CHAPTER 08

Plate Girder

8.1 Introduction

- For large spans, rolled sections do not provide the sufficient section modulus to resist the applied bending moment. Thus in such cases, it is required to increase the section modulus being provided by a section. This can be achieved by any one of the following methods:
  (a) Providing two or more I-sections connected properly
  (b) Plate girder
  (c) Truss girder
- Plate girder has moment resisting capacity which lies between rolled I-sections and truss girders.

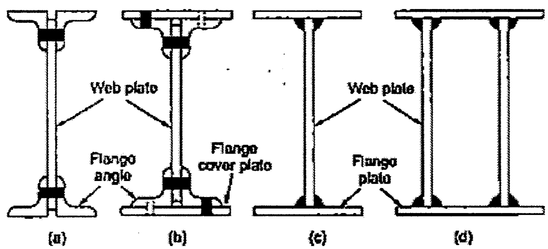


Fig. 8.1 Some typical arrangements of plate girder

- The main advantage with plate girder is that it can be built to suit the any desired proportion and of the cross section with properties which are exactly needed.
- Plate girders are built up flexural members and are preferred for spans greater than 10 m.

NOTE
1. Riveted and bolted plate girders are used for spans varying from 15 m to 30 m. However welded plate girders can be used for spans up to 100 m. It is to be noted that, as the span increases, truss girder offers more economy than the plate girder.
2. Welded plate girders are more economical than riveted or bolted plate girders due to appreciable decrease in the self-weight of the plate girder and are also more efficient and simple.

- The design of plate girder is similar to that of beam design. It involves proportioning of the girder to provide sufficient section modulus to resist bending along with a web which is sufficient enough to resist shear and of sufficient stiffness. Web buckling is of great concern in case of design of plate girders. The problem of web buckling can be eliminated either by providing thick web (which is quite uneconomical) or by providing adequate stiffeners along its length thereby dividing the web of plate girder into smaller panels and reducing the span and increasing the web buckling resistance.

8.2 Elements of a Plate Girder

(a) Web plate

(b) Flange angles with or without cover plates for riveted/bolted plate girders and only flange plates for welded plate girders.

(c) Stiffeners-Bearing, transverse and longitudinal

(d) Splices for web and flange

Fig. 8.2 shows typical elements of a riveted/bolted plate girder and Fig. 8.3 shows typical elements of welded plate girder.

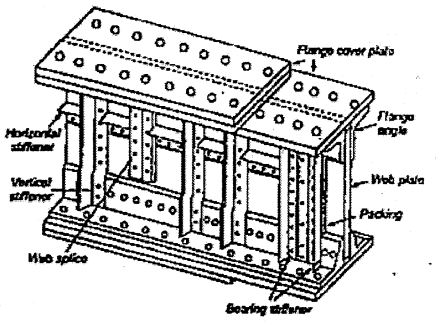


Fig. 8.2 Typical elements of a riveted/bolted plate girder

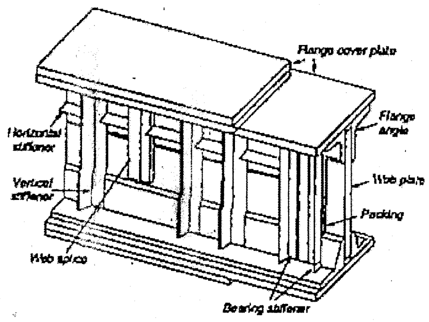


Fig. 8.3 Typical elements of welded plate girder

8.2.1 Functions of Various Elements of a Plate Girder

- **Flanges:** Flanges of required width and thickness are required to resist the flexural moment due to dead and superimposed loads coming on the plate girder by developing compressive force in one flange and tensile force in the other flange i.e. by developing an opposite resisting couple.
- **Web:** Web of required depth and thickness is required to keep the flange plates at the requisite distance and to resist shear in the beam.
- **Stiffeners:** Stiffeners are provided to safeguard the web against local buckling failure. These stiffeners are provided either in the longitudinal direction called as horizontal stiffeners or in the transverse direction called as vertical stiffeners or in both the directions.

Transverse (vertical) stiffeners are of two types viz.:

(a) **Bearing stiffener:** At the support regions, a certain portion of web acts as column i.e. compression member and there is a possibility of web buckling and hence bearing stiffeners are required.

(b) **Intermediate stiffener:** If concentrated loads are also acting on the girder (may be due to cross beams etc.) then intermediate stiffeners are also required to prevent the web from buckling.

Remember To resist the average shear stress, the thickness of the web required is quite less but the use of thinner sections for web may lead to buckling of web and thus, when thin sections for web is used there intermediate stiffeners are also provided in order to improve the buckling strength of the web.

Longitudinal (horizontal) stiffeners are provided to improve the buckling strength of the web. If only one longitudinal stiffener is provided then it is provided at a depth of $0.2d$ from the compression flange where 'd' is the depth of the web. If second longitudinal stiffener is also required then it is provided at mid depth of the web.

8.3 Self-weight of a Plate Girder

The self-weight of a plate girder can be estimated from the following empirical relation:

$$w = \frac{W}{200} \text{ kN/m}$$

where w = Factored self-weight of the plate girder

W = Total factored load on the girder

Based on this estimated value of self-weight, design moment and design shear force are computed.

8.4 Depth of Web

- Primarily the web depth depends on the loads but head room restrictions, transportation of plate girders and other factors may limit the web depth. In general, the web depth varies from $1/15^{th}$ to $1/6^{th}$ of their span.
- In practice, the web depth usually taken varies from $1/12^{th}$ to $1/8^{th}$ of the span.

8.4.1 Economical Depth of Web

It is that depth of the plate girder for which area of steel required is the minimum and thus will have minimum self-weight. A plate girder giving least depth may not be economical owing to the costs involved in fabrication, transportation, erection etc. In usual practice, a depth which is lower than the economical depth is adopted.

Let, M = Moment to be resisted by the plate girder which is assumed to be taken up entirely by the flanges

f_y = Design strength of the flange material

b_f = Width of the flange

t_f = Thickness of the flange

t_w = Thickness of the web

d = Depth of the web

Thus $$M = f_y b_f t_f d \quad \text{...(8.1)}$$

The gross cross-sectional area of the beam i.e. plate girder is,

$$A = dt_w + 2b_f t_f \quad \text{...(8.2)}$$

From Eq.(8.1), $b_f t_f = \frac{M}{f_y d}$

Substituting this value of $b_f t_f$ in (8.2)

$$A = dt_w + \frac{2M}{f_y d} \qquad \text{...(8.3)}$$

Let $k = \frac{d}{t_w}$ = a constant called as slenderness ratio of the web

Thus eq.(8.3) reduces to,

$$A = kt_w^2 + \frac{2M}{f_y k t_w} \qquad \text{...(8.4)}$$

The minimum value of t_w is obtained by differentiating eq. (8.4) w.r.t. t_w and equating it to zero i.e.

$$\frac{\partial A}{\partial t_w} = \frac{\partial}{\partial t_w}\left(kt_w^2\right) + \frac{\partial}{\partial t_w}\left(\frac{2M}{f_y k t_w}\right)$$

$$\Rightarrow \quad 0 = 2kt_w - \frac{2M}{f_y k t_w^2}$$

$$kt_w = \frac{M}{f_y k t_w^2}$$

$$t_w = \sqrt[3]{\frac{M}{f_y k^2}} \qquad \text{...(8.5)}$$

Eq. (8.3) can also be written as,

$$A = \frac{d^2}{k} + \frac{2M}{f_y d} \qquad \text{...(8.6)}$$

The optimum value of 'd' can be obtained by differentiating Eq.(8.6) w.r.t. d and equating it to zero i.e.

$$\frac{\partial A}{\partial d} = \frac{\partial}{\partial d}\left(\frac{d^2}{k}\right) + \frac{\partial}{\partial d}\left(\frac{2M}{f_y d}\right)$$

$$0 = 2\frac{d}{k} - \frac{2M}{f_y d^2} \qquad \left(\because k = \frac{d}{t_w}\right)$$

$$\frac{d}{k} = \frac{M}{f_y d^2}$$

$$d = \sqrt[3]{\frac{kM}{f_y}} \qquad \text{...(8.7)}$$

Therefore, once the depth of the girder is arrived at, general proportioning of the girder for maximum moment and shear can be done.

- For a trial girder section, $k = d/t_w$ for the web may be taken as any value ranging from 135 to 240. However, the following provision of IS 800 : 2007 are also useful:

  If $d/t_w < 67\varepsilon$ then the plate girder may designed as ordinary beam , where $\varepsilon = \sqrt{250/f_y}$

8.4.2 General Tips for Selecting the Depth of Plate Girder

A few general tips for selecting the depth of plate girder are as given below:

D/L =	1/15 to 1/25	for girders in buildings
=	1/12 to 1/18	for highway bridges
=	1/10 to 1/15	for railway bridges

where, D = Depth of the plate girder, L = Span of the girder

8.5 Web Thickness

Initially the web thickness can be assumed as 6 mm (if painted) or 8 mm (if unpainted)

(a) Minimum web thickness based on serviceability requirement

As per Cl. 8.6.1.1 of IS 800 : 2007,

(i) When transverse stiffeners are not provided and web connected to flanges along both longitudinal edges,

$$\frac{d}{t_w} \le 200\,\varepsilon_w$$

(ii) When transverse stiffness are not provided and web connected to flanges along one longitudinal edge only

$$\frac{d}{t_w} \le 90\,\varepsilon_w$$

(iii) When only transverse stiffeners are provided,

Case-1: $3d \ge c \ge d$ $\quad \frac{d}{t_w} \le 200\,\varepsilon_w$

Case-2: $0.74d \le c \le d$ $\quad \frac{c}{t_w} \le 200\,\varepsilon_w$

Case-3: $c < 0.74\ d$ $\quad \frac{d}{t_w} \le 270\varepsilon_w$

Case-4 :$c > 3d$

Here the web shall be considered as unstiffened.

(iv) When transverse stiffeners are provided along with longitudinal stiffner at one level only i.e. at 0.2d from the compression flange.

Case-1: $2.4d \ge c \ge d$ $\quad \frac{d}{t_w} \le 250\,\varepsilon_w$

Case-2: $0.74d \le c \le d$ $\quad \frac{c}{t_w} \le 250\,\varepsilon_w$

Case-3: $c < 0.74d$ $\quad \frac{d}{t_w} \le 340\,\varepsilon_w$

(v) When a second longitudinal stiffener is also provided (at neutral axis of the section)

$$\frac{d}{t_w} \le 400\varepsilon_w \qquad \text{where, } \varepsilon_w = \sqrt{250/f_{yw}}$$

where C = spacing of transverse stiffners

ε_w = Yield stress ratio of web

f_{yw} = Yield stress of the web

(b) Minimum web thickness based on compression flange buckling requirement

As per Cl. 8.6.1.2 of IS 800 : 2007, in order to avoid buckling of compression flange, the web thickness shall comply with the following requirements:

(i) When transverse stiffeners are not provided

$\frac{d}{t_w} \leq 345\varepsilon_f^2$, where, $\varepsilon_f = \sqrt{250/f_{yf}}$ = Yield stress ratio of flange

f_{yf} = Yield stress of compression flange

(ii) When transverse stiffeners are provided

Case-1 $c \geq 1.5d$ $\quad \frac{d}{t_w} \leq 345\varepsilon_f^2$

Case-2 $c < 1.5d$ $\quad \frac{d}{t_w} \leq 345\varepsilon_f$

Thus when $\varepsilon = 1$, the following observations can be made:

(a) If $k = d/t_w \leq 67$ then plate girder can be designed as ordinary beam without the requirement of any stiffener (except the end bearing stiffener). Such type of sections may be uneconomical.

(b) If $k = d/t_w$ lies between 67and 200, it can be designed as a plate girder without intermediate stiffeners. But it must be ensured that web does not buckle in shear. For k values up to 100-110, intermediate transverse stiffeners may not be required. For larger values of k, transverse stiffeners are required for web buckling consideration.

(c) For k value up to 250, longitudinal stiffener is also required.

(d) In any case, k value should not be taken more than 345 to avoid failure of compression flange.

Remember Previously, web stiffeners were invariably used but present day practice is to avoid stiffeners in order to reduce fabrication cost and time and hence it is preferable to go for k value up to 100 – 110 so that economical girder is obtained provided it is safe in web buckling due to shear without transverse stiffeners.

NOTE: It is to be noted that in all plate girders, end stiffeners are provided to transfer the load from plate girder to support.

8.6 Size of Flanges

- Flanges of riveted/bolted plate girders consist of pair of angles with or without cover plates.
- The flange angles should form as large a part of area of flange as possible and preferably not less than 1/3rd of the calculated flange area in order to keep the centroid of gravity of flange within the back of the angles and not in the flange cover plates otherwise the stability gets affected.

Angle section as flanges:

- Unequal angle sections with short legs as connected legs are preferred as this will give more moment of inertia of the section and also a large length will be available for making the connections with the flange cover plates.
- However, when shear is large then in that case either equal angles are preferred or unequal angles with long legs as connected legs (parallel to the web) is preferred.
- Cover plates are provided on the flanges when moment resisting capacity of the section has to be increased.

Remember Thickness of flange cover plates shall never be more than thickness of the flange angles in a riveted/bolted plate girder. Due to this reason only, more than one cover plate may be required. It is preferable to use all the cover plates of same thickness but if cover plates used are of different thicknesses then thickness of outer cover plate should not be more than the inner cover plates. Economy can be achieved by curtailing the length and width of cover plates as we proceed from mid-span to support region.

- It is assumed that design moment is resisted entirely by the flanges and using partial factor of safety for material for plastic section,

$$M = \frac{A_f f_y d}{\gamma_{m0}} = \frac{A_f f_y d}{1.1} \quad \ldots(8.8)$$

Thus area of flange A_f can be determined. Select $9.4\varepsilon < t_f < 13.6\, b_f\varepsilon$ so that flexural strength can be found by the formula for semi-compact section as per Cl. 8.2.1.2 of IS 800:2007as,

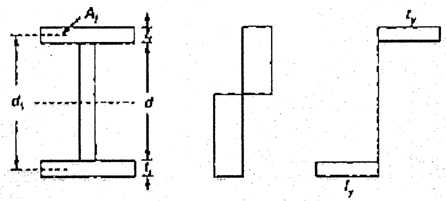
$$A_f = b_f t_f$$

i.e. $\quad 13.6\,\varepsilon t_f^2 = A_f$

Thus t_f can be determined from which flange width (b_f) can be computed as, $b_f = A_f / t_f$

8.7 Flexural Strength

- The flexural strength of a plate girder is based on tension flange yielding or the compression flange buckling.
- The buckling strength of the compression flange is governed by local buckling and lateral torsional buckling of the flange.
- The vertical buckling of the compression flange into the web can be checked by the limitations imposed on aspect ratio of the web panels by IS 800 as discussed in Section 8.5.



(a) I-section girder (b) Whole of the section resists moment $d/t_w \leq 67$ (c) Only flanges resist moment $d/t_w > 67$

Fig. 8.4 Moment and shear resistance by plate girder

- When d/t_w ratio of the web is less than or equal to ($\leq 67\varepsilon_w$) then the web is not prone to local buckling and moment carrying capacity is determined in a same way as that for restrained beams. Fig. 8.4(b) shows the stress distribution for the beam shown in Fig. 8.4(a).
- For sections with slender webs ($d/t_w > 67\varepsilon_w$) and plastic, semi-compact and compact flanges, the web is prone to shear buckling and its ability to resist flexural moment or longitudinal compression gets reduced.
- It is assumed that moment is resisted by flanges with uniform stress distribution and shear is resisted by the web.

8.8 Shear Strength of Web

- The regions located adjacent to supports or in the vicinity of concentrated loads usually control the web design as the most severe condition in terms of buckling is the case of pure shear.
- Diagonal compression of the web gives rise to shear buckling. Before buckling, the shear stress is given by the simple beam theory as, $\tau = V/dt$. Here the web is regarded as shear resistant.

- The shear strength of web of the plate girder depends on d/t_w ratio of the web and also on the spacing of intermediate stiffeners, if provided.
- Transverse stiffeners are provided to improve the shear resistance of the web as this increases the critical buckling stress of the web thereby bringing the actual shear stresses below the critical buckling stress.

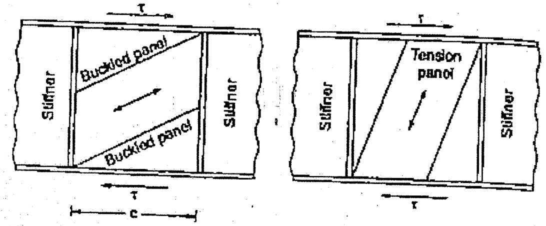
The shear capacity of the web consists of two parts viz.

(i) Shear capacity prior to the onset of buckling

(ii) Post buckling strength

Shear strength prior to onset of buckling:

- Strength before the onset of buckling comes from the elastic behaviour where stresses are totally elastic and stiffeners are only required to keep the web straight.



(a) Tension field in web panel only ignoring the bending strength of the flanges (b) Tension field in web panel including the flange contribution

Fig. 8.5 *Tension field action*

- Once the load reaches the elastic limit, the compressive shear stress makes the web to buckle. But the web panel can still carry more loads because of the action of tension field.
- The proportions of the web panel decide the amount of action of tension field.

Post buckling strength: Fig. 8.5 shows the tension field developed due to transverse stiffeners.

- The diagonal tension stresses act at the top and bottom flanges of each web panel. Their horizontal component results in compression in the flanges. However the vertical component can be assumed to be resisted by flanges or transverse stiffeners or both. If they are to be resisted by the flanges then flanges must have sufficient vertical stiffness just like in the case of bolted girders (made of flange angles and flange cover plates).

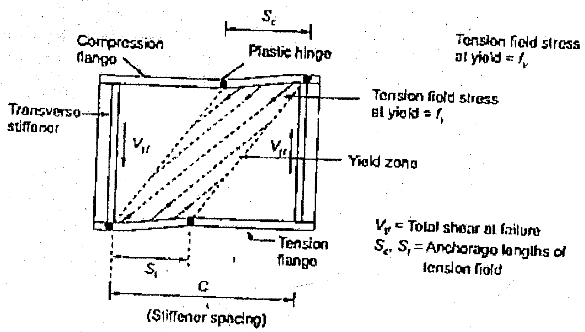


Fig. 8.6 *Web panel failure mechanism*

- In this case, the flanges act like continuous beam supported at each stiffener and loaded by the vertical components.
- The stiffeners will be designed for the reactions. Thus flanges and stiffeners are said to anchor the tension field as shown in Fig. 8.5(b).

- In case of welded plate girders, the flanges have very little flexural rigidity in the vertical direction and are not able to anchor the tension field.
- Here the flanges will deflect vertically between the stiffeners without offering much resistance.
- Thus there occurs a readjustment of diagonal tension field with parts of the web largely ineffective and stiffeners resisting the forces due to vertical component of the diagonal tension.
- Now the stiffeners begin to act as struts in addition to keep the web plate straight.
- In this post-buckling stage, it is assumed that forces developed in forming the tension field are resisted by the stiffeners and the end post. Intermediate stiffeners if not provided or provided at large spacing, the tension field action will not develop and shear capacity of the web becomes limited to the strength of the web before buckling only.
- As the ultimate load is reached, the end posts and the flanges, at the ends of the member where the shear is highest, commence to yield thereby failing by forming plastic hinges.
- This failure load is much higher than the elastic critical load. This results in gross distortions in the web and either or both the end posts and flanges.
- Thus it is necessary to make end panels sufficiently strong to anchor the longitudinal forces being set up by the creation of tension field.
- Also, so many number of intermediate stiffeners are used that form so many panels in the web of the girder. For simple plate girders, the panels close to the support are mainly under shear and panels located away from the support are mainly under flexure. Thus check for safety of the panel in shear alone as well as in shear and bending moment combined must be ensured.

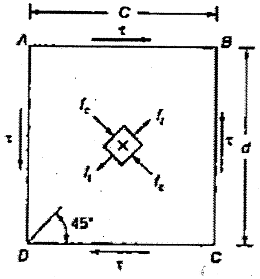


Fig. 8.7 *A typical square unbuckled shear plane*

8.8.1 Pre-buckling Behaviour of Web Panel

Let there be a square web panel subjected to vertical (and hence complimentary horizontal) shear stress τ as shown in Fig. 8.7.

- The small element shown in Fig. 8.7 is subjected to principal compressive stress along diagonal *AC* and principal tension along *BD*. If we gradually increase the load, the shear stress also increases and the plate buckles diagonally along the compression diagonal *AC*.
- If the web is adequately supported around the edges, it will not fail at this point till the stress is below the proportional limit and also it will not lose its shear resistance suddenly to zero.
- The shear stress beyond which the plate panel cannot take any further compressive stress is called as elastic critical shear stress which for simply supported condition is given by,

$$\tau_{cr,0} = k_v \frac{\pi^2 E}{12(1-\mu^2)\left(\dfrac{d}{t_w}\right)^2} \quad ...(8.9)$$

- It is assumed that after buckling and till the complete failure, the diagonal compression and hence beam shear resistance remains at a value it had at the onset of web plate buckling. This part of total shear resistance is expressed as,

$$V_1 = \tau_{cr,e}dt \qquad ...(8.10)$$

As per IS 800 resistance to shear buckling must be verified when

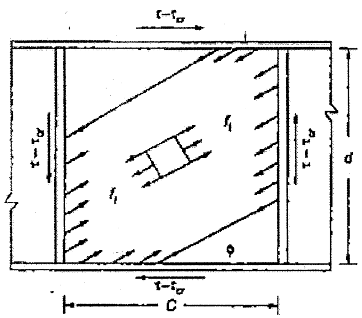
$$\frac{d}{t_w} > 67\varepsilon_w \qquad \text{for unstiffened web}$$

Do You Know?

When depth to thickness ratio (d/t_w) of the web is very low then elastic critical shear stress ($\tau_{cr,e}$) increases to such a large value that it exceeds the value of yield shear stress and here the web will yield in shear before buckling.

8.8.2 Post-buckling Behaviour of Web Panel

- After buckling, it is assumed that diagonal compression remains constant.
- Once the elastic critical stress has been reached, the compression diagonal AC (Fig. 8.7) will not resist any more load and web loses its capacity to sustain the enhanced compressive stress and a new load carrying mechanism gets developed.
- Any additional increase in shear stress beyond elastic critical stress will be supported by a tensile membrane field which is anchored to the boundaries viz. top and bottom flanges and adjacent stiffeners on either side of the web panel as shown in Fig. 8.8.



ϕ = Inclination of tension field

Fig. 8.8 Post buckling behaviour of web panel

- Using the principle of Mohr's circle, the resistance provided by shear component contribution from tension field is,

$$V_2 = \frac{f_t}{2}td\sin 2\phi \qquad ...(8.11)$$

- The stress in the web plate will be the sum of elastic critical shear stress ($\tau_{cr,e}$) and post buckled membrane tensile stress (f_t). The most critical stress condition will occur when f_t aligns with the principal diagonal tension due to $\tau_{cr,e}$ i.e. $\phi = 45°$. In general, ϕ usually remains less than 45° but the assumption of $\phi = 45°$ is quite conservative.

  Thus total shear on the web is,

$$V = V_1 + V_2 \qquad ...(8.12)$$

- Now the flanges being of finite rigidity, the pull exerted by the tensile membrane stresses in the web makes the flanges to bend inwards.
- It is essential to check the shear resistance of the web for buckling particularly in case of thin webs. Cl. 8.4.2 of IS 800 : 2007 states that this check for shear bucking of web is necessary when:

$$\frac{d}{t_w} > 67\varepsilon \text{ for web without stiffeners}$$

$$> 67\varepsilon\sqrt{\frac{K_v}{5.35}} \text{ for web with stiffeners}$$

where

$$K_v = \begin{cases} 5.35 & \text{when transverse stiffeners are provided at support} \\ 4+\dfrac{5.35}{\left(\dfrac{c}{d}\right)^2} & \text{for } \dfrac{c}{d} < 1.0 \\ 5.35+\dfrac{4.0}{\left(\dfrac{c}{d}\right)^2} & \text{for } \dfrac{c}{d} \geq 1.0 \end{cases}$$

c = Spacing of transverse stiffeners, d = Depth of web

8.9 Design of Web for Shear Buckling

There are two methods for assessing the nominal shear strength (V_n) of plate girder web with or without stiffeners viz.

(a) Simple post-critical method (b) Tension field method

8.9.1 Simple Post-critical Method

This method can be used for plate girders with or without transverse stiffeners provided web is having the transverse stiffners at supports. As per this method,

$$V_n = V_{cr} = A_v\tau_b \qquad ...(8.13)$$

where τ_b = Shear stress corresponding to web buckling which is determined as given below.

(a) $\lambda_w \leq 0.8$ (Shear yielding)

$$\tau_b = \frac{f_{yw}}{\sqrt{3}}$$

where f_{yw} = Yield stress of web material

(b) $0.8 < \lambda_w < 1.2$ (Shear buckling)

$$\tau_b = \left[1-0.8(\lambda_w-0.8)\right]\frac{f_{yw}}{\sqrt{3}}$$

(c) $\lambda_w \geq 1.2$ (Shear buckling and tension field action)

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w^2}$$

where λ_w = Non-dimensional web slenderness ratio for shear buckling stress which is given by,

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}}$$

$\tau_{cr,e}$ = Elastic critical shear stress of web which is given by,

$$\tau_{cr,e} = \frac{K_v\pi^2E}{12(1-\mu^2)\left(\dfrac{d}{t_w}\right)^2}$$

μ = Poisson's ratio for steel which is taken as 0.3

8.9.2 Tension Field Method

- This method of determining the shear buckling strength of web can be used where end and intermediate transverse stiffeners are provided.
- This method takes care of post buckling strength provided by the stiffeners.
- As the web commences to buckle, it losses it capability to resist diagonal compression and thus at this stage transverse stiffeners and the flanges come into action to resist the diagonal compression.
- The vertical component of this diagonal compression is resisted by transverse stiffeners and horizontal component by flange.
- Thus the web resists only the diagonal tension. Thus there is additional strength to resist shear buckling. Cl. 8.4.2.2 of IS 800:2007 gives the following expression for computing the shear resistance of web if intermediate and end stiffeners are provided and $b/d \geq 1$:

$$V_n = V_{tf}$$

where
$$V_{tf} = (A_v\tau_b + 0.9 w_{tf} t_w f_v \sin\phi) \leq V_p \quad ...(8.14)$$

τ_b = Buckling strength as obtained in simple post-critical buckling method

f_v = Yield strength of tension field which is obtained as,

$$f_v = \sqrt{(f_{yw}^2 - 3\tau_b^2 + \psi^2)} - \psi \quad ...(8.15)$$

$$\psi = 1.5\tau_b \sin 2\phi \quad ...(8.16)$$

ϕ = Inclination of tension field $= \tan^{-1}\left(\frac{d}{c}\right)$

w_{tf} = Width of tension field

$= d\cos\phi + (c - s_c - s_t)\sin\phi$

s_c, s_t = Anchorage lengths of tension field along the compression and tension flanges which is obtained as,

$$s = \frac{2}{\sin\phi}\sqrt{\frac{M_{fr}}{f_{yw} t_w}} \leq c \quad ...(8.17)$$

M_{fr} = Reduced plastic moment capacity of respective flange plate which is given by,

$$M_{fr} = 0.25 b_f t_f^2 f_{yf}\left[1 - \left(\frac{N_f}{b_f t_f f_{yf}/\gamma_{mo}}\right)^2\right] \quad ...(8.18)$$

N_f = Axial force in the flange

8.9.3 Collapse Behaviour of Web Panel

- On further increasing the load, the tensile membrane stresses developed in the web continues to exert a pull on the flanges and finally the resultant stress reaches the yield value and the web yields.
- On formation of four plastic hinges in the flanges, a shear mechanism gets developed as shown in Fig. 8.6.
- In order that plastic hinges get form in the flanges, the flanges must be classed as plastic sections.
- In very strong flanges, the hinges will form at the four corners of the panel.
- If flanges are compact, semi-compact or slender, the tension field will be supported entirely by the transverse stiffeners, with plastic hinges at the stiffeners and the flanges will not support the tension field.

- For very thick web girders, the web will yield before buckling.
- Another important aspect of post buckling behaviour of web in the design of plate girder (but not considered in the tension field theory) is the performance of plate girder after buckling of web in flexure. This type of buckling even tough does not lead to collapse of the entire member as long as compression flange is capable of resisting the added longitudinal compression force which the web is not able to absorb after it buckles and thus this puts an upper limit on d/t_w as 340 on the slenderness of the web of the plate girders.

8.10 Design of End Panels

8.10.1 Shear Buckling Design Method

- In order to assess the nominal shear strength (V_n) of the web of the plate girder with or without intermediate stiffeners then there are two methods viz. the **simple post critical method** and the **tension field method.**

1. **Simple Post-critical Method**

This method can be used for web of the I-section girders with or without intermediate transverse stiffeners subject to the condition that web is having transverse stiffeners at supports. The nominal shear strength is given by,

$$V_n = V_{cr}$$

where V_{cr} = Shear force that causes buckling in the web $= A_v\tau_b = dt_w\tau_b$ ...(8.19)

τ_b = Shear stress corresponding to web buckling (see Fig. 8.9) which is given as,

$$= \begin{cases} \dfrac{f_{yw}}{\sqrt{3}} & \text{for } \lambda_w \leq 0.8 \quad \text{(shear yielding)} \\ [1 - 0.8(\lambda_w - 0.8)]\dfrac{f_{yw}}{\sqrt{3}} & \text{for } 0.8 < \lambda_w < 1.2 \quad \text{(shear buckling)} \\ \dfrac{f_{yw}}{\sqrt{3}\lambda_w^2} & \text{for } \lambda_w \geq 1.2 \quad \text{(shear buckling and tension field action)} \end{cases} \quad ...(8.20)$$

λ_w = non – dimensional web slenderness ratio for shear buckling stress $= \sqrt{\dfrac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}}$

$\tau_{cr,e}$ = elastic critical shear stress of the web $= \dfrac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$

μ = Poisson's ratio

$$k_v = \begin{cases} 5.35 & \text{when transverse stiffners are provided only at supports} \\ 4 + \dfrac{5.35}{(c/d)^2} & \text{for } c/d < 1 \\ 5.35 + \dfrac{4}{(c/d)^2} & \text{for } c/d \geq 1 \end{cases} \quad ...(8.21)$$

c = Spacing of transverse stiffeners, d = Depth of the web

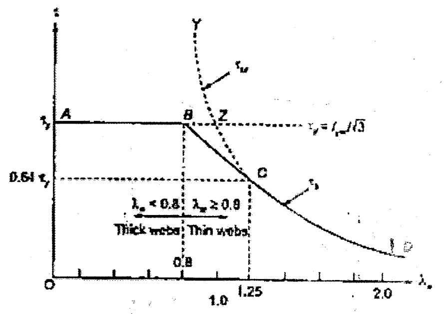


Fig. 8.9 Shear buckling strength, τ_b

Remember

1. The shear strength of the web is a function of shear yield of web and shear buckling of web depending on the depth to thickness (d/t_w) ratio of the web and λ_w is a function of d/t_w.
2. Thin web fails due to shear buckling and thick web fails due to shear yielding.

2. Tension Field Method

- This method is based on post – buckling shear strength of the web provided with intermediate transverse stiffeners in addition to transverse stiffeners at the supports subject to the condition that panels adjacent to the panel under tension field action or the end posts provide anchorage for the tension fields and if $c/d > 1$.
- Here, the shear resistance developed consists of shear strength due to primary buckling strength of the plate, the tension field action and from the plastic moment capacity of the flanges.
- The diagonal tension field is composed of a central part that anchors on the transverse stiffeners and two additional anchors due to flanges as shown in Fig. 8.6.
- For flanges to form plastic hinges, the flange sections should be classed as plastic. Compact, semi-compact and slender flange sections are not able to develop plastic moment capacity and thus the tension field will be entirely supported by transverse stiffeners and obviously flanges will not support the tension field.
- The contribution of tension field is very significant for slender webs since in them, the elastic critical stress ($\tau_{cr,e}$) is very small making the webs prone to buckling but thick webs fail due to shear yielding before buckling.

In the tension field theory, the nominal shear resistance (V_n) is given by,

$$V_n = V_{tf}$$

where $V_{tf} = [A_v\tau_b + 0.9\, w_{tf} t_w f_v \sin\phi] \le V_p$ ...(same as Eq. 8.14)

τ_b = Shear stress corresponding to web buckling i.e. buckling strength of the web material

f_v = Yield strength of the tension field

$$= \sqrt{\left[f_{yw}^2 - 3\tau_b^2 + \psi^2\right]} - \psi \qquad \text{...(same as Eq. 8.15)}$$

$$\psi = 1.5\tau_b \sin 2\phi \qquad \text{...(same as Eq. 8.16)}$$

ϕ = Inclination of the tension field which depends on the aspect ratio (c/d) of the web panel

$$= \tan^{-1}\left(\frac{d}{c}\right)$$

w_{tf} = width of the tension field = $d\cos\phi + (c - s_c - s_t)\sin\phi$, f_{yw} = Yield stress of the web

s_c, s_t = Anchorage lengths of the tension field (Fig. 8.6) along the compression and tension flanges respectively which are obtained as,

$$s = \frac{2}{\sin\phi}\sqrt{\left(\frac{M_{fr}}{f_{yw}t_w}\right)} \le c \qquad \text{...(same as Eq. 8.17)}$$

M_{fr} = Reduced plastic moment capacity of the respective flange plate (irrespective of any edge stiffener) after taking into account the axial force N_f in the flange, due to overall bending and any external axial force in the cross-section

$$= 0.25 b_f t_f^2 f_{yf}\left[1 - \left(\frac{N_f}{b_f t_f f_{yf}/\gamma_{m0}}\right)^2\right] \qquad \text{...(same as Eq. 8.18)}$$

b_f, t_f = Width and thickness of the relevant tension flange respectively, f_{yf} = Yield stress of the flange

8.10.2 Design of End Panel

- The end panel may be designed either by post-critical method or by tension field method.
- Consider the intermediate panels 2 and 3 as shown in Fig. 8.10. Horizontal component of the tension field in panels 2 and 3 will cancel out each other but the vertical component will be resisted by the transverse stiffeners. Due to this balancing action of intermediate panels, the interior panels are said to be anchored by the neighboring panels.
- However in panel 1 there is no adjacent web to the left of it for balancing the horizontal component of the tension field from panel 2 i.e. there is no anchorage for panel 1 to the left of it.

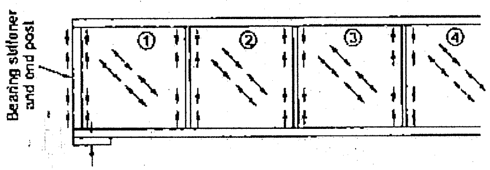


Fig. 8.10 Tension field forces in end panel

In order to deal with this situation in panel 1, there are two prevalent methods viz.

(a) Design the end panel by conventional method i.e. **post critical method** there by preventing the formation of tension field in end panel. This is more useful method. The stiffener provided at the end of this panel is referred to as **end bearing stiffener** as shown in Fig. 8.11. This end panel has been found to anchor the neighboring panels. End panels when designed by this method do not utilize the post buckling strength of the end panel thereby giving smaller aspect ratio (c/d) for the end panel.

(b) This method involves stiffening the left end of end panel using tension field action. But the end post will also have to be designed and checked for bending induced by tension field. This leads to a firm post as shown in Fig. 8.12 which is generally not preferred.

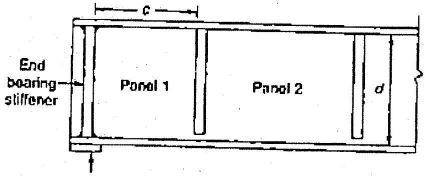
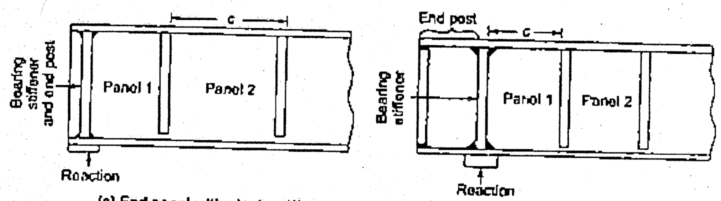


Fig. 8.11 End panel being designed without considering tension field action but provided with bearing stiffener



(a) End panel with single stiffener

(b) End panel with double stiffener

Fig. 8.12 End panel being designed considering tension field action (provided with end post)

- When post critical method is used for designing the end panel then end panel along with stiffeners has to be checked as a beam which spans in between the flanges of the plate girder and is required to resist the shear force R_{tf} and a moment of M_{tf} resulting due to tension field anchor force H_q.
- The moment M_{tf} induces a tension in the inner stiffeners and compression in the outer stiffeners. The depth of this beam is c i.e. the spacing of stiffeners. The associated force (F_{tf}) with moment (M_{tf}) is given by,

$$F_{tf} = \frac{M_{tf}}{c}$$

The end stiffener must be able to resist the reaction and compressive force F_{tf} due to the moment M_{tf}.

- When tension field method is used, the end panel is provided by either a single stiffener in the form of bearing stiffener (which in fact acts as bearing stiffener and also as end post as shown in Fig. 8.12(a) or as double stiffener as shown in Fig. 8.12(b). In the later case i.e. end panel with double stiffener, the two stiffeners along with web projecting beyond the end support forms the end post.
- In case a single stiffener is provided then it will have to act as bearing stiffener as well as end post. This end post is rigidly connected to the flanges with full strength welds and must be able to resist the reaction along with moment from the anchor forces which is taken as $(2/3)M_{tr}$.

Remember

1. The width and thickness of the end post must not exceed the width and thickness of the flange.
2. Single stiffener end post acts as both bearing stiffener and load carrying stiffener.

- In case double stiffener is provided then bearing stiffener is designed for the compressive forces due to the external reaction only. The two stiffeners along with the portion of web projecting beyond the end support (as shown in Fig. 8.12(b) form a rigid end post to provide necessary anchorage for the tension field in the end panel. The inner bearing stiffener is checked for bearing at the end and for buckling at the center.
- The tension field develops anchor forces resulting in resultant longitudinal shear R_{tf} and moment M_{tf} which are given by,

$$R_{tf} = \frac{H_q}{2}, \quad M_{tf} = \frac{H_q d}{10}$$

where H_q = Longitudinal anchorage force $= 1.25V_p\sqrt{\left(1-\frac{V_{cr}}{V_p}\right)}$

$$F_{tf} = M_{tf}/c$$

V_p = Plastic shear resistance under pure shear $= \frac{d t_w f_y}{\sqrt{3}}$

Where, d = Depth of the web

NOTE

If the actual factored shear force V in the panel designed by tension field method is less than the design shear strength V_{tf} then H_q may be reduced by

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}}$$

where V_{tf} = Basic shear strength of the panel using tension field method as given by Eq. (8.14).

V_{cr} = Critical shear strength of the panel using post critical method as given by Eq. (8.19).

Do You Know

1. Tension field does not ordinarily get fully developed in an end panel.
2. Panels with opening area exceeding 10% of the recommended minimum panel dimension should be designed without using tension field method and the adjacent panels should be designed as end panels.

8.11 Stiffeners

- When web of the plate girder is not sufficient enough to carry the loads. In that case, stiffeners are provided which perform certain specific functions which are discussed below.
- In general, there are only two types of stiffener viz. intermediate stiffener and bearing stiffener but as per IS 800:2007 the following types of stiffeners are identified based on purpose for which a stiffener is being used:

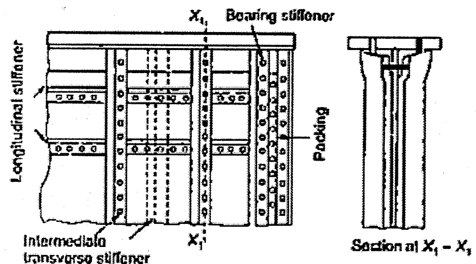


Fig. 8.13 (a) Stiffeners for bolted plate girder

(a) **Intermediate and longitudinal stiffener:** To improve the buckling strength of a slender web due shear.

(b) **Load carrying stiffener:** To prevent web buckling due to concentrated load.

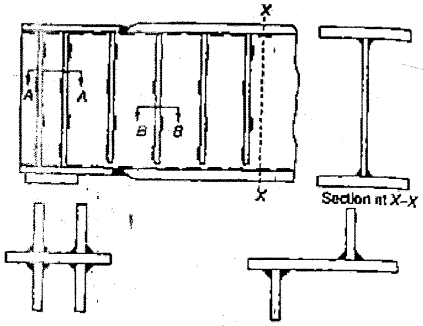
(c) **Bearing stiffener:** To prevent local crushing of the web due to concentrated loads or reactions.

(d) **Torsional stiffener:** To provide lateral restraint to beams and girders at support.

(e) **Diagonal stiffener:** To provide a sort of local reinforcement to web under shear and bearing.

(f) **Tension stiffener:** To transfer tensile forces applied to the web through a flange.

- Plate girder is assumed to be supported over the edges of the stiffeners.
- For riveted/bolted connections, angle sections are used as shown in **Fig. 8.13 (a)** and for welded connections, flat sections are used as shown in Fig. 8.13(b).



Transverse stiffener section at B-B

Fig. 8.13 (b) *Stiffeners for welded plate girder*

8.11.1 General requirements for a stiffener

(a) **Outstand of web stiffeners:** The outstand of web stiffener from the face of the web should not exceed $20t_q\varepsilon$. When the outstand of the web is between $14t_q\varepsilon$ and $20t_q\varepsilon$, then stiffener design is based on core section with an outstand of $14t_q\varepsilon$ where t_q is the stiffener thickness.

(b) **Stiff bearing length:** As shown in Fig. 8.14, the stiff bearing length (b_1) of any element is that length that cannot deform considerably in bending. In order to determine the stiff bearing length (b_1), the dispersion of load through a steel bearing element is taken as 45° passing through the solid material such as bearing plates, flange plates etc.

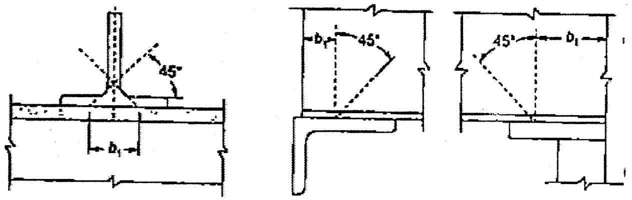


Fig. 8.14 *Stiff bearing length b_1*

(c) **Eccentricity:** In case a load or a reaction is applied at an eccentricity to the center line of the web or the CG of the stiffener does not lie on the center line of the web, there eccentricity of loading must be accounted for in the stiffener design.

(d) **Buckling resistance of stiffeners:**

- The buckling resistance (F_{qd}) of the stiffener is based on design compressive stress (f_{cd}) of a strut (curve c), the radius of gyration being taken about the axis parallel to the web.
- The effective section is the full area of the stiffener plus an effective length of the web on each side of the stiffener center line subject to a maximum of 20 times the web thickness multiplied by thickness of the web.
- The design strength to be used must be the minimum of value obtained for buckling about the web of the stiffener.
- The effective length of intermediate transverse stiffener used for computing the buckling resistance (F_{dq}) is taken as 0.7 times the actual length of the stiffener.
- The effective length for load carrying web stiffener to be used for computing the buckling resistance (F_{xd}) assumes that the flange through which the load or reaction is applied is effectively restrained against local movement relative to the other flange and is taken as:
  - (i) 0.7L when flange is restrained against rotation in the plane of the stiffener (due to other structural elements) :
  - (ii) L when flange is not restrained, where L is the length of the stiffener.

8.11.2 Intermediate Web Stiffener

- It is a stability stiffener that provides check against the diagonal buckling of thin web in plate girder. As per requirement, transverse (vertical) and/or longitudinal (horizontal) stiffeners can be provided.

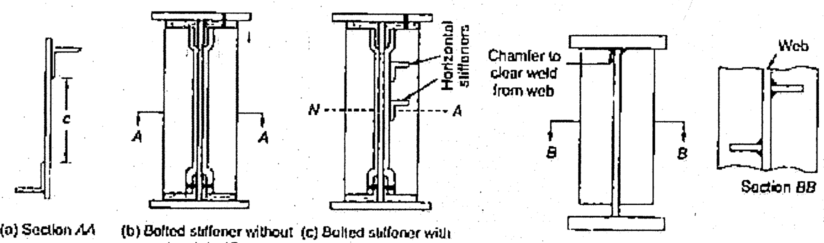
Intermediate transverse (vertical) stiffener: Theoretically, intermediate transverse stiffeners are not required when the calculated shear stress in the web is less than the critical buckling shear stress of the web of the plate girder since the web will not buckle and failure of web will occur due to shear yielding of the web. Here tension field will not develop and stiffeners are not required and the web will completely be shear resistant.

- The purpose of intermediate transverse stiffener is that it increases the buckling resistance of the web.
- Before buckling of web takes place, the normal and shear stress in the web are the same irrespective of the fact that whether stiffeners are provided or not.
- If however stiffeners are at all provided then these will remain unstressed in this case. But due to their contact with the web (due to welding or bolts/rivets), the influence of change in the section at their line of contact (i.e. stiffener and web) and if any point load is applied to the flanges at their edges then stiffener will get stressed.
- Once the buckling of web has taken place, the transverse stiffeners become the primary load carrying members quite essential for the stability of the plate girder. Thus these transverse stiffeners must be capable enough of resisting the unbalanced vertical component of the diagonal tension as well.

The following conditions must be satisfied while proportioning a transverse stiffener.

(a) It must be sufficiently stiff so that it does not get deform considerably as the web tends to buckle.

(b) It must be strong enough to withstand the shear transmitted by the web.



(a) Section AA (b) Bolted stiffener without horizontal stiffener (c) Bolted stiffener with horizontal stiffener (d) Welded intermediate stiffener

Fig. 8.15 Intermediate Stiffeners

Remember: When stiffeners are provided, the aspect ratio (c/d) should be chosen in within a range of 1.2 to 1.6. When end panel is designed near support without using the tension field theory then this aspect ratio (c/d) should be in the range of 0.6 to 1.

Spacing: Spacing of intermediate stiffener if provided should be as specified in Section 8.5.

Outstand of stiffener: Intermediate stiffeners are not designed as compression members but a width to thickness ratio limit must be adhered to in order to avoid local buckling. The outstand of the stiffener should comply with the provisions as specified in Section 8.11(a).

Minimum Stiffness: Transverse web stiffeners when not subjected to external loads and moments should have a moment of inertia I_s

(a) about the center line of web if stiffeners are on both sides of the web and

(b) about the face of the web if stiffeners are provided alternatively i.e. single stiffener on one side of the web

such that

$$I_s \geq 0.75\, d t_w^3 \qquad \text{if } c/d \geq \sqrt{2} \qquad \text{...(8.22)}$$

$$I_s \geq 1.5\, d^3 t_w^3/c^2 \qquad \text{if } c/d < \sqrt{2} \qquad \text{...(8.23)}$$

where d = Depth of the web

t_w = Minimum required thickness of the web for spacing using tension field theory

c = Actual spacing of stiffeners

Additional stiffeners will be required if these are subject to lateral loads and/or moments due to eccentricity of transverse loads w.r.t. to the web.

Buckling check:

- This check is required for intermediate transverse stiffeners only when tension field theory is used for webs.
- Stiffeners not subjected to external loads or moments should be checked for a buckling force of,

$$F_q = \frac{V - V_{cr}}{\gamma_{m0}} \leq F_{dq} \qquad \text{...(8.24)}$$

where F_{dq} = Design resistance of the intermediate stiffeners

V = Factored shear force adjacent to the stiffener

V_{cr} = Shear buckling resistance of the web panel designed without using tension field theory i.e. by using simple post critical method

- If however stiffeners are subjected to external loads and moments then they must comply with the conditions of load carrying stiffener. In addition to that, they must satisfy the following interaction relation.

$$\frac{F_q - F_x}{F_{dq}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1 \qquad \text{...(8.25)}$$

If however, $F_q < F_x$ then $(F_q - F_x)$ is taken as zero.

Here F_q = Stiffener force

F_{dq} = Design resistance of the intermediate web stiffener corresponding to buckling about an axis parallel to the web

F_x = External load or reaction on the stiffener

F_{xd} = Design resistance of the load carrying stiffener corresponding to buckling about an axis parallel to the web

M_q = Moment on the stiffener due to eccentrically applied load and transverse load, if any.

M_{yq} = Yield moment capacity of the stiffener on the basis of elastic modulus about its centroidal axis parallel to the web

- Unless intermediate stiffeners are required to serve as bearing stiffeners, these intermediate stiffeners are not required to bear against the tension flange and thus their length can be kept somewhat less than the web depth (d). By doing so, the close fit fabrication problem can be dispensed with.

8.11.3 Intermediate Longitudinal (Horizontal) Stiffener

- When the web is subjected to bending then intermediate longitudinal stiffener increases the buckling resistance of the web considerably.
- Longitudinal stiffeners are provided between the vertical stiffeners.
- The moment of inertia required for the first horizontal stiffener is,

$$I = c t_w^3 \qquad \text{...(8.26)}$$

where c = actual spacing of vertical stiffeners, t_w = Minimum required thickness of the web

- The moment of Inertia of the second stiffener is given by,

$$I = d_2 t_w^3 \qquad \text{...(8.27)}$$

where d_2 = Twice the distance from compression flange to the neutral axis

8.11.4 Connection of Intermediate Stiffeners to Web

- Intermediate transverse stiffener not subjected to external loads or moments should be connected to the web to withstand a shear between the transverse stiffener and web of a value not less than $t_w^2/5b_s$ (kN/m) where t_w is the thickness of web (in mm) and b_s is the outstand width of the stiffener (in mm).
- For stiffener subjected to external loading, then the shear between the web and stiffener due to such external loading has to be added to the above value.

8.11.5 Load Carrying Stiffener

- The load carrying stiffeners are provided when compressive force applied through the flange exceeds the buckling strength ($F_{cd,w}$) of the unstiffened web (i.e. the web alone).
- The effective length of the web to determine the slenderness ratio is computed.
- The cross sectional area of load carrying stiffener is equal to $(b_1 + n_1)t_w$ where,

b_1 = Width of stiff bearing on the flange.

n_1 = Dispersion of the load through the web at 45° to the level of half the depth of the cross section

- The web of the plate girder is like a column only and thus its buckling strength is computed as per the procedure described in Ch. 5 using the buckling curve c. The lesser of the two values viz. the design strength of the web material or the stiffener material is used for computing the buckling resistance.

Buckling check: The external load or reaction (F_x) on the stiffener must not exceed the buckling resistance (F_{xd}) of the stiffener. If the stiffener also acts as intermediate stiffener then it must be checked for buckling under the combined loads.

Bearing check: Load carrying web stiffeners acting as intermediate transverse stiffeners should be of sufficient size in order to have bearing strength of the stiffener (F_{psd}) not less than the load transmitted where,

$$F_{psd} = \frac{f_{yq} A_q}{0.8\gamma_{m0}} \geq F_x \qquad ...(8.28)$$

where F_x = External load or reaction

f_{yq} = Yield strength of the stiffener

A_q = Area of the stiffener in contact with the flange

8.12 Bearing Stiffeners

- The purpose of bearing stiffener is to transfer the concentrated load on the girder and heavy reactions at the supports to the full depth of the web.
- These stiffeners are required when web has insufficient strength for web yielding, web crippling or web buckling.
- If stiffeners are used to resist the full concentrated load, limit states like web crippling, web yielding and web buckling are not required to be checked.
- Bearing stiffener when provided at the support is called as end bearing stiffener.

Bearing stiffener must be placed:

(a) tight fitted to the loaded flange but need not to be welded or connected to it.

(b) straight and it is not joggled or crimped as shown in Fig. 8.16(a).

(c) in pairs of two or four angles symmetrically placed on either side of the web for riveted or bolted plate girder as shown in Fig. 8.16(b), (c) and (d) and two or four plates as shown in Fig. 8.16(e).

- Bearing stiffeners are designed for the applied load or the induced reaction less the load carrying capacity of the web and is given by,

$$F_w = \frac{(b_1 + n_2) t_w f_{yw}}{\gamma_{m0}} \qquad ...(8.29)$$

where b_1 = Stiff bearing length

n_2 = Length that is obtained by dispersion through the flange of the web junction at an assumed slope of 1:2.5 to the plane of the flange

t_w = Thickness of the web

f_{yw} = Yield stress of the web

- Often bearing stiffeners are required to prevent the sideways buckling of the web. Sideways buckling must be checked when compression flange is not restrained against movement relative to the tension flange.

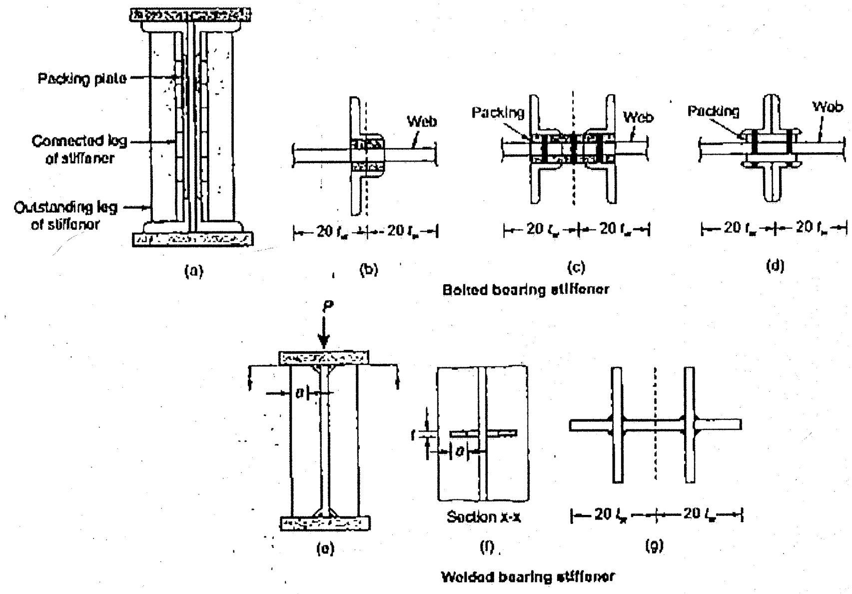


Fig. 8.16 A typical arrangement of bearing stiffeners

8.13 Stiffener Connections

With web: Stiffeners that carry load or the reactions applied through a flange should be well connected to the web by sufficient weld (length and size) or fasteners to transfer a design force which is equal to lesser of:

(a) tension capacity of the stiffener and

(b) sum of the forces applied at the two ends of the stiffener which they act in the same direction or the larger of the forces when they act in opposite directions.

With Flanges

In tension: Stiffeners that are required to resist the tension should be connected to the flange transmitting the load by continuous weld or non-slip fasteners.

In compression: Stiffeners that are required to resist the compression should either be fitted against the loaded flange or connected by continuous weld or non-slip fasteners.

The stiffener should be connected or fitted against both the flanges when

(a) load is applied directly over the support

(b) it forms the end stiffener of the stiffened web

(c) it acts as a torsional stiffener

8.14 Diagonal Stiffeners

Diagonal stiffeners should be designed to carry a portion of applied shear and bearing that exceeds the web capacity. Where the strength of web and stiffener differs, there the design strength is taken as described in Section 8.12.

8.15 Tension Stiffener

Tension stiffeners are designed to carry a portion of applied load or reaction less the web capacity as given by eq. for bearing stiffeners. Where the strength of web and stiffener differs, there the design strength is taken as described in Section 8.12.

8.16 Torsional Stiffener

When bearing stiffeners are required to provide the torsional restraint at the supports of the beam then they must comply with the following criteria:

(a) All the conditions of Section 8.12.

(b) Moment of inertia of the stiffener section about the center line of web I_s should be such that

$$I_s \geq 34\alpha_s D^3 T_{cf} \quad \text{...(8.30)}$$

where

$$\alpha_s = \begin{cases} 0.006 & \text{for } \dfrac{L_{LT}}{r_y} \leq 50 \\ \dfrac{0.3}{L_{LT}/r_y} & \text{for } 50 < \dfrac{L_{LT}}{r_y} \leq 100 \\ \dfrac{30}{\left(L_{LT}/r_y\right)^2} & \text{for } \dfrac{L_{LT}}{r_y} > 100 \end{cases} \quad \text{...(8.31)}$$

where D = Overall depth of the beam at support

T_{cf} = Maximum thickness of the compression flange in the span under consideration

KL = Laterally unsupported effective length of the compression flange of the beam

r_y = Radius of gyration of the beam about minor axis

Illustrative Examples

Example 8.1. Design a welded plate girder of span 20 m which is laterally restrained throughout. It carries a uniform load of 105 kN/m excluding its self-weight. Use steel of grade Fe410. No intermediate stiffeners one to be provided.

Solution:

For Fe410 grade steel, f_u = 410 N/mm²

f_y = 250 N/mm²

$$\epsilon = \epsilon_w = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

Design forces

Total superimposed load = 105 kN/m (Also known as live load)

∴ Factored superimposed load = 1.5 × 105 = 157.5 kN/m

$$\text{Self weight of plate girder} = \frac{wL}{400} \frac{(105\times 20)20}{400} = 105 \text{ kN}$$

∴ Self weight of plate girder per meter length = $\dfrac{105}{20}$ = 5.25 kN/m

∴ Factored self weight per metre length = 1.5 × 5.25 = 7.875 kN/m

∴ Total factored load = 157.5 + 7.875 = 165.375 kN/m

∴ Factored shear force = $165.375 \times \dfrac{20}{2}$ = 1653.75 kN

Factored bending moment (M) = $\dfrac{165.375\times 20^2}{8}$ = 8268.75 kNm

Design of Web

Optimum depth of plate girder (d) = $\left(\dfrac{Mk}{f_y}\right)^{1/3}$

For having the value of k, $\dfrac{d}{t_w}(=k)$ should be known.

∵ No intermediate stiffeners are to be provided.

∴ From serviceability criterion,

$$\frac{d}{t_w} \leq 200\,\epsilon = 200 \times 1 = 200$$

and from flange buckling criterion, $\dfrac{d}{t_w} \leq 345\,\epsilon_f^2 = 345 \times 1^2 = 345$

Thus $\dfrac{d}{t_w} \leq 200$

Let, $\dfrac{d}{t_w}$ = 180 (say)

∴ $k = \dfrac{d}{t_w} = 180$

∴ Optimum depth of plate girder (d)

$$= \left(\frac{Mk}{f_y}\right)^{1/3} = \left(\frac{8268.75\times 10^6 \times 180}{250}\right)^{1/3}$$

= 1812.41 mm ≃ 1820 mm (say)

∴ $\dfrac{d}{t_w}$ = 180

⇒ $t_w = \dfrac{d}{180} = \dfrac{1820}{180}$ = 10.11 mm ≃ 12 mm(say)

∴ Size of web plate = 1820 × 12 mm.

Design of flanges

Flanges resist the bending moment and shear is resisted by the web.

Flange area required to resist the bending moment (A_f)

$$= \frac{M\gamma_{m0}}{f_y d} = \frac{8268.75\times 10^6 \times 1.1}{250\times 1820} = 19990.38 \text{ mm}^2$$

Let flange width is taken as 1/3rd of depth of girder

$$\therefore \quad b_f = \frac{1820}{3} = 606.67 \text{ mm} \simeq 610 \text{ mm}$$

$\therefore$ Flange thickness, $(t_f)\dfrac{A_f}{b_f} = \dfrac{19990.38}{610} = 32.77 \text{ mm} \simeq 50 \text{ mm(say)}$

$\therefore$ Size of flange plate = 610×50 mm

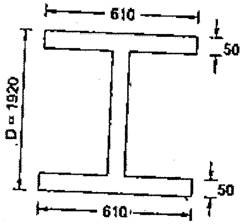
Type of flange i.e., flange classification

For plastic flange, $\dfrac{b}{t_f} \leq 8.4\varepsilon = 8.4 \times 1 = 8.4$

Outstand of flange $(b) = \dfrac{b_f - t_w}{2} = \dfrac{610-12}{2} = 299$ mm

$$\therefore \quad \frac{b}{t_f} = \frac{299}{50} = 5.98 < 8.4\varepsilon \qquad \text{(OK)}$$

Thus flange plates can be classified as plastic section.



Check of adequacy of flange plates in bending

Plastic section modulus of the girder section is given by,

$$Z_p = b_f t_f\left(\frac{d}{2} - \frac{t_f}{2}\right) + b_f t_f\left(\frac{D}{2} - \frac{t_f}{2}\right)$$

$$= 2b_f t_f\left(\frac{D - t_f}{2}\right)$$

$$= 610 \times 50\,(1920 - 50) = 57.035 \times 10^6 \text{ mm}^3$$

$\therefore$ Moment carrying capacity of the flange

$$M_d = \frac{Z_p f_y}{\gamma_{m0}} = \frac{57.035 \times 10^6 \times 250}{1.1} \text{ Nmm}$$

$$= 12962.5 \text{ kNm} > 8268.75 \text{ kNm} \qquad \text{(OK)}$$

Thus flange plates are sufficient enough to carry the design moment.

Check for adequacy of the web in shear

The adequacy of the web in shear is checked as per simple post critical buckling.

$$\frac{d}{t_w} = \frac{1820}{12} = 151.67$$

$$200\varepsilon = 200 \times 1 = 200$$

$$\therefore \quad \frac{d}{t_w} < 200\varepsilon$$

and also $\dfrac{d}{t_w} < 345\varepsilon_f^2 = 345 \times 1^2 = 345$

Critical elastic shear stress of the web $(\tau_{cr,e})$

$$= \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\dfrac{d}{t_w}\right)^2}$$

Here, $k_v = 5.35$ when transverse stiffeners are provided only at the supports i.e., no intermediate transverse stiffeners are provided.

Let Poisson's ratio $(\mu) = 0.3$

and $E = 2 \times 10^5$ N/mm^2

$$\therefore \quad \tau_{cr,e} = \frac{5.35\pi^2 \times 2 \times 10^5}{12(1-0.3^2)(151.67)^2} = 42.04 \text{ N/mm}^2$$

Non-dimensional web slenderness ratio for shear buckling stress

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 42.04}} = 1.85 > 1.2$$

Shear stress corresponding to web buckling for $\lambda_w > 1.2$ is given by

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.85^2} = 42.17 \text{ N/mm}^2$$

$\therefore$ Shear force corresponding to web buckling

$$V_{cr} = d t_w \tau_b$$
$$= 1820 \times 12 \times 12.17 \text{ N}$$
$$= 920.99 \text{ kN}$$
$$< 1653.75 \text{ kN} \qquad \text{(Not OK)}$$

$\therefore$ Web section is unsafe is shear and needs to be revised.

Let thickness of the web is revised from 12 m to 16 mm

$$\therefore \quad \frac{d}{t_w} = \frac{1820}{16} = 113.75$$

$\therefore$ Elastic critical shear stress $(\tau_{cr,e})$

$$= \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\dfrac{d}{t_w}\right)^2}$$

$k_v = 5.35$ for no intermediate transverse stiffeners

$$\therefore \quad \tau_{cr,e} = \frac{5.35\pi^2 \times 2 \times 10^5}{12(1-0.3^3)(113.75)^2} = 74.74 \text{ N/mm}^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 74.74}} = 1.39 > 1.2$$

$$\therefore \quad \tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.39^2} = 74.71 \text{ N/mm}^2$$

$\therefore$
$$V_{cr} = dt_w\tau_b = 1820 \times 16 \times 74.71 \text{ N}$$
$$= 2175.56 \text{ kN}$$
$$> 1653.75 \text{ kN}$$

$\therefore$ Web section of 1820 × 16 mm is safe in shear. (OK)

Check for lateral torsional buckling

Since the girder is laterally restrained throughout and thus there possibility of lateral torsional buckling.

Connection of flange to web

For each flange to web connection, there will be two weld lengths on either side of the web

$$q_w = \frac{VA_f\bar{y}}{2I}$$

$$I = \frac{b_f D^3}{12} - (b_f - t_w)\frac{d^3}{12} = \frac{610 \times 1920^3}{12} - (610-16)\frac{1820^3}{12}$$

$$= 3.598 \times 10^{11} = 2.984 \times 10^{11} = 0.614 \times 10^{11} \text{ mm}^4 = 614 \times 10^8 \text{ mm}^4$$

$\therefore$
$$q_w = \frac{1653.75 \times 610 \times 50 \times \left(\frac{1820}{2} + \frac{50}{2}\right)}{2 \times 614 \times 10^8} = 0.384 \text{ kN/mm}$$

Let size of weld, $S = 8$ mm

$\therefore$ $kS = 0.7 \times 8 = 5.6$ mm

$\therefore$ Strength of shop weld per unit length (f_{wd})

$$= \frac{5.6 \times 250}{\sqrt{3} \times 1.25} \text{ N/mm}$$

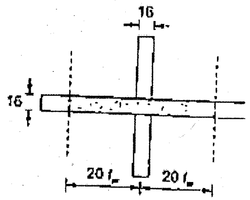
$$= 0.647 \text{ kN/m}$$
$$> 0.384 \text{ kN/mm}$$ (OK)

Design of end bearing stiffener

Bearing stiffeners are designed for the applied load or reaction less the local capacity of the web (F_w) which is given by

$$F_w = \frac{(b_1 + n_2)t_w f_{yw}}{\gamma_{m0}} \quad \text{where } b_1 = 125 \text{ mm}$$

$$n_2 = 50 \times 2.5 = 125 \text{ mm}$$



$\therefore$
$$F_w = (125 + 125)\frac{16 \times 250}{1.1} \text{ N}$$
$$= 909.1 \text{ kN}$$
$$< 1653.75 \text{ kN}$$

Thus stiffeners for bearing are required.

Try two flats as stiffener, one on either side of the web.

Maximum width of flat that can be accommodated

$$= \frac{610 - 16}{2} = 297 \text{ mm}$$

Let 16 mm thick flats are used.

Maximum permissible outstand $= 20\, t_f\epsilon = 20 \times 16 \times 1 = 320$ mm

Maximum effective outstand $= 14\, t_q\,\epsilon = 14 \times 16 \times 1 = 224$ mm

$\therefore$ Try flat of size 224 × 16 mm

Check for stiffener against buckling

Effective stiffners area $(A) = 2 \times 224 \times 16 + 2(20 \times 16) \times 16 = 17408 \text{ mm}^2$

Moment of inertia of the stiffener about centre line of web,

$$I_s = 2\left[\frac{16 \times 224^3}{12} + 16 \times 224 \times \left(\frac{224}{2} + \frac{16}{2}\right)^2\right]$$

$$= 2\left[14.986 \times 10^6 + 51.6096 \times 10^6\right] = 133.19 \times 10^6 \text{ mm}^4$$

$\therefore$ Radius of gyration $(r_x) = \sqrt{\frac{I_s}{A}} = \sqrt{\frac{133.19 \times 10^6}{17408}} = 87.47$ mm

Slenderness ratio $(\lambda) = \frac{0.7 \times 1820}{87.47} = 14.56$

Buckling curve to be used is curve *c*.

$\therefore$ For curve c, $\lambda = 14.56$ and $f_y = 250 \text{ N/mm}^2$

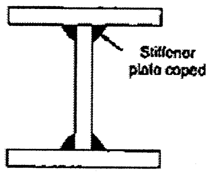
Design compressive stress $f_{cd} = 225.67 \text{ N/mm}^2$

$\therefore$ Buckling resistance $(P_d) = f_{cd}A$
$$= 225.67 \times 17408 \text{ N} = 3928.46 \text{ kN}$$
$$> 1653.75 \text{ kN}$$ (OK)

$\therefore$ Bearing stiffener is safe in compression (buckling)

Check for stiffener against bearing capacity

The bearing stiffener is required to accommodate the weld (fillet) required for joining web to the flange plate. Thus actual width available for stiffener flat will be less than the actual width available. Let a length of 15 mm is used for coping the weld.



$\therefore$ Width available for bearing $= 224 - 15 = 209$ mm

Bearing strength of stiffener, $F_p = \frac{A_q f_{yq}}{\gamma_{m0}} \geq (V - F_w)$

Area of stiffener in contact with flange $= 2(209 \times 16)$

$$A_q = 6688 \text{ mm}^2$$
$$V - F_w = (1653.75 - 909.1) = 744.65 \text{ kN}$$

$\therefore$
$$F_p = \frac{6688 \times 250}{1.1} \text{ N} = 1520 \text{ kN} > 744.65 \text{ kN}$$ (OK)

Thus bearing stiffener is safe against bearing capacity.

Check for torsional resistance provided by the bearing stiffener

Particularly from logistics and erection point of view, the plate girder must be safe for torsional resistance.

$$I_s \geq 0.34\alpha_s D^3 T_{cf}$$

$$I_y = 2\left(\frac{t_f b_f^3}{12}\right)+\left(\frac{d t_w^3}{12}\right) = \frac{2\times50\times610^3}{12}+\frac{1820\times16^3}{12} = 1892.13\times10^6\ m^4$$

$$A = 2(610\times50)+1820\times16 = 90120\ mm^2$$

$$\therefore \quad r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1892.13\times10^6}{90120}} = 144.899\ mm$$

$$\therefore \quad \text{Slenderness ratio}\ (\lambda_y) = \frac{L_{LT}}{r_y} = \frac{20\times1000}{144.899} = 138.03 > 100$$

$$\therefore \quad \alpha_s = \frac{30}{(L_{LT}/r_y)^2} = \frac{30}{(138.03)^2} = 1.575\times10^{-3}$$

$$\therefore \quad I_s \geq 0.34\,(1.575\times10^{-3})\,(1920)^3\,50 = 189.51\times10^6\ m^4$$

$$I_{s(provided)} = \frac{16\times(2\times224)^3}{12} = 119.89\times10^6\ mm^4$$

$$= 189.51\times10^6\ mm^4 \qquad \text{(Not safe)}$$

∴ Increase thickness of bearing stiffners to 18 mm.

Connection of end stiffener

On either side of the stiffener plate, there will be two weld lengths along the depth of web.

$$b_s = 224 - 15 = 209\ mm$$

Tension capacity of one flat (T_{dn})

$$= \frac{0.9A_n f_u}{\gamma_{m1}} = \frac{0.9(209\times16)410}{1.25}\ N = 987.15\ kN$$

$$\text{Shear force per unit length}\ (q_1) = \frac{987.15}{2(1820-2\times15)} = 0.276\ kN/mm$$

Let size of weld (S) = 8 mm

$$\therefore \quad kS = 0.7\times8 = 5.6\ mm$$

∴ Strength of weld per unit length assuming shop welding

$$f_{wd} = \frac{5.6\times250}{\sqrt{3}\times1.25}\ N = 0.647\ kN/m$$

$$> 0.276\ kN/mm \qquad \text{(OK)}$$

∴ Provide 8 mm size fillet weld to connect end bearing stiffener to the web plate.

Example 8.2 Design a plate girder of span 30 m using steel of grade Fe410 subjected to following loads at working conditions.

Dead loads:	UDL w_d = 16 kN/m
Concentrated load,	w_{1d} = 150 kN at 7 m from left end.
Concentrated load	w_{2d} = 150 kN at 7 m from right end.
Live load:	UDL w_L = 30 kN/m
Concentrated load	w_{1L} = 360 kN at 7 m from left end.
Concentrated load	w_{2L} = 360 kN at 7 m from right end.

Solution:

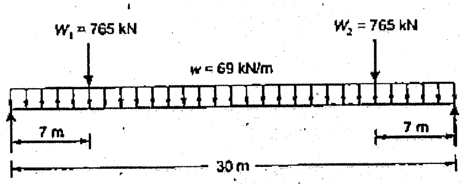
For Fe 410 steel, f_u = 410 N/mm², f_y = 250 N/mm²

Factored loads

$$\text{Total UDL}\ (w) = (w_d + w_L)\,1.5 = (16+30)1.5 = 69\ kN/m$$

$$w_1 = (w_{1d} + w_{2d})\,1.5 = (150+360)1.5 = 765\ \text{kN at 7 m from left end}$$

$$w_2 = (w_{2d} + w_{3L})\,1.5 = (150+360)1.5 = 765\ \text{kN at in llow right end}$$



$$\text{Factored bending moment}\ (M) = \frac{69\times30^2}{8}+765\times7 = 13117.5\ kNm$$

$$\text{Factored shear force}\ (V) = \frac{69\times30}{2}+765 = 1800\ kN$$

Depth of Web

Optimum depth of plate girder,

$$d = \left(\frac{MK}{f_y}\right)^{1/3}$$

Now when transverse stiffeners are provide and $3d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\epsilon = 200 \qquad \text{where}\ \epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\therefore \quad k = \frac{d}{t_w} = 200$$

$$\therefore \quad d = \left(\frac{Mk}{f_y}\right)^{1/3} = \left(\frac{13117.5\times10^6\times200}{250}\right)^{1/3} = 2189.34\ mm$$

Let d = 2250 mm

Now, $\frac{d}{t_w} = 200$

$$\Rightarrow \quad t_w = \frac{d}{200} = \frac{2250}{200} = 11.25\ mm \approx 16\ mm\ \text{(say)}$$

∴ Provide a web plate of size 2250 × 16 mm.

Design of flanges

Flange area required to resist the bending moment (A_f)

$$= \frac{M\gamma_{m0}}{f_y d} = \frac{13117.5\times10^6\times1.1}{250\times2250} = 25652\text{mm}^2$$

Let flange width is taken as $\frac{1}{3^{rd}}$ of depth of girder.

$$\therefore \quad b_f = \frac{2250}{3} = 750 \text{ mm}$$

$$\therefore \quad \text{Flange thickness } (t_f) = \frac{A_f}{b_f} = \frac{25652}{750} = 34.2 \text{ mm} \approx 50 \text{ mm (say)}$$

$\therefore$ Provide flange plate of size = 750 × 50 mm

Flange classification

$$\text{Outstand of flange } (b) = \frac{b_f - t_w}{2} = \frac{750-16}{2} = 367 \text{ mm}$$

$$\therefore \quad \frac{b}{t_f} = \frac{367}{50} = 7.34 < 8.4\epsilon \qquad \left[= 8.4\sqrt{\frac{250}{f_y}} = 8.4\sqrt{\frac{250}{250}} = 8.4\right]$$

$\therefore$ Flange section is plastic.

$$\text{Also} \quad \frac{d}{t_w} = \frac{2250}{16} = 140.625 > 126$$

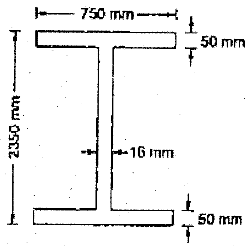
$\therefore$ Web is slender and it needs to be strengthen.

$$\text{Also} \quad \frac{d}{t_w} = 140.625 > 67\epsilon$$

$\therefore$ Web also requires check for shear buckling.

Check of adequacy of flange plates in bending

Plastic section modulus of the girder section is



$$Z_p = b_f t_f\left(\frac{D}{2} - \frac{t_f}{2}\right)\times 2$$

$$= b_f t_f (D - t_f)$$

$$= 750 \times 50\,(2350 - 50)$$

$$= 86.25 \times 10^6 \text{ mm}^3$$

$\therefore$ Moment carrying capacity of flange (M_d)

$$= \frac{Z_p f_y}{\gamma_{m0}} = \frac{86.25\times10^6\times250}{1.1}\text{Nmm}$$

$$= 19602.3 \text{ kNm}$$

$$> M (= 13117.5 \text{ kNm}) \qquad \text{(OK)}$$

Check for adequacy of web in shear

$$\text{Now,} \quad \frac{d}{t_w} = \frac{2250}{16} = 140.625 < 200\epsilon \qquad \text{where } \epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

(Serviceability criteria)

$$\text{and} \quad \frac{d}{t_w} = 140.625 < 345\,\epsilon^2 \qquad \text{(Compression flange buckling criteria)}$$

Critical elastic shear stress of web ($\tau_{cr,e}$)

$$= \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} \qquad \text{where, } \mu = 0.3,\ E = 2\times10^5 \text{ N/mm}^2$$

Let intermediate transverse stiffeners are provided at 2500 mm

$$\therefore \quad c = 2500 \text{ mm} \quad \text{and} \quad d = 2250 \text{ mm}$$

$$\therefore \quad \frac{c}{d} = \frac{2500}{2250} = 1.11 > 1$$

$$\therefore \quad k_v = 4 + \frac{5.35}{(c/d)^2} = 4 + \frac{5.35}{(1.11)^2} = 8.342$$

$$\therefore \quad \tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} = \frac{8.342\pi^2(2\times10^5)}{12(1-0.3^2)(140.625)^2} = 76.25 \text{ N/mm}^2$$

Non-dimensional web slenderness ratio for shear buckling stress,

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3}\times76.25}} = 1.376 > 1.2$$

$\therefore$ Shear stress corresponding to web buckling,

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w^2} = \frac{250}{\sqrt{3}\times1.376^2} = 76.23 \text{ N/mm}^2$$

$\therefore$ Shear force corresponding to web buckling

$$V_{cr} = A_v \tau_b = d t_w \tau_b$$

$$= 2250 \times 16 \times 76.23 \text{ N} = 2744.28 \text{ kN}$$

$$> 1800 \text{ kN} (= V)$$

Thus $V < V_{cr}$

$\Rightarrow$ End panel need not to be checked for tension field action.

Connection of flange to web

For each flange to web connection, there will be two weld lengths on either side of the web

$$q_w = \frac{VA_f\bar{y}}{2I}$$

$$I = \frac{b_f D^3}{12} - (b_f - t_w)\frac{d^3}{12} = \frac{750\times2350^3}{12} - (750-16)\frac{2250^3}{12}$$

$$= 8111.17\times10^8 - 6967.27\times10^8 = 1143.9\times10^8 \text{ mm}^4$$

$$\therefore \quad q_w = \frac{(1800\times1000)(2250\times16)\left(\frac{2250}{2}+\frac{50}{2}\right)}{2\times1143.9\times10^8}$$

$$= 325.73 \text{ N/mm} = 0.32573 \text{ kN/mm}$$

Let size of weld $(S) = 8$ mm

$\therefore$ Throat thickness of weld $(t) = 0.7S = 0.7 \times 8 = 5.6$ mm

$\therefore$Per mm strength of weld $(f_{wd}) = (1 \times t)\dfrac{f_u}{\sqrt{3}\gamma_{mw}}$

$= (1 \times 5.6)\dfrac{410}{\sqrt{3} \times 1.25}$ N/mm (Assuming shop weld $\gamma_{mw} = 1.25$)

$= 1060.48$ N/mm $= 1.06048$ kN/m

$> q_w (= 0.32573$ kN/mm$)$ (OK)

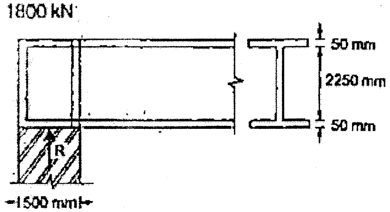
Design of bearing stiffener

At support, force to be carried by bearing stiffener = 1800 kN

Let bearing length at support $(b_1) = 500$ mm

Load capacity of web (F_w)

$$= (b_1 + n_2)t_w \frac{f_{yw}}{\gamma_{m0}}$$



Dispersion length of web

$= n_2 = 2.5 \times 50 = 125$ mm

$$\therefore \quad F_w = (500 + 125)16 \times \frac{250}{1.1} \text{N}$$

$= 2272.73$ kN $> V(= 1800$ kN$)$

Thus no bearing stiffener is required.

Design of intermediate transverse stiffener

Intermediate transverse stiffeners are provided to improve the buckling strength of slender web. Due to diagonal compression of shear, web buckling occurs. The transverse web stiffeners not being acted by concentrated load are so designed that the moment of inertia of the stiffener cross-section about an axis parallel to web is not less than that specified as per Cl 8.7.2.4 of IS 800 : 2007.

If $\dfrac{c}{d} \geq \sqrt{2}$ $\qquad I_s \geq 0.75\, d t_w^3$

If $\dfrac{c}{d} < \sqrt{2}$ $\qquad I_s \geq \dfrac{1.5 d^3 t_w^3}{c^2}$

Here, $c = 2500$ mm, $d = 2250$ mm

$$\therefore \quad \frac{c}{d} = \frac{2500}{2250} = 1.11 < \sqrt{2}$$

$$\therefore \quad \text{Minimum required, } I_s = \frac{1.5 d^3 t_w^3}{c^2}$$

Now it is assumed earlier that $3d > c > d$

$$c = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$\dfrac{d}{t_w} < 200\epsilon$ (From serviceability requirement)

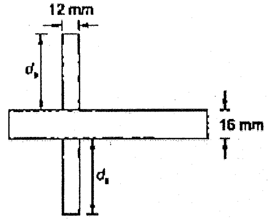
$$\Rightarrow \quad t_{w,\,reqd} > \frac{d}{200} = \frac{2250}{200} = 11.25 \text{ mm}$$

Now, $c < 1.5\, d$ i.e., 2500 mm $< 1.5 \times 2250$ (= 3375 mm)

$\dfrac{d}{t_w} < 345\epsilon$ (from compression flange buckling criteria)

$$\Rightarrow \quad t_{w,\,reqd} = \frac{d}{345} = \frac{2250}{345} = 6.5 \text{ mm}$$

Thus provide 12 mm thick stiffener.



$$\therefore \quad \text{Minimum required, } I_s = 1.5\frac{d^3 t_w^3}{c^2} = \frac{1.5 \times 2250^3 \times 12^3}{2500^2}$$

$= 4.72392 \times 10^6$ mm^4

$$\therefore \quad I_s \text{ provided} = \frac{12(2d_s)^3}{12} = 8d_s^3$$

$\therefore \quad 8d_s^3 \geq 4.72392 \times 10^6$

$\Rightarrow \quad d_s = 83.9$ mm $= 100$ mm (say)

Provide 2 nos. 100×12 mm size plate as intermediate stiffeners on each side of the web.

Check for buckling resistance of intermediate transverse stiffeners

Factored shear force $(V) = 1800$ kN

As determined earlier, $\tau_b = 76.23$ N/mm^2

$\therefore \quad V_{cr} = \tau_b A_v$

$= 76.23 \times 2250 \times 16$ N $= 2744.28$ kN

$> V(= 1800$ kN$)$

Thus buckling check for intermediate transverse stiffener is not required.

Connection of intermediate transverse stiffeners to web

This connection is to be designed for per mm shear along each component of stiffener.

$$Q = \frac{t_w^2}{5b_s} \text{ kN/m}$$

where, t_w = Thickness of web = 16 mm

b_s = Outstand of intermediate transverse stiffener

= 100 mm

$$\therefore \quad Q = \frac{16^2}{5 \times 100} = 0.512 \text{ kN/mm}$$

Let size of weld = S

$\therefore$ Throat thickness of weld $(t) = 0.7S$

$\therefore$Per mm strength of weld $(T_{dw}) = \dfrac{(1 \times t)f_u}{\sqrt{3}\gamma_{mw}} = \dfrac{0.7S(410)}{\sqrt{3} \times 1.25}$ (Assuming shop welding, $\gamma_{mw} = 1.25$)

$= 132.56\, S$ N/mm $= 0.13256\, S$ kN/mm

Thus $\quad T_{dw} \geq Q$

$\Rightarrow \quad 0.13256\, S \geq 0.512$

$\Rightarrow \quad S \geq 3.86$ mm

Thus provide a weld of size $(S) = 6$ mm

Objective Brain Teasers

Q.1 The optimum depth of a plate girder is given by the following expression:

(a) $\left(\dfrac{M}{f_y k^2}\right)^{1/3}$ (b) $\left(\dfrac{M}{f_y k^2}\right)^{2/3}$

(c) $\left(\dfrac{Mk^2}{f_y}\right)^{1/3}$ (d) $\left(\dfrac{Mk^2}{f_y}\right)^{4/3}$

Q.2 As per IS 800:2007, for plate girder with horizontal and vertical stiffeners, the larger and smaller unsupported clear dimension of the web panel must not exceed respectively:

(a) 200 t_w and 80 t_w (b) 180 t_w and 90 t_w
(c) 250 t_w and 120 t_w (d) 270 t_w and 200 t_w

Q.3 For unstiffened plate girder with clear distance d between the flanges, the thickness of web for serviceability requirement for the condition web connected to flange along one longitudinal direction only, must not be less than:

(a) $d/80$ (b) $d/100$
(c) $d/90$ (d) $d/85$

Q.4 In case of bolted plate girder, the angle section that is used should be:

(a) unequal with long leg connected.
(b) unequal with short leg connected.
(c) equal angle
(d) data insufficient

Q.5 When a plate girder is designed without using the tension field theory, the aspect ratio for end panel should lie in the range of:

(a) 0.6 to 1.0 (b) 0.3 to 0.8
(c) 1.0 to 1.5 (d) 0.2 to 0.6

Q.6 In order to prevent the plate girder from being getting flexible normal to their plane of web particularly during the fabrication and erection process, the following limits are adhered to:

(a) $d/t_w < 270$
(b) $c/d < 3$
(c) Both (a) and (b)
(d) Neither (a) nor (b)

Q.7 Which of the following type of failure is more prone to take place when web of the plate girder is made to thick?

(a) Local buckling
(b) Shear buckling
(c) Flexural buckling
(d) Shear yielding

Q.8 The advantage of end panel design by post critical method is that:

(a) it makes use of tension field action.
(b) avoid formation of tension field in the need panel.
(c) effectively makes use of end post.
(d) All of the above

Q.9 Which of the following statements is correct?

(a) The outstand of stiffeners of thickness t_q must not be more than $14t_q$
(b) MOI of first horizontal stiffener must not be less than $d_2^2 t^3$
(c) Horizontal stiffeners are provided when d/t lies between 90 and 300.
(d) MOI of second horizontal stiffener must not be less than $4ct_w^3$

Q.10 On both sides, the length of web in compression acting as part of bearing stiffener is:

(a) 12 t_w (b) 25 t_w
(c) 20 t_w (d) 16 t_w

Q.11 End posts are provided in plate girders:

(a) to carry the shear force due to reactions
(b) to carry the tension field developed
(c) to prevent buckling of flanges
(d) All of the above

Q.12 The load carrying mechanism in web of plate girder post-buckling is analogous to

(a) A truss (b) N-truss
(c) Pratt truss (d) k-truss

Q.13 Which of the following statements (s) is(are) correct?

1. Intermediate stiffeners are load carrying members in LSM but are not load carrying members in WSM.
2. LSM permits the use of thin webs as compared to WSM.
3. LSM restricts the use of deep thin webs whereas this is not so in WSM.

(a) 1 and 3 (b) 2 and 3
(c) 2 only (d) 1 and 2

Q.14 The connection of vertical stiffeners to web of plate girder is designed for

(a) Moment of $\dfrac{t_w}{7b_s}$ (b) Shear of $\dfrac{t_w^3}{5b_s}$

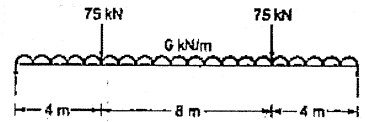
(c) Moment of $\dfrac{t_w^3}{5b_s}$ (d) Shear of $\dfrac{t_w^3}{7b_s}$

Answers

1. (a)	2. (d)	3. (c)	4. (b)	5. (a)
6. (c)	7. (d)	8. (b)	9. (a)	10. (c)
11. (b)	12. (c)	13. (d)	14. (b)	

Conventional Practice Questions

Q.1 Design a plate girder of span 16 m loaded as shown in figure below, use steel of grade Fe 410.



Q.2 Design the section of a plate girder with restrained compression flange, and carrying a total uniformly distributed load of 12 kN/m over a span of 10 m. Provide stiffeners also if required.

Q.3 Design a welded plate girder of 30 m span subjected to a uniformly distributed load of 20 kN/m.

■■■■