6.3 Behaviour of steel beams

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

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

$$M \le M_d$$

Where M_d = design bending strength of the section

6.3.1 Design strength in bending (Flexure)

The behaviour of members subjected to bending demonstrated in Fig 6.1

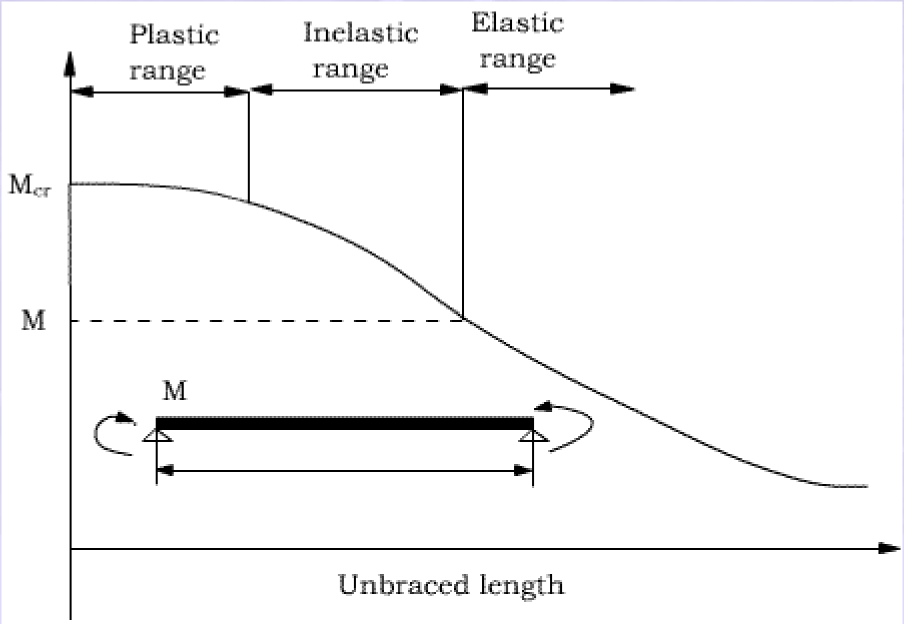

Fig 6.1 Beam buckling behaviour

This behaviour can be classified under two parts:

- When the beam is adequately supported against lateral buckling, the beam failure occurs by yielding of the material at the point of maximum moment. The beam is thus capable of reaching its plastic moment capacity under the applied loads. Thus the design strength is governed by yield stress and the beam is classified as laterally supported beam.

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

Beams which fail by flexual yielding

Type1: Those which are laterally supported

The design bending strength of beams, adequately supported against buckling (laterally supported beams) is governed by yielding. The bending strength of a laterally braced compact section is the plastic moment $M_{p.}$ If the shape has a large shape factor (ratio of plastic moment to the moment corresponding to the onset of yielding at the extreme fiber), significant inelastic deformation may occur at service load, if the section is permitted to reach M_p at factored load. The limit of $1.5M_y$ at factored load will control the amount of inelastic deformation for sections with shape factors greater than 1.5. This provision is not intended to limit the plastic moment of a hybrid section with a web yield stress lower than the flange yield stress. Yielding in the web does not result in significant inelastic deformations.

Type2: Those which are laterally shift

Lateral-torsional buckling cannot occur, if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane. Thus, for shapes bent about the minor axis and shapes with $I_z = I_y$ such as square or circular shapes, the limit state of lateral-torsional buckling is not applicable and yielding controls provided the section is compact.

6.3.1.1 Laterally supported beam

When the lateral support to the compression flange is adequate, the lateral buckling of the beam is prevented and the section flexual strength of the beam can be developed. The strength of I-sections depends upon the width to thickness ratio of the compression flange. When the width to thickness ratio is sufficiently small, the beam can be fully plastified and reach the plastic moment, such section are classified as compact sections. Howeverprovided the section can also sustain the moment during the additional plastic hinge rotation till the failure mechanism is formed. Such sections are referred to as plastic sections. When the compression flange width to thickness ratio is larger, the compression flange may buckle locally before the complete plastification of the section occurs and the plastic moment is reached. Such sections are referred to as non-compact sections. When the width to thickness ratio of the compression flange is sufficiently large, local buckling of compression flange may occur even before extreme fibre yields. Such sections are referred to as slender sections.

The flexural behaviour of such beams is presented in Fig. 6.2. The section classified as slender cannot attain the first yield moment, because of a premature local buckling of the web or flange. The next curve represents the beam classified as 'semi-compact' in which, extreme fibre stress in the beam attains yield stress but the beam may fail by local buckling before further plastic redistribution of stress can take place towards the neutral axis of the beam. The factored design moment is calculated as per Section **8.2** of the code.

The curve shown as 'compact beam' in which the entire section, both compression and tension portion of the beam, attains yield stress. Because of this plastic redistribution of stress, the member o attains its plastic moment capacity (M_p) but fails by local buckling before developing plastic mechanism by sufficient plastic hinge rotation. The moment capacity of such a section can be calculated by provisions given in Section **8.2.1.2**. This provision is for the moment capacity with low shear load.

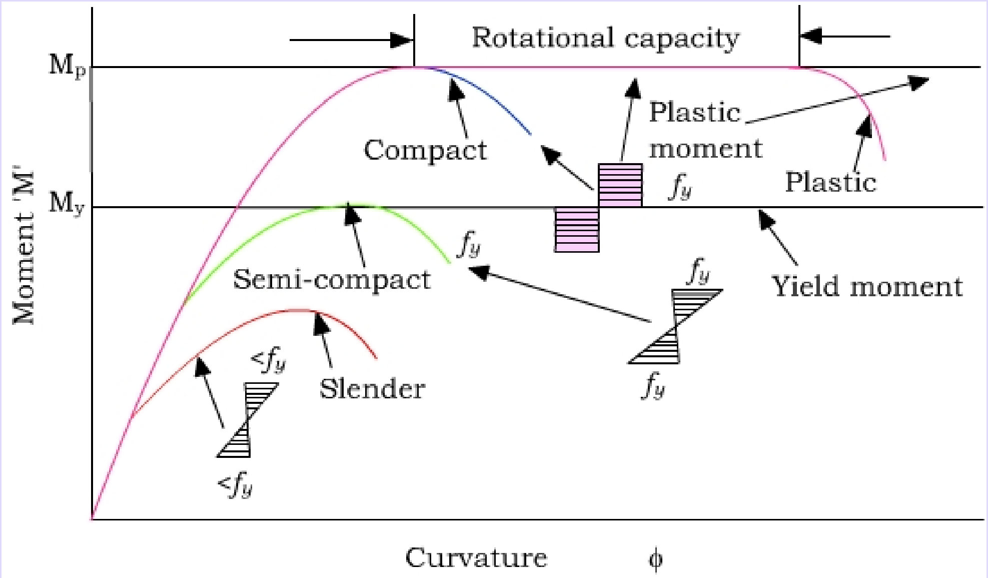

Fig 6.2 Flexual member performance using section classification

Low shear load is referred to the factored design shear force that does not exceed $0.6V_d$, where V_d is the design shear strength of cross section as explained in **8.2.1.2** of the code.

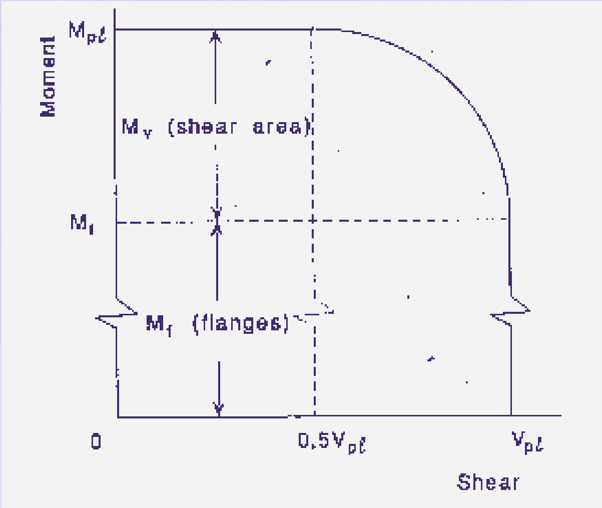

Fig.6.3 Interaction of high shear and bending moment

6.3.1.1.1 Holes in the tension zone

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

6.3.1.1.2 Shear lag effects

The simple theory of bending is based on the assumption that plane sections remain plane after bending. But, the presence of shear strains causes the section to warp. Its effect in the flanges is to modify the bending stresses obtained by the simple theory, producing higher stresses near the junction of a web and lower stresses at points away from it (Fig. 6.4). This effect is called 'shear lag'. This effect is minimal in rolled sections, which have narrow and thick flanges and more pronounced in plate girders or box sections having wide thin flanges when they are subjected to high shear forces, especially in the vicinity of concentrated loads. The provision with regard to shear lag effects is given in **8.2.1.5.**

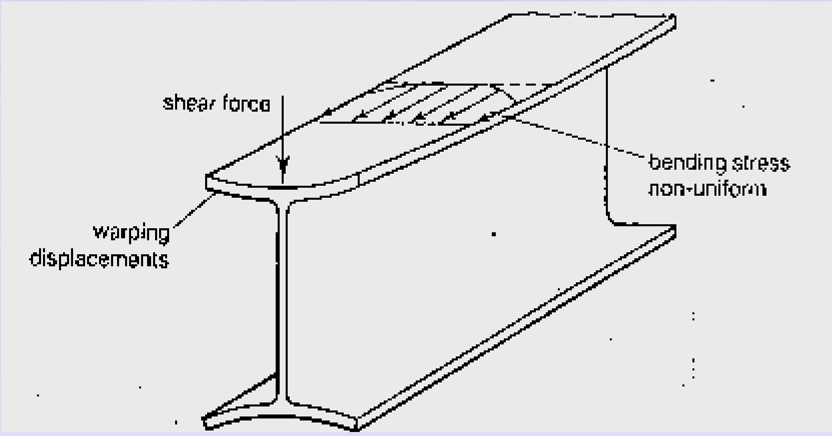

Fig.6.4 Shear Lag effects

6.3.2 Laterally unsupported beams

Under increasing transverse loads, a beam should attain its full plastic moment capacity. This type of behaviour in a laterally supported beam has been covered in

Section **8.2.1**. Two important assumptions have been made therein to achieve the ideal beam behaviour.

They are:

- The compression flange of the beam is restrained from moving laterally; and
- Any form of local buckling is prevented

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

6.3.2.1 Lateral-torsional buckling of beams

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

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

- Distance between lateral supports to the compression flange.
- Restraints at the ends and at intermediate support locations (boundary conditions).
- Type and position of the loads.
- Moment gradient along the length.
- Type of cross-section.

- Non-prismatic nature of the member.
- Material properties.
- Magnitude and distribution of residual stresses.
- Initial imperfections of geometry and loading.

They are discussed here briefly:

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

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

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

For a beam with a particular maximum moment-if the variation of this moment is non-uniform along the length (Fig. 6.5) the load carrying capacity is more than the beam with same maximum moment uniform along its length.

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

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

The effect of residual stresses is to reduce the lateral buckling capacity. If the compression flange is wider than tension flange lateral buckling strength increases and if the tension flange is wider than compression flange, lateral buckling strength decreases. The residual stresses and hence its effect is more in welded beams as compared to that of rolled beams.

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

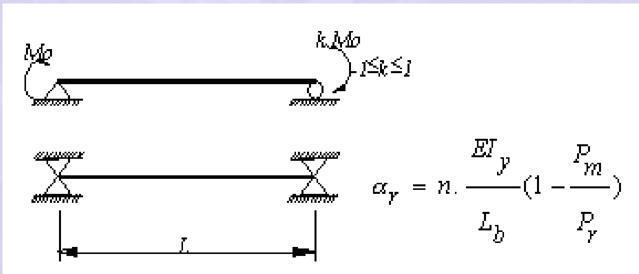

Fig 6.5 Beam subjected to Non-uniform moment

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

If the non-dimensional slenderness $\lambda_{LT} \leq 0.4$, no allowance for lateral-torsional buckling is necessary. Appendix **F** of the code gives the method of calculating $M_{cr,}$, the elastic lateral torsional buckling moment for difficult beam sections, considering loading and a support condition as well as for non-prismatic members.

6.3.3 Effective length of compression flanges

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

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

6.3.4 Shear

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

$$\tau = \frac{VQ}{It}$$

where

V = shear force at the section

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

Q = moment about neutral axis of the area that is beyond the fibre at which τ is calculated and '*t*' is the thickness of the portion at which τ is calculated.

The above Equation is plotted in Fig. 6.6, which represents shear stresses in the elastic range. It is seen from the figure that the web carries a significant proportion of shear force and the shear stress distribution over the web area is nearly uniform. Hence, for the purpose of design, we can assume without much error that the average shear stress as

$$\tau_{av} = \frac{V}{t_w d_w}$$

where

t_w = thickness of the web

d_w = depth of the web

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

$$\tau_y = \frac{f_y}{\sqrt{3}} = 0.58 f_y$$

where

f_y = yield stress.

The shear capacity of rolled beams V_c can be calculated as

$$V_c \approx 0.6 f_y t_w d_w$$

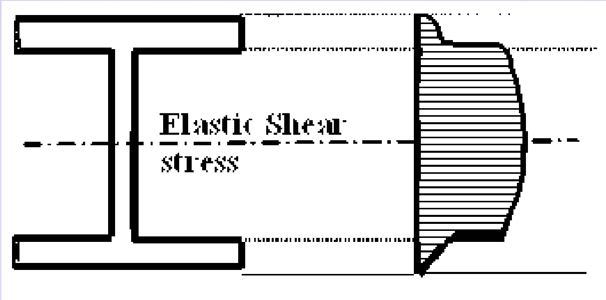

Fig 6.6 Elastic shear stresses

When the shear capacity of the beam is exceeded, the 'shear failure' occurs by excessive shear yielding of the gross area of the webs as shown in Fig 6.7. Shear yielding is very rare in rolled steel beams.

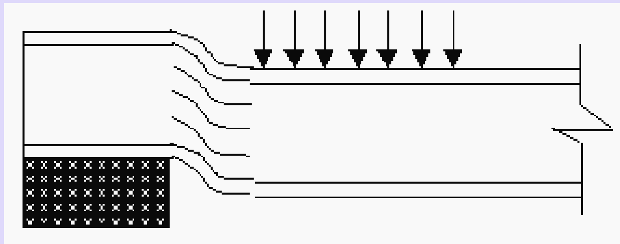

Fig 6.7 Shear yielding near support

The factored design shear force *V* in the beam should be less than the design shear strength of web. The shear area of different sections and different axes of bending are given in Section **8.4.1.1**

6.3.4.1 Resistance to shear buckling

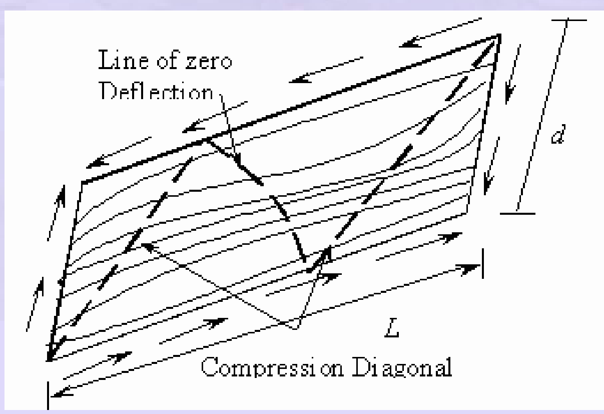

Fig 6.8 Buckling of a girder web in shear

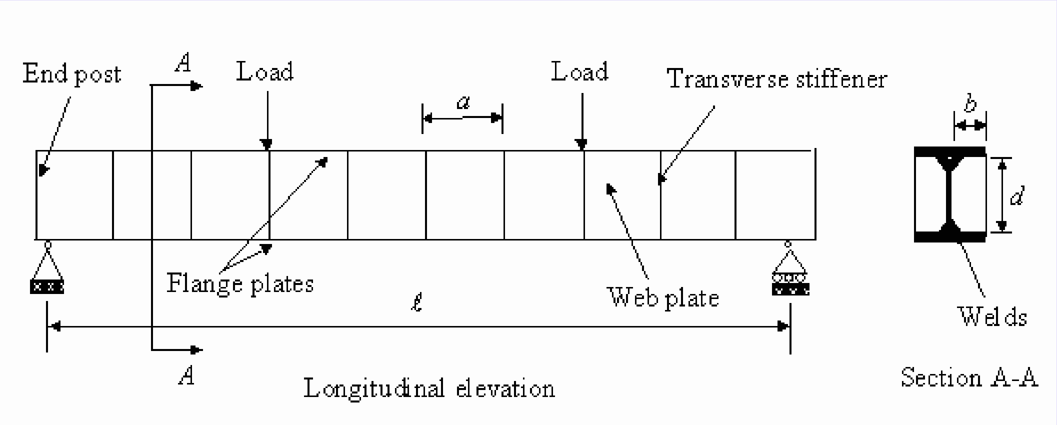

Fig 6.9 A typical plate girder

The girder webs will normally be subjected to some combination of shear and bending stresses. The most severe condition in terms of web buckling is normally the pure shear case. It follows that it is those regions adjacent to supports or the vicinity of point loads, which generally control the design. Shear buckling occurs largely as a result of the compressive stresses acting diagonally within the web, as shown in Fig.6. 8 with the number of waves tending to increase with an increase in the panel aspect ratio c */ d.*

When $d/t_w \leq 67\varepsilon$ where $\varepsilon = (250 / f_y)^{0.5}$ the web plate will not buckle because the shear stress τ is less than critical buckling stress 'τ_{cr}'. The design in such cases is similar to the rolled beams here. Consider plate girders having thin webs with $d/t_w > 67\varepsilon$. In the design of these webs, shear buckling should be considered. In a general way, we may have an un-stiffened web, a web stiffened by transverse stiffeners (Fig. 6.9) and a web stiffened by both transverse and longitudinal stiffeners (Fig. 6.10)

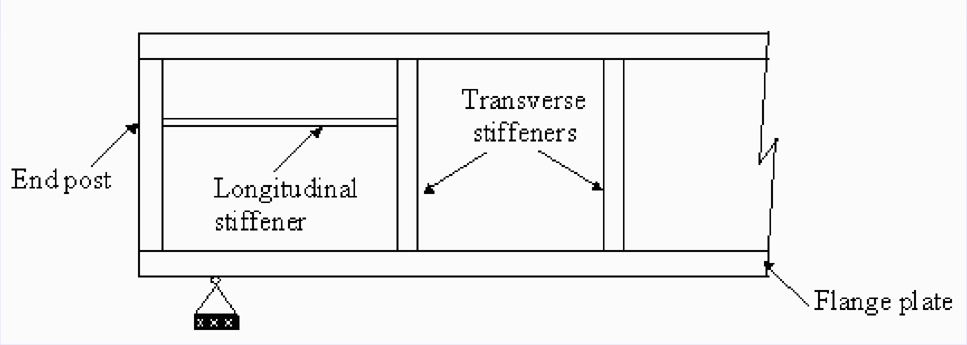

Fig 6.10 End panel strengthened by longitudinal stiffener

6.3.4.2 Shear buckling design methods

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

1. Simple Post Critical Method
2. Tension Field Method

6.3.4.2.1 Simple post critical method

It is a simplified version of a method for calculating the post-buckled member stress. The web possesses considerable post-buckling strength reserve and is shown in Fig. 6.11.

When a web plate is subjected to shear, we can visualize the structural behaviour by considering the effect of complementary shear stresses generating diagonal tension and diagonal compression. Consider an element E in equilibrium inside a square web plate with shear stress *q.* The requirements of equilibrium result in the generation of complementary shear stresses as shown in Fig.6.12. This result in the element being subjected to principal compression along the direction AC and tension along the direction BD. As the applied loading is incrementally enhanced, with corresponding increases in *q,* very soon, the plate will buckle along the direction of compression diagonal AC.

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

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

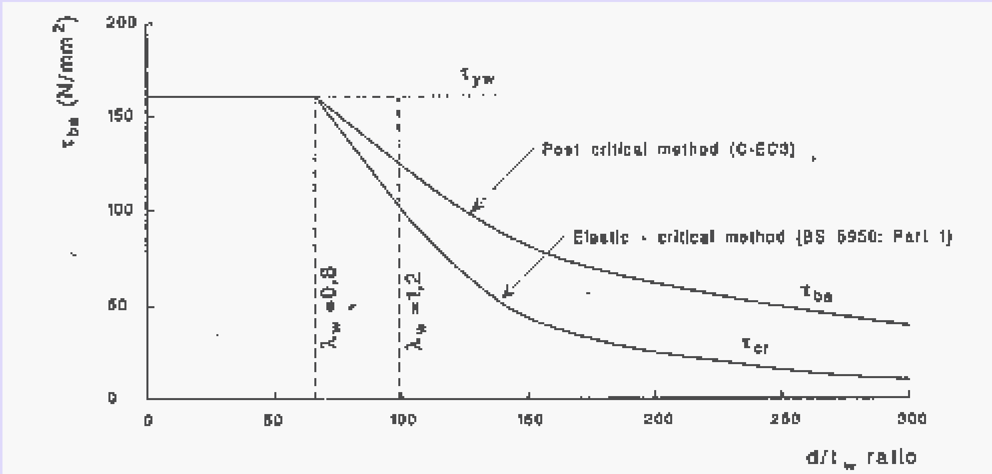

Fig. 6.11 Postbuckling reserve strength of web

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

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

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

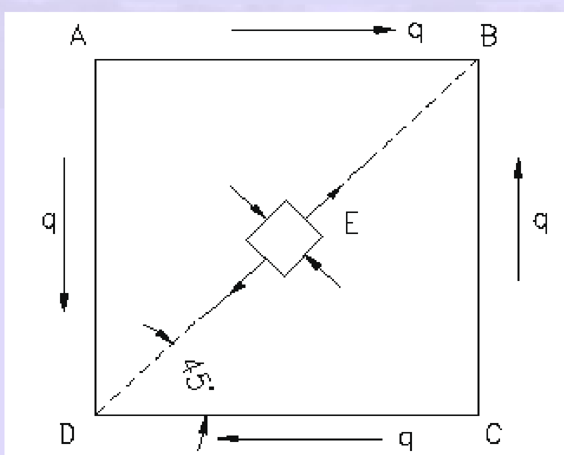

Fig 6.12 Unbuckled shear panel

When the value of (d/t) is sufficiently low (d/t < 85) τ_{cr} increases above the value of yield shear stress, and the web will yield under shear before buckling.

Based on this theory, the code gives the following values for τ_{cr} for webs, which are not too slender (Section 8.4.2.2a). The values depend on the slenderness parameter λ_w as defined in the code.

6.3.4.2.2. Tension field methods

Design of plate girders with intermediate stiffeners, as indicated in Fig. 6.10, can be done by limiting their shear capacity to shear buckling strength. However, this approach is uneconomical, as it does not account for the mobilisation of the additional shear capacity as indicated earlier. The shear resistance is improved in the following ways:

i. Increasing in buckling resistance due to reduced *c/d* ratio;

ii. The web develops tension field action and this resists considerably larger stress than the elastic critical strength of web in shear

Figure 6.13 shows the diagonal tension fields anchored between top and bottom flanges and against transverse stiffeners on either side of the panel with the stiffeners acting as struts and the tension field acting as ties. The plate girder behaves similar to an N-truss Fig.6.14.

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

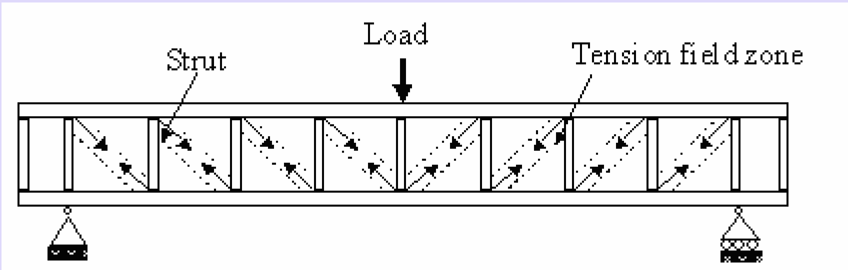

Fig 6.13 Tension field in individual sub-panel

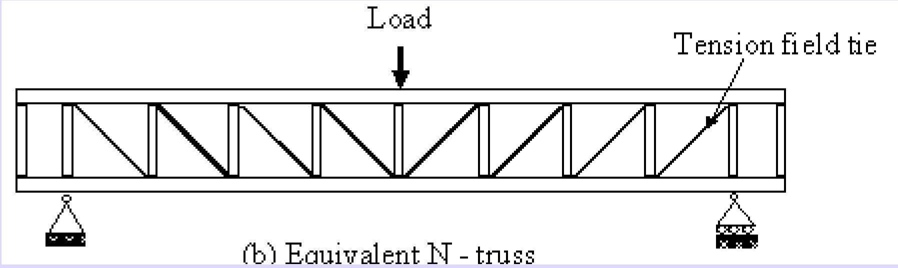

Fig 6.14 Tension field action and the equivalent N-truss

6.3.5 Stiffened web panels

For tension field action to develop in the end panels, adequate anchorage should be provided all around the end panel. The anchor force H_q required to anchor the tension field force is

$$H_q = 1.5V_{dp}\left(1 - \frac{V_{cr}}{V_{dp}}\right)^{1/2}$$

The end panel, when designed for tension field will impose additional loads on end post; hence, it will become stout (Fig 8.2 of the code). For a simple design, it may be assumed that the capacity of the end panel is restricted to V_{cr}, so that no tension field develops in it (Fig 8.1 of the code). In this case, end panel acts as a beam spanning between the flanges to resist shear and moment caused by H_q and produced by tension field of penultimate panel.

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

6.3.6 Design of beams and plate girders with solid webs.

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

Minimum web thickness – In general, we may have unstiffened web, a web stiffened by transverse stiffeners, (Fig 6.9) or web stiffened by both transverse and longitudinal stiffeners (Fig 6.10).

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

6.3.6.1 Compression flange buckling requirement

Generally, the thickness of flange plate is not varied along the span of plate girders. For non-composite plate girder the width of flange plates is chosen to be about 0.3 times the depth of the section as a thumb rule. It is also necessary to choose the breadth to thickness ratio of the flange such that the section classification is generally limited to plastic or compact section only. This is to avoid local buckling before reaching the yield stress. In order to avoid buckling of the compression flange into the web, the web thickness shall be based on recommendation given in Section **8.6.1.2** of the code.

6.3.6.2 Flanges

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

Usually, two flange angles at the top and two flange angles at the bottom are provided. These angles extend from one end to the other end of the girder. For a good proportioning, the flange angles must provide an area at least one-third of the total flange area.

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

6.3.6.3 Flange splices

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

6.3.6.3.1 Connection of flange to web

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

6.3.6.3.2 Splices in the web

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

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

6.3.7 Stiffener design

6.3.7.1 General

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

Stiffeners may be classified as

a) Intermediate transverse web stiffeners

b) Load carrying stiffeners

c) Bearing stiffeners

d) Torsion stiffeners

e) Diagonal stiffeners and

f) Tension stiffeners

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

6.3.7.2 Stiff bearing length

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

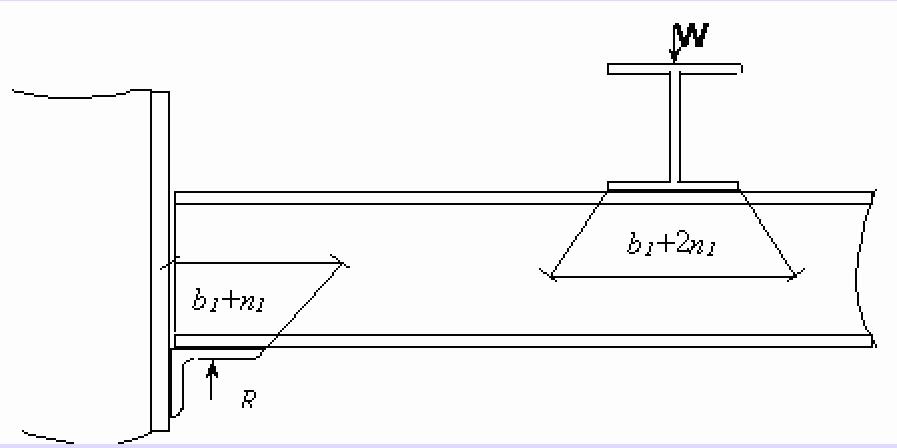

Fig 6.15 Dispersion of concentrated loads and reactions

In both cases, therefore, the load is actually spread out over a finite area by the time it passes into the web as shown in Fig.6.15. It is controlled largely by the dimensions of the plate used to transfer the load, which is itself termed "the stiff length of bearing"

Outstand of web stiffeners and eccentricity of the stiffeners is explained as per Section **8.7.1.2** to **8.7.1.5** of the code.

6.3.7.3 Design of Intermediate transverse web stiffeners

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

Intermediate transverse stiffeners must be proportioned so as to satisfy two conditions

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

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

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

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

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

6.3.7.4 Load carrying stiffeners

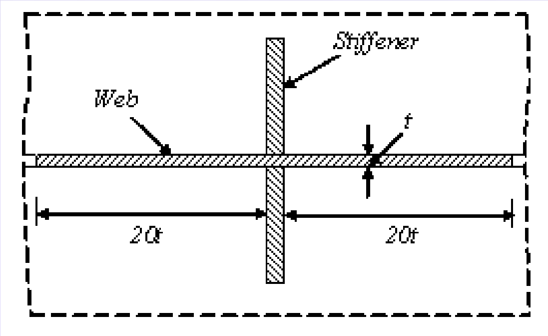

Fig.6.16 Cruciform section of the load carrying stiffener

Whenever there is a risk of the buckling resistance of the web being exceeded, especially owing to concentrated loads, load-carrying stiffeners are provided. The design of load-carrying stiffeners is essentially the same as the design of vertical stiffeners. The load is again assumed to be resisted by strut comprising of the actual stiffeners plus a length of web of 20*t* on either side, giving an effective cruciform section. Providing the loaded flange is laterally restrained, the effective length of the 'strut' may be taken as 0.7*L*. Although no separate stiffener check is necessary, a load-bearing stiffener must be of sufficient size that if the full load were to be applied to them acting independently, i.e., on a cross-section consisting of just the stiffeners, as per Fig 6.16. Then the stress induced should not exceed the design strength by more than 25%. The bearing stress in the stiffeners is checked using the area of that portion of the stiffeners in contact with the flange through which compressive force is transmitted.

6.3.7.5 Bearing stiffeners

Bearing stiffeners are required whenever concentrated loads, which could cause vertical buckling of web of the girder, are applied to either flange. Such situations occur on the bottom flange at the reactions and on the top flange at the point of concentrated loads. Figure 6.17 shows bearing stiffeners consisting of plates welded to the web. They must fit tightly against the loaded flange. There must be sufficient area of contact between the stiffeners and the flange to deliver; the load without exceeding the permissible bearing on either the flange material or the stiffeners must be adequate against buckling and the connection to the web must be sufficient to transmit the load as per Section **8.7.4** of the code.

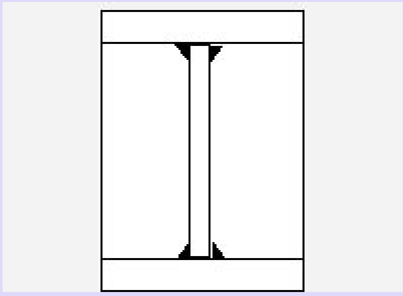

Fig. 6.17 Bearing stiffeners

The bearing stress on the contact area between stiffeners and flanges is analogous to the compressive stress at the junction of web and flange of rolled beams subjected to concentrated load.

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

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

Design of different types of stiffeners is given in the code from Sections **8.7.4** to **8.7.9**.

6.3.7.6 Connection of web of load carrying and bearing

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

6.3.7.7 Horizontal stiffeners

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

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

6.3.8 Box girders

The design and detailing of box girders shall be such as to give full advantage of its higher load carrying capacity.

Diaphragm shall be used where external vertical as well as transverse forces are to be transmitted from one member to another. The diaphragms and their fastenings shall be proportioned to distribute other force applied to them and in addition, to resist the design transverse force and the resulting shear forces. The design transverse force shall be taken as shared equally between the diaphragms.

When concentrated loads are carried from one beam to the other or distributed between the beams, diaphragms having sufficient stiffeners to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads.

6.3.9 Purlin and sheeting rails

Purlins attached to the compression flange of a main member would normally be acceptable as providing full torsional restraint; where purlins are attached to tension flange, they should be capable of providing positional restraint to that flange but are unlikely (due to the rather light purlin/rafter connections normally employed) to be capable of preventing twist and bending moment based on the lateral instability of the compression flange.

6.3.10 Bending in a non-principal plane

When deflections are constrained to a non-principal plane by the presence of lateral restraints, the principal axes bending moments are calculated due to the restraint forces as well as the applied forcs by any rational method. The combined effect is verified using the provisions in Section 9. Similarly, when the deflections are unconstrained due to loads acting in a non-principal plane, the principal axes bending moments are arrived by any rational method. The combined effects have to satisfy the requirements of Section 9.