15 PLATE GIRDERS - I

1.0 INTRODUCTION

A fabricated plate girder shown diagrammatically in Fig. 1 is employed for supporting heavy loads over long spans. The bending moments and shear forces produced in such girders are well beyond the bending and shear resistance of rolled steel girders available. In such situations the designer has the choice of one of the following solutions:

- Use two or more regularly available sections, side-by-side. This is an expensive solution and may still not satisfy deflection limitation.
- Use a cover-plated beam; i.e. weld a plate of adequate thickness to beef up each flange. This would enhance the bending resistance, in circumstances where the web has adequate shear resistance and the rolled steel section is only marginally inadequate.
- Use a fabricated plate girder, wherein the designer has the freedom (within limits) to choose the size of web and flanges, or
- Use a steel truss or a steel-concrete-composite truss.

This chapter is concerned with plate girders only. Plate girders are large I – shaped sections built up from plates, as shown in Fig. 2.

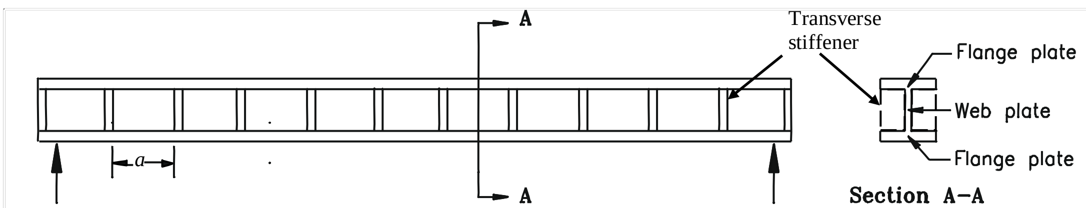

Fig. 1 Typical plate girder with intermediate and end stiffeners

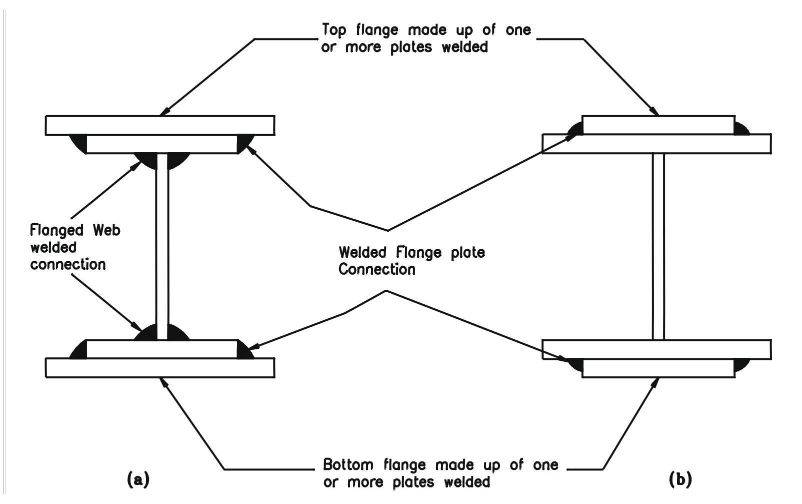

Fig. 2 Cross-section of fabricated plate girders

Nearly all plate girders built today are welded, although they may use bolted field splices. In the West, plate girders are invariably fabricated in fabrication shops, using numerically controlled welding machines [See Fig. 2(*a*)]. If the plate girders are to be fabricated in the field, the type sketched in Fig. 2(*b*) is used to minimise overhead welding.

The primary function of the flange plates is to resist bending moments by developing axial compressive and tensile stresses. The web plate resists the shear. For a given applied bending moment, the axial force decreases, as the depth of the girder (*d*) increases. From this point of view it is economical to keep the flanges as far apart as possible. This would ensure that the flanges would have to resist smaller axial forces. Thus a smaller area of cross section would suffice than would be the case if a smaller depth were chosen. However, this would also mean that the web would be deep. To reduce the self-weight (and the corresponding self-weight bending moment), the web thickness (*t*) would have to be limited to slender proportions, (the web proportions are normally expressed in terms of the web slenderness ratio, *d/t*). Slender webs (with large *d/t* values) would buckle at relatively low values of applied shear loading. (It is also important to note that webs having a span/depth ratio smaller than *12* would result in a *"deep"* beam, wherein the structural behaviour can no longer be described by conventional simple beam theories).

Efficient and economical design usually results in slender members. Hence advantage must be taken of the post buckling capacity of the web i.e. the ability of the girder to withstand transverse loads considerably in excess of the load at which the web buckles under shear. A girder of high strength to weight ratio can be designed by incorporating the post buckling strength of the web in the design method employed. This would be particularly advantageous where the reduction of self-weight is of prime importance. Examples of such situations arise in long span bridges, ship girders, transfer girders in building etc.

2.0 SHEAR RESISTANCE OF TRANSVERSELY STIFFENED PLATE GIRDERS

Webs of plate girders are usually stiffened transversely as shown in Figure 1. This helps to increase the ultimate shear resistance of the webs, as will be seen later. The stiffener spacing *(a)* influences both buckling and post buckled behaviour of the web under shear. In order to allow for this, the parameter (*a/d*) which accounts for the geometry of the web panel is important. Obviously, a long span girder will have various web panels and each panel will have different combinations of bending moments and shear forces. In a long plate girder, panels close to the support will be subjected to predominant shear and those close to the centre, to predominant bending moments.

In what follows, the effect of shear will be considered first, followed by the effect of co-existing bending moment and shear forces.

2.1 Shear resistance of a web

2.1.1 Pre-buckling behaviour (Stage 1)

When a web plate is subjected to shear, we can visualise the structural behaviour by considering the effect of complementary shear stresses generating diagonal tension and diagonal compression.

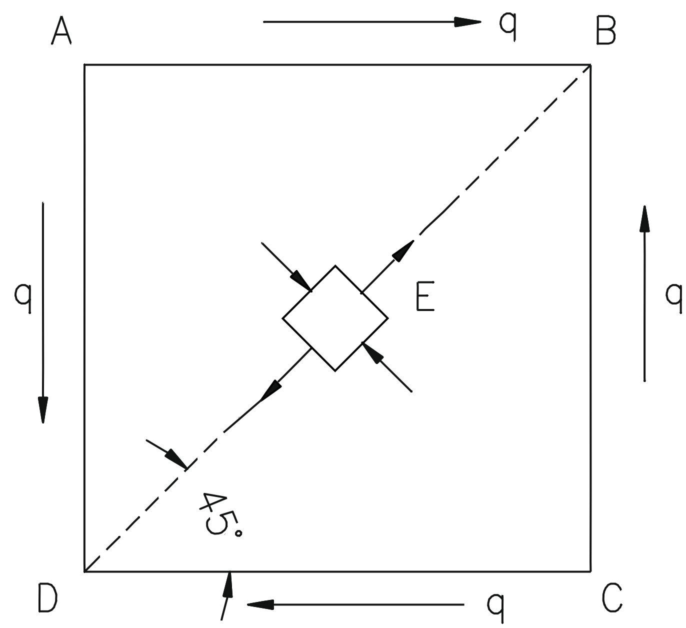

Fig. 3 Unbuckled shear panel

Consider an element *E* in equilibrium inside a square web plate subject to a shear stress *q*. The requirements of equilibrium result in the generation of complementary shear stresses as shown in Fig. 3. This results in the element being subjected to principal compression along the direction *AC* and tension along the direction of *BD*. As the applied loading is incrementally enhanced, with corresponding increases in *q*, very soon, the plate will buckle along the direction of compressive 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"* q_{cr}. The value of q_{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 restraint offered by the flanges and stiffeners we may conservatively assume them to be simply supported. The critical shear stress in such a case is given by

$$q_{cr} = k_s \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{d}\right)^2 \qquad (1)$$

where, k_s is the shear buckling coefficient given by

$$k_s = 5.35 + 4\left(\frac{d}{a}\right)^2 \quad where\ \frac{a}{d} \geq 1,\ \ i.e.\ for\ wide\ panels$$

$$k_s = 5.35\left(\frac{d}{a}\right)^2 + 4\ where\ \frac{a}{d} \leq 1,\ \ i.e.\ for\ webs\ with\ closely\ spaced\ transverse\ stiffeners$$

Values of q_{cr} for various values of web aspect ratios are tabulated in Table 1.

Table 1: q_{cr} (MPa) Values

d/t \ a/d	**1.0**	**1.5**	**2.0**	**0.5**
100	169	129	115	Buckling does not govern
125	108	83	73	Buckling does not govern
150	75	57	51	204
175	55	42	37	150
200	42	32	29	115
225	33	25	23	91
250	27	21	18	72

$E = 200{,}000\ MPa \qquad \nu = 0.3 \qquad \pi = 3.1412$

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

2.1.2 Post buckled behaviour (Stage 2)

The compression diagonal (*AC*) is unable to resist any more loading beyond the one corresponding to the elastic critical stress. Once the web has lost its capacity to sustain increase in compressive stresses, a new load–carrying mechanism is developed. Applications of any further increases in the shear load are supported by a *tensile membrane field*, anchored to the boundaries, viz. the top and bottom flanges and the adjacent stiffener members on either side of the web. The angle of inclination of the membrane stress (θ) is unknown at this stage (See Fig. 4). Thus the total state of stress in this web plate may be obtained by superimposing the post-buckled membrane tensile stresses (p_t) upon those set up when the applied shear stress reached the critical value q_{cr}.

The state of stress in the web in the post-buckled stage is shown in Fig. 5.

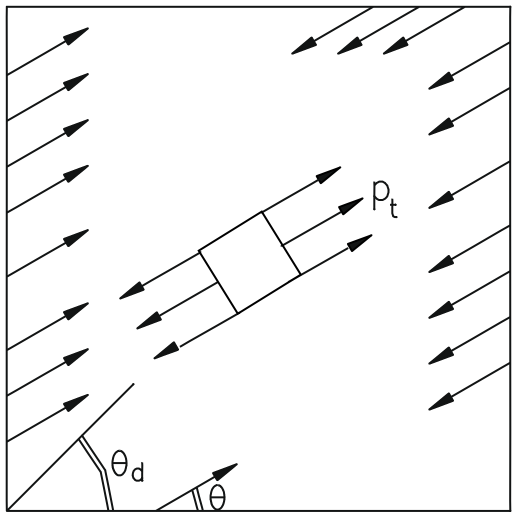

Fig. 4 Post buckled behaviour

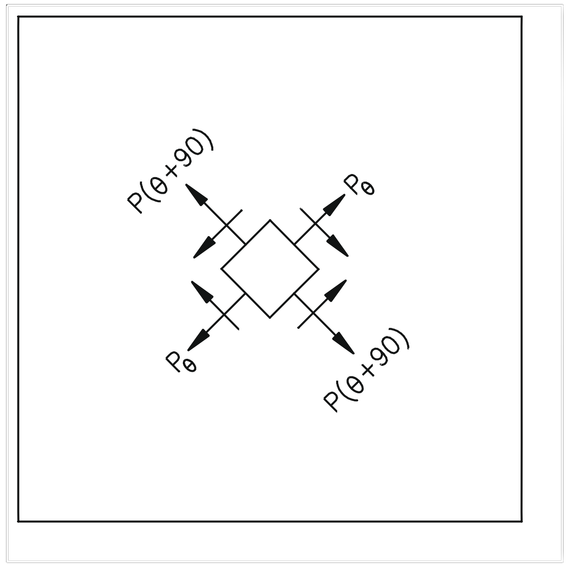

Fig. 5 State of stress in the web in the post buckled stage

Resolving these stresses in the direction along and perpendicular to the inclination θ we get,

$$p_\theta = q_{cr}.\sin 2\theta + p_t$$
$$p_{(\theta+90)} = -q_{cr}\sin 2\theta \qquad (2)$$
$$q_\theta = -q_{cr}.\cos 2\theta$$

Since the flanges are of finite rigidity, the pull exerted by the tensile membrane stresses in the web will cause the flanges to bend inwards.

2.1.3 Collapse behaviour (Stage 3)

When the load is further increased, the tensile membrane stress *(p_t)* developed in the web continues to exert an increasing pull on the flanges. Eventually the resultant stress *(p_θ)* (obtained by combining the buckling stress in Equation (1) and the membrane stress p_t) reaches the yield value for the web.

This value (of the membrane stress at yield) may be denoted by p_{yw}, and may be determined by Von-Mises yield criterion.

$$p_\theta^2 + p_{(\theta+90)}^2 - p_\theta . p_{(\theta+90)} + 3q_\theta^2 = p_{yw}^2 \qquad (3)$$

where, p_{yw} = tensile yield stress of the web

By substituting the values of p_θ, $p_{(\theta+90)}$ *and* q_θ from Equation (2), the above equation may be presented in a non-dimensional form as follows:

$$\frac{p_{yt}}{p_{yw}} = \sqrt{\left(1-\frac{q_{cr}}{q_{yw}}\right)^2\left(1-\frac{3}{4}\sin^2 2\theta\right)} - \frac{\sqrt{3}}{2}.\frac{q_{cr}}{q_{yw}}\sin 2\theta \qquad (4)$$

where, q_{cr} is obtained from Equation (1)

$$q_{yw} = \frac{p_{yw}}{\sqrt{3}} \qquad i.e.\, shear\ yield = \frac{tensile\ yield}{\sqrt{3}} \qquad (5)$$

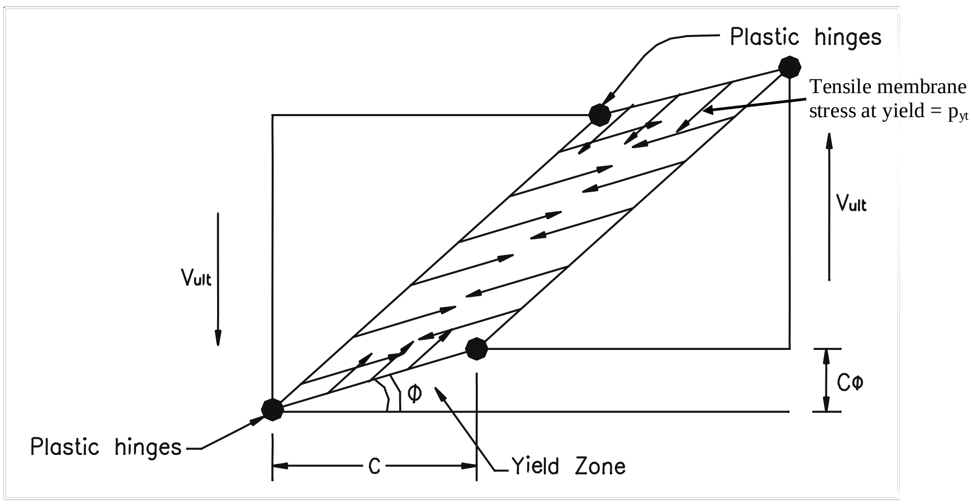

Fig. 6 Collapse of the panel

Once the web has yielded, final collapse of the girder will occur when four plastic hinges are formed in the flanges as shown in Fig. 6. The plastic moment capacity of the flange plate is M_{pf}.

By using the virtual work method, Rockey and his team at Cardiff have shown that the failure load can be computed from

$$V_S = q_{cr}.d.t + p_{yt}.t\sin^2\theta\left(d\cot\theta - a + c\right) + 4\frac{M_{pf}}{c} \qquad (6)$$

where c = distance between the hinges given by

$$c = \frac{2}{\sin\theta}\sqrt{\frac{M_{pf}}{p_{yt}.t}} \qquad (7)$$

This equation can be non-dimensionalised by using the shear load required to produce yielding in the entire web $(V_{yw} = q_{yw}.\ d.t.)$

$$\frac{V_S}{V_{yw}}=\frac{q_{cr}}{q_{yw}}+\sqrt{3}\sin^2\theta\left(\cot\theta-\frac{a}{d}\right)\frac{p_{yt}}{p_{yw}}+4\sqrt{3}\sin\theta\sqrt{\frac{p_{yt}}{p_{yw}}\cdot\frac{M_{pf}}{d^2t.p_{yw}}} \quad (8)$$

Equation (6) or (8) can be solved if θ is known. As these equations are based on Energy Method, the correct solution will be obtained by maximising V_s with respect to θ. By a systematic set of parametric studies, Evans has established that θ is approximately equal to *2/3* of the inclination of diagonal of the web.

$$\theta \cong \frac{2}{3}\tan^{-1}\left(\frac{d}{a}\right) \quad (9)$$

Notice that Equations (6) and (8) are obtained by adding 3 quantities: web-buckling strength, post-buckling membrane strength of the web plate and the plastic moment capacity of the flange.

In this context, it must be noted that in order for the flanges to develop hinges, the flanges must be classifiable as "***plastic***" sections. If flanges can not develop plastic hinges because they are compact, semi-compact or slender, this method of analysis is NOT applicable.

2.1.4 "Weak" flanges

When a plate girder has weak flanges, M_{pf} is a small quantity in comparison with the other terms. Hence

$$\frac{V_S}{V_{yw}}=\frac{q_{cr}}{q_{yw}}+\sqrt{3}\sin^2\theta\left(\cot\theta-\frac{a}{d}\right)\frac{p_{yt}}{p_{yw}} \quad (10)$$

In this case the tension field is NOT supported by flanges.

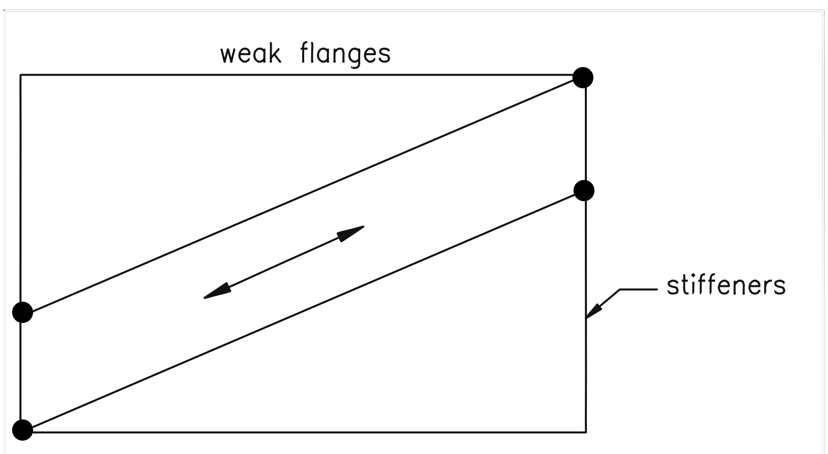

Fig. 7 Weak flange

The field anchors entirely on transverse stiffeners as shown in Fig. 7.

2.1.5 Very "Strong" flanges

When *"Very Strong"* flanges are employed, the distance *(c)* of the plastic hinge away from the end panel increases. When $c = a$, the hinges will form at the four corners, constituting a *"picture frame"* type mechanism (See Fig. 8) and the tension field angle (θ) is $45°$. Ultimate shear in this case is given by:

$$\frac{V_S}{V_{yw}} = \frac{1}{4}\frac{q_{cr}}{q_{yw}} + \frac{\sqrt{3}}{2}\sqrt{\left(1 - \frac{1}{4}\left(\frac{q_{cr}}{q_{yw}}\right)^2\right)} + 4\sqrt{3}\,\frac{d}{a}.\frac{M_{pf}}{d^2 t\, p_{yw}} \qquad (11)$$

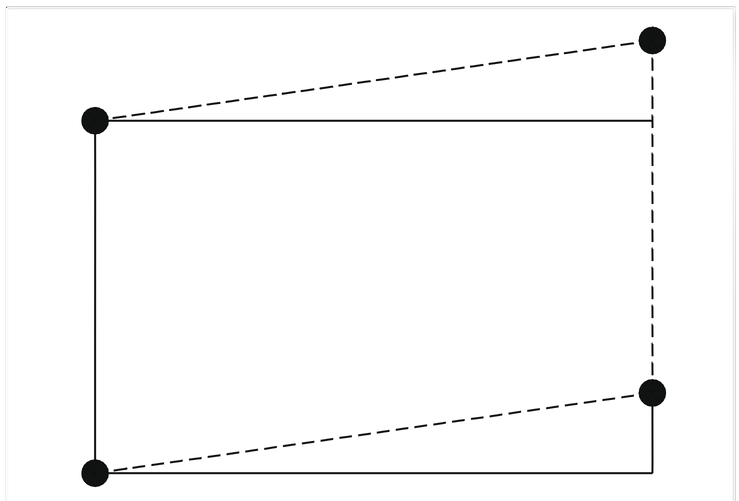

Fig. 8 Picture frame mechanism of strong flange

2.1.6 Very "Thick" webs

In case the web is thick, it will yield before buckling failure will form by a picture frame mechanism, with $q_{cr} = q_{yw}$.

$$\frac{V_S}{V_{yw}} = 1 + 4\sqrt{3}\left(\frac{d}{a}\right).\frac{M_{pf}}{d^2 t\, p_{yw}} \qquad (12)$$

2.1.7 Very "Slender" webs

Very slender webs are rarely used as they cause anxieties for the users due to high levels of buckling.

In very slender webs q_{cr}/q_{yw} is extremely small and there will be significant post-buckled tension field, the value of membrane stress p_{yt} will be very large. The general expression given in Equation (8) is valid and θ can be evaluated using equation (9).

3.0 WEBS SUBJECTED TO CO-EXISTENT BENDING AND SHEAR

When a girder is subjected to predominant bending moments and low shear, its ultimate capacity is conditioned by the interaction between the effects of the bending moment and shear force.

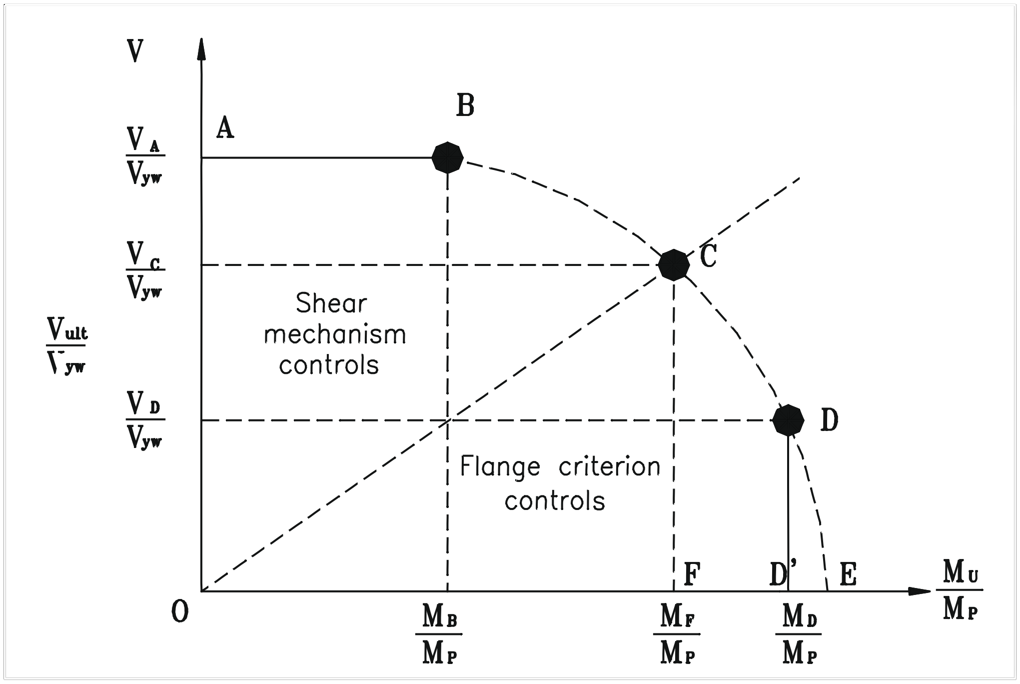

Fig . 9(a)

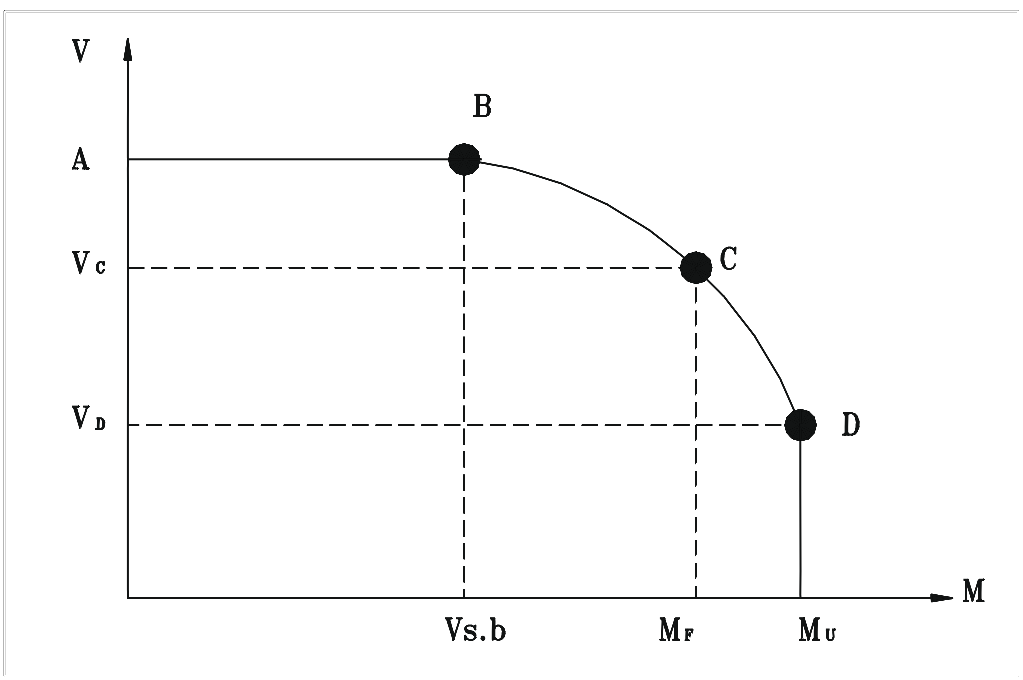

Fig. 9(b)

Fig. 9 Interaction between bending and shear effects

The interaction diagram is generally expressed in the form seen in Fig. 9, where the shear capacity is plotted in the *Y*-axis and the bending capacity in the *X*-axis. Any point in the interaction diagram shows the co-existent values of shear and bending moment that the girder can sustain. The vertical ordinates are non-dimensionalised using V_{yw} (Yield shear of the web) and the horizontal ordinates by M_p (the fully plastic moment resistance of the cross section). The portion of the curve between points *A* and *C* is the region in which the girder will fail by predominant shear, i.e. shear mechanism of the type represented in Fig. 6 will develop at collapse.

The vertical ordinate at A presents the shear capacity (V_s) given by Equation 8. This shear capacity will reduce gradually due to the presence of co-existent bending moment. Beyond point *C*, when the applied moment is high, the failure will be triggered by the collapse of flanges by one of the following: (i) by yielding of flange material or (ii) by inward buckling of the compression flange or (iii) by lateral buckling of the flange. Thus there is a distinct change in failure criterion represented by line *OC* in Fig. 9(*a*); the left of *OC* represents shear failure and the right of *OC*, flexural failure. Generally the flange failure mode will be triggered, when the applied bending moment is approximately equal to the plastic moment resistance M_F, provided by the flange plates only, neglecting the contribution from the web.

$$M_F = b_f \,.\, T \,.\, p_{yf} \left(d + T\right) \qquad (13)$$

where, b_f - Breadth of flange
T - Thickness of flange
p_{yf}- Design stress of flange
d - depth of web plate

This value represents the horizontal co-ordinate of the point *C*, i.e. the point *F*. In zone *ABC*, the presence of additional bending moment requires the following three factors to be considered.

- The reduction in the web buckling stress due to the presence of bending stresses.
- The influence of bending stresses on the value of membrane stress required causing yield in the web.
- The reduction of plastic moment capacity of flanges due to the presence of axial flange stresses caused by bending moment.

3.1 Modified web buckling stress

The modified web buckling stress due to coincident bending stress may be computed from the following interaction Equation:

$$\left(\frac{q_{crm}}{q_{cr}}\right)^2 + \left(\frac{f_{mb}}{f_{crb}}\right) = 1 \qquad (14)$$

where, q_{crm} = modified shear buckling stress in web

q_{cr} = elastic critical shear stress in web (pure shear case as defined previously)

f_{mb} = compressive bending stress in the extreme fibre at the mid panel due to the bending moment.

f_{crb} = buckling stress for the plate due to a pure bending moment given by

$$f_{crb} = 23.9\ \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{d}\right)^2 \qquad (15)$$

3.2 Modified membrane stress for web yielding

The bending membrane stress *(p_{yt})* to be added to the critical shear stress *(q_{cr})* was calculated in the pure shear case, using Equations (4) and (1) respectively. The modified expression for the membrane stress p_{ytm}, in the presence of applied bending moment is given by

$$p_{ytm} = -\frac{1}{2}A + \frac{1}{2}\sqrt{\left(A^2 - 4\left(p_b^2 + 3q_{crm}^2 - p_{yw}^2\right)\right)} \qquad (16)$$

where, $A = 3\, q_{crm} \sin 2\theta + p_b \sin^2\theta - 2p_b \cos^2 \theta$

p_b = The value of bending stress, which varies over both the depth and width of the web panel.

As p_{ytm} varies for various values of p_b, it may be necessary to compute p_{ytm} at a number of locations in order to compute the resultant of the membrane stresses. This could be time-consuming. To simplify the design calculations an adequately accurate approximate procedure is suggested a little later.

3.3 Reduction of plastic moment capacity of flanges

When high axial forces are developed in the flanges due to bending moments, their effects in reducing plastic moment capacity of flange plates must be taken into account. From plasticity theory, the reduced capacity *(M'_{pf})* is given by

$$M'_{pf} = M_{pf}\left[1 - \left(\frac{p_f}{p_{yt}}\right)^2\right] \qquad (17)$$

where, p_f is the average axial stress for the portion of the flange between hinges.

3.4 Design procedure

The simplified design procedure suggested by Rockey et. al (*1978*) is validated by them by experiments and parametric studies. This procedure is summarised below:

The shear load capacity at point *C* of the interaction diagram may be obtained approximately from an empirical relationship given below.

$$\left(\frac{V_c}{V_{yw}}\right) = \frac{q_{cr}}{q_{yw}} + \frac{p_{yt}}{p_{yw}} \sin\left(\frac{4}{3}\theta_d\right)\left[0.554 + 36.8\,\frac{M_{pf}}{M_F}\right]\left[2 - \left(\frac{b}{d}\right)^{\frac{1}{8}}\right] \qquad (18)$$

This equation gives the vertical ordinate of the point *C* in the interaction diagram [Fig. 9(*a*)]. The horizontal ordinate as stated previously is given by the value of M_F (See Equation 13).

The interaction diagram is constructed in stages as follows [See Fig. 9(*b*)]:

(i) Between *A* and *B*, the curve is horizontal. The horizontal ordinate *B* is given by maximum bending moment in the end panel given by $V_s.b$, but limited to a value of $0.5M_P$.

(ii) Between *B* and *C*, the curve may be straight (for simplicity). The moment corresponding to *C* is given by

$$M_F = b_f \,.\, T \,.\, p_{yf}\ (d + T)$$

(iii) The point *D* represents nearly the ultimate capacity of the flanges (M_u) and the shear values when high bending is present. This is discussed in the next section.

3.5 Webs subjected to pure bending

The region beyond *C* of the interaction diagram represents a high bending moment, so the failure is by bending moment, rather than by shear mode. In a thin walled girder, the web subjected to compressive bending stress will buckle, thereby losing its capacity to carry further compressive stresses. The compression flange will therefore carry practically all the compressive stresses, as the web is unable to be fully effective. Consequently the girder is unable to develop full plastic moment of resistance (M_p) of the cross section.

If no lateral buckling occurs (e.g. by provision of adequate lateral supports), the girder will fail by inward collapse of compression flange at an applied moment (M_u) which is approximately equal to the moment required to produce first yield in the extreme fibres of compression flange. This moment is – of course – reduced because of the effects of web buckling. Though the concept is simple, the resulting calculations are complex. The research of Cooper in *1971* enables the ultimate moment capacity to be determined by a simple formula:

$$\frac{M_u}{M_y} = 1 - 0.0005\,\frac{A_w}{A_f}\left[\frac{d}{t} - 5.7\sqrt{\frac{E}{p_{yf}}}\right] \geq \frac{M_F}{M_y} \qquad (19)$$

M_y = Bending moment required to produce yield in the extreme fibre of flange assuming fully effective web (i.e. neglecting web buckling)

This value of M_u is the moment required to produce yield in the extreme fibres of the flange. The corresponding stresses in the web will be below yield. (Point *D* in the interaction diagrams). The ordinate of *D* can be calculated approximately from

$$\frac{V_D}{V_C} = \sqrt{\frac{M_p - M_u}{M_{pw}}} \qquad (20)$$

where, M_p is the fully plastic moment capacity of the complete cross section
M_{pw} = plastic moment resistance of the web plate alone.
$= 0.25\ td^2.\ p_{yw}$
The complete interaction diagram can now be drawn.

4.0 ULTIMATE BEHAVIOUR OF TRANSVERSE WEB STIFFENERS

The shear failure mechanism described so far has been extensively verified by experiments. Before post-buckling action in webs can develop the members in the boundary (viz. the flanges and the stiffeners) must be able to support the forces imposed on them by the web tension field.

The transverse stiffeners play an important role in allowing the full ultimate capacity of the girder to be achieved (a) by increasing the web buckling stress (b) by supporting the tension field after web buckling and (c) by preventing the tendency of flanges to get pulled towards each other. The stiffeners must therefore possess sufficient rigidity to ensure that they remain straight, while restricting buckling to the individual web panels.

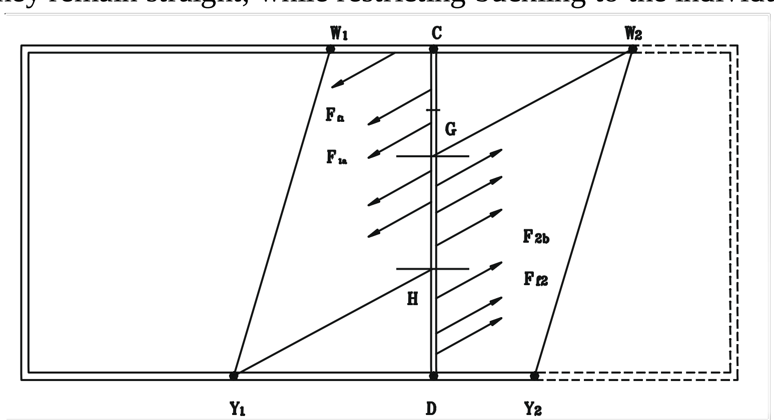

Fig. 10 Force imposed on transverse stiffeners by tension field

4.1 Analysis of loads imposed on the transverse stiffener

Fig. 10 represents the loads acting on typical stiffener *CD* positioned between two adjacent panels, each of which have developed shear failure mechanism. This is perhaps the most critical form of loading of an intermediate stiffener.

Let us consider the loads on the intermediate stiffener

The resultant of the loads acting on portion W_1C of top flanges is F_{fw}, inclined at an angle of θ_1 and DY_2 of bottom flange is $F_{f.2}$, inclined at an angle of θ_2. The vertical component of these forces will tend to pull the flanges together and the stiffener will resist this by developing end loads (V_C and V_D).

$$V_C = -F_{f.1}\sin\theta_1$$
$$V_D = +F_{f.2}\sin\theta_2 \qquad (21)$$

Moreover, the loading imposed directly upon the stiffener by web tension field can be split into *3* zones: the top part *CG* is subject to a pull to the left by the left hand panel, the bottom part *HD* is similarly subject to a pull to the right by the right hand panel. The part *GH* is pulled to the left and to the right (these two forces more or less balancing each other). Thus the central region remains virtually unloaded by the tension field action.

The vertical component of forces on zones CG and *DH* are respectively obtained as

$$V_1 = -p_{yt1}.t.CG.\sin\theta_1.\cos\theta_1$$
$$V_2 = -p_{yt2}.t.HD.\sin\theta_2.\cos\theta_2 \qquad (22)$$

Thus once the ultimate shear loading and the geometry of failure mechanism has been determined, calculations of forces on the stiffeners by employing Equations (21) and (22) may be made.

Unfortunately, the actual behaviour of stiffeners (as evidenced by experimental studies by Rockey in 1981 and Puthali in 1979) is somewhat different from the simple model described above.
Firstly, a portion of the web plate acts with the stiffeners in resisting the axial load, despite the fact the web has theoretically yielded due to tension field. The effective cross section is in the form of a *T* section or a cruciform section, if the stiffeners are on both sides of the web plate.

Secondly the theory explained previously assumed that the axial loading is applied to the stiffener cross section at its mid thickness. The true position of load application is unknown and some degree of eccentricity of application of loads is inevitable.

Thus the stiffener is subjected to both axial load *(P)* and bending moment arising due to the eccentricity of the applied load from the centroidal axis ($\bar{x}$) and is given by $P\bar{x}$. There will–inevitably be–some imperfections (δ_o) in the stiffener giving rise to consequent moments of $P\delta_o$. The disturbing action on the stiffener due to the web buckling is difficult to quantify but nevertheless is present.

Horne (*1979*) has proposed a suitable expression to define the combination of axial load and bending moment (making allowance for the above complexities) which can be sustained by a stiffener:

$$\frac{M}{M_{ps}} = 1.0 - \frac{p_{ys}\, t_s \left(b_s - \bar{x} + 0.5t\right)^2}{M_{ps}} \frac{P}{P_s} \qquad (23)$$

where, M_{ps} = Full plastic moment capacity of the section where there is no axial loading
P_s = Squash load (i.e. full axial yield load)
b_s = Width of stiffener
t_s = Thickness of stiffener
p_{ys} = Design stress of stiffener
t = Thickness of web

For any girder, the axial load to which the stiffener is subjected can be computed from Equations (21) and (22). Then, provided the co-existent moment is less than the allowable moment defined by Equation (23), the stiffener will be able to support the loads to which it is subjected.

The theory governing the design of stiffeners is given above; but the design codes make simplifications to ease the task of the designers and enable quick sizing of the stiffeners.

5.0 GENERAL BEHAVIOUR OF LONGITUDINALLY STIFFENED GIRDERS

In order to obtain greater economy and efficiency in the design of plate girders, slender webs are often reinforced both longitudinally and transversely. The longitudinal stiffeners are generally located in the compression zones of the girder. The main function of the longitudinal stiffeners is to increase the buckling resistance of web. The longitudinal stiffener remains straight thereby subdividing the web and limiting the web buckling to smaller web panels. In the past it was usually thought that the resulting increase in ultimate strengths could be significant. Recent studies have shown that this is not always the case, as the additional cost of welding the longitudinal stiffeners invariably offsets any economy resulting in their use.

The main effect of longitudinal stiffeners is to increase the elastic critical buckling strength. Studies by Rockey et al have shown that a longitudinally reinforced plate girder subject predominantly to shear would develop a collapse mechanism, similar to the one described previously, provided the stiffeners remained rigid up to failure. Once a tension field develops it extends over the complete depth of the girder. In other words, once one of the sub panels has buckled, the post buckling tension field develops over the whole depth of the web panel and the influence of the stiffeners may be neglected. Thus the equations established previously are valid, keeping in mind q_{cr} values are enhanced due to the smaller panel dimensions.

In the design of longitudinal stiffeners, linear buckling theories are often used to establish the minimum value of stiffener rigidity required to ensure that the longitudinal stiffeners remain straight at first buckling. This would ensure that the buckling is limited to individual sub-panels of the web. This invariably involves the provision of stiffeners of substantial sizes, just so that they would remain straight without themselves buckling.

Normally the rigidity of such stiffeners has to be increased by *4-6* times, to satisfy this requirement. The heavy stiffeners thus designed, though adequate, will naturally increase the weight of steel used and therefore the cost. It seems that it is more sensible to increase the web thickness in these cases.

In general, the extra cost of providing longitudinal stiffeners is rarely justified. In western countries numerically controlled machines are used extensively for fabricating plate girders. In these countries the provision of longitudinal (and transverse) stiffeners involves manual welding and contributes to the rise in the cost of fabrication. Indeed, in Scandinavian countries, the current trend is to eliminate or minimise the use of transverse stiffeners as far as possible. The use of longitudinal stiffeners has also been discontinued for this reason in these countries. In other words, if they can help it, they do not use any stiffeners at all!

Longitudinal stiffeners are rarely – if ever – used in buildings. They are sometimes used in bridges, particularly when the Elastic Design is employed. Fig. 11 shows a typical plate girder with longitudinal and transverse stiffeners.

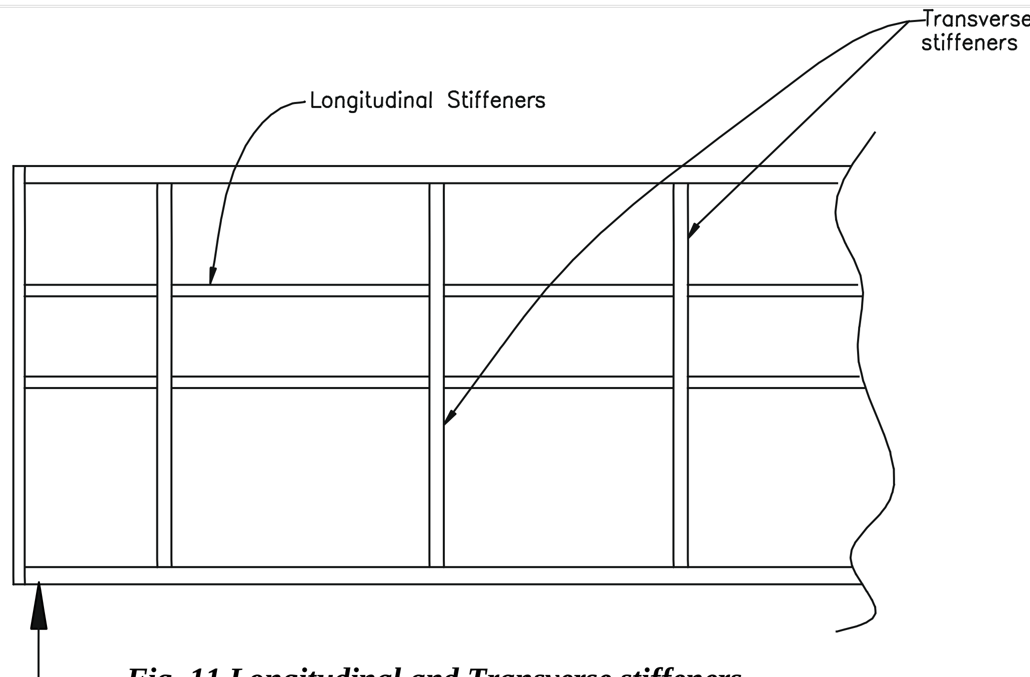

Fig. 11 Longitudinal and Transverse stiffeners

6.0 CONCLUSIONS

This chapter has considered the ultimate behaviour of plate girders in some detail. Fundamental theoretical relationship based on buckling and post-buckling theories have been established. In some case, semi-empirical procedures have been suggested to ease the tedium of lengthy calculations. Transverse stiffeners have been considered in some depth. The use of longitudinal stiffeners has also been described and the reasons for not using these extensively have been discussed.

7.0 REFERENCES

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