DSS-MOD-V-PLATE GIRDER

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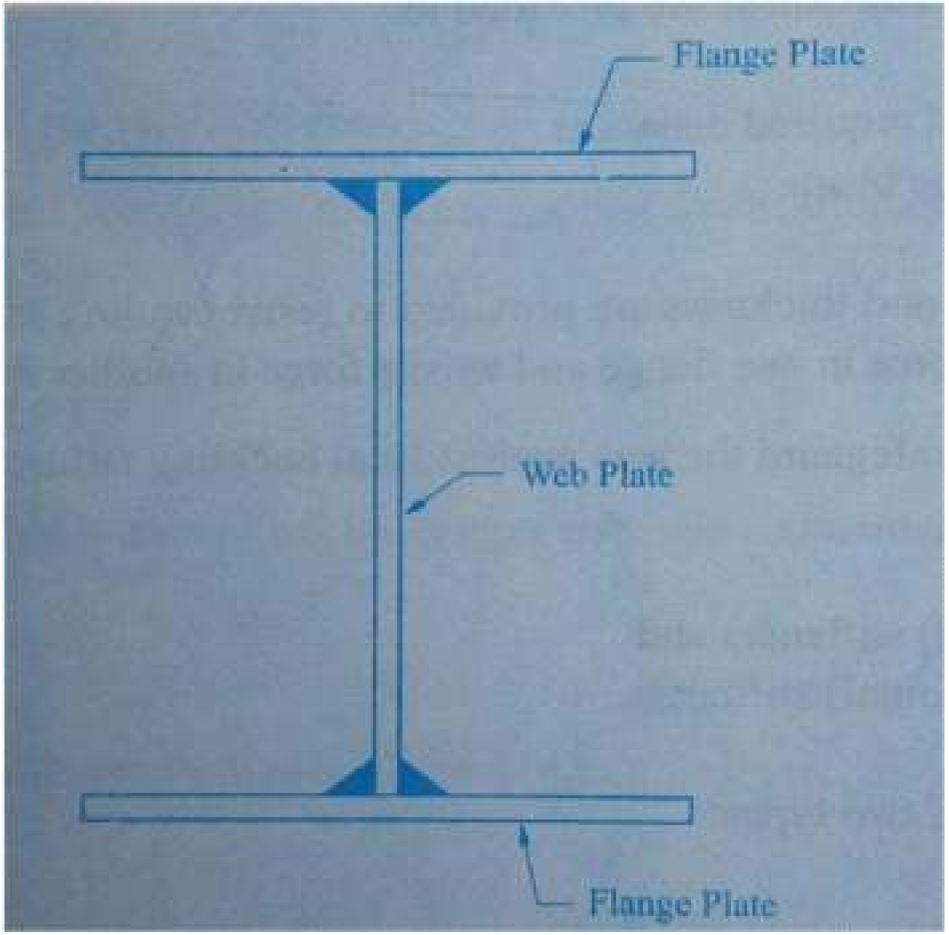
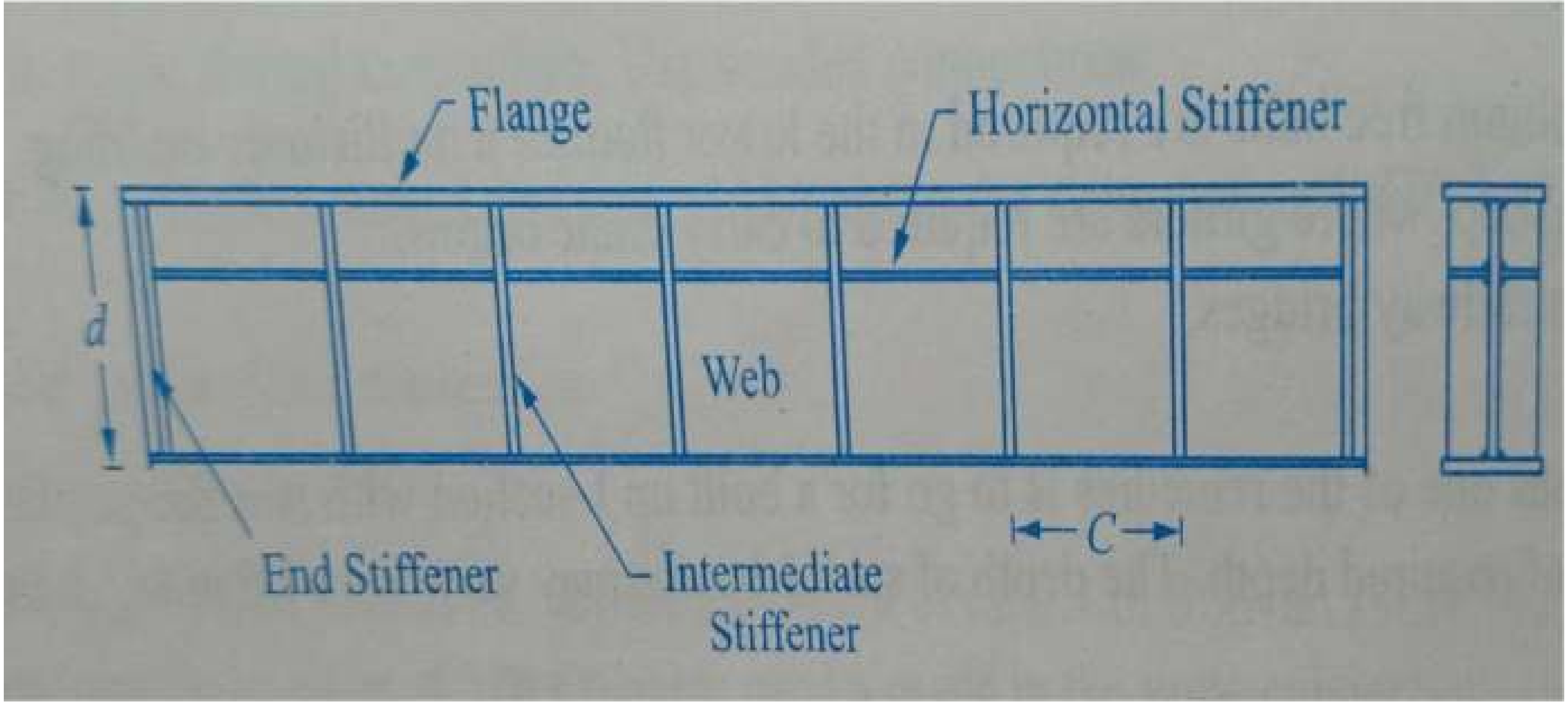
- A plate girder is a beam built up from plate elements to achieve a more efficient arrangement of material than is possible with rolled beams.
- Plate girders are economical where the spans are long enough to permit saving in cost by proportioning for the particular requirements.
- Plate girders may be of riveted, bolted, or welded constructions.

Difference between Beam and Girder

- A plate girder is actually a deep beam. The limit states with regard to beams are still applicable for plate girders.
- All rolled W (wide-flange shapes) have "compact" webs and only a few sections have "non-compact" flanges.
- When the web is "non-compact" or "slender", provisions should be given to account for local and bend-buckling of the web.
- In general, plate girder webs are typically "slender".
- The flexural and shear strengths of a plate girder are largely related to the flange and web respectively.



Flange Plate
Web Plate
Flange Plate


Flange
Horizontal Stiffener
d
Web
End Stiffener
Intermediate Stiffener
C

ELEMENTS OF PLATE GIRDER

1. WEB
2. FLANGE
3. STIFFENERS

Webs of required depth and thickness are provided to

a) Keep flange plates at required distances.
b) Resist the shear in beam.

Flanges of required width and thickness are provided to resist bending moment developing compressive force in one flange and tensile force in another flange.

Stiffeners are provided to safeguard the web against local buckling. Stiffeners are provided along transverse(vertical) and longitudinal(Horizontal) direction.

Transverse Stiffeners are of two types.

A) End Bearing Stiffeners and B)intermediate stiffeners

A) End Bearing Stiffeners-These are provided to transfer the load from beam to support. Near the support, certain portion of web is subjected to compression, so there is possibility of crushing of web.Web needs some stiffeners to transfer load to support.

B)Intermediate stiffeners: To resist shear, the required thickness of web may be very small. As thickness is small. To avoid buckling, intermediate stiffeners may be required.

Many times horizontal stiffeners are provided to incease buckling strength of web. If only one horizontal stiffener will be provided then it should be provided at a depth 0.2 d from compression flange (d=depth of web). If another horizontal stiffener is to be provided, it should be at mid of web. Generally web, flange and stiffeners all are plates.

SELF WEIGHT OF PLATE GIRDER

w=W/200 kN/m, w= factored self weight, W=Factored Total load on girder

ECONOMICAL DEPTH:

Assuming the moment to be taken by flange only

$M=f_y*b_f*t_f*d$

$b_f*t_f*d_f=M/f_y$ (b_f*d_f= Area of flange, bf, df= breadth and depth of flange

A=2*bf*tf+d*tw= 2M/(fy*d)+d*tw

Taking $d/t_w=k$

$A= 2M/(f_y*d)+d^2/k$

To make area minimum,, differentiating w.r.t. d,

$0=-2M/(fy*d^2)+2d/k$

$d^3=Mk/f_y$

$d=[Mk/f_y]^{1/3}$

$\varepsilon=1$, and $[k=d/tw]\leq 67$, design the beam as ordinary beam

$\varepsilon=1$, and $[k=d/tw]=$ 67 to 200, , plate girder without intermediate stiffener. Generally k from 100-110, intermediate stiffeners may not be required, for higher values it is required.

$\varepsilon=1$, and $[k=d/tw]=250$, longitudinal stiffeners required.

K should not be taken more than 345.

Another practical guide line,

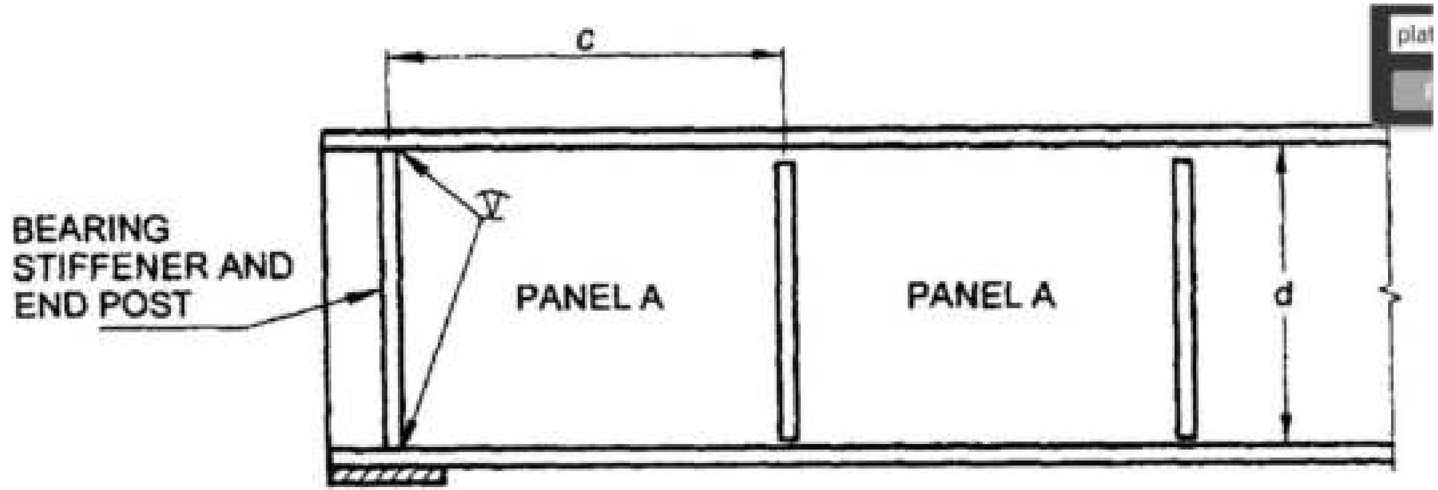
D/L=1/15 to 1/25 for girders in building.

D/L=1/12 to 1/18 for girders in highway bridge.

D/L=1/10to 1/15 for girders in railway bridge.

D depth of girder including flange.

L equivalent span length



8.6 Design of Beams and Plate Girders with Solid Webs

8.6.1 *Minimum Web Thickness*

The thickness of the web in a section shall satisfy the following requirements:

8.6.1.1 *Serviceability requirement*

a) When transverse stiffeners are not provided,

$$\frac{d}{t_w} \leq 200\varepsilon$$ (web connected to flanges along both longitudinal edges)

$$\frac{d}{t_w} \leq 90\varepsilon$$ (web connected to flanges along one longitudinal edge only).

b) When only transverse stiffeners are provided (in webs connected to flanges along both longitudinal edges),

1) when $3d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\ \varepsilon$$

2) when $0.74\ d \leq c < d$

$$\frac{c}{t_w} \leq 200\varepsilon_w$$

3) when $c < d$

$$\frac{d}{t_w} \leq 270\varepsilon_w$$

4) when $c > 3d$, the web shall be considered as unstiffened.

c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange) according to **8.7.13** (a)

1) when $2.4d \geq c \geq d$

$$\frac{d}{t_w} \leq 250\varepsilon_w$$

2) when $0.74\,d \leq c \leq d$

$$\frac{c}{t_w} \leq 250\varepsilon_w$$

3) when $c < 0.74\,d$

$$\frac{d}{t_w} \leq 340\varepsilon_w$$

d) When a second longitudinal stiffener (located at neutral axis is provided)

$$\frac{d}{t_w} \leq 400\varepsilon_w$$

where

d = depth of the web,

t_w = thickness of the web,

c = spacing of transverse stiffener (*see* Fig. 12 and Fig. 13),

ε_w = yield stress ratio of web = $\sqrt{\frac{250}{f_{yw}}}$, and

f_{yw} = yield stress of the web.

MINIMUM THICKNESS OF WEB BASED ON FLANGE BUCKLING

8.6.1.2 *Compression flange buckling requirement*

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

a) When transverse stiffeners are not provided

$$\frac{d}{t_w} \le 345\varepsilon_f^2$$

b) When transverse stiffeners are provided and

1) when $c \ge 1.5\, d$

$$\frac{d}{t_w} \le 345\varepsilon_f^2$$

2) when $c < 1.5d$

$$\frac{d}{t_w} \le 345\varepsilon_f$$

where

d = depth of the web,

t_w = thickness of the web,

c = spacing of transverse stiffener (*see* Fig. 12 and Fig. 13),

ε_f = yield stress ratio of web = $\sqrt{\frac{250}{f_{yf}}}$, and

f_{yf} = yield stress of compression flange.

Size of flanges

Assuming bending is to be resisted by flange

M=Af*fy*d/1.1

Select 9.4 ε <tf<13.6 ε for semi compact,

SHEAR BUCKLING RESISTANCE OF WEB:

For thin webs, shear resistance for webs is required.'

d/tw>67 ε for a web without stiffeners

d/tw>67 ε*sqrt[kv/5.35] for a web with stiffeners

Kv=5.35 when transverse stiffeners are provided.

$$= 4.0 + \frac{5.35}{(\frac{c}{d})^2}\ for \frac{c}{d} < 1.0 \qquad\qquad = 5.35 + \frac{4.0}{(\frac{c}{d})^2}\ for \frac{c}{d} \geq 1.0$$

C, d are spacing of transverse stiffeners and depth of web respectively35 when transverse stiffeners are provided.

Vn=Vcr are calculated by

a) Simple post critical method

b) Tension field method

8.4.2.2 *Shear buckling design methods*

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) *Simple post-critical method* — The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}$$

where

V_{cr} = shear force corresponding to web buckling

$= A_v \tau_b$

where

τ_b = shear stress corresponding to web buckling, determined as follows:

1) when $\lambda_w \leq 0.8$

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

2) when $0.8 < \lambda_w < 1.2$

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

3) when $\lambda_w \geq 1.2$ $\quad \tau_b = f_{yw} / \left(\sqrt{3}\lambda_w^2\right)$

where

λ_w = non-dimensional web slenderness ratio for shear buckling stress, given by:

$$\lambda_w = \sqrt{f_{yw}/\left(\sqrt{3}\tau_{cr,e}\right)}$$

$t_{cr,e}$ = the elastic critical shear stress of the

$$\text{web} = \frac{K_v \pi^2 E}{12\left(1-\mu^2\right)\left[d/t_w\right]^2}$$

where

μ = Poisson's ratio, and

K_v = 5.35 when transverse stiffeners are provided only at supports

= $4.0 + 5.35/(c/d)^2$ for $c/d < 1.0$

= $5.35 + 4.0/(c/d)^2$ for $c/d \geq 1.0$

where c, d are the spacing of transverse stiffeners and depth of the web, respectively.

TENSION FIELD METHOD

The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and if $c/d \geq 1.0$, where c, *d are the spacing of transverse stiffeners and* depth of the web, respectively.

In the tension field method, the nominal shear resistance, V_n, *is given by:*

$$V_n = V_{tf}$$

where

$$V_{tf} = \left[A_v\, \tau_b + 0.9\, w_{tf}\, t_w\, f_v \sin\phi\right] \leq V_p$$

where

τ_b = buckling strength, as obtained from **8.4.2.2**(a)

f_v = yield strength of the tension field obtained from

$$= \left[f_{yw}^{\,2} - 3\tau_b^{\,2} + \psi^2\right]^{0.5} - \psi$$

ψ = $1.5\, \tau_b \sin 2\phi$

ϕ = inclination of the tension field

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

w_{tf} = the width of the tension field, given by:

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

w_{tf} = the width of the tension field, given by:

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

f_{yw} = yield stress of the web

d = depth of the web

c = spacing of stiffeners in the web

τ_b = shear stress corresponding to buckling of web **8.4.2.2**(a)

s_c, s_t = anchorage lengths of tension field along the compression and tension flange respectively, obtained from:

$$s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_{yw}\, t_w}\right]^{0.5} \leq c$$

where

M_{fr} = reduced plastic moment capacity of the respective flange plate (disregarding any edge stiffener) after accounting for the axial force, N_f in the flange, due to overall bending and any external axial force in the cross-section, and is calculated as:

$$M_{fr} = 0.25\, b_f\, t_f^{\,2}\, f_{yf}\left[1 - \left\{N_f / \left(b_f\, t_f\, f_{yf} / \gamma_{m0}\right)\right\}^2\right]$$

where

b_f, t_f = width and thickness of the relevant flange respectively

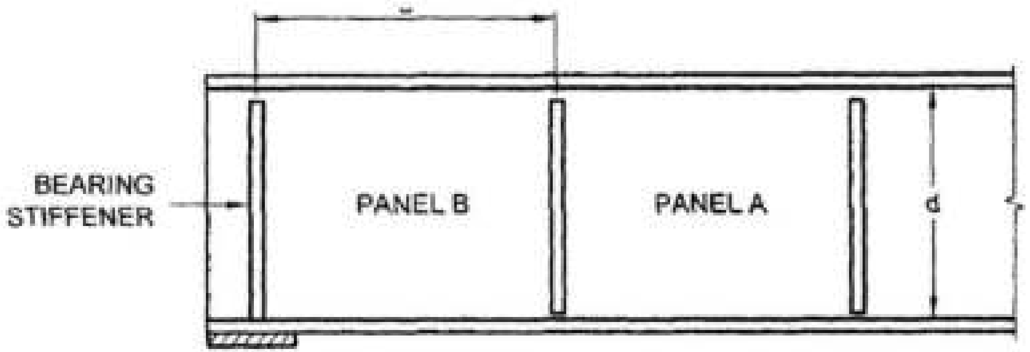
f_{yf} = yield stress of the flange

8.5 Stiffened Web Panels

8.5.1 *End Panels Design* (*see* Fig. 12)

The design of end panels in girders in which the interior panel (panel A) is designed using tension field action shall be carried in accordance with the provisions given herein. In this case the end panel should be designed using only simple post critical method, according to **8.4.2.2**(a).

Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges to resist a shear force, R_{tf} and a moment, M_{tf} due to tension field forces as given in **8.5.3**. Further, end stiffener should be capable of resisting the reaction plus a compressive force due to the moment, equal to M_{tf} (*see* Fig. 12).



NOTES

1 Panel A is designed utilizing tension field action as given in **8.4.2.2**(b).

2 Panel B is designed without utilizing tension field action as given in **8.4.2.2**(a).

3 Bearing stiffener is designed for the compressive force due to bearing plus compressive force due to the moment M_q as given in **8.5.3**.

8.5.2 *End Panels Designed Using Tension Field Action* (*see* Fig. 13 and Fig. 14)

The design of end panels in girders, which are designed using tension field action shall be carried out in accordance with the provisions mentioned herein. In this case, the end panel (Panel B) shall be designed according to **8.4.2.2**(b).

Additionally it should be provided with an end post consisting of a single or double stiffener (*see* Fig. 13 and Fig. 14), satisfying the following:

a) *Single stiffener* (*see* Fig. 13) — The top of the end post should be rigidly connected to the flange using full strength welds.

The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to 2/3 M_{tf} due to tension field forces, where M_{tf} is obtained from **8.5.3**. The width and thickness of the end post are not to exceed the width and thickness of the flange.

b) *Double stiffener* (*see* Fig. 14) — The end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment, M_{tf} due to the tension field forces as given in **8.5.3**.

8.5.3 *Anchor Forces*

The resultant longitudinal shear, R_{tf}, and a moment M_{tf} from the anchor of tension field forces are evaluated as given below:

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

where

$$H_q = 1.25\,V_p\left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$$

$$V_p = \frac{dtf_y}{\sqrt{3}}$$

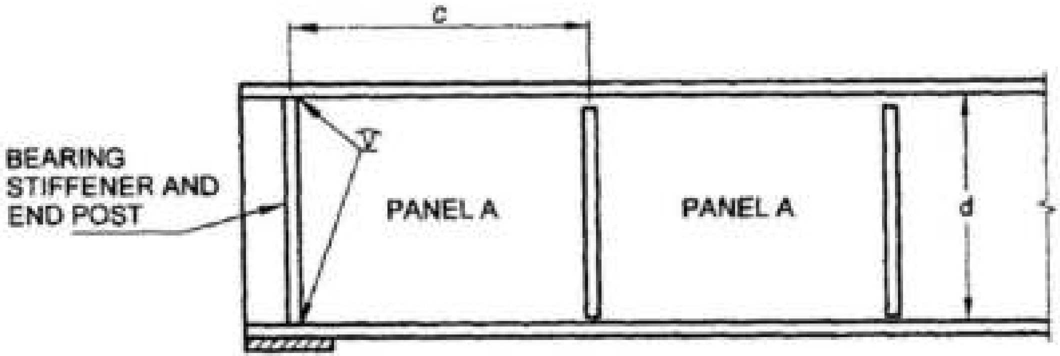
d = web depth

If the actual factored shear force, V in the panel designed using tension field approach is less than the shear strength, V_{tf} as given in **8.4.2.2**(b), then the values of H_q may be reduced by the ratio $\frac{V - V_{cr}}{V_{tf} - V_{cr}}$

where

V_{tf} = the basic shear strength for the panel utilizing tension field action as given in **8.4.2.2**(b), and

V_{cr} = critical shear strength for the panel designed utilizing tension field action as given in **8.4.2.2**(a).



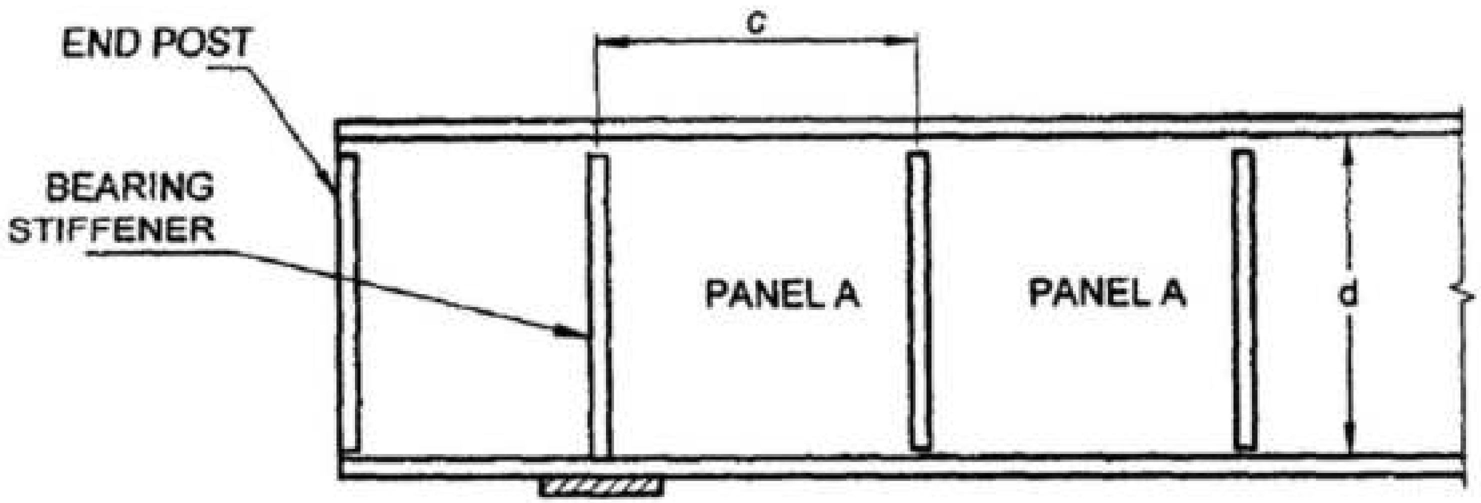
NOTES

1 Panel A is designed utilizing tension field action as given in **8.4.2.2**(b).

2 Panel B is designed utilizing tension field action as given in **8.4.2.2**(b).

3 Bearing stiffener and end post is designed for combination of compressive loads due to bearing and a moment equal to 2/3 M_q as given in **8.5.3**.

FIG. 13 END PANEL DESIGNED USING TENSION FIELD ACTION (SINGLE STIFFENER)



NOTES

1 Panel *A* is designed utilizing tension field action as given in **8.4.2.2**(b).

2 Bearing stiffener is designed for compressive force due to bearing as given in **8.4.2.2**(a).

3 End post is designed for horizontal shear R_u and moment M_u as given in **8.5.3**.

FIG. 14 END PANEL DESIGNED USING TENSION FIELD ACTION (DOUBLE STIFFENER)

10.6.1 If Simple Post Buckling Method is Used in the Design of End Panel

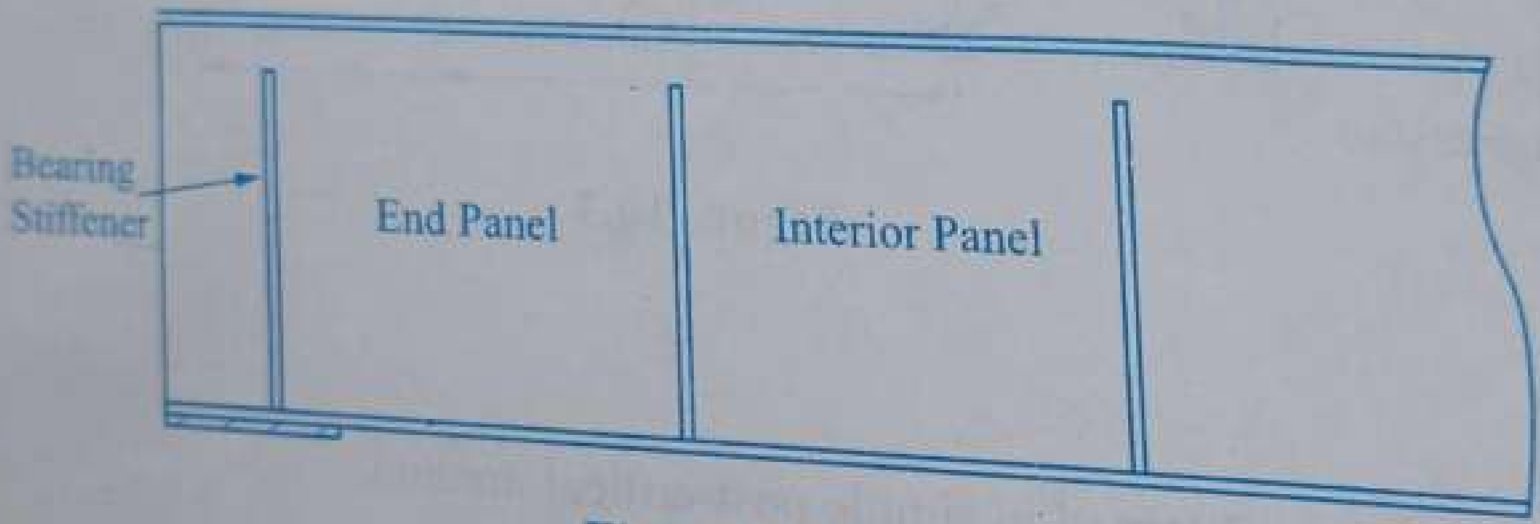


Figure 10.4

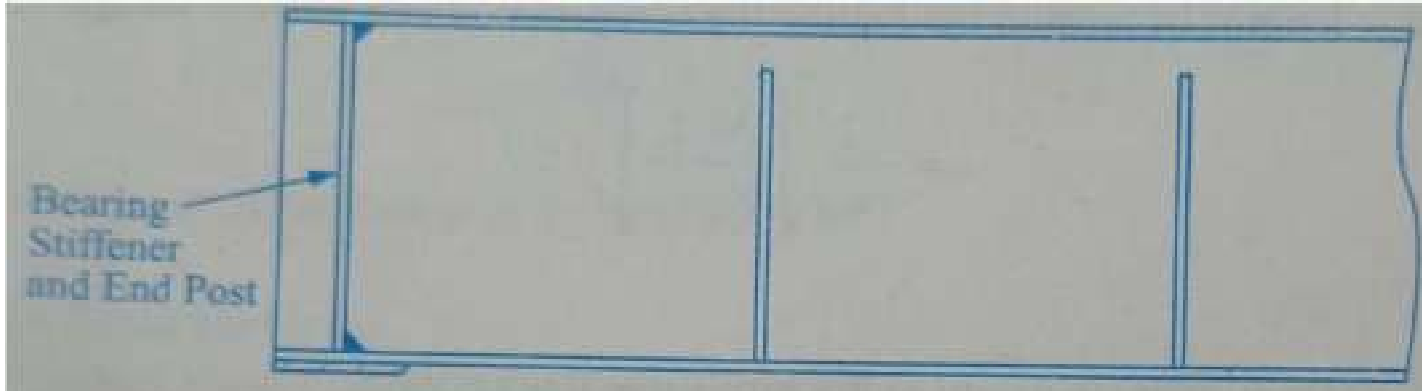


Figure 10.5

Figure 10.6

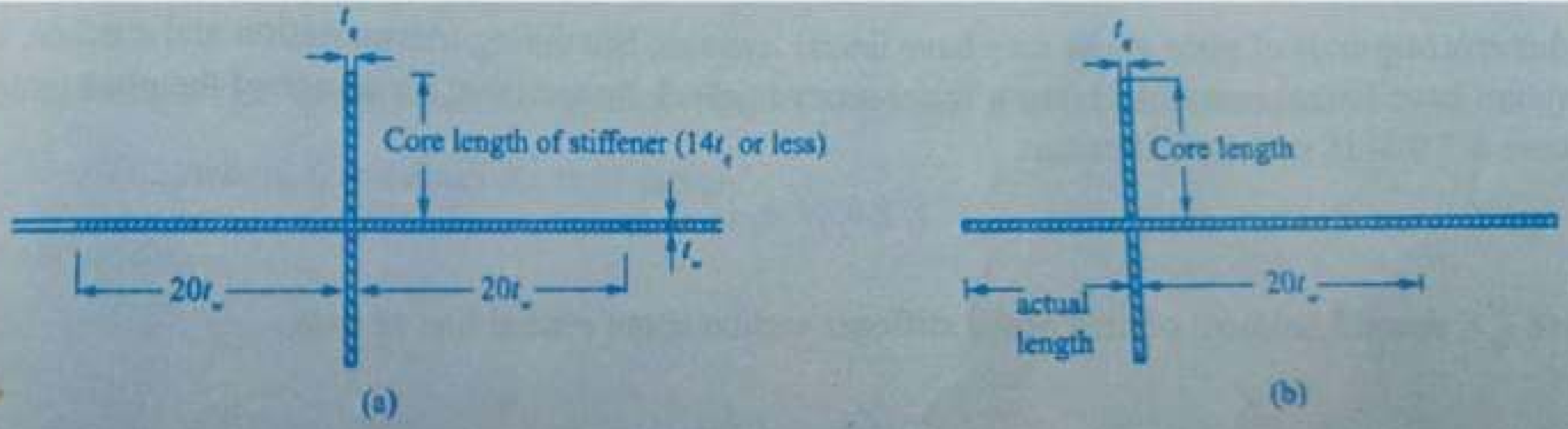

t_s
Core length of stiffener ($14t_s$ or less)
t_w
$20t_w$
$20t_w$
(a)
t_s
Core length
actual length
$20t_w$
(b)

Figure 10.7

- Procedure of Design of Plate Girder

1. SELF WEIGHT OF PLATE GIRDER, w=W/200 kN/m, w= factored self weight, W=Factored Total load on girder
2. Economical depth= $d=[Mk/f_y]^{1/3}$
3. Determine the area of the flange to resist moment. Proportion it so that b_f/t_f satisfies requirements of plastic/ compact/ semi compact.
4. Check the moment capacity of the girder.
5. Find shear resistance of web using either simple post critical method or tension field method.
6. Design the weld connecting flange plate to web plate.
7. Design the end bearing stiffeners.
8. Design the connection of stiffener.
9. Design the load carrying stiffeners. If required.
10. Design the intermediate stiffeners. If required.

Design a welded plate girder of span 24 m to carry a super imposed load 35 kN/m. Avoid using of bearing and intermediate stiffeners. Use Fe415 steel.

Solution:

1. Calculation of moment and shear Force:

Span=24 m, Super imposed load= 35 kN/m,

Factored super imposed load=35*1.5=52.5 kN/m

Total factored super imposed load=1.5*35*24=1260 kN

Self weight= 1260/200=6.3 kN/m

Total load=52.5+6.3=58.8 kN/m

Bending moment= $58.8*24^2/8=4233.6$ kNm

SF=58.8*24/2=705.6 kN

2.Depth of web Plate

Avoiding stiffeners, $d/t_w \leq 67$

Economical depth of web $d=[Mk/f_y]^{1/3}$

D={4233.6*103*67/250}1/3=1043 mm

Taking 1000 mm plate $t_w \geq [1000/67=14.92$ mm] Take $t_w=16$ mm

Web plate is 1000 mm *16 mm

3. Selection of Flange:

Assuming moment to be taken by flange alone

$[A_f*f_y*d/1.1] \geq M$

$A_f \geq M*1.1/[f_y*d]$

$A_f \geq [4233.6*106*1.1/\{250*1000\}] \geq 18628$ mm2

For making it a plastic section $b/t_f \leq 8.4$

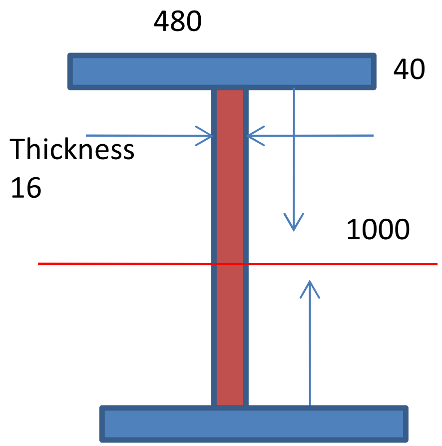
$b_f/(2*t_f) \leq 8.4$

$b_f=16.8t_f$

$A_f=b_f*t_f=16.8$ tf*tf ≥ 18628 mm^2 which gives $t_f=33.3$ mm

Taking 40 mm thick plate , $b_f=18628/40=465.7$ mm

Use flange plate=480 mm*40 mm.



4. Checking Moment Capacity of Girder

Md=[Zp*fy/1.1] ≤ 1.2*Ze*fy/1.1

Md= =[Zp*fy/1.1] =[480*40*(1000+40/2+40/2)*250/1.1]=4538.182 kN ≤ 1.2*Ze*fy/1.1

$$I_{zz} = 2*[\frac{1}{12}*480*40^3+480*40*\left(\frac{1000+40}{2}\right)^2 = $$
$$= 2*5601.28*10^6 mm^4$$

$$1.2*Z_e*\frac{f_y}{1.1} = 1.2*[I_{zz}/y_{max}]*250/1.1 ==$$
$$= 1.2*[2*5601.28*10^6/540]*250/1.1$$
$$= 5657.82\ kNm$$

Md=4538.182> [M=4233.6 kNm]

5.Shear Resisting Capacity:

$$V_d = \frac{A_v * f_{yw}}{1.1 * \sqrt{3}} = \frac{d * t_w * f_{yw}}{1.1 * \sqrt{3}} = \frac{1000 * 16 * 250}{1.1 * \sqrt{3}}$$
$$= 2099.45\ kN > 705.6\ kN$$

No stiffener is required.

6. Check for End Bearing:

Bearing strength of web: $F_w=(b_1+n_2)t_w*f_{yw}/1.1$

Assuming width of bearing 200 mm and stiff bearing length 100 mm

n2=2.5*bf=2.5*40=100 mm

F_w=(100+100)16*250/1.1=727 kN>705.6 kN

So, safe.

7. Weld Design to join web to Flange:

Shear force=705.6 kN

$$Shear\ stress\ in\ flange\ at\ the\ level\ of\ junction\ of\ web\ and\ flange$$
$$= q = \frac{F}{bI}a\bar{y}$$
$$= \frac{705.6 * 1000}{480 * 2 * 5601.28 * 10^6}[480 * 40 * \left(500 + \frac{40}{2}\right)$$
$$= 0.512\ N/mm^2$$

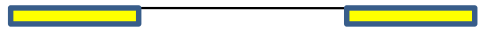
Shear force per mm length in the junction=0.512*480=245.76 N

If s is the size of the weld, providing weld on both sides

$$2 * 0.7 * s * \frac{410}{\sqrt{3} * 1.25} = 265.1\, s = 245.76\ , s = 0.92\ mm$$

As minimum weld thickness is 5 mm for web thickness 16 mm, so provide intermittent welds.

Percentage of weld=0.92*100/5=18.4 %,



Taking 20% weld length

As minimum weld length 40 mm, unweld length will be 40*4=160 mm

Maximum unweld length=12*t=12*16=192> 160 so OK>

Problem: Design the same plate girder thin web and end stiffener without intermediate stiffeners.

Solution:

1. Maximum moment=4233.6 kNmand Maximum shear force=705.6 kN.
2. Selection of depth of web

   For d/tw> 200,intermediate transverse stiffeners are required.

   For d/tw< 67,no intermediate or end transverse stiffeners are required.

Take d/tw=100, so that only end transverse stiffeners are required.

$k=d/t_w=100$ $d=[Mk/f_y]^{1/3}$

$d=[4233.6*10^6*100/250]^{1/3}=1192$ mm, provide d=1200 mm

k=100=d/tw, $1200/100=t_w=12$ mm web plate=1200 mm*12 mm plate.

3. Design of Flange

$[A_f*f_y*d/1.1] \geq M$

$A_f \geq M*1.1/[f_y*d]$

$A_f \geq [4233.6*10^6*1.1/\{250*1200\}] \geq 15523 mm^2$

For making it a plastic section $b/t_f \leq 8.4$

$b_f/(2*t_f) \leq 8.4$

$b_f=16.8t_f$

$A_f=b_f*t_f=16.8$ tf*tf ≥ 15523 mm^2 which gives $t_f=30.4$ mm, take tf=36 m

bf=15523/36=431 mm

Use flange plate=440 mm*36 mm

4. Checking Moment Capacity of Girder

Md=[Zp*fy/1.1] ≤ 1.2*Ze*fy/1.1

M_d= =[Z_p*fy/1.1] =[440*36*(1200+36/2+36/2)*250/1.1]=4449.6 kNm

1.2*Ze*fy/1.1

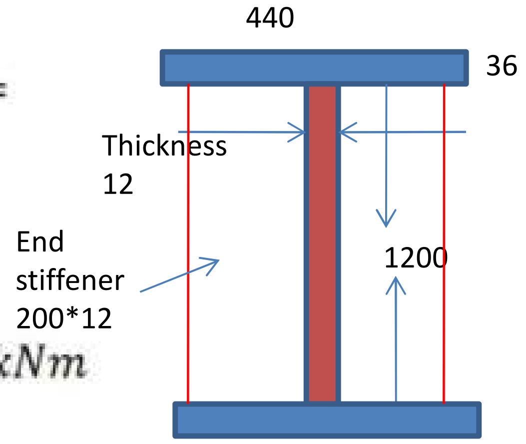
$$I_{zz} = 2 * [\frac{1}{12} * 440 * 36^3 + 440 * 36 * \left(\frac{1200+36}{2}\right)^2 =$$

$$= 1.21 * 10^{10} mm^4$$

$$= 1.2 * Z_z * 250/1.1 = 1.2 * [I_{zz}/y_{max}] * 250/1.1 ==$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w} = \frac{f_y 250}{\sqrt{3} 1.22^2} = 1.296 [97\ N/mm^2$$

$$= 1.2 * [1.21 * 10^{10}/636] * 250/1.1 = 5189.9\ kNm$$



M_d=4449.6> [M=4233.6 kNm]

5. Shear Resistnace of web:

Trannsverse stiffener only providedkv=5.35

d/tw>67

Check for shear bucklingusing simple post critical method

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)(\frac{d}{t_w})^2} = \frac{5.35 * \pi^2 2 * 10^5}{12(1-0.3^2)(\frac{1200}{12})^2}$$

$$= 96.7\ N/mm^2$$

Calculations:

As λ_w=1.22 > 1.2

$$\lambda_w = \sqrt{\frac{f_{yw}}{\tau_{cr}\sqrt{3}}} = \sqrt{\frac{250}{96.7 * \sqrt{3}}} = 1.22$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w{}^2} = \frac{250}{\sqrt{3}1.22^2} 96.97\ N/mm^2$$

V_{cr}=d*t_w*τb=1200*12*96.97=1396.44 kN

V_d=V_{cr}/γ_{m0}=1396.44/1.1=1269.49>705.6 kN

6.LOAD CAPACITY OF WEB

As per clause 8.7.4 load capacity of web Fw=(b1+n2)tw*fyw/γ_{m0}

b1=0, n2=2*2.5*36=180 mm

Fw=(0+180)12*250/1.1=545.45 kN<705.6 kN

Hence end stiffeners are required.

7.DESIGN OF END STIFFENERS

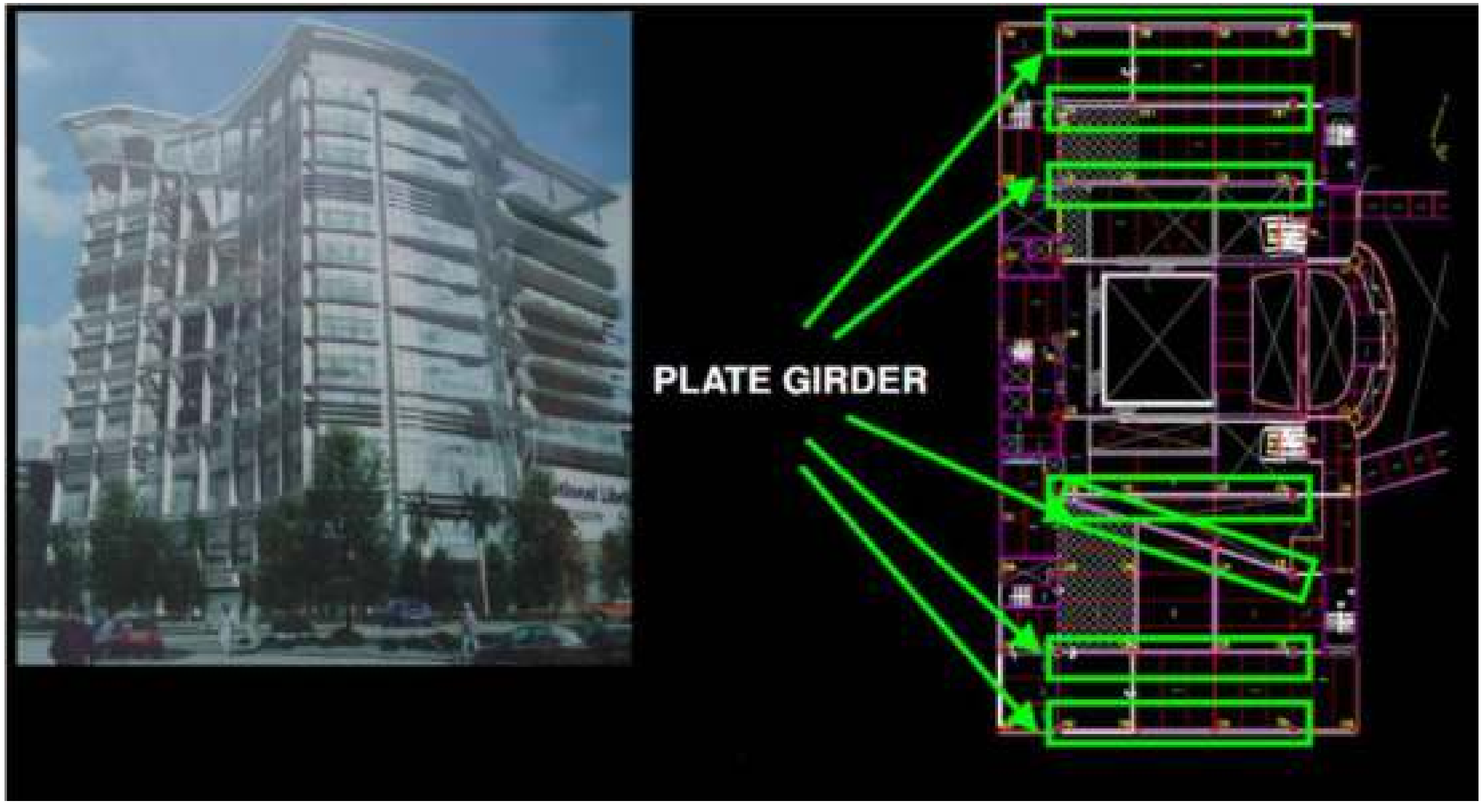
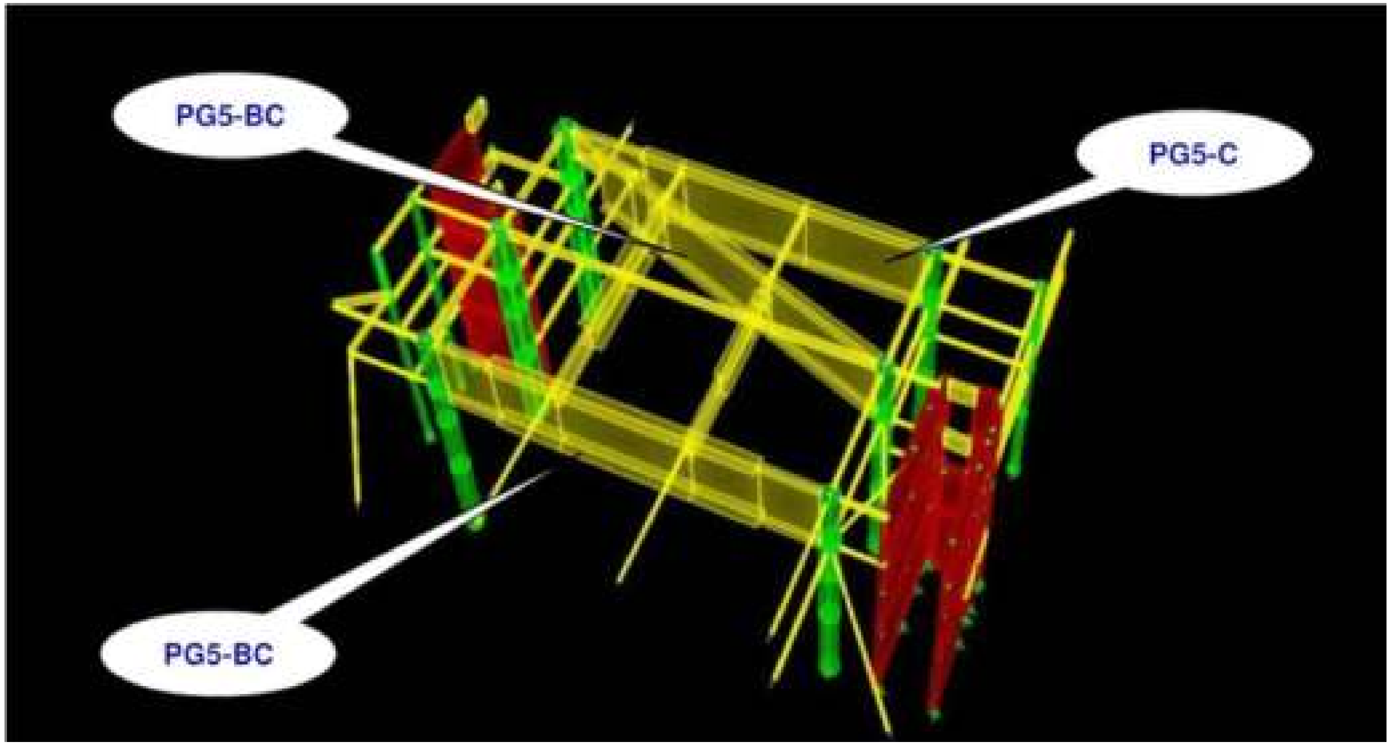


PLATE GIRDER



PG5-BC
PG5-C
PG5-BC

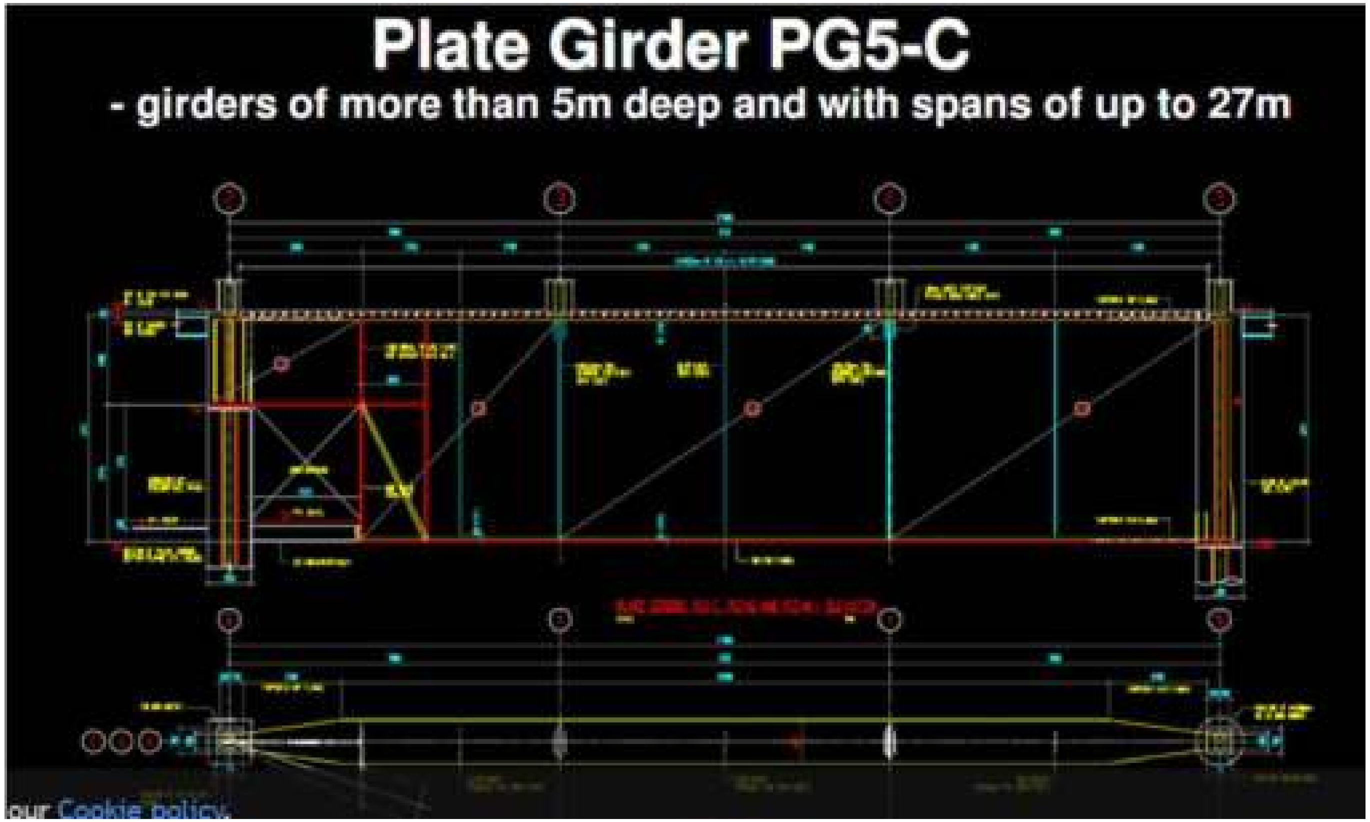
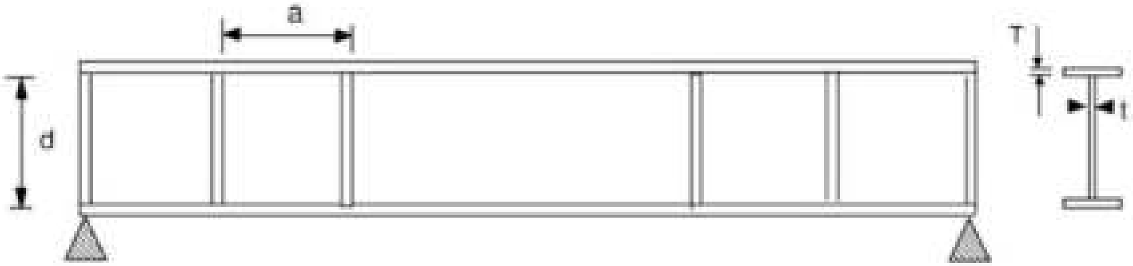


Plate Girder PG5-C
- girders of more than 5m deep and with spans of up to 27m
our Cookie policy.

Design of Plate Girders

Efficient design → thick flanges and thin deep webs

-but web cannot be so thin that serviceability and flange buckling are affected



Section Slenderness

Clause 5.2.2

For a section with flat compression plate elements, the section slenderness (λ_s) shall be taken as the value of the plate element slenderness (λ_e) for the element of the cross-section which has the greatest value of λ_e / λ_{ey}

$$\boxed{\lambda_e = \left(\frac{b}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)}}$$

Read carefully Clause 5.2.2

Design of Web

Unstiffed webs

- web shear
- combined shear and bending
- web buckling
- web bearing.

Stiffened webs:

- transverse, intermediate stiffener proportioning
- end stiffeners
- end posts
- axial loads on stiffeners
- longitudinal stiffeners.

Clause 5.10.1, 5.10.4, 5.10.5, 5.10.6

Table 5.9 Minimum web thickness, $k_y = \sqrt{\left(\frac{f_y}{250}\right)}$

Arrangement	Minimum thickness t_w
Unstiffened web bounded by two flanges:	$k_y d_1/180$
Ditto for web attached to one flange (Tee):	$k_y d_1/90$
Transversely stiffened web:	
when $1.0 \leq s/d_1 \leq 3.0$ (See Note 4 also)	$k_y d_1/200$
$0.74 < s/d_1 \leq 1.0$	$k_y s/200$
$s/d_1 \leq 0.74$	$k_y d_1/270$
Web having transverse and one longitudinal stiffener:	
when $1.0 \leq s/d_1 \leq 2.4$	$k_y d_1/250$
$0.74 \leq s/d_1 \leq 1.0$	$k_y s/250$
$s/d_1 < 0.74$	$k_y d_1/340$
Webs having two longitudinal stiffeners and $s/d_1 \leq 1.5$	$k_y d_1/400$
Webs containing plastic hinges	$k_y d_1/82$

Notes:
1. The above limits are from Clauses 5.10.1, 5.10.4, 5.10.5 and 5.10.6 of AS 4100.
2. d_1 is the clear depth between the flanges
3. s is the spacing of transverse stiffeners
4. When $s/d_1 > 3.0$ the web panel is considered unstiffened.

Shear Capacity of Webs

Clause 5.10.2

$$\boxed{V^* \leqslant \phi V_w}$$

ϕ = capacity reduction factor = 0.9

$$V_w = 0.6 f_y A_w \quad \text{for} \quad \frac{k_y d_1}{t_w} \leqslant 82$$

$$k_y = \sqrt{\left(\frac{f_y}{250}\right)}$$

A_w : the effective area of the web: $A_w = (d_1 - d_d)\, t_w$

d_1 : the clear depth of the web,

d_d : height of any holes up to a height of $0.1d_1$ for

unstiffened webs ($0.3d_1$ *if web is stiffened) and*

t_w : is the web thickness.

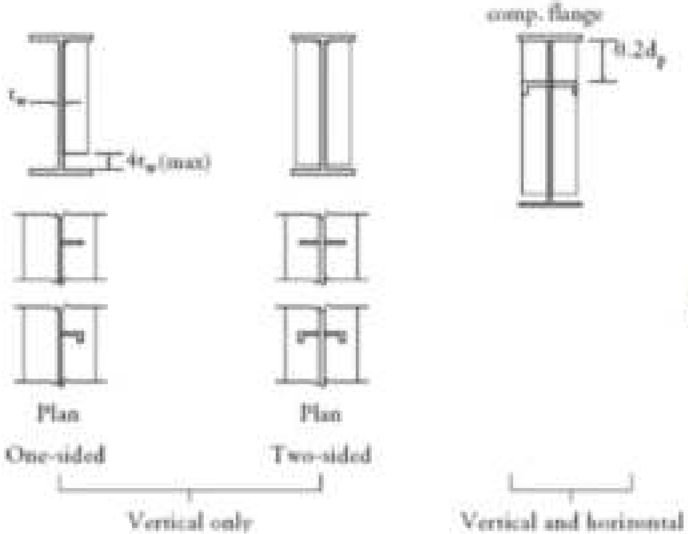
Buckling Capacity of Unstiffed Web

Clause 5.11.5.1

$$V_b = \alpha_v (0.6 f_y A_w)$$

$$\boxed{V^* \leqslant \phi V_b}$$

$$\alpha_v = \left(\frac{82 t_w}{d_1 k_y}\right)^2$$



Method of stiffening

Shear and bending interaction for unstiffened webs

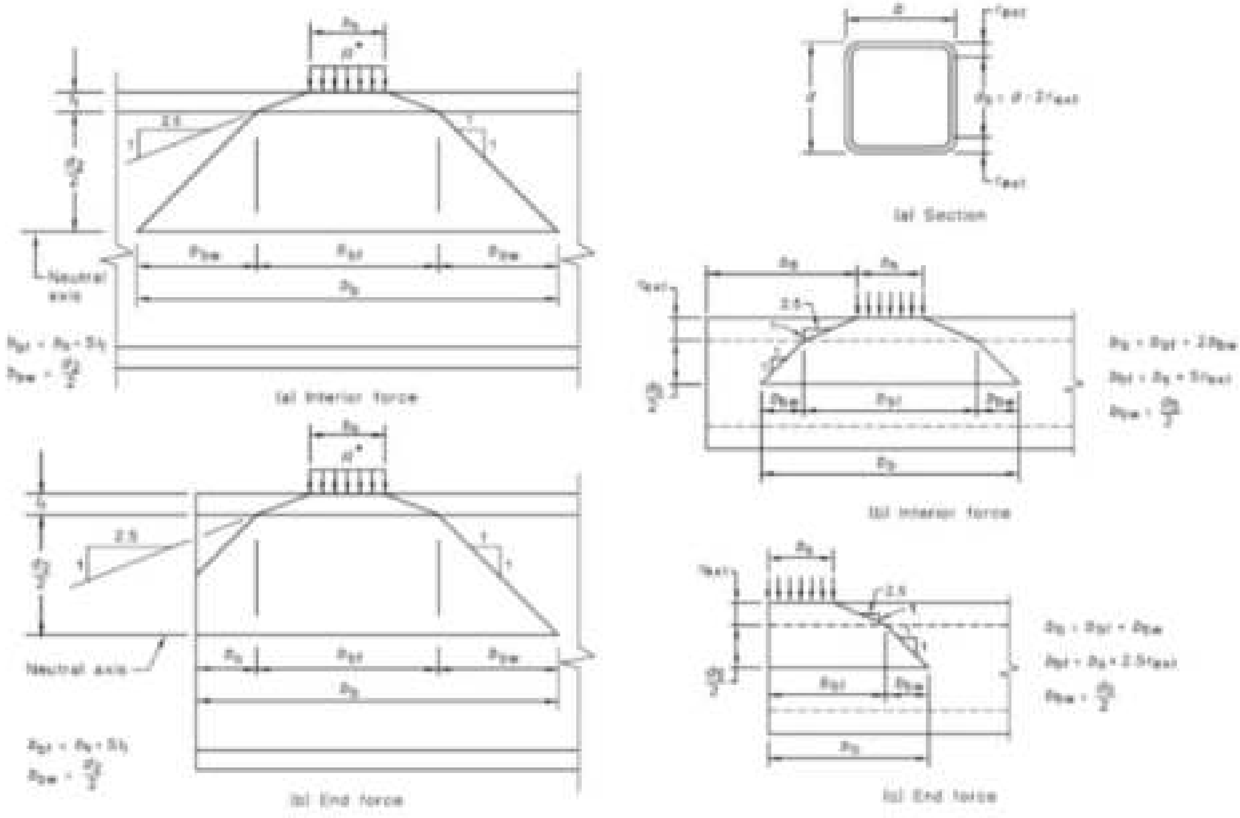
Clause 5.12.2, 5.12.3

$$\alpha_{vm} = \left[2.2 - \frac{1.6M^*}{(\phi M_s)}\right] \qquad \text{for } 0.75\phi M_s < M^* \leq \phi M_s$$

$$\alpha_{vm} = 1.0 \qquad \text{for } M^* \leq 0.75\phi M_s$$

α_{vm}: Reduction factor in shear capacity

Bearing Capacity of Unstiffed Web



Bearing Capacity of Unstiffed Web

5.13.2 Bearing capacity The design bearing force (R^*) on a web shall satisfy—

$$R^* \leq \phi R_b$$

where

ϕ = the capacity factor (see Table 3.4)

R_b = the nominal bearing capacity of the web under concentrated or patch loading, which shall be taken as the lesser of its nominal bearing yield capacity (R_{by}) defined in Clause 5.13.3, and its nominal bearing buckling capacity (R_{bb}) defined in Clause 5.13.4.

Bearing Capacity of Unstiffed Web

- Bearing yield capacity (Clause 5.13.3)

$$R_{by} = 1.25 b_{bf} t_w f_y$$

and

$$R^* \leq \phi R_{by}$$

where t_w is the thickness of the relevant web of an open section (e.g. UB or WB), f_y is the design yield stress and R^* is the design bearing or reaction force.

The dispersion width is determined as follows:

$$b_{bf} = b_s + 5t_f \text{, or}$$

$$b_{bf} = b_s + 2.5t_f + b_d \text{, whichever is the lesser.}$$

where b_s *is the length of the stiff bearing,* t_f *the flange thickness and* b_d *is the remaining* distance to the end of the beam.

Bearing Capacity of Unstiffed Web

- Bearing buckling capacity (Clause 5.13.4)

  The nominal bearing buckling capacity (R_{bb}) *of a web without* transverse stiffeners shall be taken as the axial load capacity determined in accordance with Section 6 (Members subject to Axial Compression) using α_b equals 0.5 and k_f *equals 1.0 for a compression member of area* $t_w b_b$ *and slenderness ratio* l_e/r *equals* $2.5d_1/t_w$ *where* b_b *is the total bearing width obtained by dispersions at a slope of 1:1 from* b_{bf} *to the neutral axis, if available, as shown in Figure 5.13.1.1.*

Bearing Capacity of Unstiffed Web

- Bearing buckling capacity (Clause 5.13.4)

Table 5.10 Web slenderness ratios for unstiffened webs

	I-section beam	Hollow section
Effective width	b_b	b_b
Interior bearing slenderness ratio	$\frac{l_e}{r} = \frac{2.5d_1}{t_w}$	$\frac{l_e}{r} = \frac{3.5d_5}{t_w}$ (for $b_d \geqslant 1.5d_5$)
End bearing slenderness ratio	$\frac{l_e}{r} = \frac{2.5d_1}{t_w}$	$\frac{l_e}{r} = \frac{3.8d_5}{t_w}$ (for $b_d < 1.5d_5$)

Notes: 1. d_1 = clear depth between flanges

2. t_w = thickness of web.

3. b_b = total bearing width dispersion at neutral axis (Figure 5.11(b))—see SSTM [2003b] or Figure 5.13.1.3 of AS 4100 to calculate this for RHS/SHS.

4. d_5 = flat width of RHS/SHS web.

$$\phi R_{bb} = \phi \alpha_c A_w f_y$$

where α_c is determined in Section 6 of AS 4100

Design of Load Bearing Stiffeners

- Bearing yield capacity (Clause 5.14.1)

$R^* \leq \phi R_{sy}$

where

R^* = the design bearing force or design reaction, including the effects of any shear force applied directly to the stiffener

ϕ = the capacity factor (see Table 3.4)

R_{sy} = the nominal yield capacity of the stiffened web

= $R_{by} + A_s f_{ys}$

R_{by} = the nominal bearing yield capacity (see Clause 5.13.3)

A_s = the area of the stiffener in contact with the flange

f_{ys} = the yield stress of the stiffener.

Design of Load Bearing Stiffeners

- Bearing buckling capacity (Clause 5.14.2)

$$R^* \leq \phi R_{sb}$$

R_{sb} = *the nominal buckling capacity of the stiffened web, determined in accordance with* Section 6 using b equals 0.5 and k_f *equals 1.0 for a compression member whose radius of* gyration is taken about the axis parallel to the web.

Design of Load Bearing Stiffeners

- Bearing buckling capacity (Clause 5.14.2)

The effective section of the compression member shall be taken as the area of the stiffener, together with a length of web on each side of the centreline not greater than the lesser of—

$$\frac{17.5t_w}{\sqrt{\left(\frac{f_y}{250}\right)}} \text{ and } \frac{s}{2}, \text{ if available.}$$

The effective length (l_e) of the compression member used in calculating the buckling capacity (R_{sb}) shall be determined as either—

$$l_e = 0.7d_1$$

where the flanges are restrained by other structural elements against rotation in the plane of the stiffener, or—

$$l_e = d_1$$

Torsional End Restraints

Clause 5.14.5 of AS 4100 requires the second moment of area of a pair of stiffeners, I_s, about the centreline of the web satisfies:

$$I_s \geqslant \frac{\alpha_t d^3 t_f R^*}{1000 F^*}$$

where

$$\alpha_t = \frac{230}{\left(\frac{l_e}{r_y}\right)} - 0.60 \quad \text{and } 0 \leqslant \alpha_t \leqslant 4$$

d = depth of section

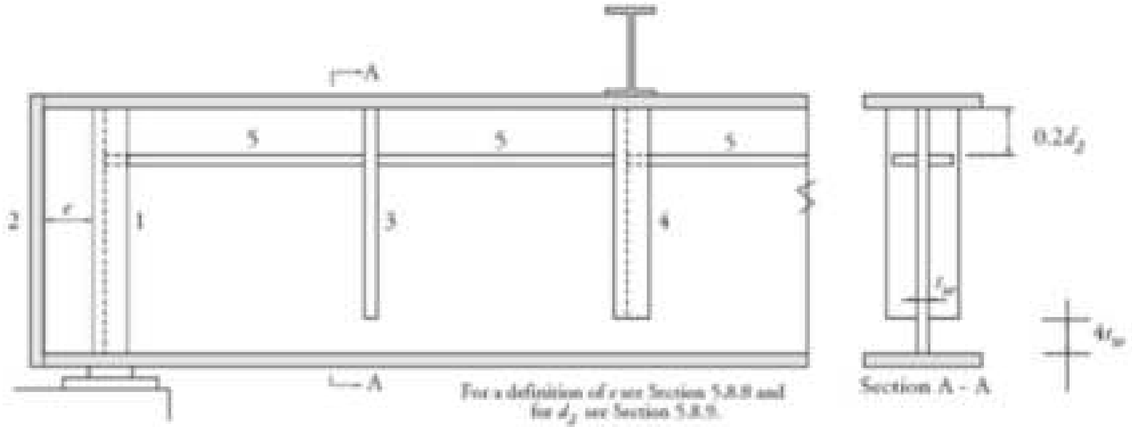
t_f = thickness of critical flange (see Section 5.4.1)

R^* = design reaction at the support/bearing

F^* = total design load on the member between supports

$\left(\frac{l_e}{r_y}\right)$ = load-bearing stiffener slenderness ratio (Clause 5.14.2)

Design of Intermediate Transverse Stiffeners



Web stiffeners: 1 - load bearing stiffener; 2 - end plate; 3 - intermediate transverse stiffener; 4 - load bearing stiffener; 5 - longitudinal stiffener.

Design of Intermediate Transverse Stiffeners

- Minimum area (Clause 5.11.3)

$$A_s \geqslant 0.5\gamma A_w (1 - \alpha_v) c_k \left(\frac{V^*}{(\phi V_u)}\right)$$

where

$$c_k = k_s - \frac{k_s^2}{\sqrt{(1 + k_s^2)}}$$

$$k_s = \frac{s}{d_p} \quad (s = \text{spacing between stiffeners and } d_p = \text{depth of web})$$

γ = 2.4 for a single plate stiffener (one side of the web)
= 1.8 for a single angle stiffener
= 1.0 for a pair of stiffeners (one each side of the web)

$$\alpha_v = \left(\frac{82\, t_w}{d_p k_y}\right)^2 \left(\frac{a}{k_s^2} + b\right) \leqslant 1.0$$

$$k_y = \sqrt{\left(\frac{f_y}{250}\right)}$$

a = 0.75 and b = 1.0 for $1.0 \leqslant k_s \leqslant 3.0$

a = 1.0 and b = 0.75 for $k_s \leqslant 1.0$

Design of Intermediate Transverse Stiffeners

- Buckling capacity (Clause 5.11.4)

$$V^* \leq \phi(R_{sb} + V_b)$$

where

ϕ = capacity reduction factor = 0.9

R_{sb} = nominal buckling capacity of the stiffener

and V_b is the nominal shear buckling capacity:

$$V_b = \alpha_v \alpha_d \alpha_f (0.6 f_y A_w)$$

A_w = gross sectional web area = $d_1 \times t_w$

$\alpha_v = \left(\frac{82 t_w}{d_p k_y}\right)^2 \left(\frac{a}{k_s^2} + b\right) \leq 1.0$

t_w = web thickness

d_p = depth of web or deepest web panel

$k_y = \sqrt{\left(\frac{f_y}{250}\right)}$

$k_s = \frac{s}{d_p}$

s = horizontal spacing between stiffeners

$a = 0.75$ and $b = 1.0$ for $1.0 \leq k_s \leq 3.0$

$a = 1.0$ and $b = 0.75$ for $k_s \leq 1.0$

$\alpha_d = 1 + \frac{1 - \alpha_v}{1.15 \alpha_v \sqrt{(1 + k_s^2)}}$; or

= 1.0 for end web panels with end posts with specific she buckling conditions (see Clause 5.15.2.2 of AS 4100)

$\alpha_f = 1.0$; or $= 1.6 - \frac{0.6}{\sqrt{\left[1 + \left(\frac{40 b_{fo} t_f^2}{d_p^2 t_w}\right)\right]}}$ with specific b_{fo} conditions (see below)

b_{fo} = flange restraint factor, taken as the least of the following:

$= \frac{12 t_f}{k_y}$; or

= distance from the mid-plane of the web to the nearest edge at a flange (or zero if no flange present); or

= half the clear distance between webs for two or more webs

Connection to Web

(Clause 5.15.8)

Welds or other fasteners connecting each intermediate transverse stiffener not subject to external loads are required by Clause 5.15.8 of AS 4100 to transmit a minimum shear force in kN/mm of:

$$v_w^* = \frac{0.0008 t_w^2 f_y}{b_{es}}$$

where b_{es} is the stiffener outstand from the web face and t_w is the web thickness.

End Posts

(Clause 5.15.8)

When an end post is required, it shall be formed by a load bearing stiffener and a parallel end plate. The load bearing stiffener shall be designed in accordance with Clause 5.14, and shall be no smaller than the end plate. The area of the end plate (A_{ep}) *shall* satisfy

$$A_{ep} \geq \frac{d_1[(V^*/\phi) - \alpha_v V_w]}{8 e f_y}$$

α_v : given in Clause 5.11.5.2

V_w : *given in Clause 5.11.4*

e : *distance between the end plate and load bearing stiffener.*

Design of Longitudinal Web Stiffeners
(Clause 5.16)

A longitudinal stiffener at a distance of $0.2d_2$ from the compression flange should have a second moment of area, I_s, about the face of the web not less than:

$$I_s = 4a_w t_w^2 \left\{ \left(1 + \frac{4A_s}{a_w}\right)\left(1 + \frac{A_s}{A_w}\right) \right\}$$

where

$a_w = d_2 t_w$

A_s = area of the stiffener

d_2 = twice the clear distance from the neutral axis to the compression flange

A second horizontal stiffener, if required, should be placed at the neutral axis and should have an I_s not less than:

$$I_s = d_2 t_w^3$$

PLATE GIRDER

Dr. G.C.BEHERA

Design a welded plate girder of span 24 m to carry a super imposed load 35 kN/m. Avoid using of bearing and intermediate stiffeners. Use Fe415 steel.

Solution:

1. Calculation of moment and shear Force:

Span=24 m, Super imposed load= 35 kN/m,

Factored super imposed load=35*1.5=52.5 kN/m

Total factored super imposed load=1.5*35*24=1260 kN

Self weight= 1260/200=6.3 kN/m

Total load=52.5+6.3=58.8 kN/m

Bending moment= $58.8*24^2/8$=4233.6 kNm

SF=58.8*24/2=705.6 kN

2.Depth of web Plate

Avoiding stiffeners, $d/t_w \leq 67$

Economical depth of web $d=[Mk/f_y]^{1/3}$

$D=\{4233.6*10^6*67/250\}1/3$=1043 mm

Taking 1000 mm plate $t_w \geq [1000/67=14.92$ mm] Take t_w=16 mm

Web plate is 1000 mm *16 mm

3. Selection of Flange:

Assuming moment to be taken by flange alone

$[A_f*f_y*d/1.1] \geq M$

$A_f \geq M*1.1/[f_y*d]$

$A_f \geq [4233.6*106*1.1/\{250*1000\}] \geq 18628$ mm2

For making it a plastic section $b/t_f \leq 8.4$

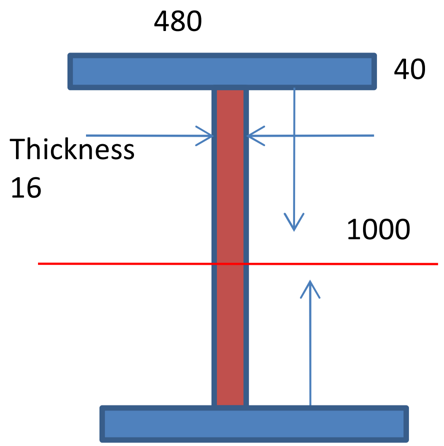
$b_f/(2*t_f) \leq 8.4$

$b_f=16.8t_f$

$A_f=b_f*t_f=16.8\ tf*tf \geq 18628\ mm^2$ which gives $t_f=33.3$ mm

Taking 40 mm thick plate , $b_f=18628/40=465.7$ mm

Use flange plate=480 mm*40 mm.



4. Checking Moment Capacity of Girder

Md=[Zp*fy/1.1] ≤ 1.2*Ze*fy/1.1

Md= =[Zp*fy/1.1] =[480*40*(1000+40/2+40/2)*250/1.1]=4538.182 kN ≤ 1.2*Ze*fy/1.1

$$I_{zz} = 2*[\frac{1}{12}*480*40^3+480*40*\left(\frac{1000+40}{2}\right)^2 = 2*5601.28*10^6 mm^4$$

$$1.2*Z_e*\frac{f_y}{1.1} = 1.2*[I_{zz}/y_{max}]*250/1.1 == 1.2*[2*5601.28*10^6/540]*250/1.1 = 5657.82\ kNm$$

Md=4538.182> [M=4233.6 kNm]

5.Shear Resisting Capacity:

$$V_d = \frac{A_v * f_{yw}}{1.1 * \sqrt{3}} = \frac{d * t_w * f_{yw}}{1.1 * \sqrt{3}} = \frac{1000 * 16 * 250}{1.1 * \sqrt{3}}$$
$$= 2099.45\ kN > 705.6\ kN$$

No stiffener is required.

6. Check for End Bearing:

Bearing strength of web: $F_w=(b_1+n_2)t_w*f_{yw}/1.1$

Assuming width of bearing 200 mm and stiff bearing length 100 mm

n2=2.5*bf=2.5*40=100 mm

F_w=(100+100)16*250/1.1=727 kN>705.6 kN

So, safe.

7. Weld Design to join web to Flange:

Shear force=705.6 kN

$$Shear\ stress\ in\ flange\ at\ the\ level\ of\ junction\ of\ web\ and\ flange$$
$$= q = \frac{F}{bI} a\bar{y}$$
$$= \frac{705.6 * 1000}{480 * 2 * 5601.28 * 10^6} [480 * 40 * \left(500 + \frac{40}{2}\right)$$
$$= 0.512\ N/mm^2$$

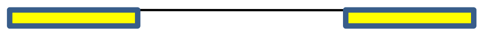
Shear force per mm length in the junction=0.512*480=245.76 N

If s is the size of the weld, providing weld on both sides

$$2 * 0.7 * s * \frac{410}{\sqrt{3} * 1.25} = 265.1\, s = 245.76\ , s = 0.92\ mm$$

As minimum weld thickness is 5 mm for web thickness 16 mm, so provide intermittent welds.

Percentage of weld=0.92*100/5=18.4 %,



Taking 20% weld length

As minimum weld length 40 mm, unweld length will be 40*4=160 mm

Maximum unweld length=12*t=12*16=192> 160 so OK>

Problem: Design the same plate girder thin web and end stiffener without intermediate stiffeners.

Solution:

1. Maximum moment=4233.6 kNmand Maximum shear force=705.6 kN.
2. Selection of depth of web

   For d/tw> 200,intermediate transverse stiffeners are required.

   For d/tw< 67,no intermediate or end transverse stiffeners are required.

Take d/tw=100, so that only end transverse stiffeners are required.

$k=d/t_w=100$ $d=[Mk/f_y]^{1/3}$

$d=[4233.6*10^6*100/250]^{1/3}$=1192 mm, provide d=1200 mm

k=100=d/tw, 1200/100=t_w=12 mm web plate=1200 mm*12 mm plate.

3. Design of Flange

$[A_f*f_y*d/1.1] \geq M$

$A_f \geq M*1.1/[f_y*d]$

$A_f \geq [4233.6*10^6*1.1/\{250*1200\}] \geq 15523mm^2$

For making it a plastic section $b/t_f \leq 8.4$

$b_f/(2*t_f) \leq 8.4$

$b_f=16.8t_f$

$A_f=b_f*t_f$=16.8 tf*tf ≥ 15523 mm^2 which gives t_f=30.4 mm, take tf=36 m

bf=15523/36=431 mm

Use flange plate=440 mm*36 mm

4. Checking Moment Capacity of Girder

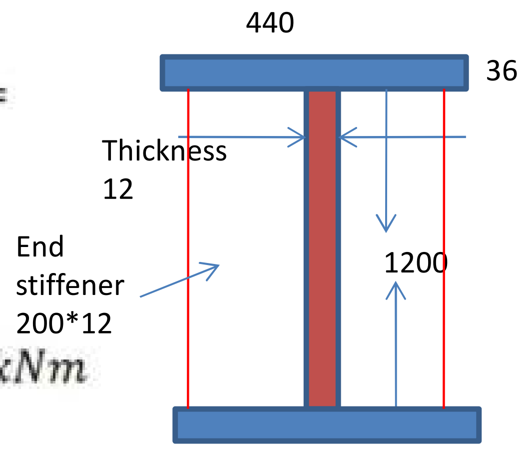
Md=[Zp*fy/1.1] ≤ 1.2*Ze*fy/1.1

M_d= =[Z_p*fy/1.1] =[440*36*(1200+36/2+36/2)*250/1.1]=4449.6 kNm

1.2*Ze*fy/1.1

$$I_{zz} = 2 * [\frac{1}{12} * 440 * 36^3 + 440 * 36 * \left(\frac{1200+36}{2}\right)^2 =$$
$$= 1.21 * 10^{10} mm^4$$

$$1.2 * Z_e * \frac{f_y}{1.1} = 1.2 * [I_{zz}/y_{max}] * 250/1.1 ==$$
$$= 1.2 * [1.21 * 10^{10}/636] * 250/1.1 = 5189.9\ kNm$$



M_d=4449.6> [M=4233.6 kNm]

5. Shear Resistnace of web:

NO Trannsverse stiffener only provided kv=5.35

d/tw>67

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w{}^2} = \frac{250}{\sqrt{3}1.22^2} 96.97\ N/mm^2$$

Check for shear buckling using simple post critical method

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)(\frac{d}{t_w})^2} = \frac{5.35 * \pi^2 2 * 10^5}{12(1-0.3^2)(\frac{1200}{12})^2}$$
$$= 96.7\ N/mm^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\tau_{cr}\sqrt{3}}} = \sqrt{\frac{250}{96.7 * \sqrt{3}}} = 1.22$$

Calculations:

As λ_w=1.22 > 1.2

$$\lambda_w = \sqrt{\frac{f_{yw}}{\tau_{cr}\sqrt{3}}} = \sqrt{\frac{250}{96.7 * \sqrt{3}}} = 1.22$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w{}^2} = \frac{250}{\sqrt{3}1.22^2} 96.97\ N/mm^2$$

$V_{cr}=d*t_w*\tau b$=1200*12*96.97=1396.44 kN

$V_d=V_{cr}/\gamma_{m0}$=1396.44/1.1=1269.49>705.6 kN

6.LOAD CAPACITY OF WEB

As per clause 8.7.4 load capacity of web Fw=(b1+n2)tw*fyw/γ_{m0}

b1=0, n2=2*2.5*36=180 mm

Fw=(0+180)12*250/1.1=545.45 kN<705.6 kN

Hence end stiffeners are required.

7.DESIGN OF END STIFFENERS

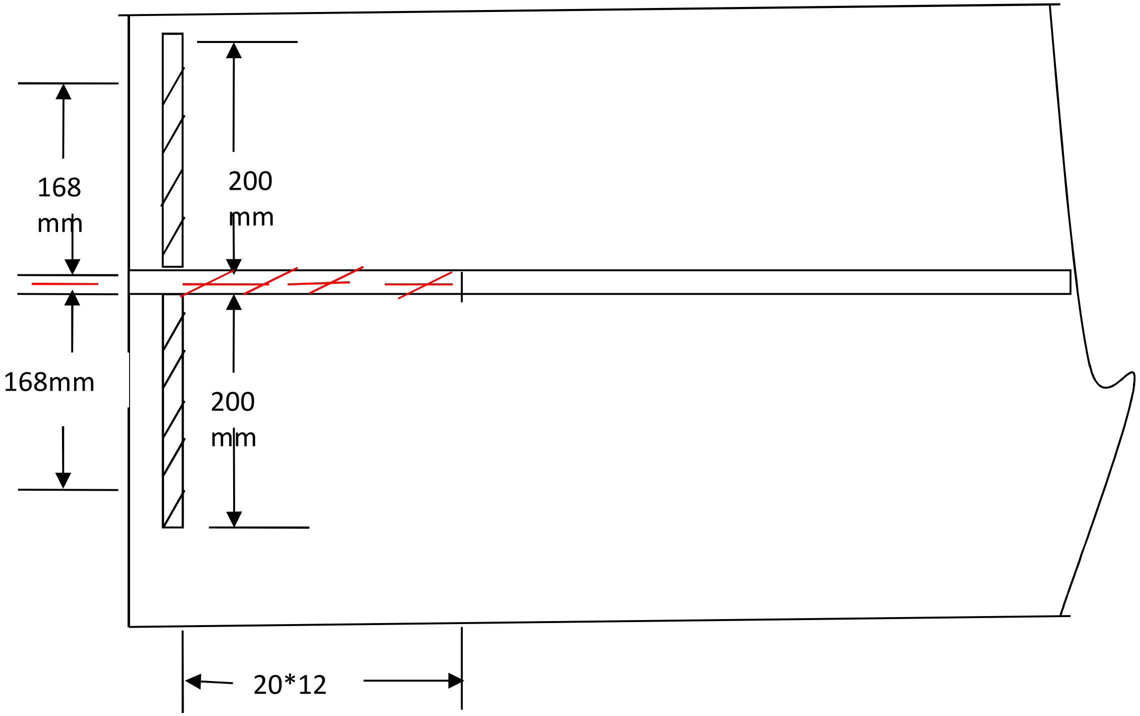
Outstand of flange=[440-12]/2=214 mm

Take Two 200*12 mm plates

Effective end bearing=14*tW=14*12=168 mm

Core area of stiffener on each side=168*12=2016 mm^2

Effective area of web=20*tw=20*12=240 mm^2



Area of buckling resistance=A=2[168*12]+20*12*12=6912 mm^2

I_z=1/12*[12*(168+168+12)^3]+1/12*[{200*12}*(12)^3]=42.17*10^6 mm^4

r=sqrt(I_z/A)=78.12 mm

λ=KL/r=0.7*1200/78.12=10.75

For λ=KL/r=0.7*1200/78.12=10.75, fcd=227+[(224-227)*(10.5-10)/(20-10)=226.78 N/mm^2

buckling resistance=A*fcd=6912*226.78=1567.5 kN>705.6 kN

Hence stiffener is safe.

Check for bearing capacity: Cl.8.7.5.2

Area of stiffener in contact with flange=2*200*12=4800 mm2

$F_{psd}=A_q*f_{qp}/(0.8*\gamma_{mo})$

F_{psd}=4800*250/(0.8*1.1)=1363.63 kN

Check torsional resistance provided by Stiffener:

As per Cl.8.7.4, during transport if torsional resistance is to provided by stiffeners only,

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

For girder Iy= $2*(1/12)*36*440^3+(1/12)*1200*12^3=511.277*10^6$ mm^4

A=2*36*440+1200*12=46080 mm2

ry=sqrt(Iy/A)=sqrt(511.277*106/46080)=105.3 mm

$\lambda=kl/r_y=24*1000/105.3=227.92$

$\alpha_s=30/\ \lambda^2=30/(227.92)^2=5.779*10^{-4}$

$I_s \geq [0.34\alpha_s D^3 T_{cf}=0.34*5.779*10^{-4}*(1272)\ ^3*36=14.548*10^6]$

I_s= second moment of inertia about x-x=1/12*{12*$(200+12+200)^3=69.93*10^6$ mm^4

$I_s=69.93*10^6$ $mm^4 \geq 14.548*10^6$

For torsional resistance it is ok.

Weld design:

Shear force=V=705.6 Kn

Iz=1.21*1010 mm4

Shear stress $\mathit{Shear\ force\ in\ flange\ at\ the\ level\ of\ junction}$

$$\mathit{Shear\ stress\ in\ flange\ at\ the\ level\ of\ junction\ of\ web\ and\ flange}$$
$$= q = \frac{F}{bI} a\bar{y}$$
$$\mathit{Shear\ force\ \ unit\ length} = q = \frac{F}{bI} a\bar{y} * b = \frac{F}{I} a\bar{y}$$
$$= \frac{705.6 * 1000}{1.218 * 10^{10}} [440 * 36 * \left(600 + \frac{36}{2}\right)$$
$$= 570.71 N/mm$$
$$2 * 0.7 * s * \frac{410}{\sqrt{3} * 1.25} = 265.1\, s = 570.71\ , s = 2.153\ mm$$

Use 5mm weld.

If 5 mm weld is to be used, then percentage of weld for intermittent weld=2.153*100/5.0=43.06

Provide 50% weld and leave another 50%

Taking 40 mm minimum weld length, leave another 40 mm which is less maximum un-weld length16*12=192 mm

9. Weld connecting stiffener and web:

$I=1/12*d*t^3$

$A=t_w*d$

$r=sqrt(I/A)=t_w/sqrt(12)$

$\lambda=0.7d/r=2.42d/t_w$ $\quad \lambda=2.42*1200/12=242$

n1=D/2=[1200+2*36]/2=636 mm

Fcd for λ=242, =26.2+[24.3-26.2]*2/10=25.82 N/mm^2

Assuming b1=0

Area of web resisting shear=636*12

Shear taken by web=636*12*25.82=197.08 kN

Shear through weld=705.6-197.08=508.52 kN

Length of weld=1200-2*12=1176 mm

Shear per mm=508.52/1176=0.432kN/mm

Additional shear=tw^2/5*bs==12*12/5*200=0.144 kN/mm

Total shear=0.432+0.144=0.576 kN/mm

$$2 * 0.7 * s * \frac{410}{\sqrt{3} * 1.25} = 265.1\, s = 570.71\ , s = 2.171 mm$$

Minimum weld length will be 5 mm.

If 5 mm weld is to be used, then percentage of weld for intermittent weld=2.171*100/5.0= 43.06

Provide 50% weld and leave another 50%