). Such tables are of great use to designers for selecting a suitable section for a member.

COLD ROLLING AND COLD FORMING:

Cold rolling, as the term implies involves reducing the thickness of unheated material into thin sheets by applying rolling pressure at ambient temperature. The common colds rolled products are coils and sheets. Cold rolling results in smoother surface and improved mechanical properties. Cold rolled sheets could be made as thin as 0.3 mm.

Cold forming is a process by which the sheets (hot rolled / cold rolled) are folded in to desired section profile by a series of forming rolls in a continuous train of roller sets. Such thin shapes are impossible to be produced by hot rolling. The main advantage of cold-formed sheets in structural application is that any desired shape can be produced. In other words it can be tailor-made into a particular section for a desired member. These cold formed sheet steels are basically low carbon steel

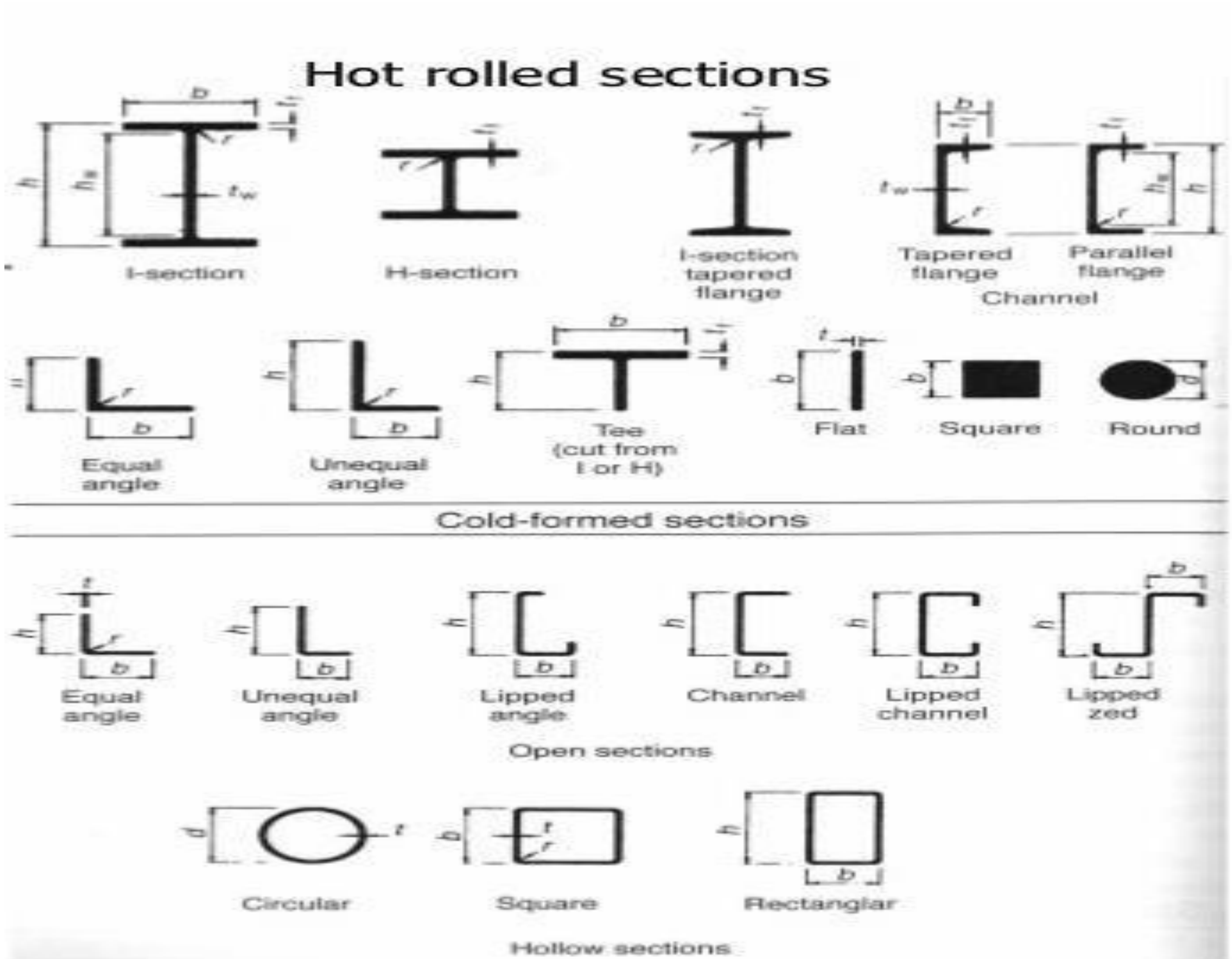

Fig 4. Hot rolled and cold formed structural sections

LIMIT STATE DESIGN:

INTRODUCTION TO LIMIT STATE DESIGN:

Civil Engineer has to ensure that the structures and facilities he designs are (i) **fit for their purpose (ii) safe and (iii) economical and durable**. Thus safety is one of the paramount responsibilities of the designer. However, it is difficult to assess at the design stage how safe a proposed design will actually be. There is, in fact, a great deal of uncertainty about the many factors, which influence both safety and economy.

The uncertainties affecting the safety of a structure are due to

- Uncertainty about loading
- Uncertainty about material strength
- Uncertainty about structural dimensions and behaviour.

These uncertainties together make it impossible for a designer to guarantee that a structure will be absolutely safe. All that the designer can ensure is that the risk of failure is extremely small, despite the uncertainties. An illustration of the statistical meaning of safety is given in Fig.5.

Let us consider a structural component (say, a beam) designed to carry a given nominal load. Bending moments (B.M.) produced by loads is first computed. These are to be compared with the resistance or strength (R.M.) of the beam. But the resistance (R.M.) itself is not a fixed quantity, due to variations in material strengths that might occur between nominally same elements.

The statistical distribution of these member strengths (or resistances) will be as sketched in (a).Similarly, the variation in the maximum loads and therefore load effects (such as bending moment) which different structural elements (all nominally the same) might encounter in their service life would have a distribution shown in (b).

The uncertainty here is both due to variability of the loads applied to the structure, and also due to the variability of the load distribution through the structure. Thus, if a particularly weak structural component is subjected to a heavy load which exceeds the strength of the structural component, clearly failure could occur. Unfortunately it is not practicable to define the probability distributions of loads and strengths, as it will involve hundreds of tests on samples of components.

Normal design calculations are made using a single value for each load and for each material property and taking an appropriate safety factor in the design calculations. The single value used is termed as Characteristic Resistance of material.

Characteristic resistance of a material (such as Concrete or Steel) is defined as that value of resistance below which not more than a prescribed percentage of test results may be expected to fall. (For example the characteristic yield stress of steel is usually defined as that value of yield stress below which not more than 5% of the test values may be expected to fall).

In other words, this strength is expected to be exceeded by 95% of the cases. Similarly, the characteristic load is that value of the load, which has an accepted probability of not being exceeded during the life span of the structure. Characteristic load is therefore that load which will not be exceeded 95% of the time.

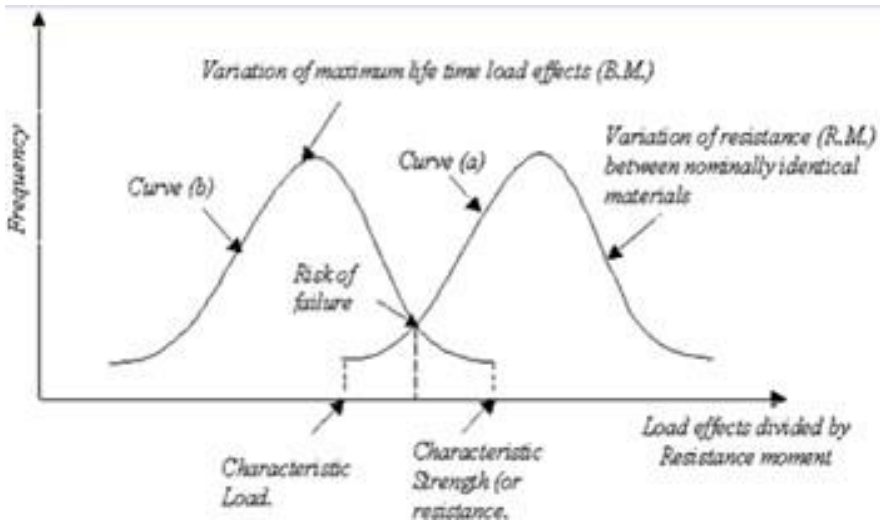

Fig5. Statistical meaning of safety

Most structural designs are based on experience. If a similar design has been built successfully elsewhere, there is no reason why a designer may not consider it prudent to follow aspects of design that have proved successful, and adopt standardized design rules.

- ❖ In the Working Stress Method (WSM) of design, the first attainment of yield stress of steel was generally taken to be the onset of failure as it represents the point from which the actual behaviour will deviate from the analysis results.
- ❖ Also, it was ensured that non-linearity and buckling effects were not present. It was ensured that the stresses caused by the working loads are less than an allowable stress obtained by dividing the yield stress by a factor of safety.

- ❖ The factor of safety represented a margin for uncertainties in strength and load.
- ❖ The value of factor of safety in most cases is taken to be around 1.67. In general, each member in a structure is checked for a number of different combinations of loads.
- ❖ Some loads vary with time and this should be taken care of. It is unnecessarily severe to consider the effects of all loads acting simultaneously with their full design value, while maintaining the same factor of safety or safety factor.
- ❖ Using the same factor of safety or safety factor when loads act in combination would result in uneconomic designs.

A typical example of a set of load combinations is given below, which accounts for the fact that the dead load, live load and wind load are all unlikely to act on the structure simultaneously at their maximum values: (Stress due to dead load + live load) ≤ allowable stress (Stress due to dead load + wind load) ≤ allowable stress (Stress due to dead load + live load + wind) ≤ 1.33 times allowable stress.

Limitations:

- ❖ In practice there are severe limitations to this approach.
- ❖ The major limitation stems from the fact that yielding at any single point does not lead to failure.
- ❖ This means that the actual factor of safety is generally different from the assumed factor of safety and varies from structure to structure.
- ❖ There are also the consequences of material nonlinearity, non-linear behaviour of elements in the post-buckled state and the ability of the steel components to tolerate high local stresses by yielding and redistributing the loads.

The elastic theory does not consider the larger safety factor for statically indeterminate structures which exhibit redistribution of loads from one member to another before collapse. These are addresses in a more rational way in Limit State Design

ANALYSIS PROCEDURES AND DESIGN PHILOSOPHY:

An improved design philosophy to make allowances for the shortcomings in the Working Stress Method was developed in the late 1970’s and has been extensively incorporated in design standards and codes. The probability of operating conditions not reaching failure conditions forms the basis of Limit State Method (LSM). The Limit State is the condition in which a structure would be considered to have failed to fulfill the purpose for which it was built.

Two limit states:

- **Limit State of Collapse** is a catastrophic state, which requires a larger reliability in order to reduce the probability of its occurrence to a very low level.
- **Limit State of Serviceability** refers to the limit on acceptable service performance of the structure. Not all the limit states can be covered by structural calculations.

  For example, corrosion is covered by specifying forms of protection (like painting) and brittle fracture is covered by material specifications, which ensure that steel is sufficiently ductile.

  The major innovation in the Limit State Method is the introduction of the **partial safety factor** format which essentially splits the factor of safety into two factors
  - one for the material
  - one for the load.

Table.1 Types of limit states

Limit State of Strength	**Limit State of Serviceability**
Yielding, Crushing and Rupture	Deflection
Stability against buckling, overturning and sway	Vibration
Fracture due to fatigue	Fatigue checks (including reparable damage due to fatigue)
Brittle Fracture	Corrosion

In accordance with these concepts, the safety format used in Limit State s based on probable maximum load and probable minimum strengths, so that a consistent level of safety is achieved.

Partial Saftey factors can be of two types.

- γ_f = partial safety factor for load (load factor)
- γ_m = partial safety factor for material strength

Both the partial safety factors for load and material are determined on a ‘probabilistic basis’ of the corresponding quantity. It should be noted that γ_f makes allowance for possible deviation of loads and also the reduced possibility of all loads acting together. On the other hand γ_m allows for uncertainties of element behaviour and possible strength reduction due to manufacturing tolerances and imperfections in the material.

- The partial safety factor for steel material failure by yielding or buckling γm_0 is given as 1.10 while for ultimate resistance it is given as $\gamma m1=1.25$.
- For bolts and shop welds, the factor is 1.25 and for field welds it is 1.50.

Strength is not the only possible failure mode. Excessive deflection, excessive vibration, fracture etc. also contribute to Limit States. Fatigue is also an important design criterion for bridges, crane girders etc. Thus the following limit states may be identified for design purposes: Collapse Limit States are related to the maximum design load capacity under extreme conditions.

- The partial load factors are chosen to reflect the probability of extreme conditions, when loads act alone or in combination.
- Stability shall be ensured for the structure as a whole and for each of its elements. It includes overall frame stability against overturning and sway, uplift or sliding under factored loads.
- Serviceability Limit States are related to the criteria governing normal use.
- Axial loads are used to check the adequacy of the structure. These include Limit State of Deflection, Limit State of Vibration, Limit State of Durability and Limit State of Fire Resistance.
- Load factor, γf, of value equal to unity shall be used for all loads leading to serviceability limit states.
- Fatigue Limit State is important where distress to the structure by repeated loading is a possibility.
- Stress changes due to fluctuations in wind loading normally need not be considered.
- Fatigue design shall be as per Section 13 of this code IS800:2007. When designing for fatigue, the load factor for action, γf, equal to unity shall be used for the load causing stress fluctuation and stress range.
- The design considerations for Durability, Fire Resistance and Fatigue have already been discussed in the previous chapter.
- The above limit states are provided in terms of partial factors, reflects the severity

of the risks.

An illustration of partial safety factors suggested in the revised IS: 800 for ultimate load conditions is given in Table 2. The basic load values are specified in IS 875:1987- Except for earthquake load. The dead load which includes the self weight of the member and the weight of any permanent fixture such as a wall can be obtained by knowing the unit weight of the materials.

Live loads for residential buildings are given as 3 kN/m^2 and the office buildings as 4 kN/m^2 . Wind load may be worked out based on the basic wind speed at the place and permeability of the build as described in IS 875-part3, 1987. The calculation of loads is given in IS 1893-2002.

Table 2: Partial safety factors (Cl.5.3.3)

Combination	Limit State of Strength					Limit state of Serviceability			
	DL	LL		WL/EL	AL	DL	LL		WL/EL
		Leading	Accom-panying				Leading	Accom-panying	
DL+LL+CL	1.5	1.5	1.05	-	-	1.0	1.0	1.0	-
DL+LL+CL +WL/EL	1.2 1.2	1.2 1.2	1.05 0.53	0.6 1.2	-	1.0	0.8	0.8	0.8
DL+WL/EL	1.5 (0.9)*	-	-	1.5	-	1.0	-	-	1.0
DL+ER	1.2 (0.9)	1.2	-	-	-	-	-	-	-
DL+LL+AL	1.0	0.35	0.35	-	1.0	-	-	-	-

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

* When action of different live loads is simultaneously considered, the leading live load is one which causes the higher load effects in the member/section and all other live loads are classified as accompanying.

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

Note: The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in Section 4.

LOADS ON STRUCTURES:

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

a) Dead loads;

b) Imposed loads (live load, crane load, snow load, dust load, wave load, earth pressure, etc);

c) Wind loads;

d) Earthquake loads;

e) Erection loads;

f] Accidental loads such as those due to blast, impact of vehicles, etc; and

g) Secondary effects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components, eccentric connections, rigidity of joints

- ***Dead Loads*** — The self-weights of all permanent constructions and installations including the self-weight of all walls, partitions, floors, roofs, and other permanent fixtures acting on a member.

- ***Imposed (Live) Load*** — The load assumed to be produced by the intended use or occupancy including distributed, concentrated, impact, vibration and snow loads but excluding, wind, earthquake and temperature loads.

- ***Wind Loads*** — Load experienced by member or structure due to wind pressure acting on the surfaces.

- ***Earthquake Loads*** — The inertia forces produced in a structure due to the ground movement during an earthquake.

- ***Erection Loads*** — The actions (loads and eformations) experienced by the structure exclusively during erection.

- ***Accidental Loads*** — Loads due to explosion, impact of vehicles, or other rare loads for which thestructure is considered to be vulnerable as per the user.

LOCAL BUCKLING AND SECTION CLASSIFICATION:

Introduction:

- ✓ Sections normally used in steel structures are I-sections, Channels or angles etc. which are called open sections, or rectangular or circular tubes which are called closed sections.
- ✓ These sections can be regarded as a combination of individual plate elements connected together to form the required shape.
- ✓ The strength of compression members made of such sections depends on their slenderness ratio.
- ✓ Higher strengths can be obtained by reducing the slenderness ratio
  i.e. by increasing the moment of inertia of the cross-section. Similarly, the strengths of beams can be increased, by increasing

the moment of inertia of the cross-section. For a given cross- sectional area, higher moment of inertia can be obtained by making the sections thin-walled.

- ✓ As discussed earlier, plate elements laterally supported along edges and subjected to membrane compression or shear may buckle prematurely.
- ✓ Therefore, the buckling of the plate elements of the cross section under compression/shear may take place before the overall column buckling or overall beam failure by lateral buckling or yielding. This phenomenon is called **local buckling**.
- ✓ Thus, local buckling imposes a limit to the extent to which sections can be made thin-walled.

Consider an I-section column, subjected to uniform compression [Fig. 6(a)]. Therefore, in open sections such as I sections, the flanges which are outstands tend to buckle before the webs which are supported along all edges. Further, the entire length of the flanges is likely to buckle in the case of the axially compressed member under consideration, in the form of waves.

On the other hand, in closed sections such as the hollow rectangular section, both flanges and webs behave as internal elements and the local buckling of the flanges and webs depends on their respective width-thickness ratios. In this case also, local buckling occurs along the entire length of the member and the member develops a ‘chequer board’ wave pattern [Fig. 6(b)].

In the case of beams, the compression flange behaves as a plate element subjected to uniform compression and, depending on whether it is an outstand or an internal element, undergoes local buckling at the corresponding critical buckling stress. However, the web is partially under compression and partially under tension. Even the part in compression is not under uniform compression.

Therefore the web buckles as a plate subjected to inplane bending compression. Normally, the bending moment varies over the length of the beam and so local buckling may occur only in the region of maximum bending moment.

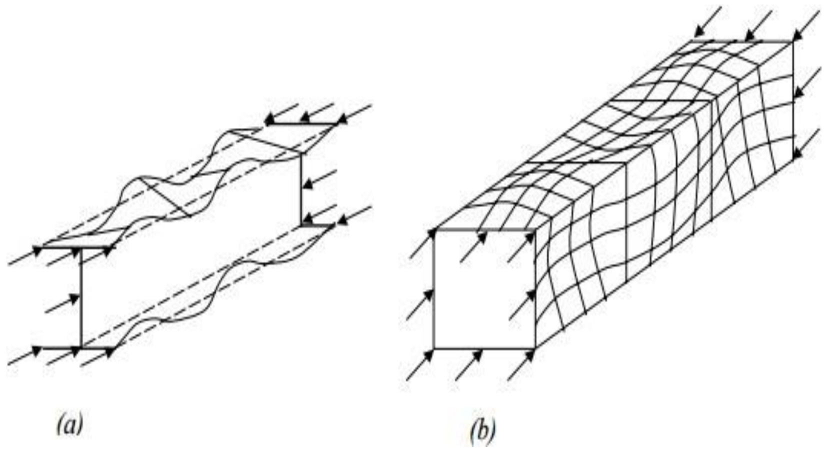

Fig. 6 Local buckling of Compression Members

Local buckling has the effect of reducing the load carrying capacity of columns and beams due to the reduction in stiffness and strength of the locally buckled plate elements. Therefore it is desirable to avoid local buckling before yielding of the member. Most of the hot rolled steel sections have enough wall thickness to eliminate local buckling before yielding.

However, fabricated sections and thin-walled cold-formed steel members usually experience local buckling of plate elements before the yield stress is reached. It is useful to classify sections based on their tendency to buckle locally before overall failure of the member takes place. For those cross-sections liable to buckle locally, special precautions need to be taken in design.

However, it should be remembered that local buckling does not always spell disaster. Local buckling involves distortion of the cross-section. There is no shift in the position of the cross-section as a whole as in global or overall buckling. In some cases, local buckling of one of the elements of the cross section may be allowed since it does not adversely affect the performance of the member as a whole. In the context of plate bucking, it was pointed out that substantial reserve strength exists in plates beyond the point of elastic buckling. Utilization of this reserve capacity may also be the objective of design.

Therefore, local buckling may be allowed in some cases, provided due care is taken to estimate the reduction in the capacity of the section due to it.

BASIC CONCEPTS OF PLASTIC THEORY:

Before attempting the classification of sections, the basic concepts of plastic theory will be introduced. More detailed descriptions can be found in subsequent chapters.

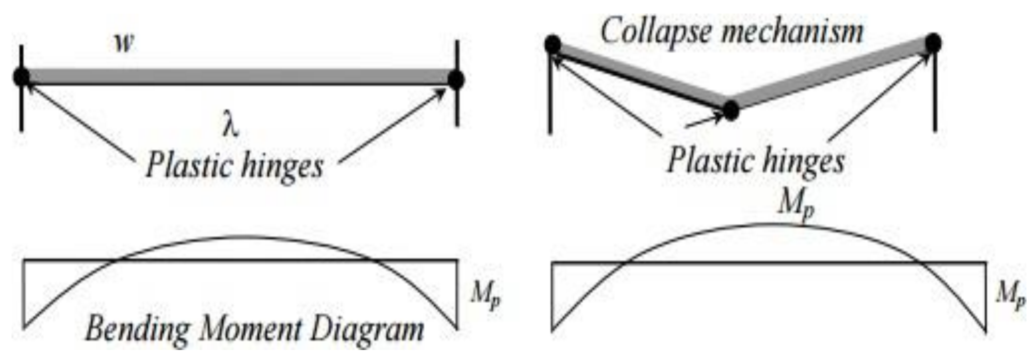

Fig. 7 Formation of a Collapse Mechanism in a Fixed Beam

Consider a beam with both ends fixed and subjected to a uniformly distributed load of w per meter length as shown in Fig. 7(a). The elastic bending moment at the ends is $wl^2/12$ and at mid-span is $wl^2/24$, where l is the span.

The stress distribution across any cross section is linear As w is increased gradually, the bending moment at every section increases and the stresses also increase. At a section close to the support where the bending moment is maximum, the stresses in the extreme fibers reach the yield stress.

The moment corresponding to this state is called the first yield moment My, of the cross section. But this does not imply failure as the beam can continue to take additional load. As the load continues to increase, more and more fibers reach the yield. Eventually the whole of the cross section reaches the yield stress .The moment corresponding to this state is known as the plastic moment of the cross section and is denoted by Mp.

The ratio of the plastic moment to the yield moment is known as the shape factor since it depends on the shape of the cross section. The cross section is not capable of resisting any additional moment but may maintain this moment for some amount of rotation in which case it acts like a plastic hinge. If this is so, then for further loading, the beam, acts as if it is simply supported with two additional moments Mp on either side, and continues to carry additional loads until a third plastic hinge forms at mid-span when the bending moment at that section reaches Mp.

The beam is then said to have developed a collapse mechanism and will collapse as shown in Fig 7(b). If the section is thin-walled, due to local buckling, it may not be able to sustain the moment for additional rotations and may collapse either before or soon after attaining the plastic moment. It may be noted that formation of a single plastic hinge gives a collapse mechanism for a simply supported beam.

The ratio of the ultimate rotation to the yield rotation is called the rotation capacity of the section. The yield and the plastic moments together with the rotation capacity of the cross-section are used to classify the sections.

SECTION CLASSIFICATION:

Sections are classified depending on their moment-rotation characteristics (Fig. 8). The codes also specify the limiting width-thickness ratios β = b/t for component plates, which enables the classification to be made.

Plastic cross-sections: Plastic cross-sections are those which can develop their full plastic moment Mp and allow sufficient rotation at or above this moment so that redistribution of bending moments can take place in the structure until complete failure mechanism is formed (b/t ≤ β1) (see Fig. 9).

Compact cross-sections: Compact cross-sections are those which can develop their full-plastic moment Mp but where the local buckling prevents the required rotation at this moment to take place (β1< b/t < β2).

Semi-compact cross-sections: Semi-compact cross-sections are those in which the stress in the extreme fibers should be limited to yield stress because local buckling would prevent the development of the full-plastic moment Mp. Such sections can develop only yield moment My (β2< b/t ≤ β3).

Slender cross-sections: Slender cross-sections are those in which yield in the extreme fibers cannot be attained because of premature local buckling in the elastic range (β3 < b/t).

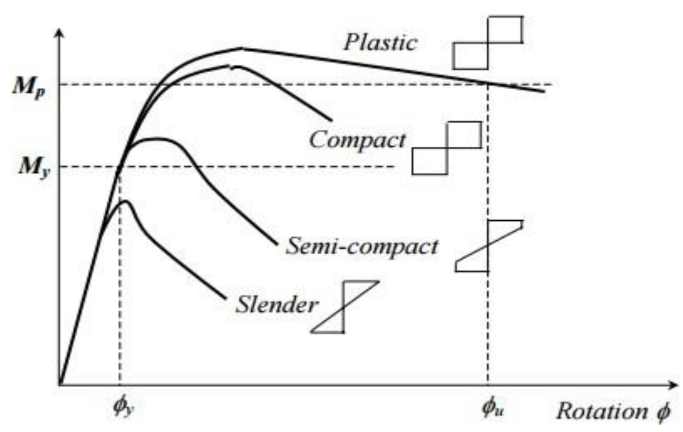

Fig. 8 Section Classification based on Moment-Rotation Characteristics

It should be remembered that even for steels with a large yield plateau, some strain hardening effects are likely to take place and the maximum moment is likely to be larger than Mp for plastic and compact sections. In such cases, the rotation capacity may be taken as the ratio of the rotation when the moment capacity drops back to Mp to the rotation at yield.

The relationship between the moment capacity Mu and the compression flange slenderness b/t indicating the β limits is shown in Fig. 9. In this figure, the value of Mu for semi-compact sections is conservatively taken as My. In the above classification, it is assumed that the web slenderness d/t is such that its buckling before yielding is prevented. It should be noted that the entire web may not be in uniform compression and if the neutral axis lies in the web, a part of the web may actually be in tension.

In this case, the slenderness limits are somewhat relaxed for the webs. Since the above classification is based on bending, it cannot be used for a compression member. The only criterion required is whether the member is slender or not. However, in practice, it is considered to be prudent to use compact or plastic sections for members carrying predominantly compressive loads.

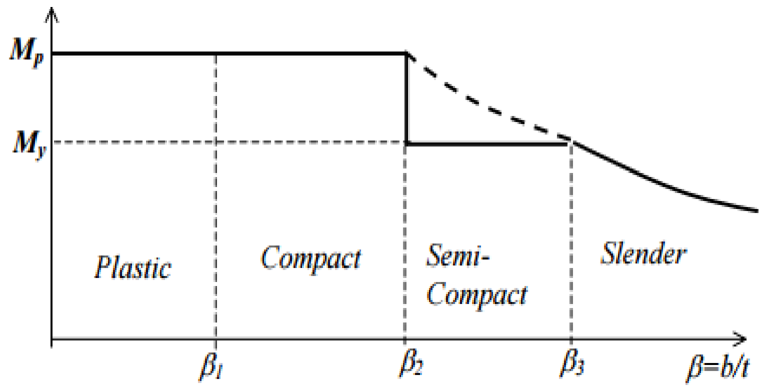

Fig 9. Section classification based on b/t ratio

LIMITS ON WIDTH-THICKNESS RATIO:

If the flanges and webs of cross-sections are considered to be plates under compression, their limiting width-thickness ratios can be obtained by equating the critical buckling stress to the yield stress. However, such an approach disregards a number of factors such as the actual support restraint provided by the adjoining plate element and the residual stresses and initial imperfections.

Therefore, the limiting width-thickness ratios β1 , β2 and β3 are useful for designers and are normally arrived at by validation in the testing laboratory. The limiting width-thickness ratios for different sections as per IS: 800, 2007 are given in Table 3.

The various extents of widths and thicknesses for different cross sections have been defined in Fig 10. Local buckling can be prevented, by controlling the width-thickness ratio. One way of doing this is by adopting higher thickness of the plate.

This method is adopted in rolled steel sections. However in the case of built-up sections and cold-formed sections, longitudinal stiffeners are provided which divide the total width into a number of smaller widths. The buckling of stiffened plates is beyond the scope of this chapter.

Table 3.Limits on width to thickness ratio of plate elements

<table>
<tr><th colspan="3" rowspan="2">Compression element</th><th rowspan="2">Ratio</th><th colspan="3">Class of Section</th></tr>
<tr><th>Plastic (β_1)</th><th>Compact (β_2)</th><th>Semi-compact (β_3)</th></tr>
<tr><td rowspan="2">Outstanding element of compression flange</td><td colspan="2">Rolled section</td><td>b/t_f</td><td>9.4ε</td><td>10.5ε</td><td>15.7ε</td></tr>
<tr><td colspan="2">Welded section</td><td>b/t_f</td><td>8.4ε</td><td>9.4ε</td><td>13.6ε</td></tr>
<tr><td rowspan="2">Internal element of compression flange</td><td colspan="2">Compression due to bending</td><td>b/t_f</td><td>29.3ε</td><td>33.5ε</td><td rowspan="2">42ε</td></tr>
<tr><td colspan="2">Axial compression</td><td>b/t_f</td><td colspan="2">Not applicable</td></tr>
<tr><td rowspan="4">Web of an I-H-or box section [c]</td><td colspan="2">Neutral axis at mid-depth</td><td>d/t_w</td><td>84ε</td><td>105ε</td><td>126ε</td></tr>
<tr><td rowspan="2">Generally</td><td>If r_1 is negative:</td><td>d/t_w</td><td rowspan="2">$\frac{84\varepsilon}{1+r_1}$
but $\leq 42\varepsilon$</td><td>$\frac{105.0\varepsilon}{1+r_1}$</td><td rowspan="3">$\frac{126.0\varepsilon}{1+2r_2}$
but $\leq 42\varepsilon$</td></tr>
<tr><td>If r_1 is positive :</td><td>d/t_w</td><td>$\frac{105.0\varepsilon}{1+1.5r_1}$
but $\leq 42\varepsilon$</td></tr>
<tr><td colspan="2">Axial compression</td><td>d/t_w</td><td colspan="2">Not applicable</td></tr>
<tr><td colspan="3">Web of a channel</td><td>d/t_w</td><td>42ε</td><td>42ε</td><td>42ε</td></tr>
<tr><td colspan="3">Angle, compression due to bending
(Both criteria should be satisfied)</td><td>b/t
d/t</td><td>9.4ε
$9.4\ \varepsilon$</td><td>10.5ε
10.5ε</td><td>15.7ε
15.7ε</td></tr>
<tr><td colspan="3">Single angle, or double angles with the components separated, axial compression
(All three criteria should be satisfied)</td><td>b/t
d/t
$(b+d)/t$</td><td colspan="2">Not applicable</td><td>15.7ε
15.7ε
25ε</td></tr>
<tr><td colspan="3">Outstanding leg of an angle in contact back-to-back in a double angle member</td><td rowspan="2">d/t</td><td rowspan="2">9.4ε</td><td rowspan="2">10.5ε</td><td rowspan="2">15.7ε</td></tr>
<tr><td colspan="3">Outstanding leg of an angle with its back in continuous contact with another component</td></tr>
<tr><td>Circular tube subjected to moment or axial compression</td><td colspan="2">CHS or built by welding</td><td>D/t</td><td>$44\varepsilon^2$</td><td>$55\varepsilon^2$</td><td>$88\varepsilon^2$</td></tr>
<tr><td colspan="3">Stem of a T-section, rolled or cut from a rolled I-or H-section</td><td>D/t_f</td><td>8.4ε</td><td>9.4ε</td><td>18.9ε</td></tr>
<tr><td colspan="7">Note 1: Section having elements which exceeds semi-compact limits are to be taken as slender cross sections
Note2: $\varepsilon=(250/f_y)^{1/2}$
Note 3: Check webs for shear buckling in accordance when $d/t > 67\ \varepsilon$. Where, b is the width of the element may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate, t is the thickness of element, d is the depth of the web, D mean diameter of the element,
Note 4: Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favorable classification.
Note 5: The stress ratio r_1 and r_2 are defined as
$r_1=\frac{\text{actual average axial compressive stress}}{\text{design compressive stress of web alone}}$, $r_2=\frac{\text{actual average axial compressive stress}}{\text{design compressive stress of overall section}}$</td></tr>
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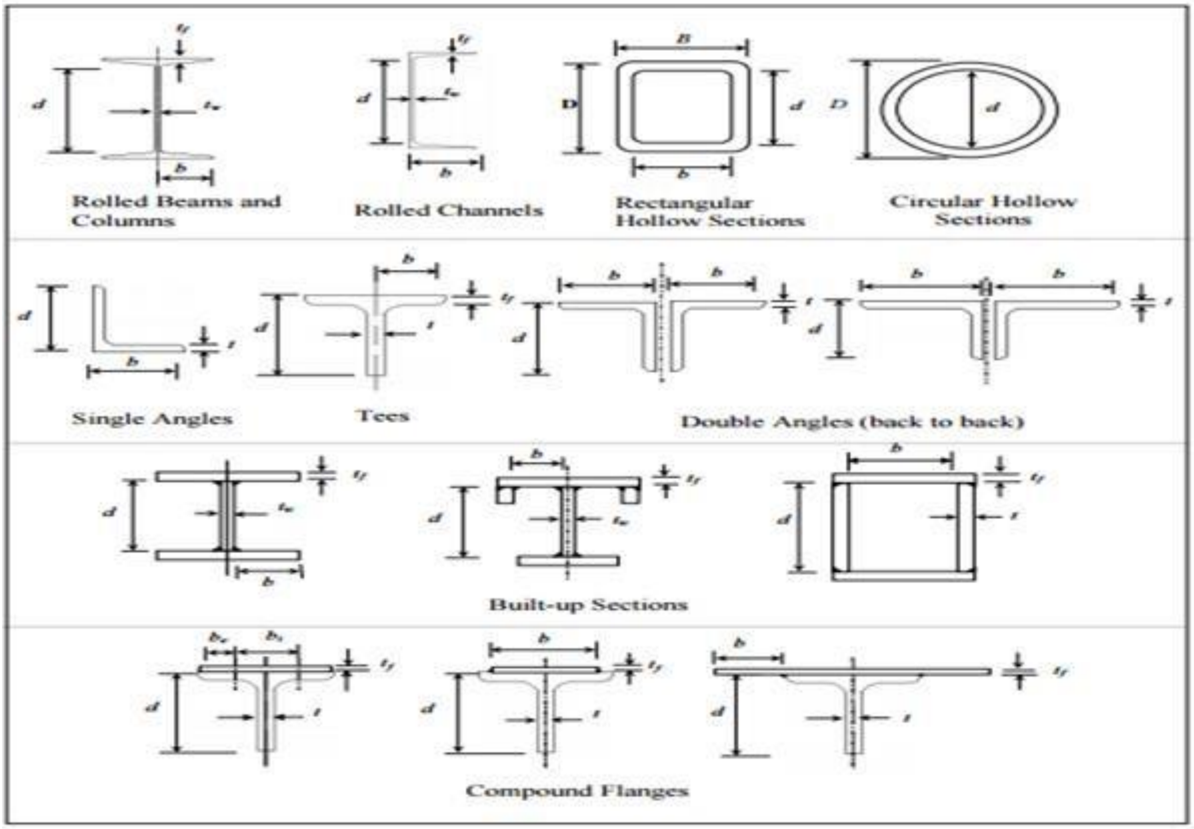

Fig 10.Dimensions of sections

It may be noted that semi-compact and slender members cannot be used in plastic design. In fact, only plastic sections can be used in indeterminate frames forming plastic collapse mechanisms while compact sections can be used in simply supported beams failing after reaching Mp at one section.

In elastic design, semi-compact sections may be used with the understanding that they will fail at My. Slender sections also have a stiffness problem and are normally not preferred in hot-rolled structural steel work. However, they are extensively used in cold-formed members and the manufacturer’s literature may be consulted while using them. Plate girders are usually designed taking advantage of the tension field approach to achieve economy.

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SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT – II- CONNECTIONS – SCI1303

CONNECTIONS

Advantages of Bolted Connections

1. The erection of the structure can be speeded up.
2. Less skilled workers are required
3. Cheaper compared to riveted connections due to reduced labour and equipments required.

Disadvantages of Bolted Connections

1. Cost of material is high, double the cost of rivets
2. The tensile strength is reduced because of area reduction at the root of the thread and due to stress concentration.
3. Normally, these are of a loose fit excepting turned bolts and hence their strength is reduced.
4. When subjected to vibration or shock, bolts may get loose.

Types of Bolts

i. Unfinished Bolts
ii. High Strength Bolts

Unfinished bolts: The bolts are available in 5 mm to 36 mm diameter designated as M5 to M36. The ratio of net area to nominal plain shank area of bolt is 0.78.

The bolts are available in Grade 4.6 and 8.8

Here in Grade 4.6

4 indicates the 1/100th the nominal ultimate tensile strength of steel

0.6 indicates the ratio of Yield stress to Ultimate tensile Strength

Thus Ultimate Tensile Strength of Class 4.6 bolt is 400 N/mm^2 and yield strength is 240 N/mm^2 (0.6 x 400).

These bolts are called as Bearing- Type Joints as the force is transferred by bearing and interlocking of bolts.

High Strength Bolts: These Bolts are also called as friction type bolts

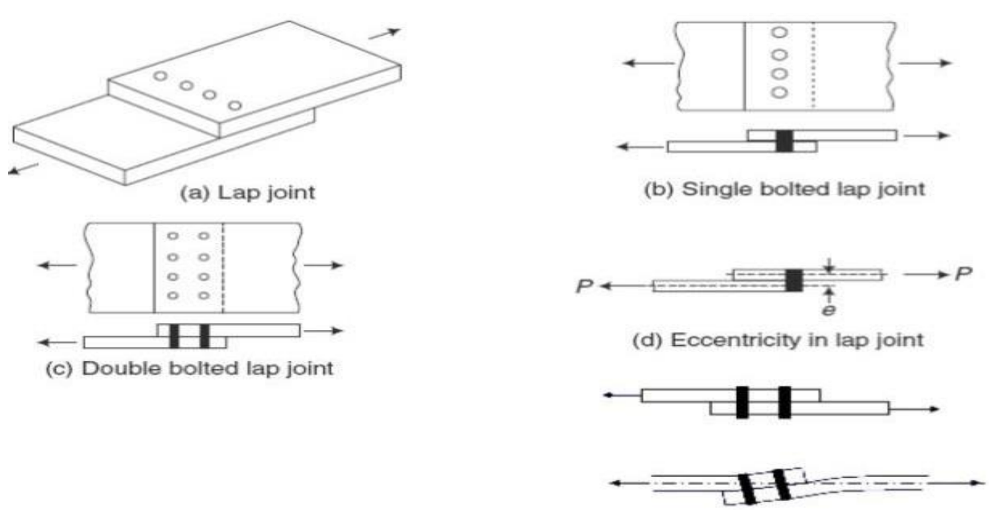
Advantages of High Strength Bolts (or) High Strength Friction Bolts (HSFG)

i. HSFG bolt can provide a rigid joint. There is no slip between the elements connected.
ii. Large tensile forces are developed in bolts, which provide large clamping force to the connected elements.
iii. Bolts are not subjected to shear or bearing
iv. The possibility of failure at net section is minimized
v. No stress concentration in holes therefore the fatigue strength is more.
vi. Few person are required to make connections, thus the cost is minimized
vii. Alterations are easy to make.
viii. For same, Strength lesser bolts are required as compared to rivets/ ordinary bolts which bring overall economy.

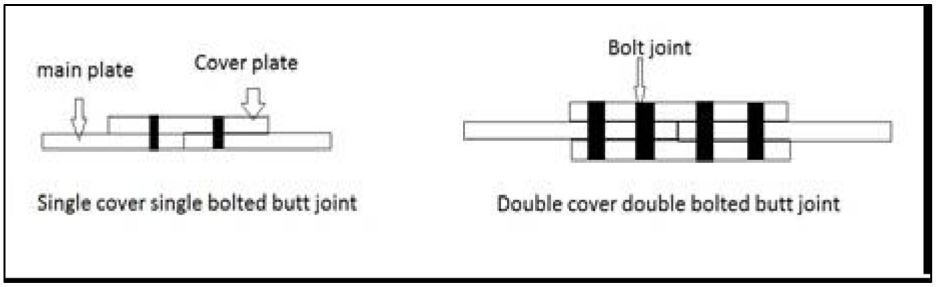
<u>Types of Bolted Joints</u>

i. Lap Joint
    - Single Bolted Lap Joint
    - Double Bolted Lap Joint   Lap Joints in Single Shear
    - Eccentricity in Lap Joint

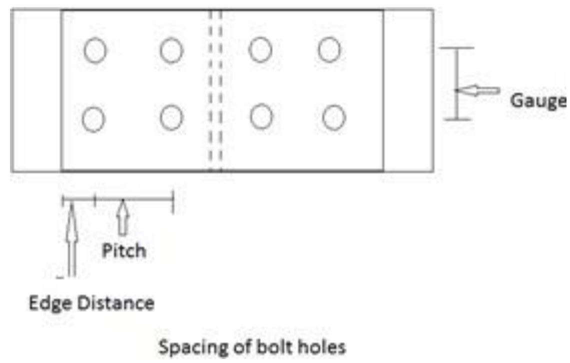
ii. Butt Joint
    - Single Cover Single Bolted Butt Joint
    - Single Cover Double Bolted Butt Joint
    - Double Cover Single Bolted Butt Joint   Butt Joints in Double Shear.
    - Double Cover Double Bolted Butt Joint



TYPES OF LAP JOINTS



TYPES OF BUTT JOINT



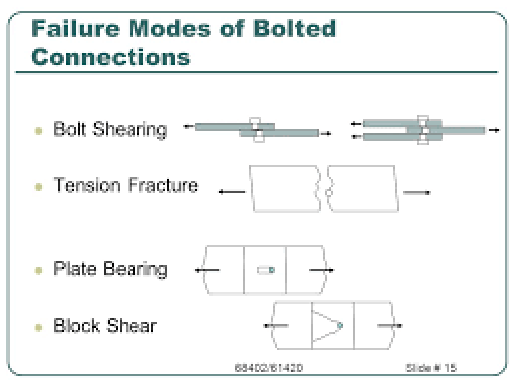
Terminology for Bolted Joints

Pitch: It is the distance between the centers of two consecutive bolts along a row

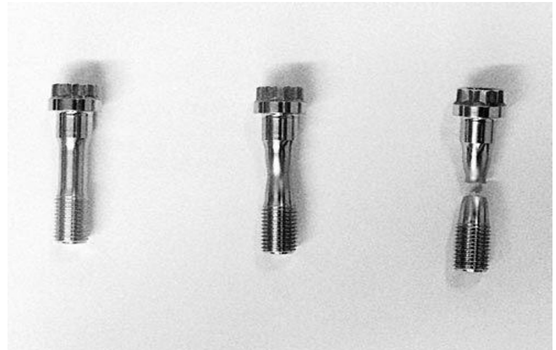
Gauge Length: It is the distance between adjacent bolt lines or distance between the back of the rolled section and the bolt line, or centre to centre distance between two consecutive bolts measured along the width of the member or connection.

Failure of Bolts

i. Shear Failure of Bolts
ii. Bearing Failure of Bolts
iii. Bearing Failure of Plates
iv. Tension Failure of Bolts
v. Tension or Shear Failure of Plates
vi. Block Shear Failure.



FAILURE OF BOLT IN DOUBLE SHEAR



FAILURE OF BOLTS DUE TO TENSION

Advantages of Welded Connections.

i. Welded Designs offer the opportunity to achieve a more efficient use of materials. Welding is the only process that produces one piece construction.
ii. The speed of fabrication and erection helps compress production schedules
iii. Welding saves weight and reduces cost.
iv. No deductions are there for holes; thus the gross area is effective in carrying loads
v. Welded Joints are better in fatigue, impact and vibration loads

Welding Defects

1. Incomplete Fusion
2. Incomplete Penetration
3. Porosity
4. Slag Inclusion
5. Under Cutting

Assumptions in Analysis of Welded Joints

1. The welds connecting the various parts are homogenous, isotropic and elastic elements
2. The parts connected by the weld are rigid and therefore deformations are neglected.
3. Only stresses due to external loads are considered. The Effects of residual stresses, stress concentrations and shape of the welds are neglected.

Fillet Vs Butt Weld

Fillet Weld is preferred in comparison to butt weld for the following reasons.

1. A fillet weld saves the operation of veeing and finishing the ends of the members
2. In Case of butt weld, members are fabricated slightly long and cut exactly to have a close fit in field. This is uneconomical.
3. Fillet Welds have lower residual stress.

Welded Joints vs Bolted vs Riveted Joints

1. Welded Joints are economical because the need for splice plates and bolts/ rivets is eliminated.
2. Welded Joints are more rigid (due to continuity of section at joints)
3. Strength of welded Joint is same as the parent metal.
4. It's possible to connect tubular sections with welded joints, which are economical
5. Due to the fusion of two metals joined, a continuous structure is formed
6. Process of Welding is faster compared to bolting and riveting
7. More Skilled labour is required as compared to bolting or riveting
8. Inspection of welding is difficult and expensive, whereas bolted and riveted can be inspected by tapping with hammer.
9. Efficiency of Welding is higher than bolted or welded. Its even possible to achieve 100 % efficiency by proper welding.
10. Possibility of brittle failure is higher in case of welding than compared to bolted or riveted

Solved Problems

Question 1

Calculate the strength of a 20 mm diameter bolt of grade 4.6 for the following cases. The main plates to be joined are 12mm thick.

a) Lap Joint
b) Single Cover Butt Joint; the cover plate being 10 mm thick
c) Double Cover Butt Joint; each of the cover plate being 8 mm thick.

For Fe 410 grade of steel fu = 410 Mpa = 410 N/mm2
For bolts of grade 4.6: fub = 400 Mpa = 400 N/mm2
γ_{mb} = partial safety factor for the material of bolts = 1.25
A_{nb} = Net tensile stress area of 20 mm diameter bolt = $0.78 \times \frac{\pi D^2}{4} = 245\ mm^2$

(a) The bolts will be in single shear and bearing
Diameter of bolts d = 20 mm
The strength of bolt in single shear
$V_{sb} = A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_b} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26$ kN
The Strength of bolt in bearing,
$V_{pb} = 2.5\, k_b\, bt \frac{f_u}{\gamma_{mb}}$

For bolt of diameter 20 mm , diameter of opening do = 22 mm and e= 33 mm. Assume pitch= 50 mm (2.5 x 20 mm)

K_B is the least of

$\frac{e}{3d_o} = \frac{33}{3 \times 22} = 0.5$

$\frac{p}{3d_o} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.5$

$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

And 1

Hence $k_b = 0.5$

$V_{pb} = 2.5 \text{ x } 0.5 \text{ x } 12 \text{ x } \frac{400}{1.25} \text{ x } 10^{-3} = 96$ kN.

The strength of bolt will be the minimum of the bolt in shear and bearing will be 45.26 kN.

b) The thickness of plate t= 10 mm.
The strength of bolt in bearing

$V_{pb} = 2.5\, k_b\, bt \frac{f_u}{\gamma_{mb}} = 2.5 \text{ x } 0.5 \text{ x } 20 \text{ x} 10 \text{ x } \frac{400}{1.25} \text{x } 10^{-3} = 80$ kN.

The minimum strength of bolt in bearing and shear is 45.26 kN.

c) The bolt will be in single shear and bearing

The strength of bolt in double shear.

$V_{sb} = 2 \text{ x } A_{nb} \frac{f_{ub}}{\sqrt{3}\, \gamma_b} = 2 \text{ x} 245 \text{ x } \frac{400}{\sqrt{3} \times 1.25} \text{ x } 10^{-3} = 90.52$ kN

The bolt strength in bearing.

$V_{pb} = 2.5\, k_b\, bt \frac{f_u}{\gamma_{mb}} = 2.5 \text{ x } 0.5 \text{ x } 12 \text{ x } \frac{400}{1.25} \text{ x } 10^{-3} = 96$ kN

The Minimum strength of bolt in shear and bearing is 90.52 kN.

Question 2

Two plates of 16 mm and 14 mm thickness are joined together by a groove weld. The joint is subjected to a factored load tensile load of 430 kN. Due to some reasons the effective length of weld that was provided was 175 mm, Check the safety of the joint if

a) Single V groove joint is provided
b) Double V Groove Joint is provided

a) Assume the plates are to be shop welded. ($\gamma_{mw} = 1.25$)

For the 410 grade of steel, $f_y = 250$ Mpa

Throat thickness, $t_e = (5/8)t = (5/8) \times 14 = 8.75$ mm
For Shop weld partial safety factor for material

Effective length of weld $L_w = 175$ mm

$$T_{dw} = L_w t_e \frac{f_y}{\gamma_{mw}} = 175 \times 8.75 \times \frac{250}{1.25} \times 10^{-3} = 306.25 \text{ kN} < 430 \text{ kN}$$

Which is inadequate.

b) In case of double groove V weld complete penentration of the weld takes place. Therefore as per specification,

Throat Thickness = $t_e = 14$ mm.

$$T_{dw} = L_w t_e \frac{f_y}{\gamma_{mw}} = 175 \times 14 \times \frac{250}{1.25} \times 10^{-3} = 490 \text{ kN} > 430 \text{ kN}$$

The provided Section is adequate.

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SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT –III- Design Of Tension Members – SCI1303

DESIGN OF TENSION MEMBERS

Types of Tension members – Net area – Net effective sections – concept of shear lag – Design strength of simple and compound members – Use of lug angles – Design of tension splices.

CODE BOOK: IS 800: 2007
DESIGN AID: Indian Standard Handbook No.1 (otherwise known as) Steel Tables.
REFERENCE BOOK: Limit State Design of Steel Structures by S K Duggal
International Codal Reference : Euro Code 3 (Not Allowed in Examination Hall)

TENSION MEMBERS (IS 800:2007 Cl 6.1)

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads upto the ultimate load, at which stage they may fail by rupture at a critical section.

Types of Tension members

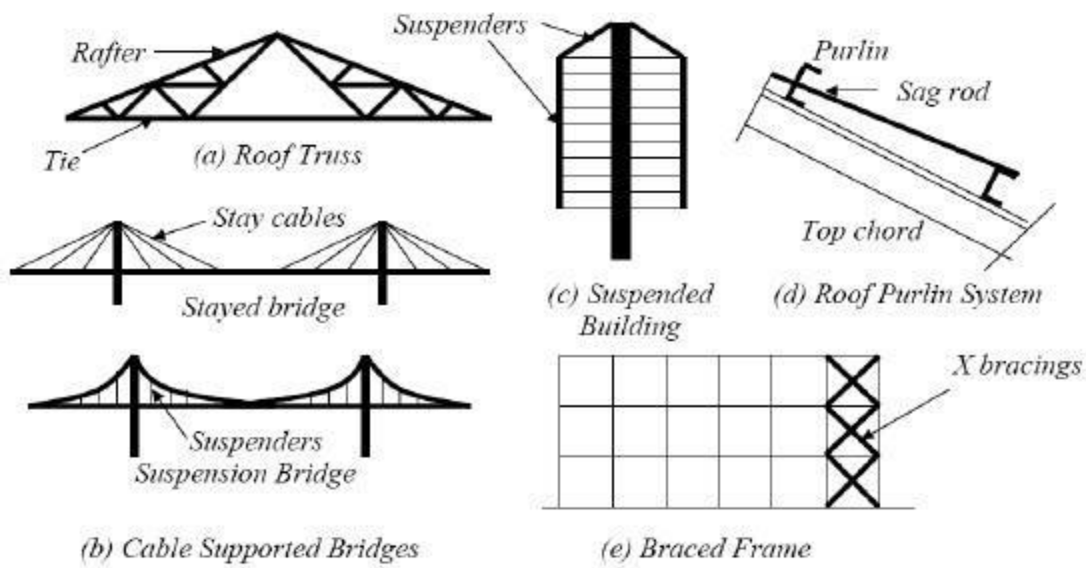

Fig. 1 Tension Members in Structures

Ties of trusses [Fig 1(*a*)], suspenders of cable stayed and suspension bridges [Fig.1 (*b*)], suspenders of buildings systems hung from a central core [Fig.1(*c*)] (such buildings are used in earthquake prone zones as a way of minimising inertia forces on the structure), and sag rods of roof purlins [Fig 1(*d*)] are other examples of tension members. Tension members are also encountered as bracings used for the lateral load resistance. In X type bracings [Fig.1 (*e*)] the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice versa. Hence, such members may have to be designed to resist tensile and compressive forces.

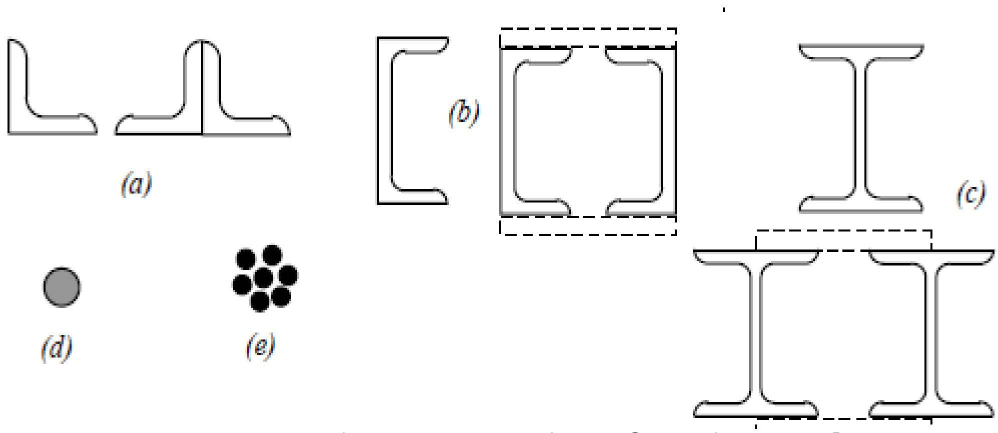

Fig. 2 Cross Sections of Tension Members

The tension members can have a variety of cross sections. The single angle and double angle sections [Fig 2(*a*)] are used in light roof trusses as in industrial buildings. The tension members in bridge trusses are made of channels or I sections, acting individually or built-up [Figs. 2(*c*) and 2(*d*)]. The circular rods [Fig.2 (*d*)] are used in bracings designed to resist loads in tension only. They buckle at very low compression and are not considered effective. Steel wire ropes [Fig.2 (*e*)] are used as suspenders in the cable suspended bridges and as main stays in the cable-stayed bridges.

Net area & Net effective sections

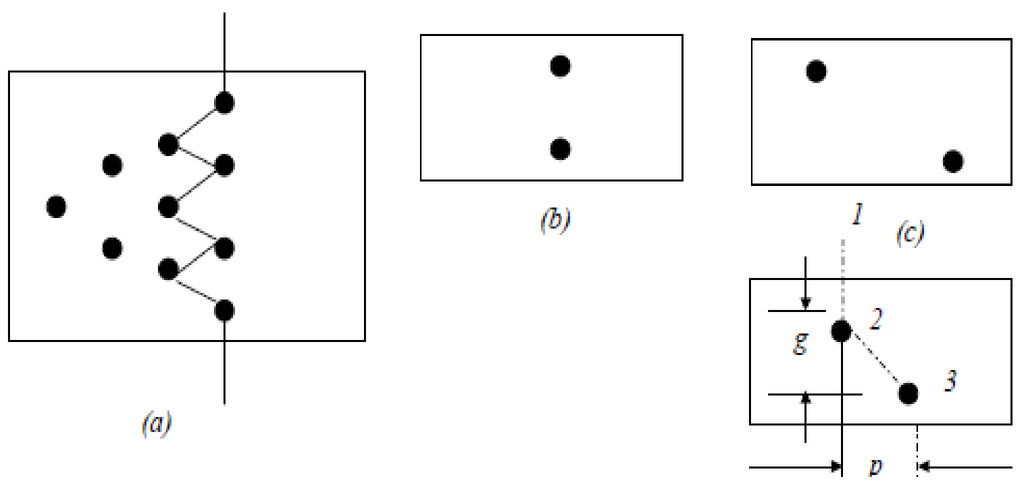

Fig. 3.Plates with Bolt Holes under Tension

plates have more than one hole for the purpose of making connections. These holes are usually made in a staggered arrangement [Fig.3 (*a*)]. Let us consider the two extreme arrangements of two bolt holes in a plate, as shown in Fig.3 (*b*) & 3(*c*). In the case of the arrangement shown in Fig.3 (*b*), the gross area is reduced by two bolt holes to obtain the net area. Whereas, in arrangement shown in Fig.3c, deduction of only one hole is necessary, while evaluating the net area of the cross section.

Obviously the change in the net area from the case shown in Fig.3(*c*) to Fig.3 (*b*) has to be gradual. As the pitch length (the centre to centre distance between holes along the direction of the stress) p, is decreased, the critical cross section at some stage changes from straight section [Fig.3(*c*)] to the staggered section 1-2-3-4 [Fig.3 (*d*)]. At this stage, the net area is decreased by two bolt holes along the staggered section, but is increased due to the inclined leg (2-3) of the staggered section.

The net effective area of the staggered section 1-2-3-4 is given by

$$A_n=(b-2d+p^2/4g)t$$

When multiple holes are arranged in a staggered fashion in a plate as shown in Fig.6 (a), the net area corresponding to the staggered section in general is given by

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

where, *n* is the number of bolt holes in the staggered section [*n* = 7 for the zigzag section in Fig.6(*a*)] and the summation over $p^2/4g$ is carried over all inclined legs of the section [equal to *n-1* = 6 in Fig.3(*a*)]. Normally, net area of different staggered and straight sections have to be evaluated to obtain the minimum net area to be used in calculating the design strength in tension.

concept of shear lag

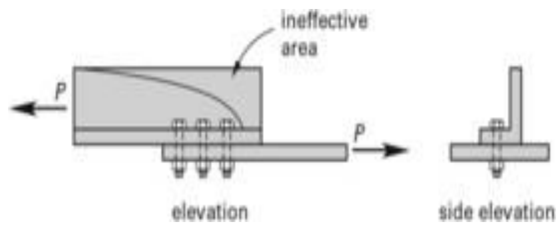

Fig.4 Shear lag effect shown on angle

The in plane shear deformation effect by which concentrated forces tangential to the surface of a plate gets distributed over the entire section perpendicular to the load over a finite length of the plate along the direction of the load.

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Design strength of simple and compound members

The design strength of tension member is the lowest of the following:

Design Strength Due to Yielding of Gross Section

Design Strength Due to Rupture of Critical Section

Design Strength Due to Block Shear

Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension,*Tdg*,as governed by yielding of gross section, is given by

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

Where

fy = yield stress of the material,

Ag = gross area of cross-section, and

g_0= partial safety factor for failure in tension by yielding .

Design Strength Due to Rupture of Critical Section

Plates

The design strength in tension of a plate, *Tdn,* as governed by rupture of net cross-sectional

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

area, *An,* at the holes is given by

where

g_{ml}= partial safety factor for failure at ultimate stress (see Table 5 in IS 800:2007),

fu = ultimate stress of the material, and
An = net effective area of the member given

Single Angles

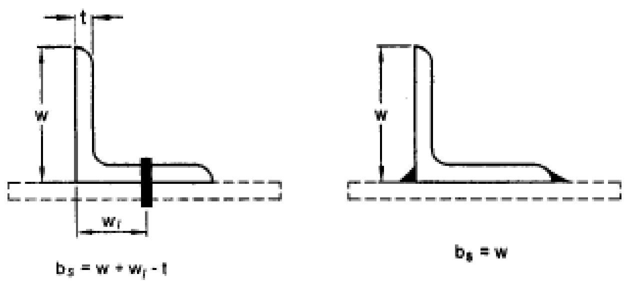


Fig. 5 Angles with single leg Connections

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

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

Where

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u \gamma_{m0} / f_y \gamma_{m1}) \\ > 0.7$$

Where

w = outstand leg width,
b_s, = shear lag width, as shown in Fig. 5, and
L_C =length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

For preliminary sizing, the rupture strength of net section may be approximately taken as:

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

Where

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

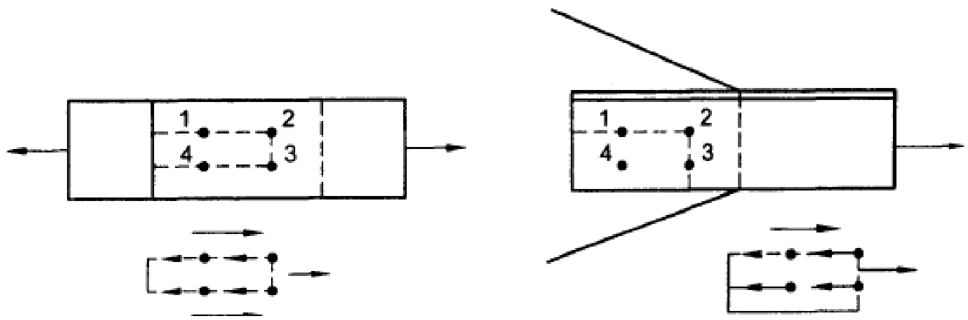
Other Section

The rupture strength, *Tdn,* 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 ,where β 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.

Design Strength Due to Block Shear

Bolted Connections

The block shear strength, *Tdb* of connection shall be taken as the smaller of,



6APlate 6B Angle

Fig.6 Block shear failure

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

or

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

A_{vg},A_{vn}= minimum gross and net area in shear along bolt line parallel to external force,respectively(1-2 and 3-4 as shown in Fig. 6A and 1-2 as shown in Fig. 6B),

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle,end bolt line, perpendicular to the line of force, respectively (2-3 as shown inFig. 6B), and

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

Welded Connection

The block shear strength, Tdb shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

Use of lug angles

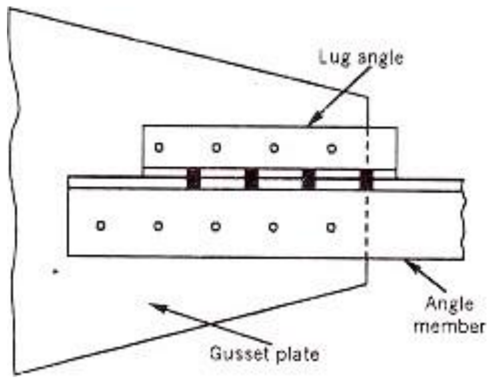


Fig 7 Lug Angle

A larger length of the tension member and the gusset plate may be required sometimes to accommodate the required number of connection rivets. But this may not be feasible and economical. To overcome this difficulty lug angles are used in conjunction with main tension members at the ends. It provides extra gauge lines for accommodating the rivets and thus enables to reduce the length of the connection. They are generally used when the members are of single angle, double angle or channel sections.

Main objectives of the lug angles

1. They produce eccentric connections, due to rivets placed along lug angle. The centroid of the rivet system of the connection shifts, causing eccentric connection and bending moments.
2. Stress distribution in the rivets connecting lug angles is not uniform. It is preferred to put a lug angle at the beginning of the connection where they are more effective and not at the middle or at the end of the connection.
3. Rivets on the lug angles are not as efficient as those on the main member. The Out-standing leg of the lug angle usually gets deformed and so the load shared by the rivets on the lug angles is proportionately less.

Is 800:2007 specification for lug angles are(clause 10.12)

The lug angles and their connections to the gusset or other supporting member shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the member, and the attachment of the lug angle to the main angle shall be capable of developing a strength not less than 40 percent in excess of the force in the outstanding leg of the angle.In the case of channel members and the like,the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10 percent in

excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20 percent in excess of that force.In no case shall fewer than two bolts, rivets or equivalent welds be used for attaching the lug angle to the gusset or other supporting member.

The effective connection of the lug angle shall,as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the main member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.Where lug angles are used to connect an angle member, the whole area of the member shall be taken as effective not withstanding the requirements of Section 6 of this standard.

Design of tension splices.

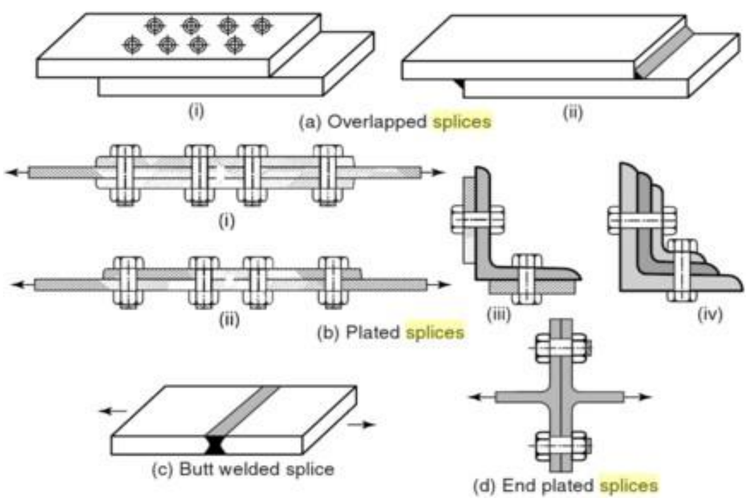


Fig.8 Typical splices in tension member

Splicing of tension members is necessary when the required length of the member is more than the length available or when the member has different cross-sections for different parts of its length. If actual member is to be of greater length, two or more lengths shall have to be spliced at the joints.when tension members of different thickness are to be connected, filler plates may be used to bring the members in level. The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor,β_{Pk}given by:

$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$

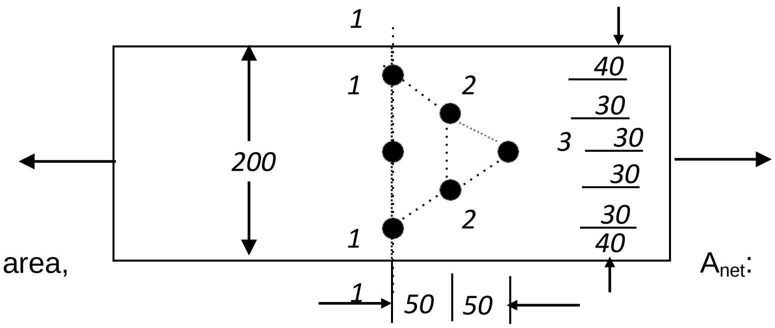
t_{pk} =thickness of packing plate.

PROBLEM 1:

Determine the design tensile strength of the plate (200 X 10 mm) with the holes as shown below, if the yield strength and the ultimate strength of the steel used are 250 MPa and 420 MPa and 20 mm diameter bolts are used.

f_y = 250 MPa

f_u = 420 MPa



Calculation of net area, A_{net}:

$$A_n\,(section\ 11) = (200 - 3*21.5)*10 = 1355\ mm^2\ (governs)$$

$$A_n\,(section\ 1221) = \left(200 - 4*21.5 + \frac{2*50^2}{4*30}\right)*10 = 1557\ mm^2$$

$$A_n\,(section\ 12321) = \left(200 - 5*21.5 + \frac{4*50^2}{4*30}\right)*10 = 1758\ mm^2$$

T_d is lesser of

i. $A_g.f_y / \gamma_{gm1} = \frac{200*10*250/1.10}{1000}$ = 454.55 kN

ii. $0.9.A_n.f_u / \gamma_{gm1} = \frac{0.9*1355*420/1.25}{1000}$ =409.75 kN

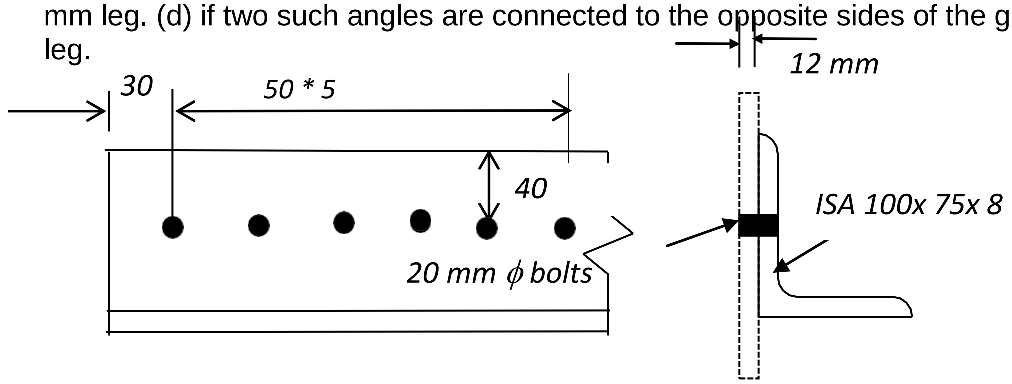
Therefore T_d = 409.75 kN

Efficiency of the plate with holes $= \frac{T_d}{A_g f_y/\gamma_{m0}} = \frac{409.75}{454.55} = 0.90$

PROBLEM 2:

Analysis of single angle tension members

A single unequal angle 100x 75x 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 nos. 20 mm diameter bolts to transfer tension. Determine the design tensile strength of the angle. (a) if the gusset is connected to the 100 mm leg, (b) if the gusset is connected to the 70 mm leg, (c) if two such angles are connected to the same side of the gusset through the 100 mm leg. (d) if two such angles are connected to the opposite sides of the gusset through 100 mm leg.



a) <u>The 100mm leg bolted to the gusset</u> :

A_{nc} = (100 - 8/2 - 21.5) *8 = 596 mm^2.

A_{go} = (75 - 8/2) * 8 = 568 mm^2

A_g = ((100-8/2) + (75 – 8/2)) * 8 =1336 mm^2

<u>Strength as governed by tearing of net section:</u>

β = 1.4 – 0.076 (w/t) (f_y/f_u) (b_s/L_c) ; (b_s = w + w_1 – t = 75 + 60 – 8 =127)

β = 1.4 – 0.076 * (75 / 8) * (250 / 420) * (127 / 250) = 1.18

T_{dn} = 0.9 A_{nc} f_u / γ_{m1} +β A_{go} f_y /γ_{m0}

= 0.9 * 596 * 420 / 1.25 + 1.18 * 568 * 250 / 1.10

= 333145 N (or) 333.1 kN

Strength as governed by yielding of gross section:

T_{dg} = A_g f_y /γ_{m0}

=1336 * 250 / 1.10 = 303636 N (or) 303.6 kN

Block shear strength

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

= {(5*50 + 30)*8*250 / ($\sqrt{3}$ * 1.1) + 0.9*(40 - 21.5/2)* 8*420 / 1.25

or

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

= {0.9*(5*50 + 30 – 5.5*21.5)*8*420 /($\sqrt{3}$ = 298648 N = 298.65 kN

*1.25) + 40*8*250 / 1.1}

The design tensile strength of the member = 298.65 kN

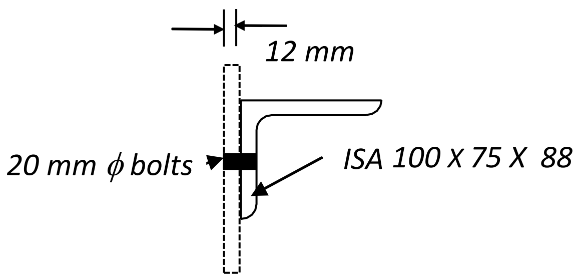
The efficiency of the tension member, is given by

$$\eta = \frac{T_d}{A_g f_y} = \frac{298.5 * 1000}{(100 + 75 - 8) * 8 * 250/1.10} = 0.983$$

b) The 75 mm leg is bolted to the gusset:

A_{nc} = (75 - 8/2 - 21.5) * 8 = 396 mm^2

A_{go} = (100 - 8/2) * 8 = 768 mm^2



Strength as governed by tearing of net section:

ISA 100x75x8

$\beta = 1.4 - 0.076\ (w/t)\ (f_y/f_u)\ (b_s/L_c)$; ($b_s = w + w_1 - t = 100 + 40 - 8 = 132$)

$\beta = 1.4 - 0.076 * (100 / 8) * (250 / 420) * (132 / 250) = 1.101$

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

$= 0.9 * 396 * 420 / 1.25 + 1.101 * 768 * 250 / 1.10$

$= 312000$ N (or) 312.0 kN

Strength as governed by yielding of gross section:

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

$= 1336 * 250 / 1.10 = 303636$ N (or) 303.6 kN

Block shear strength:

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

$= \{(5*50 + 30)*8*250 / (\sqrt{3} * 1.1) + 0.9*(35 - 21.5/2)* 8*420 / 1.25$

The design tensile strength of the member = 289.60 kN

The efficiency of the tension member, is given by

$$\eta = \frac{T_d}{A_g f_y} = \frac{289.6 * 1000}{(100 + 75 - 8) * 8 * 250/1.10} = 0.954$$

<u>Even though the tearing strength of the net section is reduced, the block shear failure still governs the design strength</u>.

The efficiency of the tension member is 0.954

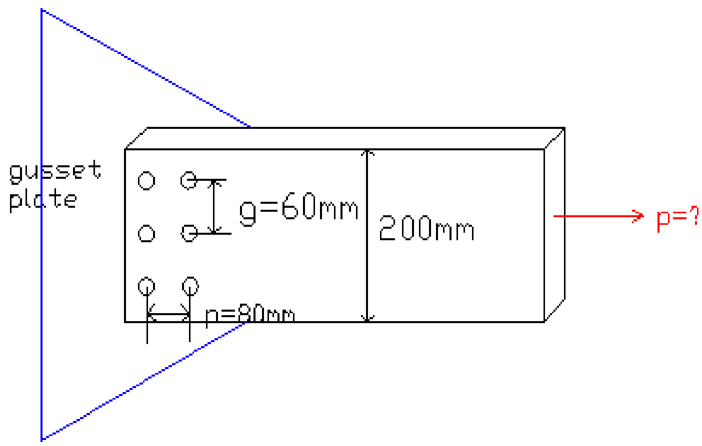
<u>Note</u>: The design tension strength is more some times if the longer leg of an unequal angle is connected to the gusset (when the tearing strength of the net section governs the design strength).

An understanding about the range of values for the section efficiency, η, is useful to arrive at the trial size of angle members in design problems.

(c & d)The double angle strength would be twice single angle strength as obtained above in case (a)

T_d = 2 * 298.65 = 597.30 kN

Problem 3:A tension member of flat plate having size 200mm x 10mm.It is connected to agusset plate with 6-bolt (3 bolt in each column of bolt) with chain riveting. Diameter of 16mmand gauge is 60mm, pitch of 80mm determine the axial tensile load.



Solution:

Agross **= b x t**

= 200 x 10

= 2000 mm^2

Anet = (b-(n x d)) x t

= (200- (3 x (16+2))) x 10

= (200-54) x10

=1460 mm^2

As per **LSM**

(i) According to **Yielding criteria**

Pu

= (fy/1.1) x Agross

= (250/1.1) x 2000

= 454.54 kN

(ii) According to **Rupture criteria**

Pu **= (0.9fu/ γ m1) x Anet**

= (0.9x410/1.25) x 1460

= 431 kN

Adopt lesser value, Pu = 431 kN.

SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

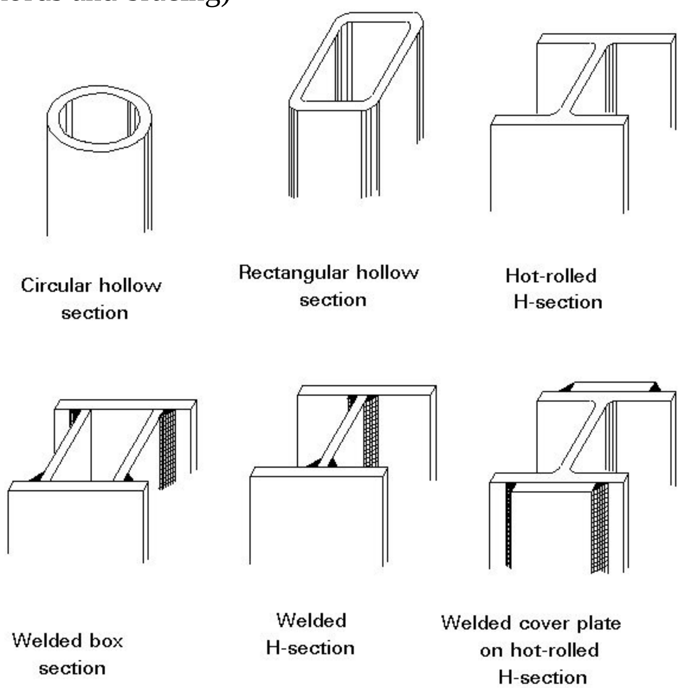
UNIT – IV- Compression Members – SCI1303

COMPRESSION MEMBERS

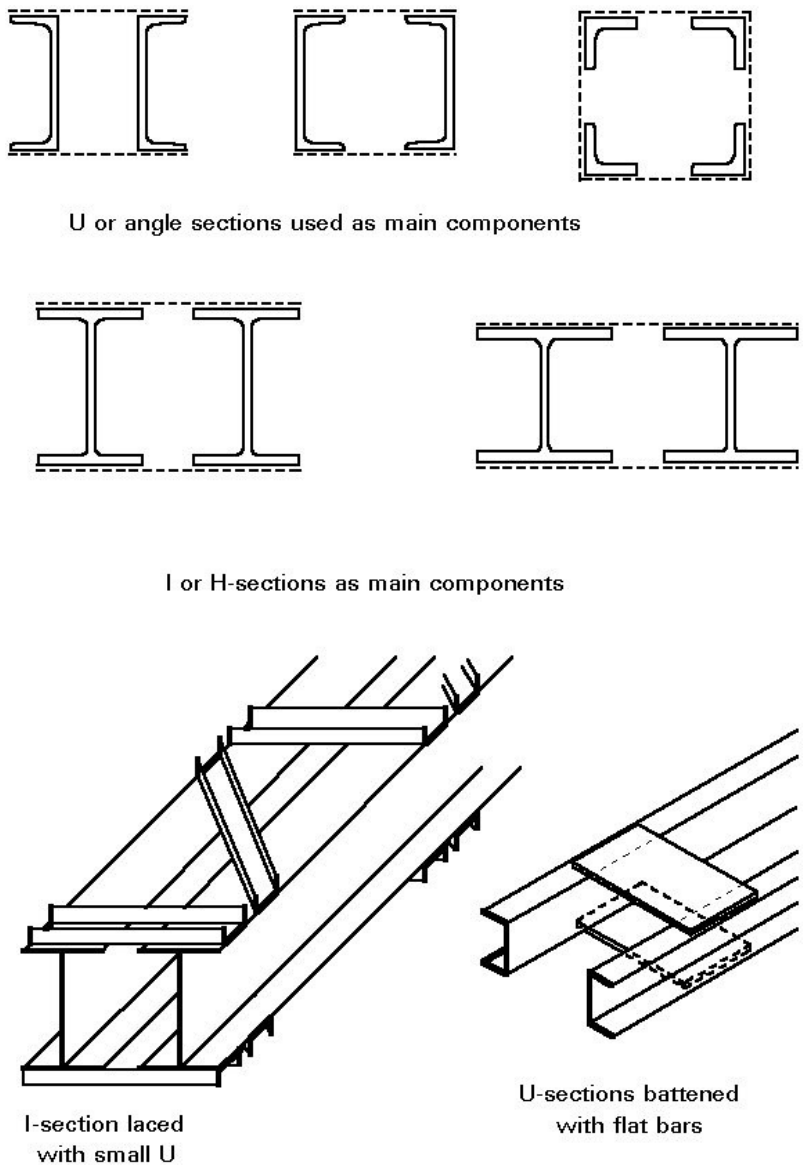
Introduction

Steel Compression members

- Building columns
- Frame Bracing
- Truss members (chords and bracing)



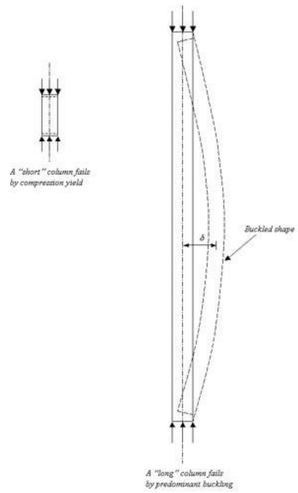
Different C/S of Columns



Built up Columns

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. (See Fig.1). The strut is said to have "buckled"



Short vs Long Columns

Buckling behaviour is thus characterized by large deformations developed in a direction (or plane) normal to that of the loading that produces it. When the applied loading is increased, the buckling deformation also increases. Buckling occurs mainly in members subjected to compressive forces. If the member has high bending stiffness, its buckling resistance is high. Also, when the member length is increased, the buckling resistance is decreased. Thus the buckling resistance is high when the member is short or "stocky" (i.e. the member has a high bending stiffness and is short) conversely, the buckling resistance is low when the member is long or "slender". Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load.

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 Robertsons 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.

Buckling

Elastic (Euler) buckling

Inelastic buckling

Buckling modes

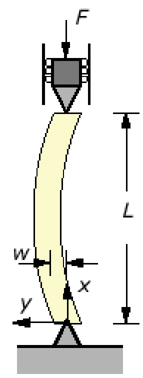
Overall buckling

Flexural buckling

Torsional buckling

Torsional-flexural buckling

Local buckling



Simply supported column subjected to axial load F

The design compressive strength Pd, of a member is given by

$$P < P_d$$

Where

$P_d = A_c f_{cd}$

Ac- Cross sectional area of the section as per code 7.3.2
Fcd-Design compressive stress obtained as per code 7.1.2.1.
The design compressive stress, fcd , of axially loaded compression members shall be calculated using the following equation:

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

where

ϕ = $0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 \Big/ \pi^2 E}$$

f_{cc} = Euler buckling stress = $\dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r;

α = imperfection factor given in Table 7;

χ = stress reduction factor (*see* Table 8) for different buckling class, slenderness ratio and yield stress

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

λ_{m0} = partial safety factor for material strength.

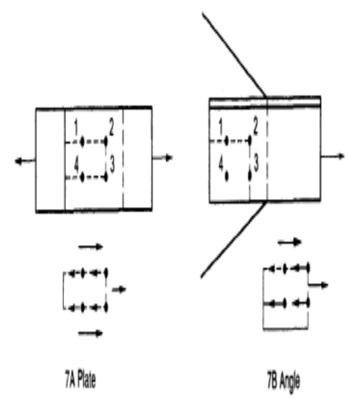


Fig. 7 Block Shear Failure

Effective Length of Compression Members

The effective length KL, is calculated from the actual length L, of the member, considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from centre-to-centre of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end, shall be taken as the actual length.

Effective Length

Where the boundary conditions in the plane of buckling can be assessed, the effective length, KL can be calculated on the basis of Table 11. Where frame analysis does not consider the equilibrium of a framed structure in the deformed shape (second-order analysis or advanced analysis), the effective length of compression members in such cases can be calculated using the procedure given in D-1. The effective length of stepped column in single storey buildings can be calculated using the procedure given in D-2.

Eccentric Beam Connection

In cases where the beam connections are eccentric in plan with respect to the axes of the column, the same conditions of restraint as in concentric connection shall be deemed to apply, provided the connections are carried across the flange or web of the columns as the case may be, and the web of the beam lies within, or in direct contact with the column section, Where practical difficulties prevent this, the effective length shall be taken as equal to the distance between points of restraint, in non-sway frames.

Compression Members in Trusses

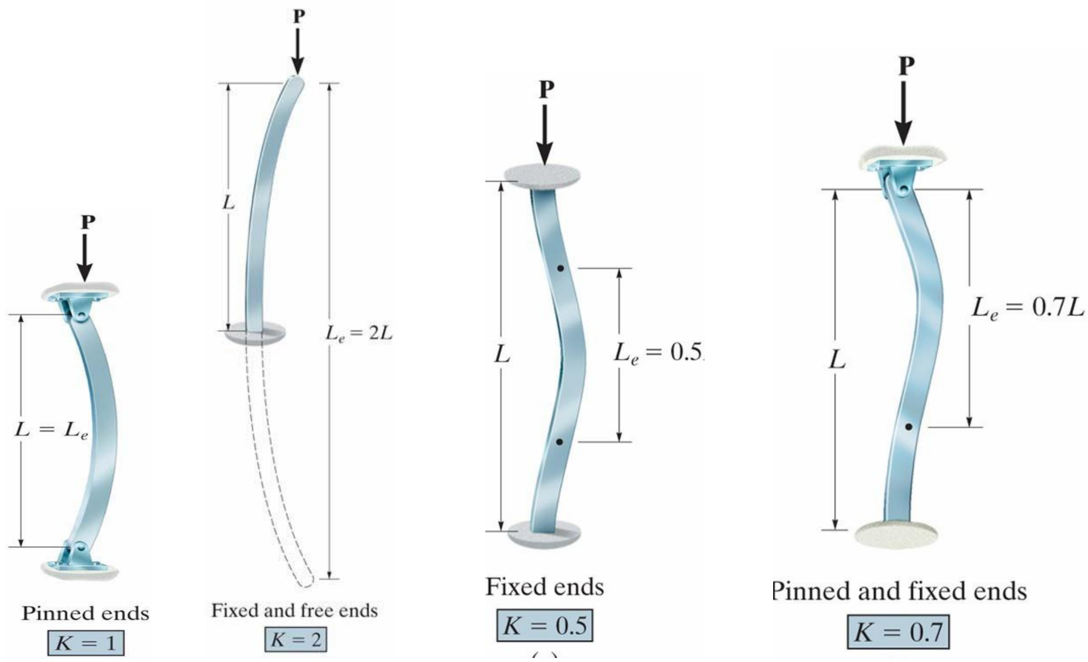
In the case of bolted, riveted or welded trusses and braced frames, the effective length, KL, of the compression members shall be taken as 0.7 to 1.0 times the distance between centres of connections, depending on the degree of end restraint provided. In the case of members of trusses, buckling in the plane perpendicular to the plane of the truss, the effective length, KL shall be taken as the distance between the centres of intersection.

Elastic Buckling

Equilibrium equation

Internal moment + applied moment = 0

Different end Conditions give different lengths for equivalent half-sine wave



The classification of different sections under different buckling class a, b, c or d, is given in Table 10.IS800-2007 The stress reduction factor X, and the design\ compressive stress~,~, for different buckling class, yield stress, and effective slenderness ratio is given in Table 8 for convenience. The curves corresponding to different buckling class are presented in nondimensional form, the following figure.

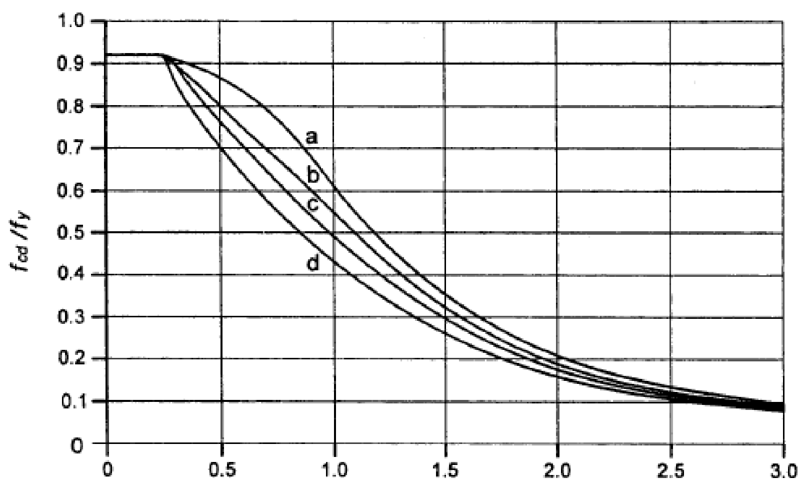


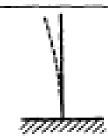
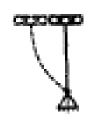
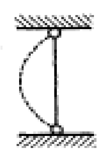
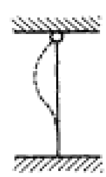
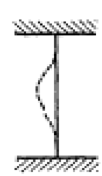
Table 7 Imperfection Factor, α

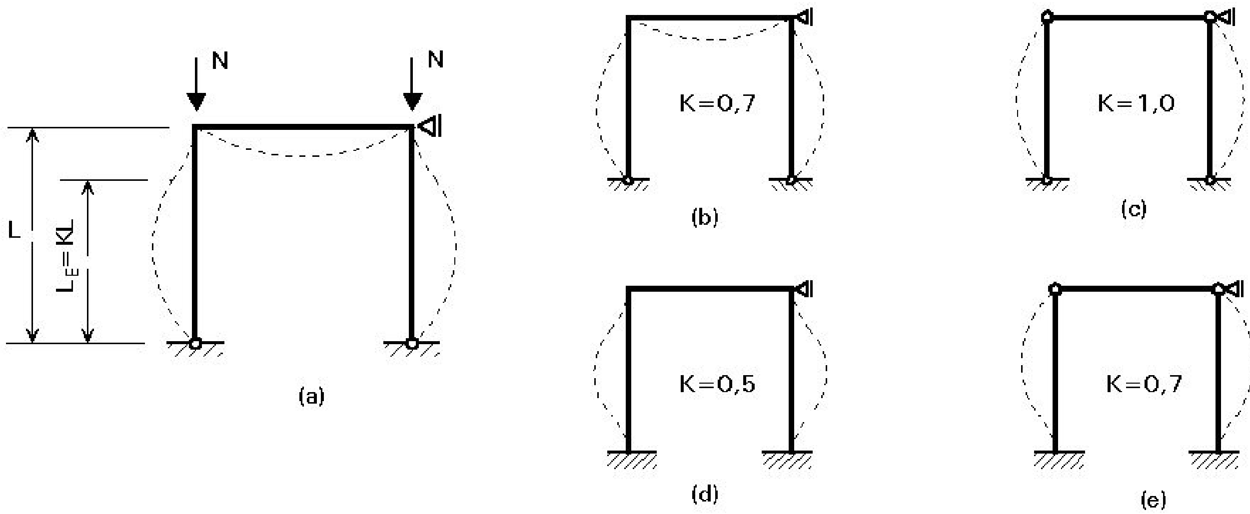
(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Table 11 Effective Length of Prismatic Compression Members

(*Clause* 7.2.2)

Boundary Conditions				**Schematic Representation**	**Effective Length**
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L



Buckling of a Column sway frame

Column Curve

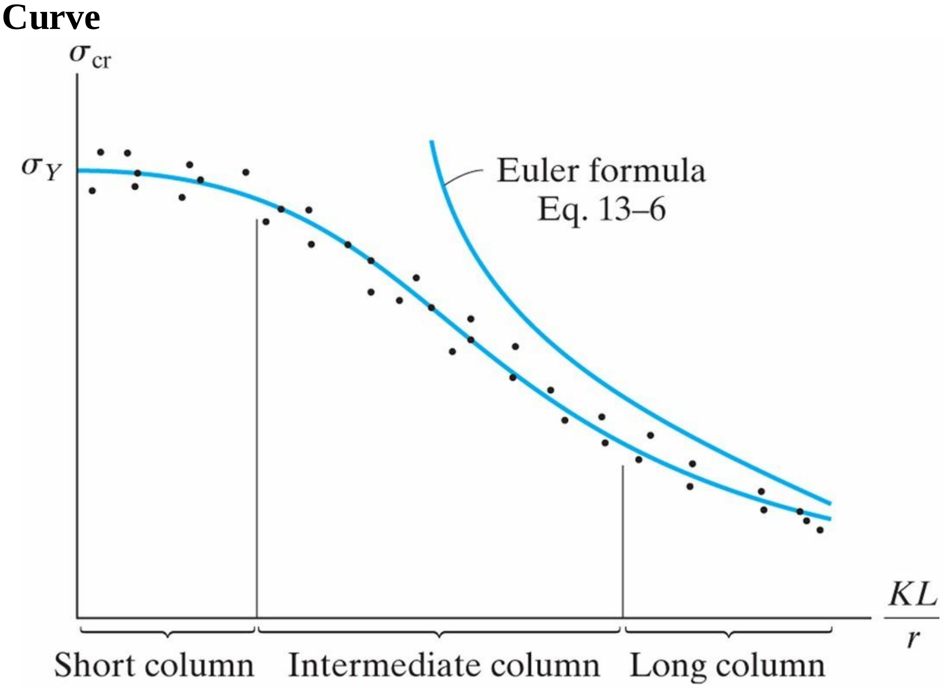
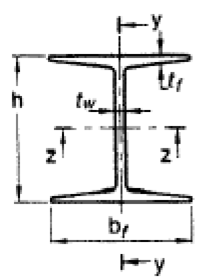
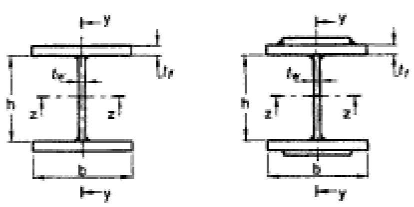
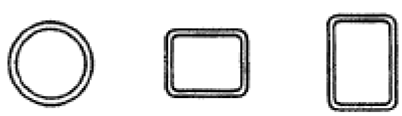
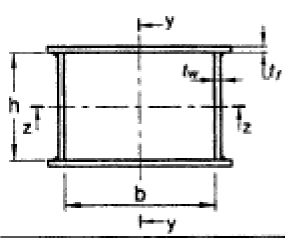
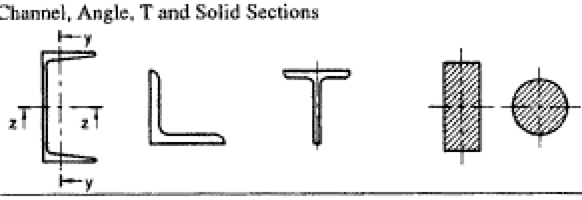
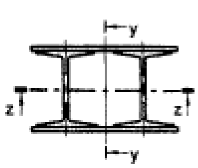


Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z y-y	c c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

Design of steel Column

Thickness of Plate Elements

Classification of members on the basis of thickness of constituent plate elements shall satisfy the widththickness ratio requirements specified in Table 2.

Effective Sectional Area, A,

Except as modified in 3.7.2 (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

Eccentricity for Stanchions and Columns

For the purpose of determining the stress in a stanchion or column section, the beam reactions orsimilar loads shall be assumed to be applied at an eccentricity of 100 mm from the face of the section or at the centre of bearing whichever dimension gives the greater eccentricity, and with the exception of the following two cases:

a) In the case of cap connection, the load shall be assumed to be applied at the face of the column or stanchion section or at the edge of packing, if used towards the span of the beam.

b) In the case of a roof truss bearing on a cap, no eccentricity be taken for simple bearings without connections capable of developing any appreciable moment. In case of web member connection with face, actual eccentricity is to be considered.

In continuous columns, the bending moments due to eccentricities of loading on the columns at any floor may be divided equally between the columns above and below that floor level, provided thaL the moment of inertia of one column section, divided by its effective length does not exceed 1.5 times the correspoi~ding value of the other column. Where this ratio is exceeded, the bending moment shall be divided in proportion to the moment of inertia of the column sections divided by their respective effective lengths.

Where the ends of compression members areprepared for bearing over the whole area, they shall bespliced to hold the connected members accurately inposition, and to resist bending or tension, if present. Such splices should maintain the intended member stiffness about each axis. Splices should be located as close to the point of inflection as possible. Otherwise their capacity should be adequate to carry magnified moment.

The ends of compression members faced for bearing shall invariably be machined to ensure perfectcontact of surfaces in bearing.

Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which the members are subjected.

Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members being jointed, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress shall be accounted for.

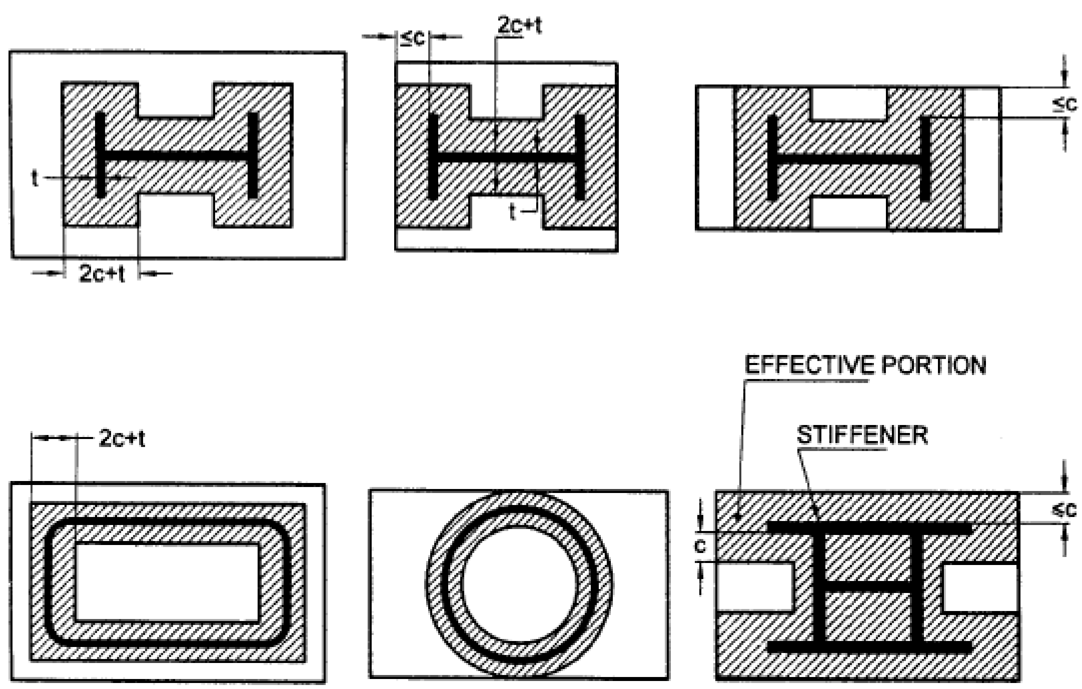
Column Bases

General

Column bases should have sufficient stiffness and strength to transmit axial force, bending moments and shear forces at the base of the columns to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should be provided wherever necessary, Shear resistance at the proper contact surface between steel base and concrete/grout may be calculated using a friction coefficient of 0.45.

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure should not exceed the bearing strength equal to 0.6~C~,wheref,~ is the smaller of characteristic cube strength of concrete or bedding
material.

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection c of the base plate beyond the Pace of the column and gusset may be taken as effective in transferring the column load as given in below figure such that bearing pressure on the effective area does not exceed bearing capacity of concrete base.



Gusseted Bases

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc, in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without ex~eeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact. Where the ends of the column shaft and the gusset plates are not faced for complete bearing, the weldings, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

Column and base plate connections

Where the end of the column is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected.

Slab Bases

Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection,

The minimum thickness, t,, of rectangular slab bases, supporting columns under axial compression shall be

$$ts= \sqrt{2.5w(a^2 - 0.3b^2)\gamma mo/fy} > tf$$

where

w = uniform pressure from below on the slab base under the factored load axial compression;

a, b = larger and smaller projection, respectively tf of the slab base beyond the rectangle circumscribing the column; and

tf = flange thickness of compression member.

When only the effective area of the base plate is used as in 7.4.1.1, IS 800-2007 C^2 may be used in the above equation (see Fig. 9) instead of $(a^2 - 0.3b^2)$.

When the slab does not distribute the column load uniformly, due to eccentricity of the load etc, special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

Angle Struts

Single Angle Struts

The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentrically by connecting one of its legs to a gusset or adjacent member.

Concentric loading

When a single angle is concentrically loaded in compression, the design strength may be evaluated using 7.1.2. IS800-2007

Loaded through one leg

The flexura! torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, $_{\gamma}\lambda c$ as given below:

$$\lambda_e = \sqrt{k_1 + k_2\,\lambda_{vv}^2 + k_3\,\lambda_\varphi^2}$$

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}} \text{ and } \lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio ($250/f_y)^{0.5}$.

Constant values

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1)]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

[1)] Stiffeness of in-plane rotational restraint provided by the gusset/connecting member.
For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.

Double Angle Struts

For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, *KL*, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, *KL*, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values based on 7.1.2, The angles shall be connected together over their lengths so as to satisfy the requirements of 7.8 and 10.2.5.

Continuous Members

Double angle continuous struts such as those forming the flanges, chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with 7.2.4.

Combined Stresses

In addition to axial loads, if the struts carry loads which cause transverse bending, the combined bending and axial stresses shall be checked in accordance with 9.3. For determining the permissible axial and bending stresses, the effective length shall be taken in accordance with the 7.2 and 8.3.

Laced Columns

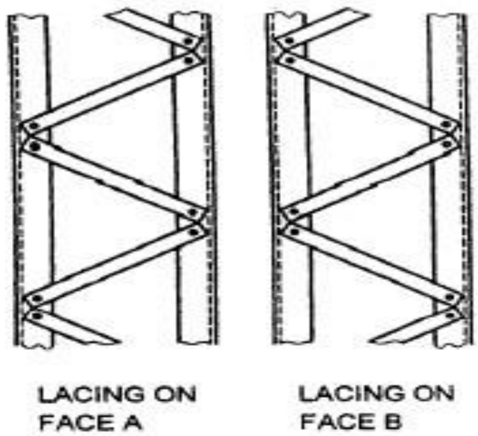
Members comprising two main components laced and tied, should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing (see Fig. IOA and 10B).

As far as practicable, the lacing system shall be uniform throughout the length of the column.

Except for tie plates as specified in 7.7, double laced systems (see Fig. 10B) and single laced systems (see Fig. IOA) on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut (see Fig. 10C), unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

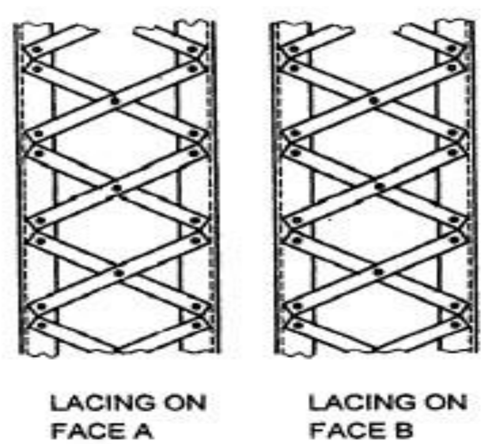
Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.

The The effective slenderness ratio, *(KZA-).*, of laced columns shall be taken as 1.05 times the *(KVr)o*, the actual maximum slenderness ratio, in order to account for shear deformation effects.



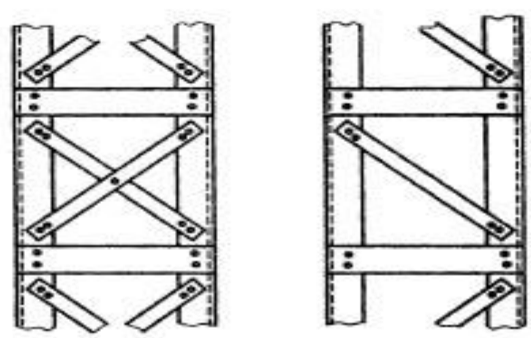
PREFFERED LACING
ARRANGEMENT

10A Single Laced System



PREFFERED LACING
ARRANGEMENT

10B Double Laced System



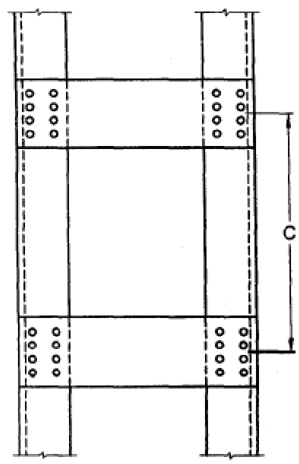
10C Double Laced and Single Laced System Combined with Cross Numbers

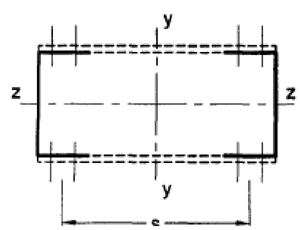
Battened Columns

Compression members composed of two main components battened should preferably have the, individual members of the same cross-section and symmetrically disposed about their major axis. Where practicable, the compression members should have a\ radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of the batten (seeFig. 11).

Battened compression members, not complying with the requirements specified in this section or those subjected to eccentricity of loading, applied moments or lateral forces in the plane of the battens (see Fig. 11), shall be designed according to the exact theory of elastic stability or empirically, based on verification by tests.

NOTE— If the columnsection is subjectedto eccentricityor other moments about an axis perpendicular to battens, the
battens and the column section should be specially designed
for such moments and shears.
7.7.1.3 The battens shall be placed opposite to each
other at each end of the member and at points where
the member is stayed in its length and as far as
practicable, be spaced and proportioned uniformly
throughout. The number of battens shall be such that
the member is divided into not less than three bays within its actual length from centre-to-centre of end
connections.





EXAMPLE 3.1 Determine the buckling strength of a W 12 x 50 column. Its length is 20 ft. For major axis buckling, it is pinned at both ends. For minor buckling, is it pinned at one end and fixed at the other end.

Solution

Step I. Visualize the problem xy

Figure 2. (a) Cross-section; (b) major-axis buckling; (c) minor-axis buckling

- For the W12 x 50 (or any wide flange section), x is the major axis and y is the minor axis. Major axis means axis about which it has greater moment of inertia ($I_x > I_y$)

Figure 3. (a) Major axis buckling; (b) minor axis buckling **Step II. Determine the effective lengths**

- According to Table C-C2.1 of the AISC Manual (see page 16.1 - 189):

- For pin-pin end conditions about the minor axis

  $K_y = 1.0$ (theoretical value); and $K_y = 1.0$ (recommended design value)

- For pin-fix end conditions about the major axis

  $K_x = 0.7$ (theoretical value); and $K_x = 0.8$ (recommended design value)

- According to the problem statement, the unsupported length for buckling about the major (x) axis = $L_x = 20$ ft.
- The unsupported length for buckling about the minor (y) axis = $L_y = 20$ ft.
- Effective length for major (x) axis buckling = $K_x L_x = 0.8 \times 20 = 16$ ft. = 192 in.
- Effective length for minor (y) axis buckling = $K_y L_y = 1.0 \times 20 = 20$ ft. = 240 in.

Step III. Determine the relevant section properties

- For W12 x 50: elastic modulus = E = 29000 ksi (constant for all steels)
- For W12 x 50: $I_x = 391$ in^4. $I_y = 56.3$ in^4 (see page 1-21 of the AISC manual)

Step IV. Calculate the buckling strength

- Critical load for buckling about x - axis = $P_{cr-x} = \frac{\pi^2 E I_x}{(K_x L_x)^2} = \frac{\pi^2 \times 29000 \times 391}{(192)^2}$

$$P_{cr-x} = 3035.8 \text{ kips}$$

- Critical load for buckling about y-axis = $P_{cr-y} = \frac{\pi^2 E I_y}{(K_y L_y)^2} = \frac{\pi^2 \times 29000 \times 56.3}{(240)^2}$

$$P_{cr-y} = 279.8 \text{ kips}$$

- Buckling strength of the column = smaller (P_{cr-x}, P_{cr-y}) = <u>Pcr = 279.8 kips</u>

  <u>Minor (y) axis buckling governs.</u>

Notes:

- Minor axis buckling usually governs for all doubly symmetric cross-sections. However, for some cases, major (x) axis buckling can govern.
- Note that the steel yield stress was irrelevant for calculating this buckling strength.

3.3 INELASTIC COLUMN BUCKLING

- Let us consider the previous example. According to our calculations $P_{cr} = 279.8$ kips. This P_{cr} will cause a uniform stress $f = P_{cr}/A$ in the cross-section
- For W12 x 50, A = 14.6 in^2. Therefore, for $P_{cr} = 279.8$ kips; f = 19.16 ksi

  The calculated value of f is within the elastic range for a 50 ksi yield stress material.

- However, if the unsupported length was only 10 ft., $P_{cr} = \left(\frac{\pi^2 E I_y}{(K_y L_y)^2}\right)$ would be calculated as 1119 kips, and f = 76.6 kips.
- <u>The member would yield before buckling.</u> This value of f is ridiculous because the material will yield at 50 ksi and never develop f = 76.6 kips.

- **Equation (3.1) is valid only when the material everywhere in the cross-section is in the elastic region. If the material goes inelastic then Equation (3.1) becomes useless and cannot be used.**
- What happens in the inelastic range?

Several other problems appear in the inelastic range.
- The member out-of-straightness has a significant influence on the buckling strength in the inelastic region. It must be accounted for.

- The residual stresses in the member due to the fabrication process causes yielding in the cross- section much before the uniform stress f reaches the yield stress F_y.
- The shape of the cross-section (W, C, etc.) also influences the buckling strength.
- In the inelastic range, the steel material can undergo strain hardening.

All of these are very advanced concepts and beyond the scope of CE405. You are welcome to CE805 to develop a better understanding of these issues.

• So, what should we do? We will directly look at the AISC Specifications for the strength of compression members, i.e., Chapter E (page 16.1-27 of the AISC manual).

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – V- DESIGN OF STEEL STRUCTURES I – SCI1303

DESIGN OF BEAMS
Beam classification

- **Main or Primary beams/ girders**
- **Secondary beams/joists**

- **Girders**
- **Joist**
- **Lintels**
- **Purlins**
- **Rafter**
- **Spandrels**
- **Stringers**

- **Laterally Stable**
- **Laterally Unstable**

PERMISSIBLE STRESS DESIGN

Stresses in Structures at working loads are not allowed to exceed a certain proportion of the yield stress of the material.

- Stress levels are limited to elastic range
- Leads to highly conservativesolutions.

LIMIT STATE DESIGN OF BEAMS

- In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur on it throughout itslife.

- 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.

Limit States

- Ultimate Limit States

  - (**flexure, shear, bearing, compression, torsion, lateral-torsion**)

- Serviceability Limit States

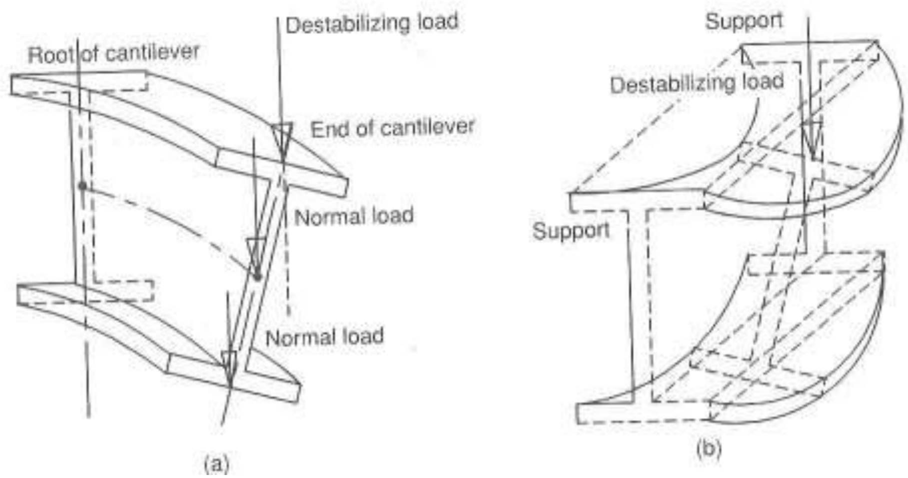
  -(**deflection, vibration, fire, durability**)

Types of Loads

- Dead loads
- Imposed loads (Live Load, Crane Load, Snow Load, Dust Load, Wave Load, Earth pressures)
- Wind Loads
- Earthquake Loads
- Erection Loads
- Accidental Loads (Blast, Impact of vehicles)
- Secondary Effects (temperature effects, differential settlements, eccentric connections, varied rigidity)

Stability of Beams

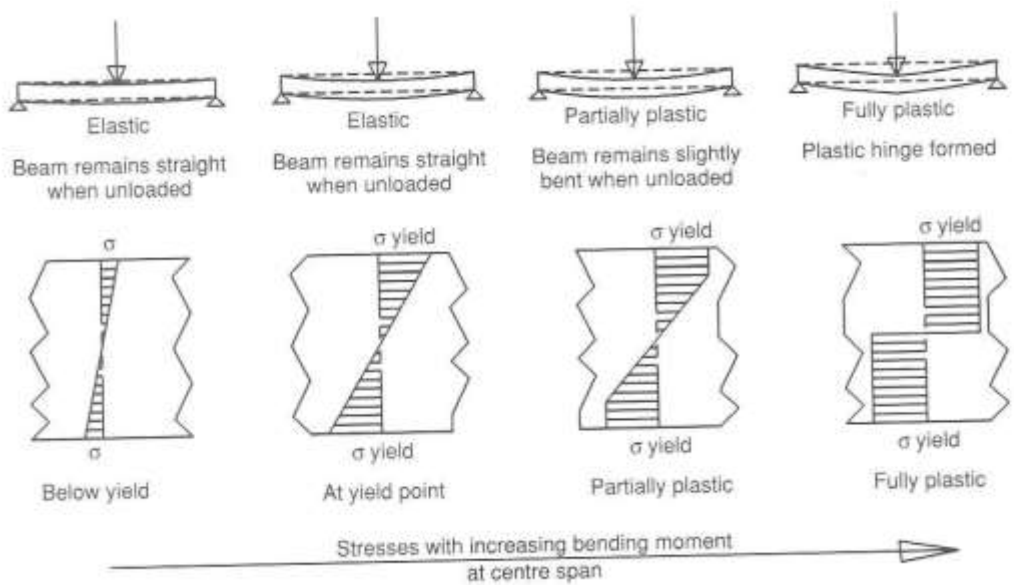
- Laterally Unrestrained Beams
- Laterally Restrained Beams



Lateral-torsional Buckling in Beams

Failure Modes in Beams

- Bending

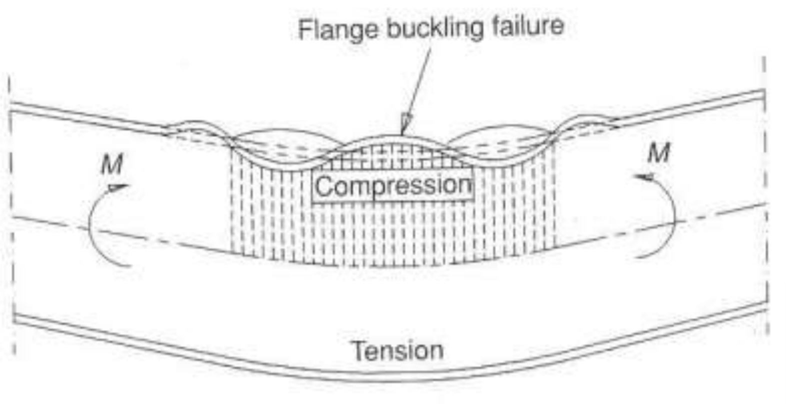


➢ **When all the beam cross-section has become plastic the beam fails by formation of a plastic hinge at the point of maximum imposed moment.**

➢ **The bending moment cannot be increased and the beam collapses as though a hinge has been inserted into the beam.**

Failure Modes in Beams…

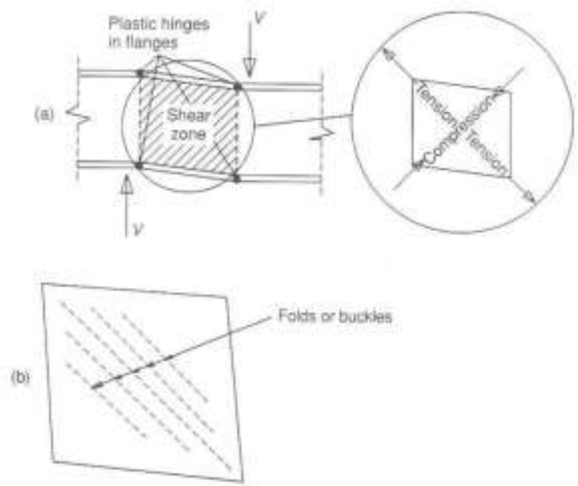
- Local buckling



Local Flange buckling failure

Failure Modes in Beams…

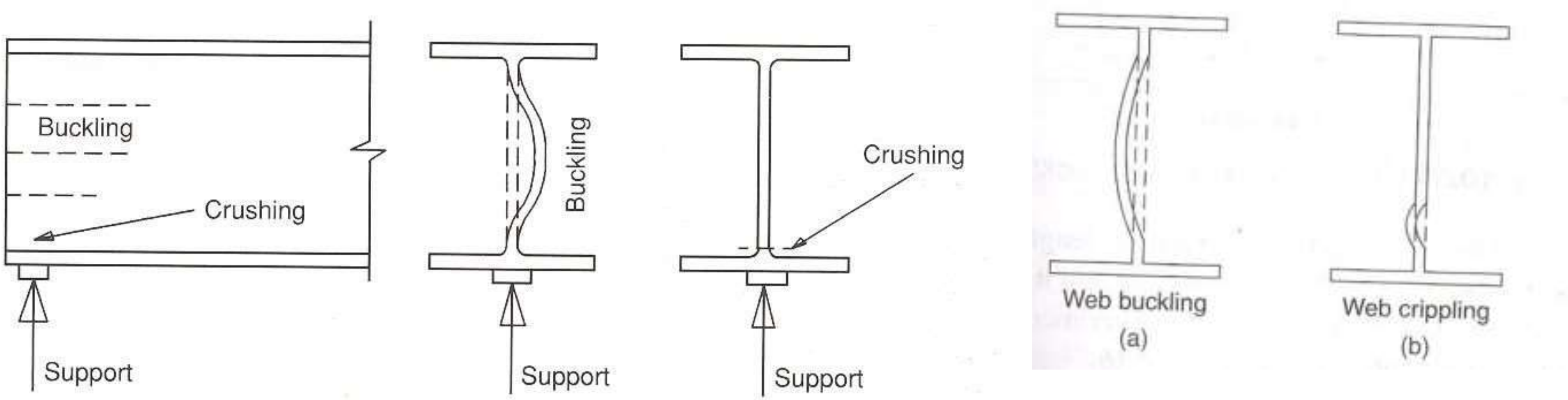
- **Shear**



During the shearing process, if the web is too thin it will fail by buckling or rippling in the shear zone as shown in fig.

Failure Modes in Beams…

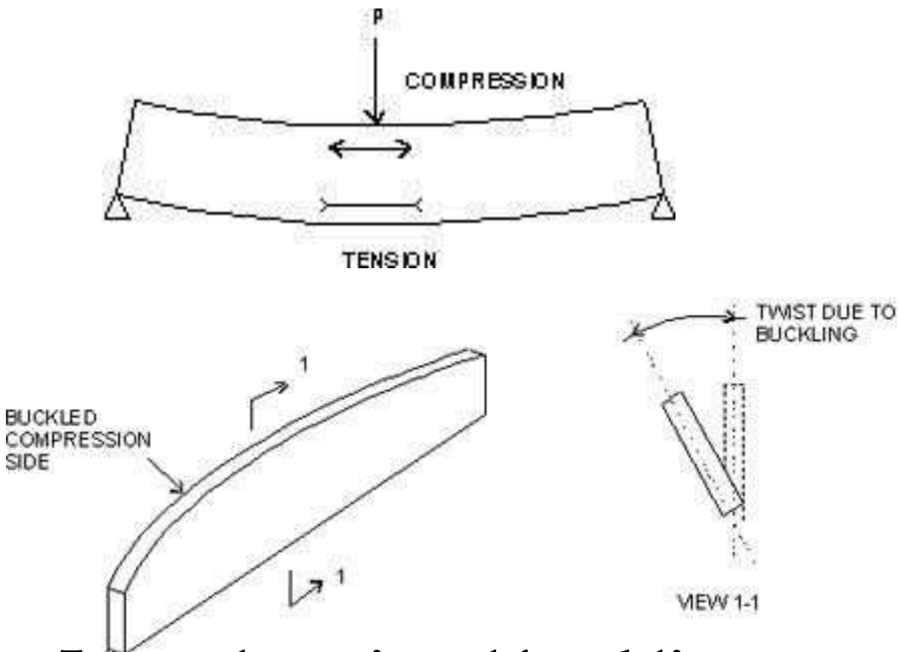
- **Web bearing and buckling**



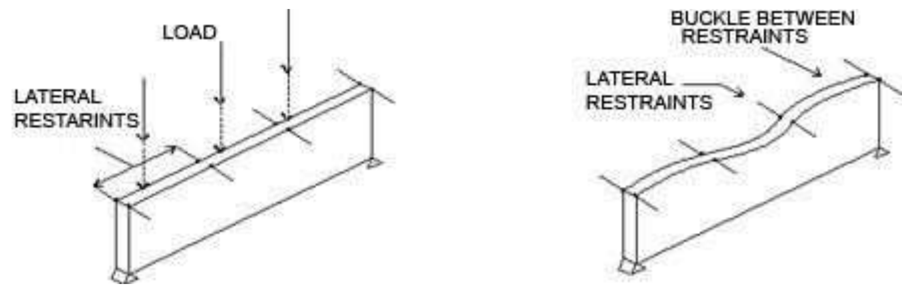
Due to high vertical stresses directly over a support or under a concentrated load, the beam web may actually crush or buckle as a result of these stresses.

Failure Modes in Beams…

- **Lateral-torsional buckling**

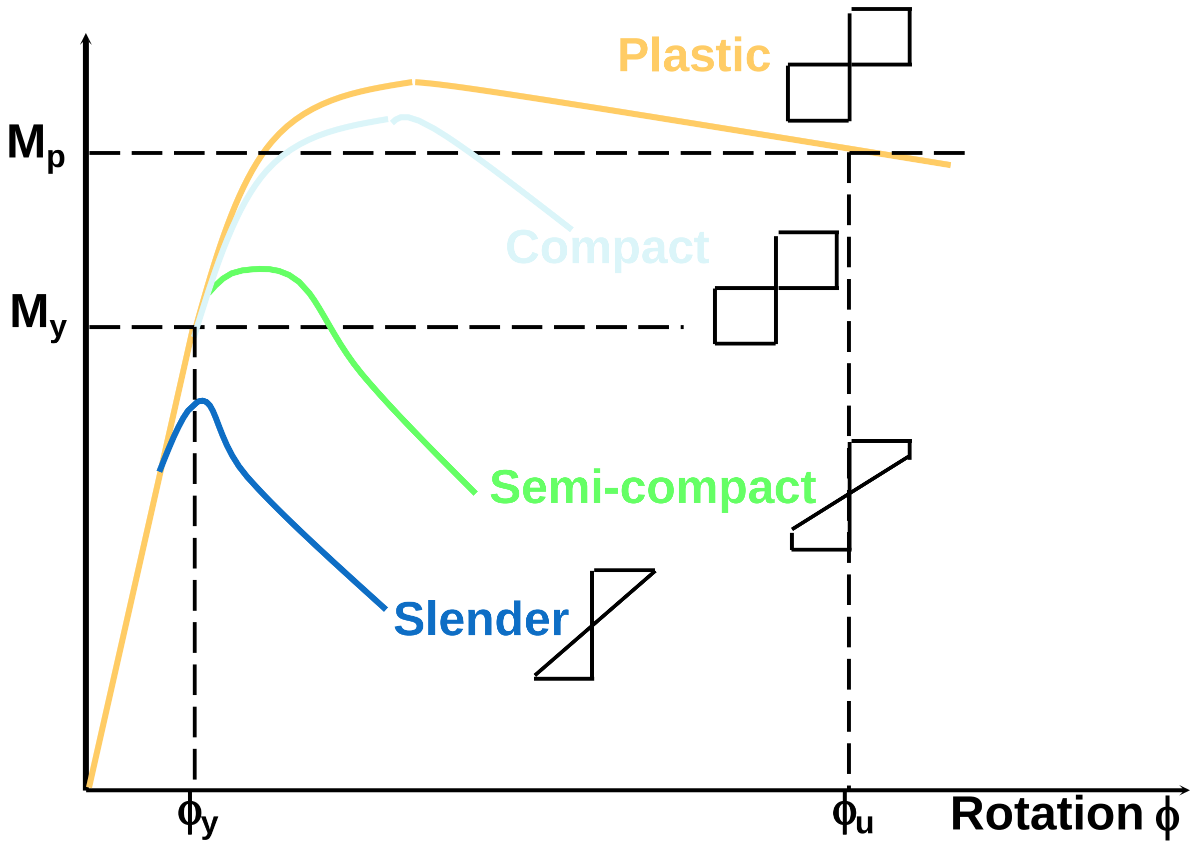


<u>Lateral torsional buckling of a simply supported beam</u>



<u>Behaviour of beam with restraints</u>

SECTION CLASSIFICATION



Section Classification based on Moment-Rotation Characteristics

Section Classification

- a) *Plastic* – Cross sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism.

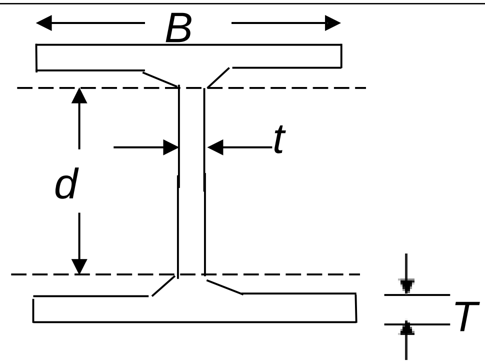
- b) *Compact* – Cross sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism.

- c) *Semi-Compact* – Cross sections, in which the extreme fibre in compression can reach, yield stress, but cannot develop the plastic moment of resistance, due to local buckling.

- d*) Slender* – Cross sections in which the elements buckle locally even before reaching yield stress. In such cases, the effective sections for design shall be calculated by deducting width of the compression plate element in excess of the semi-compact section limit.

Sectional Classification for Indian Conditions

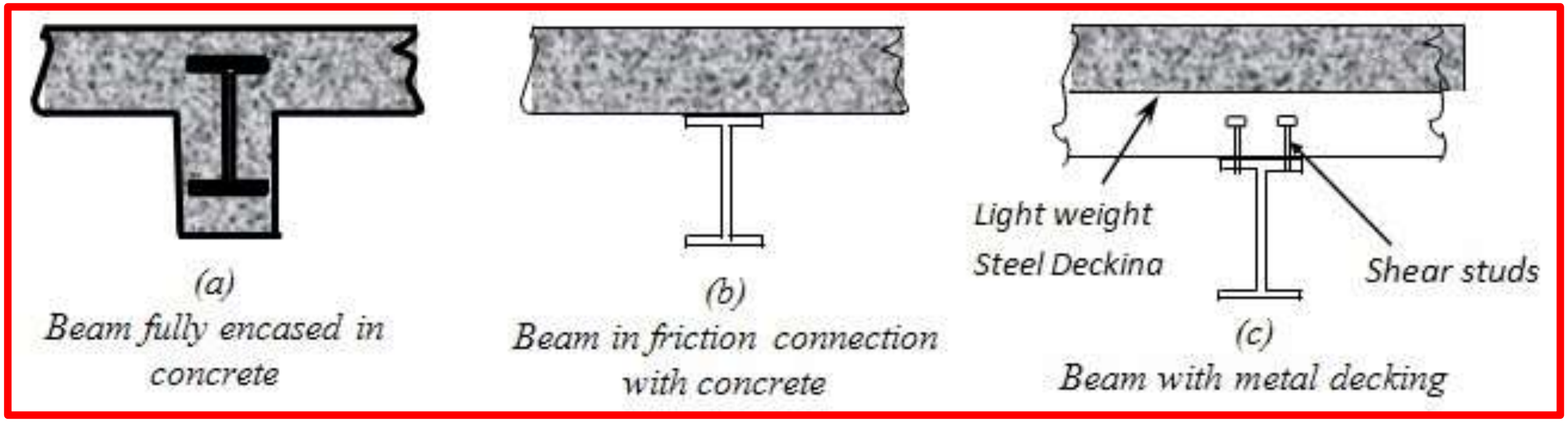
b = B/2



$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

Section type	**Flange criterion (b/T) ε**	**Web criterion (d/t) ε**
Slender	**>15.75**	**>126**
Semi-compact	**<15.75 $\geq$ 9.975**	**<126 $\geq$ 102.9**
Compact	**<9.975 $\geq$8.92**	**<102.9 $\geq$ 82.95**
Plastic	**<8.92**	**<82.95**

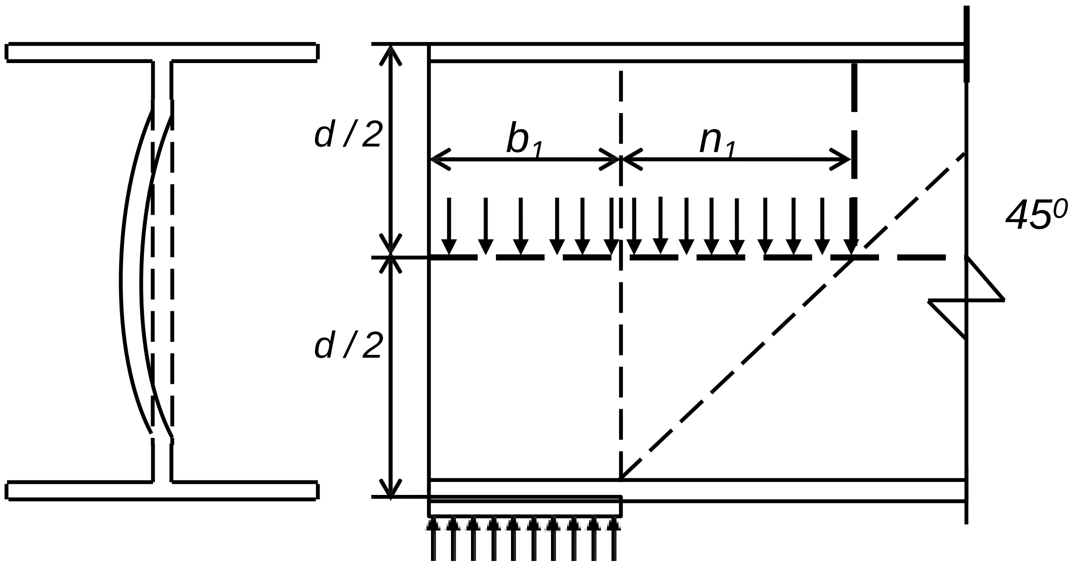
Laterally supported beams



Limit states for Laterally Restrained beams

- Limit state of flexure
- Limit state of shear
- Limit state of bearing
- Limit state of serviceability

WEB BUCKLING



Effective width for web buckling

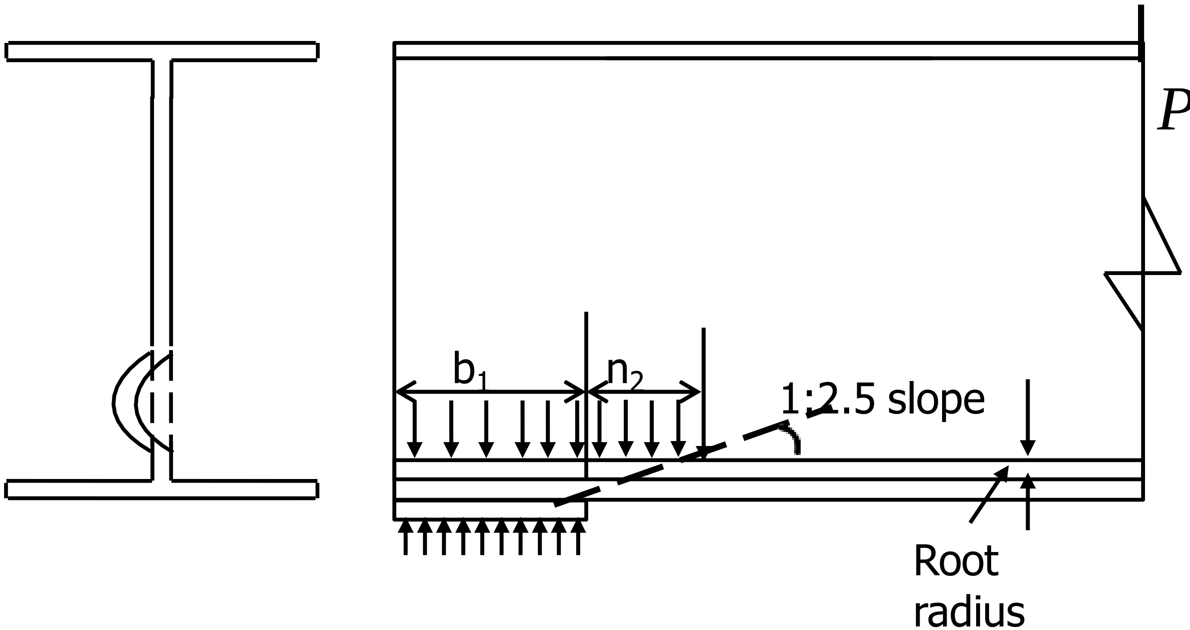
$$P_{wb} = (b_1 + n_1)\ t\ \ f_c$$

$$\lambda = \frac{L_E}{r_y} = \frac{0.7d}{r_y}$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{t^3}{12t}} = \frac{t}{2\sqrt{3}}$$

$$\frac{L_E}{r_y} = 0.7d\ \frac{2\sqrt{3}}{t} \approx 2.5\ \frac{d}{t}$$

WEB CRIPPLING



$$P_{crip} = (b_1 + n_2)\, t\, f_{yw}$$

Effective width of web bearing

Web Crippling in beams

Design Strength in Bending (Flexure)

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

Laterally Supported Beam

The design bending strength as governed by plastic strength, M_d, shall be taken as

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2\, Z_e f_y / \gamma_{m0}$$

$$M \leq M_d$$

Holes in the tension zone

$$(A_{nf} / A_{gf}) \geq (f_y/f_u)\,(\gamma_{m1}/\gamma_{m0}) / 0.9$$

Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$\boldsymbol{V \leq V_d}$$

V_d = design strength calculated as , $\boldsymbol{V_d = V_n / \gamma_{m0}}$

The nominal plastic shear resistance under pure shear is given by: $\boldsymbol{V_n = V_p}$

A_v = shear area $\quad V_p = \dfrac{A_v f_{yw}}{\sqrt{3}}$

DESIGN OF STEEL BEAMS

- **STEP 1:**

  Determination of design shear forces V and bending moments M at critical points on the element

  Table 4 (page 29) gives the factors for different load combinations

DESIGN OF STEEL BEAMS...

- **STEP 2:**

Section Modulus Required

$$Z_p \text{ (required)} = M \times \gamma_{mo} / f_y$$

– γ_{mo} **is the partial Safety Factor for materials given in Table 5 (page 30)**

Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

DESIGN OF STEEL BEAMS…

- **STEP 3:**

Selection of Suitable Section

Shape Factor (v) -

The ratio M_p/M_y is a property of the cross-section shape and is independent of the material properties.

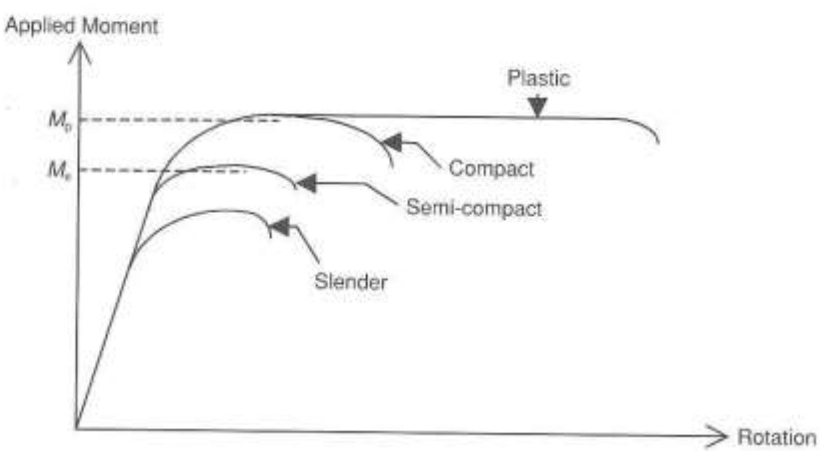
$\upsilon = M_p/M_y = Z_p/Z_e$

Hence, $Z_p = Z_e \times \upsilon$

DESIGN OF STEEL BEAMS…

- **STEP 4:**

Classification of Section **(Table 2, page 18)**



Check adequacy of the section including self-weight

DESIGN OF STEEL BEAMS…

- **STEP 5:**

Check shear Strength

Design shear Strength, $V_d = A_v \times f_{yw}/\sqrt{3}$ (cl. 8.¢, page 59)

$$V_p = \frac{A_v\, f_{yw}}{\sqrt{3}}$$

($V_d > V$)

A_v = shear area, and

f_{yw} = yield strength of the web.

If $V > 0.6\, V_d$, design for combined shear and bending (cl 9.2, page 69)

Where A_v = sheararea

f_{yw} = yield strength of web

DESIGN OF STEEL BEAMS…

- **STEP 6:**

Check Bending Capacity

- If laterally supported beam **(cl. 8.2.1, page 52)**

- If laterally unsupported beam **(cl. 8.2.2, page 54)**

Get M_d and check if $M < M_d$

DESIGN OF STEEL BEAMS…

- **STEP 7:**

Check for deflection

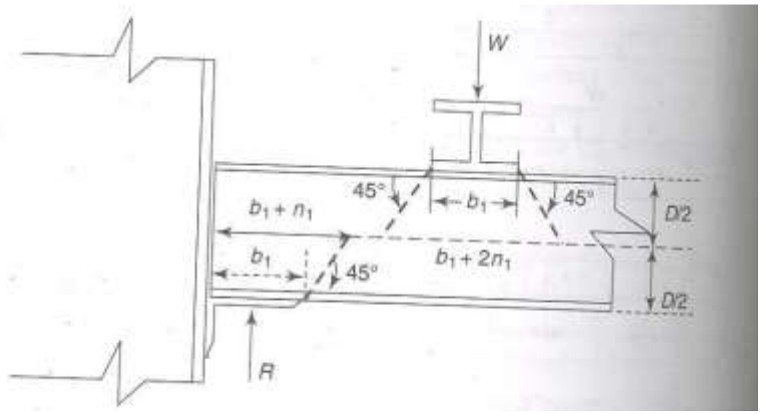
This is a serviceability limit state and hence must be calculated on the basis of unfactored imposed loads

Allowable max. deflection **–(Table 6, page31)**

DESIGN OF STEEL BEAMS…

- **STEP 8**

Check for Web Buckling **(cl. 8.7.3.1, page 67)**

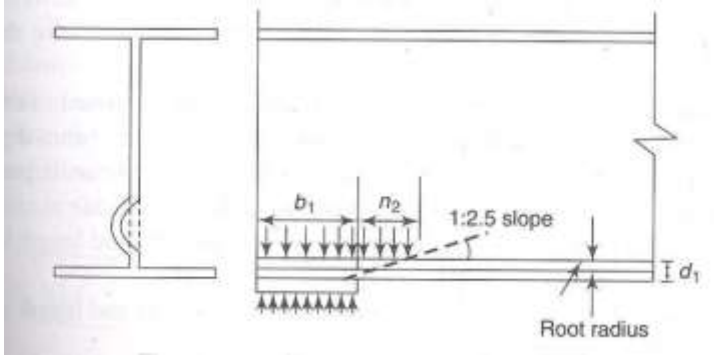


Dispersion of concentrated loads and reactions for evaluating web buckling

DESIGN OF STEEL BEAMS...

- **STEP 9**

Check for Web Bearing (cl. 8.7.4, page 67)



Exmples

A simply supported beam has an effective span of 7m and carries a uniformly distributed load of 50 kN/m (i.e DL = 25kN/m and LL = 25 kN/m). Taking f_y = 250 N/mm^2 and E = 2 x 10^5 N/mm^2, design the beam, if it is laterally supported.

Example….

- STEP 1:
- Factored Load = 1.5 x 50 = 75 kN/m (Table 4)

- STEP 2:
- Design Bending Moment = $wl^2/8$ = 459.375 kN.m

Design shear force = wl/2 = 262.5kN

Example….

- **STEP 3:**

Plastic Section modulus reqd., $Z_p = M \times \gamma_{mo}/f_y$

(cl. 8.2.1.2)

$= 459.375 \times 10^6 \times 1.1/250$ **(Table 5)**

$= 2021.25 \times 10^3\ mm^3$

Example....

- Z_p/Z_e is conservatively assumed to be 1.15

$Z_{e,\ reqd} = 2021.25 \times 10^3/1.15 = 1757.61 \times 10^3\ mm^3$

Choose ISMB 500

Depth, $h = 500$ mm ;

width of flange, $b = 172$ mm ;

Thickness of flange, $t_f = 17.2$ mm ;

Thickness of web, $t_w = 10.2$ mm ;

Depth of web, $d = h - 2(t_f+R) = 500 - 2(17.2+17) = 431.6$ mm

$I_{zz} = 45218.3 \times 10^4\ mm^4$; $Z_e = 1808.7 \times 10^3\ mm^3$

Weight of the section = 86.9 kg/m

Example....

- **STEP 4**

Section Classification **(Table 2)**

$\varepsilon = \sqrt{z\mu o / fy} = 1$

$b/t_f = 172/17.2 = 10 < 10.5\varepsilon \rightarrow$ compact

$d/t_w = 431.6/10.2 = 42.31 < 84\varepsilon \rightarrow$ plastic

Hence section is **compact**

Example....

- **STEP 5**

Check for adequacy of section

Factored self weight = 1.5 x 86.9 x 9.81/1000

= 1.279 kN/m

Total factored load = 75 + 1.279 = 76.279 kN/m

Example....

- $M_{max} = wl^2/8 = 467.21$ kN.m

Z_p(reqd.) $= 467.21 \times 10^6 \times 1.1/250$

$= 2055.72 \times 10^3\ mm^3 < 2080 \times 10^3\ mm^3$

Hence provided section isadequate

Example….

- **STEP 6**

Design Shear Force , V = wl/2

= 76.279 x 7/2 = 266.98 kN

Example....

- **STEP 7**

Design Shear Strength ,$V_d = V_n / \gamma_{mo}$

$= h \times t_w \times f_{yw} / (1.1 \times \sqrt{3})$

= μoo x 10.z x zμo /(1.1 x √3)

= 669.201 kN > 266.98 kN

Hence OK

Also $V < 0.6V_d$

Example….

- **STEP 8**

Check for Design Capacity

$d/t_w = 42.31 < 67\varepsilon$ **(cl 8.2.1.1)**

$M_d = \beta_b Z_p \times f_y/\gamma_{mo} = 1 \times 2080 \times 10^3 \times 250/1.1$

$= 472.7273$ kN.m $< 1.2 \times Z_e \times f_y/\gamma_{mo}$ (cl 8.2.1.2)

< 493.28 kN.m

Hence satisfied

Example....

- **STEP 9**

Check for Deflection (Use unfactored imposed load)

$\delta = 5wl^4/384 = 8.64mm < l/300$ **(Table 6)**

$< 23.33mm$

Hence safe

Example….

- In the previous problem the bearing length was assumed to be adequate.
- Suppose a bearing length of 75mm isprovided.
- We should check the safety of the web in bearing and buckling

Example….

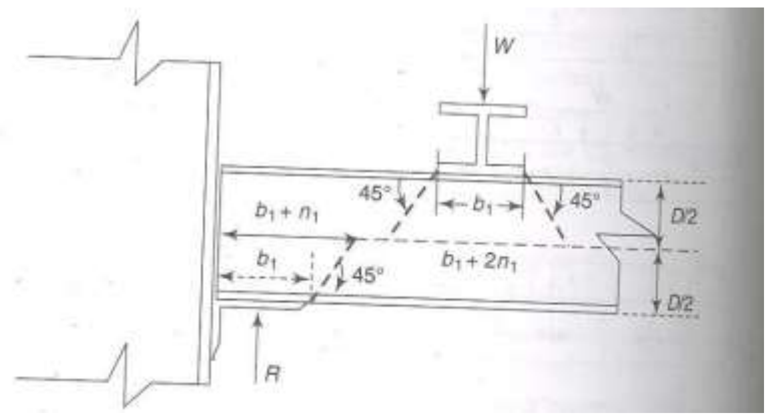
- **Web Buckling (cl. 8.7.3.1)**

$I_{eff,web} = b_1 \times t_w{}^3/12 = 75 \times 10.2^3/12 = 6632.55\ mm^4$

$A_{eff,web} = 75 \times 10.2 = 765\ mm^2$

$r = \sqrt{)_{eff,web}/ A_{eff,web}}$

$= 2.94\ mm$



Example....

- **Web Buckling...**

Effective length of the web **(cl. 8.7.1.5)**

= 0.7d = 0.7 x 431.6

Slenderness ratio λ = 0.7 x 431.6/2.94

Design comp. stress f_{cd} = 103.528 N/mm^2

(Table 9c)

Example….

$n_1 = 250mm$ (i.e 500/2)

$b_1 + n_1 = 75 + 250 = 325mm$

$A_b = 325 \times 10.2 = 3315 mm2$

Buckling resistance = $f_{cd} \times A_b$

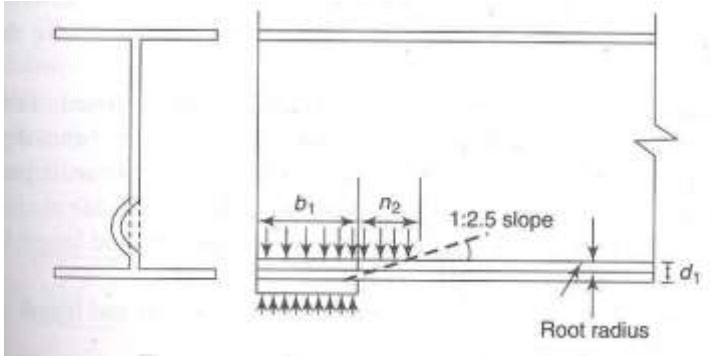
$= 343.195$ kN > 266.98 kN

Hence Ok . The web is safe against buckling.

Example....

- **Check for Web Bearing (cl. 8.7.4, page 67)**

Crippling strength of web $f_w = (b_1 + n_2)\ t_w\ f_{yw}/\gamma_{mo}$



b_1 = 75 mm;

$d_1 = t_f + R = 17.2 + 17 = 34.2$ mm

$n_2 = 2.5\ d_1 = 85.5$mm

$f_w = 372.07$ kN > 266.98 kN

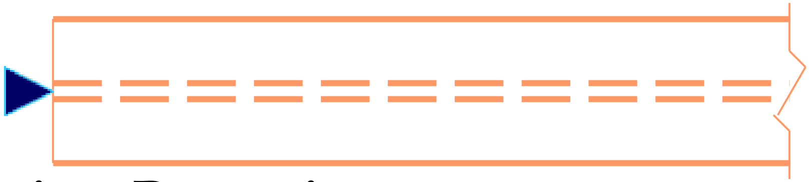
Hence Ok. Web is safe against bearing.

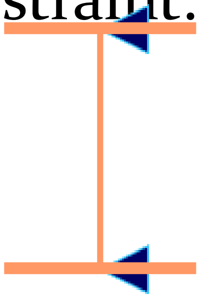
LATERAL BUCKLING OF BEAMS

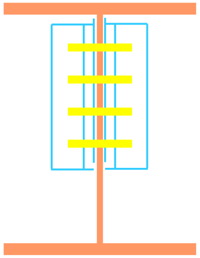
- **FACTORS TO BE CONSIDERED**
  - **Distance between lateral supports to the compression flange.**
  - **Restraints at the ends and at intermediate support locations (boundary conditions).**
  - **Type and position of the loads.**
  - **Momentgradient along the unsupported length.**
  - **Type of cross-section.**
  - **Non-prismatic nature of the member.**
  - **Material properties.**
  - **Magnitude and distribution of residualstresses.**
  - **Initial imperfections of geometry and eccentricity of loading.**

END SUPPORT CONDITIONS

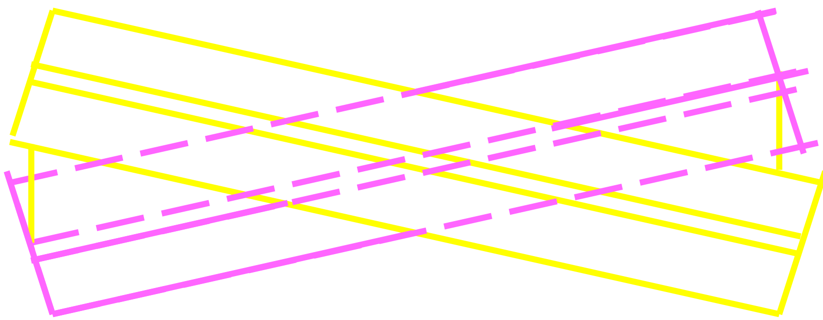
Torsional (Rotational) Restraint:

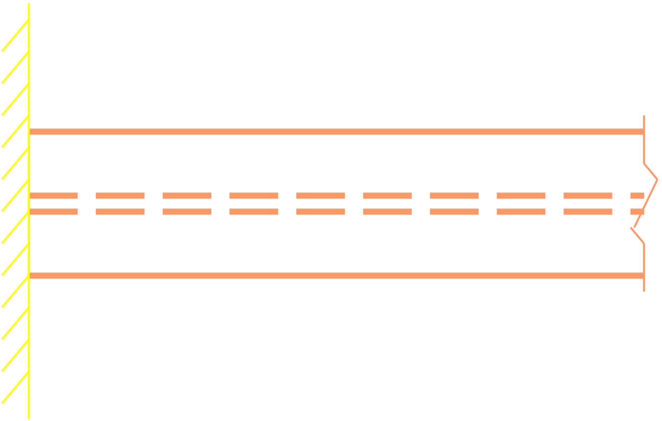






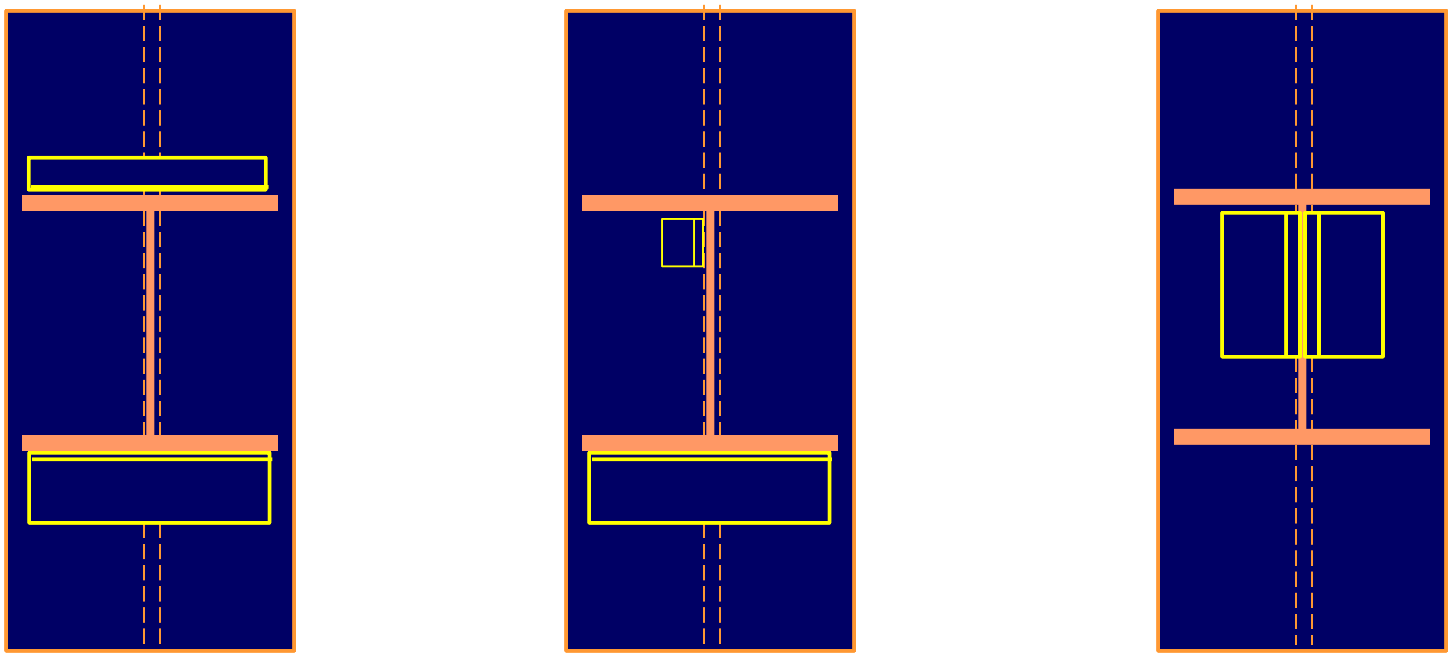
Warping Restraint:





The end restraint elements shall be capable of safely resisting, in addition to wind and other applied external forces, a horizontal force acting in the plane in a direction normal to the axis of compression flange of the beam at the level of the centroid of the flange and having a value not less than 2.5 percent of the maximum compressive force occurring in the flange.

LATERAL TORSIONAL RESTRAINT AT SUPPORTS



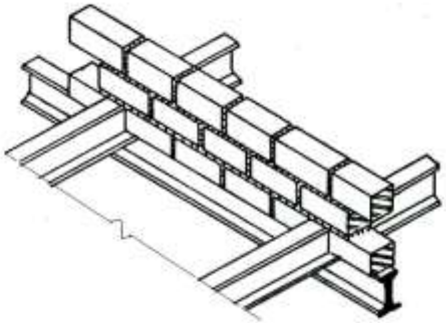
WARPING RESTRAINT AT SUPPORTS

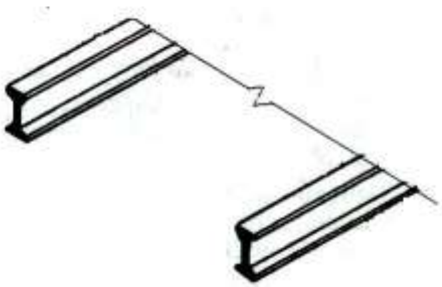
EFFECTIVE LENGTH (Simply Supported)

Effective Length of Compression Flanges

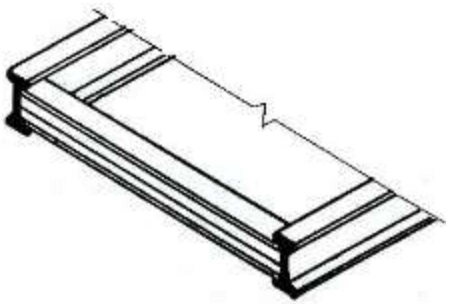
Boundary Condition	Effective Length (kL)
Each end – torsional restraint	*L*
Full torsional and Partial Warping restraint	0.85 L
Full Torsional & Warping Restraint	0.75 L

EFFECTIVE LENGTH(Cantilever)

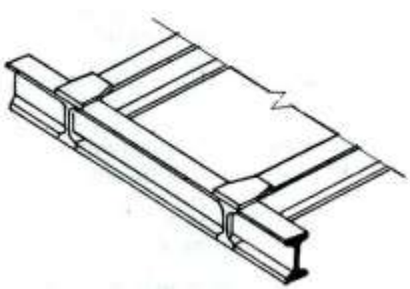




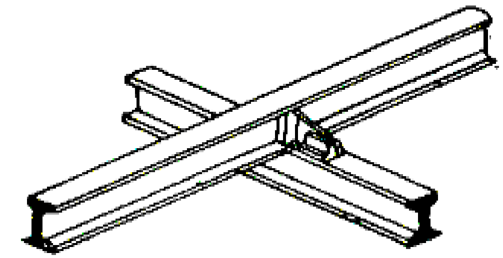
$kL = 0.85\,L$



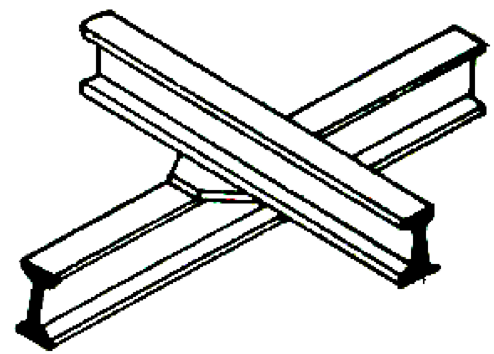
$kL = 0.75\,L$



$kL = 0.5\,L$

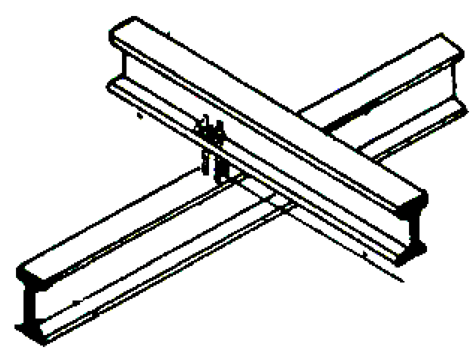


$kL = L$



$kL = 2\,L$

$kL = 3\,L$

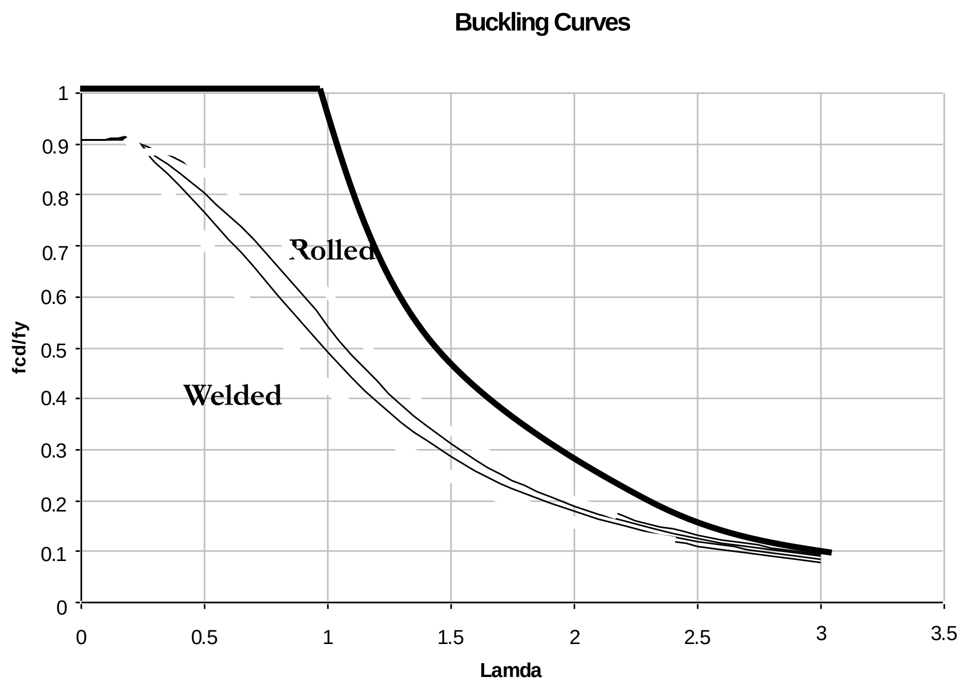


ELASTIC LATERAL BUCKLING MOMENT

$$M_{ocr} = \frac{\pi}{L}\sqrt{EI_y\, GJ} \times \sqrt{1 + \frac{\pi^2 EC_w}{GJ\, L^2}}$$

$$M_{cr} = c_1 \frac{\pi^2 EI_y}{(KL)^2} \left\{ \left[\left(\frac{K}{K_w} \right)^2 \frac{I_w}{I_y} + \frac{GI_t\,(KL)^2}{\pi^2 EI_y} + \left(c_2\, y_g - c_3 y_j \right)^2 \right]^{0.5} - \left(c_2\, y_g - c_3 y_j \right) \right\}$$

EFFECT OF RESIDUAL STRESSES & IMPERFECTIONS



IMPERFECTION FACTOR, *α*

Buckling Class	Rolled	Welded
α	0.21	0.49

Design Strength in Bending (Flexure)

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

Laterally Supported Beam

The design bending strength as governed by plastic strength, M_d, shall be taken as

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2\, Z_e f_y / \gamma_{m0}$$

$$M \leq M_d$$

Holes in the tension zone

$$(A_{nf} / A_{gf}) \geq (f_y/f_u)\,(\gamma_{m1} / \gamma_{m0}) / 0.9$$

Cont...

Laterally Unsupported Beams

The design bending strength of laterally unsupported beam is given by:

$$M_d = \beta_b Z_p f_{bd}$$

f_{bd} = design stress in bending, obtained as , $f_{bd} = \chi_{LT} f_y / \gamma_{m0}$

χ_{LT} = reduction factor to account for lateral torsional buckling given by:

$$\chi_{LT} = \frac{1}{[\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}]} \leq 1.0$$

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

α_{LT} = 0.21 for rolled section,

α_{LT} = 0.49 for welded section

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}}$$

Cont…

Elastic Lateral Torsional Buckling Moment

$$M_{cr} = \sqrt{\left\{ \left(\frac{\pi^2 E I_y}{(KL)^2} \right) \left[G I_t + \frac{\pi^2 E I_w}{(KL)^2} \right] \right\}}$$

$$M_{cr} = \frac{\beta_{LT}\, \pi^2 E I_y h}{2(KL)^2} \left[1 + \frac{1}{20} \left[\frac{KL / r_y}{h / t_f} \right]^2 \right]^{0.5}$$