PLATE GIRDER
PROPORTIONING

BY Dr. Ali Ahmed

Plate Girder

Built-up member (Beam)

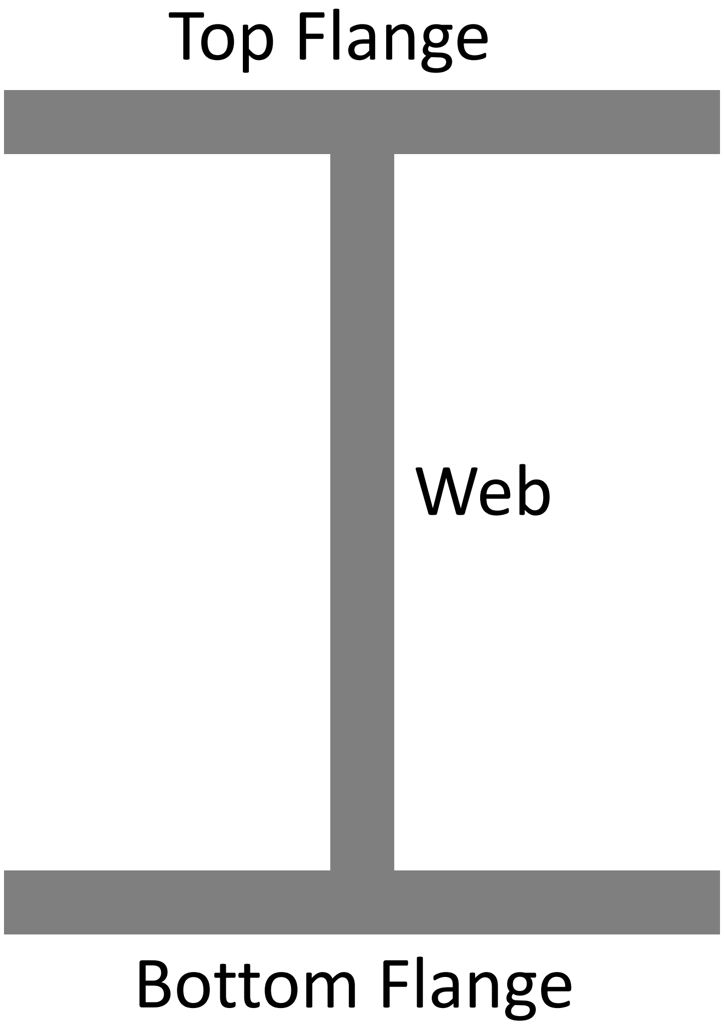


Plate Girder

Efficient cross section [Flexure in flanges & shear in web]

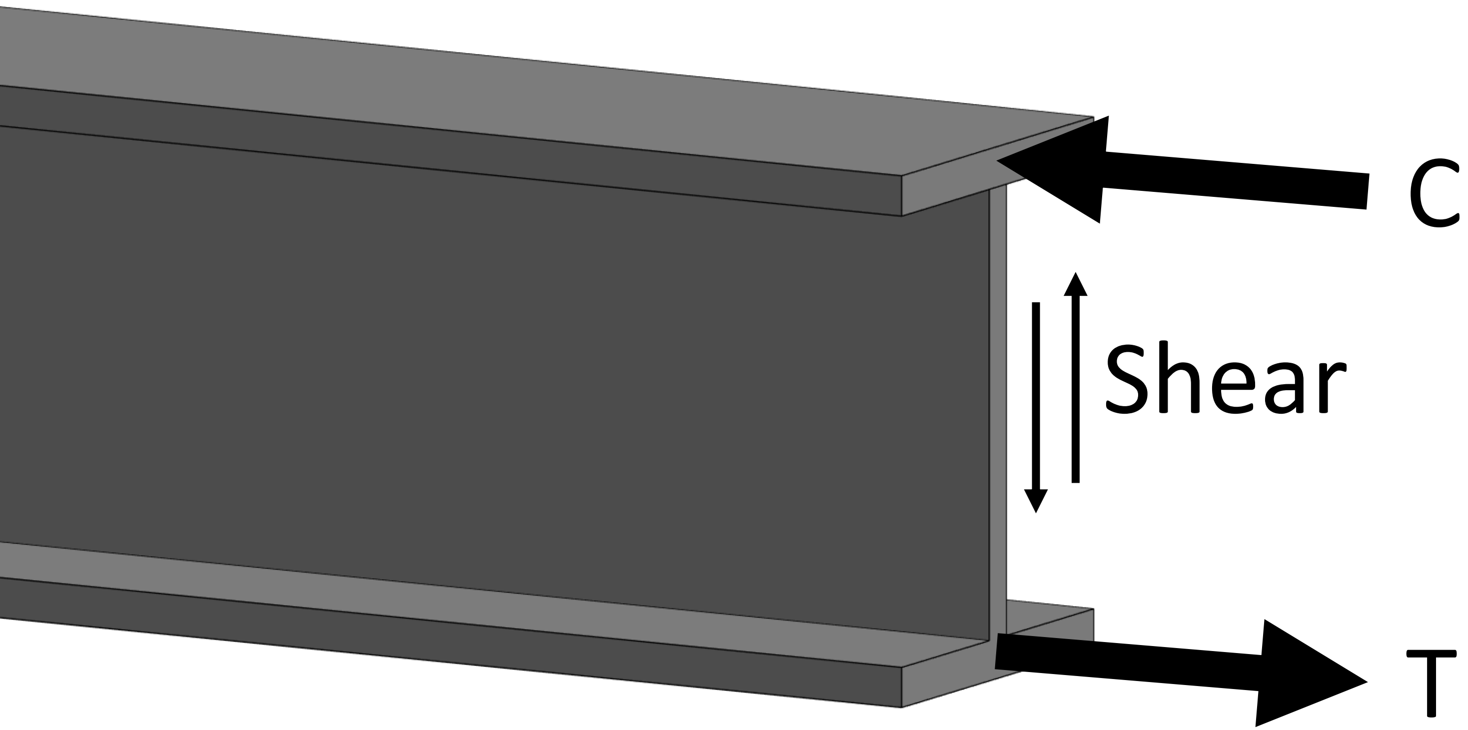


Plate Girder

- Flanges → Compact Web→ Slender
- h/tw > 161.2 [Max limit is 320 for A36 steel]
- Riveted, Bolted or Welded
- High cost as compared to Hot Rolled shapes
- Stiffeners used to provide indirect stability to non-compact web.
- Plate girders are economical where spans are long enough [proportioning for the particular requirements]

PLATE GIRDER BRIDGE ON RIVER RAVI

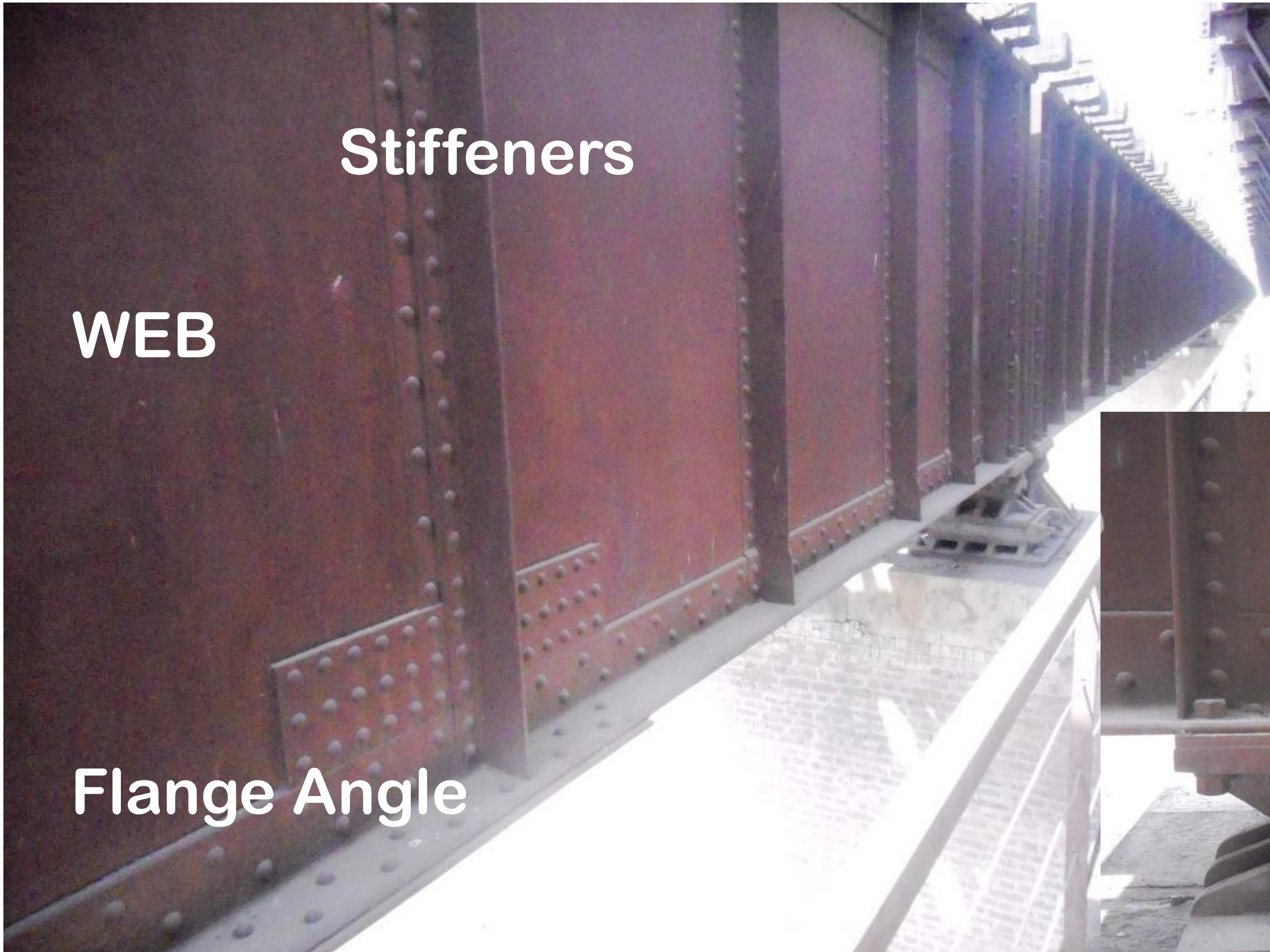


PLATE GIRDER IN BULIDING STRUCTURE

Beacon House DHA Phase VI

Designed by

(Izhar Group of Companies)

Span Lengths

- Plate girders are economical for railroad bridges, which are subjected to heavy loads, for spans in the range of 14 to 40 m.
- For highway bridges, this economical range is 24 to 46m.
- Plate girders are common for 60 m spans and have been used for many spans in excess of 120 m.
- Trusses become more economical for spans larger than the economical span range of the plate girders.
- These are also used in buildings for larger spans and heavy loads, for example, in case of heavy crane runway girders.

Differences from Ordinary Beam

1. Girder is a special type of beam.
2. Spans are bigger.
3. Loads are heavy.
4. Plate girder usually consists of built-up sections.
5. Main difference is that girder is actually a deep beam.

   The web is usually very thin as compared to its depth. Plate girders may be distinguished from beams on the basis of the web slenderness ratio h/t_w.

   Generally ,For plate girders,

$$h/t_w > 5.70\sqrt{E/F_y}$$

h/t_w equal to 161.2 for A36 steel.

Differences from Ordinary Beam

However, according to AISC F13.2,

$h/t_w \leq 0.40\, E / F_y$ for $a/h > 1.5$

(320 for A36 steel)

$$h/t_w \leq 12.0\sqrt{E/F_y}$$ for $a/h \leq 1.5$

(339 for A36 steel)

a is clear distance between stiffeners

6. Girder is a non-compact or slender section. Buckling in webs due to diagonal compression caused by shear may be allowed in such girders. Post-buckling strength is available in the presence of web stiffeners.

Types of Plate Girders

i. Welded and riveted plate girder.

ii. Box girders used for heavy loads that can also resist torsion to a large extent (Figure 6.3).

iii. Hybrid girders are shown in Figure 6.4 in which the flanges are made up of high strength steel (A514 having F_y = 350 MPa) while the web may be made of ordinary steel (A36 having F_y = 250 MPa).

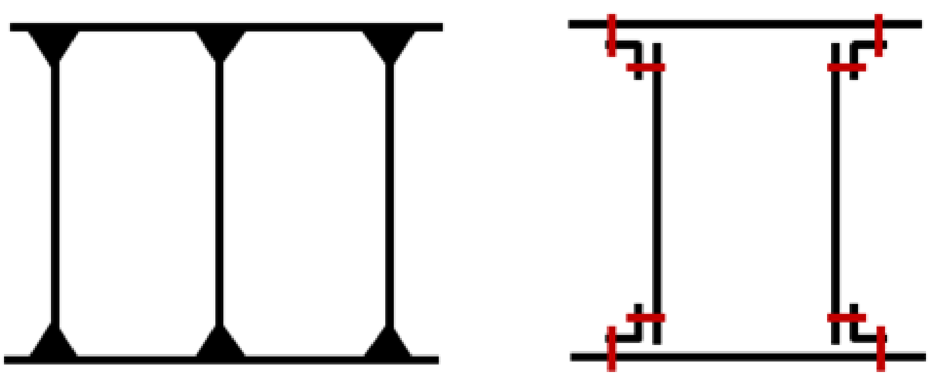


Fig. 6.3. Welded and riveted Box Girder.

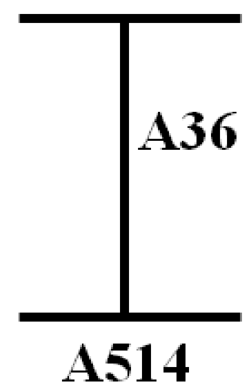


Fig 6.4. Hybrid Girder

iv. Crane bridge girder is a special type of plate girder used to support the crane loads, as shown in Fig.6.5. Crane runway girders provided in a perpendicular direction to the crane bridge at the ends may also be plate girders.

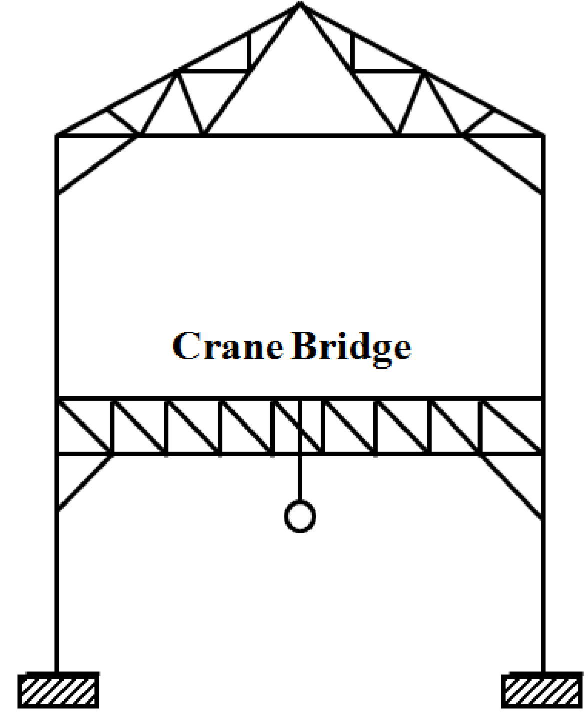
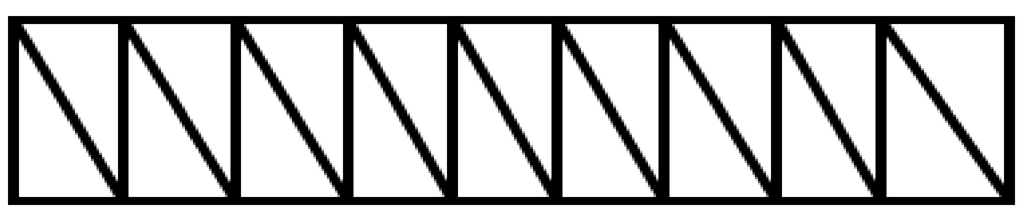


Fig 6.5. Crane Girder in Industrial Building

v. N-Type girder, as in Fig. 6.6, may also be used for special field requirements having stiffeners just like vertical and diagonal members of a truss.

Fig 6.5. A Typical N Type Girder



VAIORUS SHAPES OF PLATE GIRDER BRIDGES


CARPOOL
IS 3
OR MORE
PERSONS
PER
VEHICLE
MOL
MOL

CRANE PLATE GIRDERS

vi. Pre-stressed girder is a plate girder in which pre-stressing tendons are placed to apply moments opposite to the expected moments due to loads (Fig. 6.7). *Load carrying capacity increases and the deflections reduce by using pre-stressing.*

vii. Delta girder, shown in Fig. 6.8, may be used for more stability of the compression flange.

viii. Varying cross section girder having more depth in regions of greater moment may be economical in certain cases.

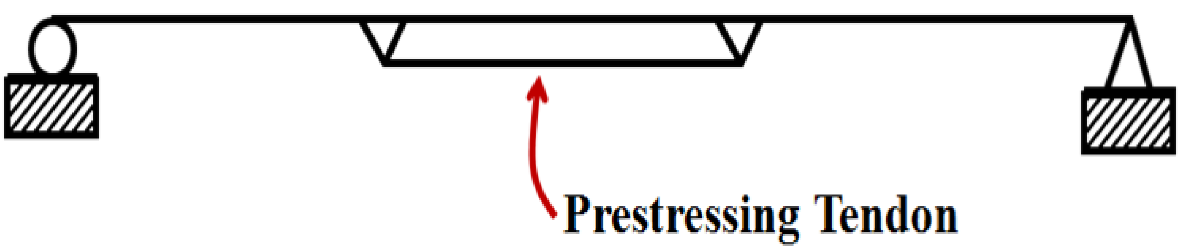


Fig 6.7. Simply Supported Prestress Girder

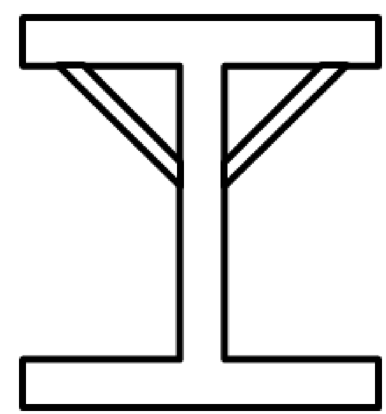


Fig 6.8. A Typical Delta Girder

Plate Girder vs Truss

1. The fabrication cost is lower than for trusses but higher than for rolled beam sections.
2. Weaker axis vs stronger axis stiffness → vibrations and impact are not serious problems for plate girders.
3. Plate girders require smaller vertical clearance than trusses.
4. Detailing for a truss and construction according to these details, especially for connections, is quite involved as compared with plate girders.
5. In case of accident, such as striking of a truck to bridge side fence, the plate girder may only bend without serious consequences.

However, a similar occasion for a truss may break few members making the structure unstable. Collapse of the structure may result under such conditions.

6. Maintenance including painting is easier in case of plate girders than in trusses.

7. Material cost for plate girders is usually higher than the material cost for the corresponding trusses. However, the labour cost may be lesser in plate girders.

PROPORTIONING OF THE SECTION

Proportioning the section means to decide the flange and web sizes and to check that whether, these sizes provide the required strength.

According to AISC, the ratio of web area to the compression flange area (a_w) must not be more than 10.

The important requirements of proportioning may be summarized as under:-

1. Section modulus must be sufficient to resist the applied bending moments at various sections of the girder.
2. The web with or without the help of the stiffeners must provide adequate shear strength.
3. Moment of inertia must be sufficient to keep the deflections within limits.
4. Lateral bending stiffness of the girder must be sufficiently large to prevent lateral-torsional buckling.
5. Stability must be provided to the non-compact web through the use of bearing and intermediate stiffeners.

Depth (*h*)

The height "*h*" (or depth) for welded plate girders is the distance between inner ends of flanges, which varies between *L*/6 to *L*/15 with most common values of *L*/10 to *L*/12.

For no depth restriction, economic design is obtained using the following: $h = 1.1(M_u)^{1/3}, (mm)$

Where, M_u = maximum factored moment in N-mm. The depth must be selected in multiples of 25 mm.

Self Weight

Approximate self-weight in *kN/m* is given by the following expressions:

$$selfweight \approx 0.013(M_u)^{0.67}$$

where M_u is in the kN-m units.

Web Thickness With Intermediate Stiffeners

Plate girder design with slender webs is valid when

$$h/t_w \geq 5.70\sqrt{E/F_y}$$

$h/t_w = 161.2$ for A36 steel.

$\therefore \quad (t_w)_{max} = h/161.2$

The minimum web thickness, $(t_w)_{min}$, should be maximum out of the following t_{w1}, t_{w2} and t_{w3}:

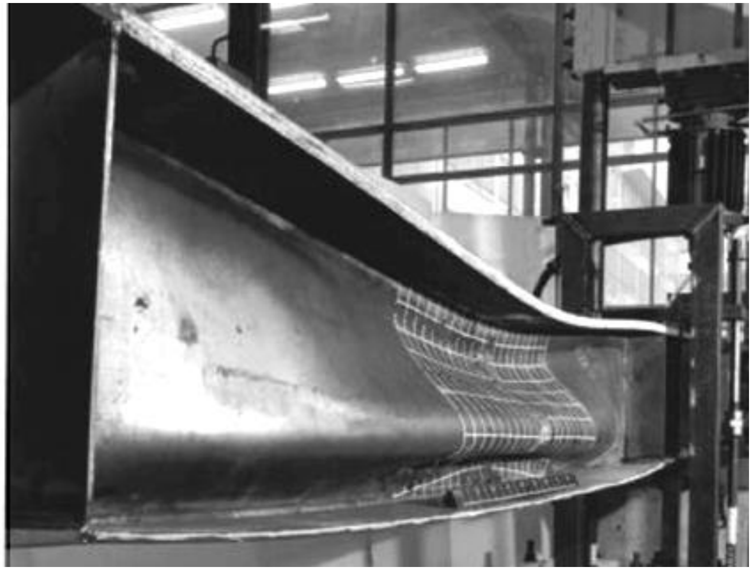
1- For corrosion resistance

t_{w1} = 10 mm is a common minimum for unsheltered girders while 6 or 8 mm are minimum values for more sheltered building girders.

2- *For vertical buckling of the compression flange*

If the web of a plate-girder is too slender, the two sides of the compression flange may buckle together in a vertical plane at a stress less than yield stress.

This behaviour is different from ordinary local flange buckling.



Denoting the clear stiffener spacing by *a*, following limitations of the web sizing may be listed that keeps the flange buckling effect within limits:

For $a/h > 1.5$ $\quad (h/t_w)_{max} = 0.4E/F_y \quad$ (320 for A36 steel)

For $a/h \leq 1.5$ $\quad (h/t_w)_{max} = 12.0\sqrt{E/F_y} \quad$ (339 for A36 steel)

The second value is more critical and may be used in cases where the value of *a/h* is not exactly known.

$$\therefore \qquad t_{w2} = h/320$$

3- To provide the required shear strength, considering 60% of yield shear strength for slender web,

$$t_{w3} = \frac{V_u}{0.6 \times \phi_v \times 0.6 F_y h} = \frac{V_u \times 1000}{0.6 \times 0.9 \times 0.6 \times 250 \;\; h}$$

$$t_{w3} = 12.4 \frac{V_u}{h}$$ for A36 steel

The web thickness close to the minimum is selected, but is rounded to the higher whole number millimetres up to 10 mm and then 12, 15, 18, 20, etc. mm.

Flange Dimensions

Flange dimensions may be decided using the following *Flange Area Formula*:

$$A_f = \frac{M_u / \phi_b}{R_{pg} F_{cr} h} - \frac{A_w}{6}$$

where M_u is in N-mm and R_{pg} is the plate girder flexural strength reduction coefficient, defined as under:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300\,a_w}\left(\frac{h_c}{t_w} - 5.70\sqrt{\frac{E}{F_{cr}}}\right) \le 1.0 \qquad a_w = \frac{h_c\, t_w}{b_{fc}\, t_{fc}} \le 10$$

Where,

h_c = twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used

b_{fc} = width of the compression flange and

t_{fc} = the thickness of the compression flange.

Assuming $R_{pg.}F_{cr} \approx 225$ MPa, the required flange area becomes:

$$A_f \approx 0.0049\frac{M_u}{h} - \frac{A_w}{6}$$ for A36 steel and M_u in N-mm

Flange width

Flange width and thickness are decided from the calculated flange area such that the flange has width/thickness ratio (λ) preferably less than or equal to λ_r.

$$\lambda = b_f / 2t_f$$

can be either kept equal to λ_p for compact flange or even greater than λ_p, however, it should be kept lesser than λ_r.

λ_p = *10.8 for A36 steel*

For doubly symmetric sections:

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{0.7 F_y}}$$ = 19 for A36 steel

$$k_c = \frac{4}{\sqrt{h / t_w}}$$

k_c value is between 0.35 and 0.76 (0.35 for slender webs)

For $\lambda = \lambda_p$:

$$b_f = \sqrt{2\lambda_p (A_f)_{req}}$$

The flange width (b_f) normally varies between *h*/3 to *h*/6 at maximum moment section.

If the above calculated flange width is lesser than *h*/6, then use $b_f = h/6$ may be used.

Greater width of the flange with lesser thickness creates problems associated with FLB and smaller width with larger thickness reduces the lateral strength of the girder, making LTB more critical.

A balance should be made between the two dimensions to achieve nearly same strengths for both FLB and LTB limit states.

The decided b_f should be rounded to the nearest multiples of 50 mm.

Further the ratio area to the compression flange area should not be more than 10.

Flange Thickness

$$t_f = \frac{(A_f)_{req}}{(b_f)_{selected}}$$

Check: $$\lambda = \frac{b_f}{2t_f} \le \lambda_r$$

The increments in flange plate thickness should be as follows:

$t_f \le 10$ mm	2 mm
10 mm $\le t_f \le 40$ mm	12,15,18,20,…,38,40 mm
$t_f > 40$ mm	5 mm

Related Definitions

1. Radius of Gyration (r_t)

The exact value of r_t is :

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_o}{d} + \frac{1}{6} a_w \frac{h^2}{h_o d}\right)}}$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}}$$

h_o = distance between the flange centroids.

r_t = radius of gyration of a section comprising of the compression flange plus 1/3rd of the compression web area, taken about an axis in the plane of the web

$$r_t \approx \sqrt{\frac{t_{fc} b_{fc}^3 / 12}{A_{fc} + A_w / 6}} = \frac{b_{fc}}{\sqrt{12(1 + a_w / 6)}}$$

Where, $A_w = h\, t_w$ and $A_{fc} = b_{fc} \times t_{fc}$

2. Compression Flange Slenderness Ratio For LTB $= L_b / r_t$
3. Bending Coefficient (C_b) defined earlier for beams.

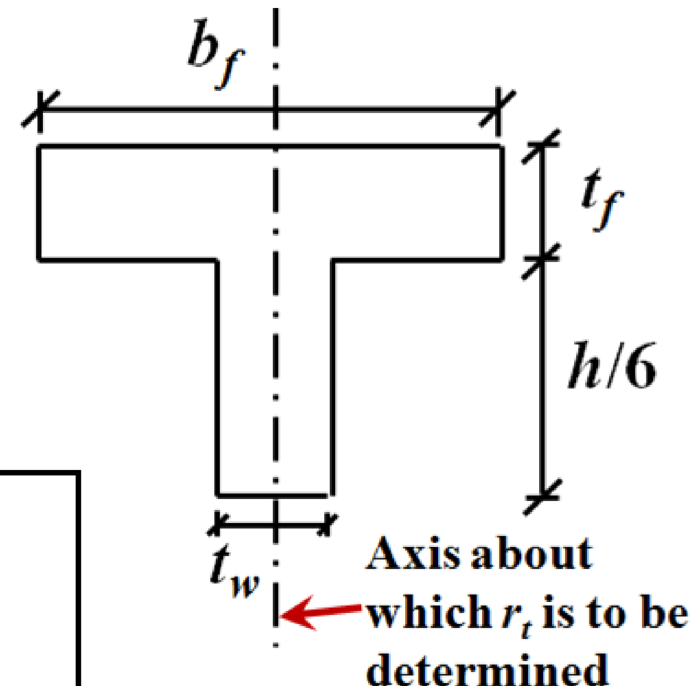


Figure 6.9. Section For Calculation of r_t

4. Elastic Section Modulus

S_{xc} = S_x with respect to the outside fibre of the compression flange.

S_{xt} = S_x with respect to the outside fibre of the tension flange.

6. Plate Girder Flexural Strength Reduction Factor (R_{pg})

This factor takes care of effect of the web instability on the local stability of the flange.

Values Of Important Parameters

For LTB Limit State

L_b = unbraced length for the compression flange

$$L_p = 1.1 r_t \sqrt{E / F_y} = 31.1 \times r_t / 1000 \text{ (m)} \quad \text{for A36 steel}$$

$$L_r = \pi r_t \sqrt{E / 0.7 F_y} = 106.2 \times r_t / 1000 \text{ (m)} \quad \text{for A36 steel}$$

For FLB Limit State:

$$\lambda = \frac{b_f}{2t_f}$$

$$k_c = \frac{4}{\sqrt{h/t_w}}$$

between 0.35 and 0.76
(0.35 for slender webs)

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}}$$

= 10.8 for A36 steel

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}}$$

= 19 for A36 steel

Where, $F_L = 0.7\,F_y$ for major axis bending of slender web built-up section.

WLB Limit State:

This limit state is considered during the stiffener design.

Critical Compression Flange Stress For LTB (F_{cr})

For $L_b \leq L_p$ $\quad F_{cr} = F_y$

For $L_p < L_b \leq L_r$ $\quad F_{cr} = C_b F_y \left[1 - 0.3 \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y$

For $L_b > L_r$ $\quad F_{cr} = \frac{C_b \pi^2 E}{(L_b / r_t)^2} \leq F_y$

Critical Compression Flange Stress For FLB (F_{cr})

For $\lambda \leq \lambda_p$ $\quad F_{cr} = F_y$

For $\lambda_p < \lambda \leq \lambda_r$ $\quad F_{cr} = F_y \left[1 - 0.3 \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right]$

For $\lambda > \lambda_r$ $\quad F_{cr} = \frac{0.9 E k_c}{\lambda^2} = \frac{63000}{\lambda^2}$ for slender web sections

Plate Girder Bending Strength Reduction Factor (Rpg)

This reduction factor takes care of reduction in bending strength due to thin or slender web.

A thin web attached to a flange may reduce the strength provided by it and also that provided by the flange indirectly.

$$R_{pg} = 1 - \frac{a_w}{1200 + 300\,a_w}\left(\frac{h_c}{t_w} - 5.70\sqrt{\frac{E}{F_y}}\right) \leq 1.0$$

$$a_w = \frac{h_c\,t_w}{b_{fc}t_{fc}}$$

Where,

a_w = ratio of web area to compression flange area.

h_c = twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used.

Nominal Flexural Strength (M_n)

If, $h/t_w \leq 5.70\sqrt{E/F_y}$ find M_n as for a regular beam with compact web.

If above condition is not satisfied, the nominal flexural strength (M_n) is the lower value obtained from limit states of tension flange yielding and compression flange buckling, as follows:

For yielding of the tension flange,

$$M_n = S_{xt} F_y / 10^6$$

For buckling of the compression flange,

$$M_n = S_{xc} R_{pg} F_{cr} / 10^6$$

where F_{cr} is taken as the smaller value for LTB and FLB limit states.

$S_{xt} = S_{xc} = S_x$ for doubly symmetrical I-sections

Cutting-Off Cover Plates In Riveted Girders Or Reducing Flange Area In Welded Girders

- In case of riveted plate girders, 2 or 3 flange plates (called *cover plates*) along with a pair of angles are selected for the flanges.
- Two of the cover plates may be cut off at locations where the bending moment is sufficiently low. Each of the plate separately must satisfy $b_f / 2t_f$ ratio limits.
- In case of welded plate girder, the flange plate can be reduced in size where moment is lesser. The reduced plate must separately satisfy the $b_f / 2t_f$ ratio limits.
- Total area of cover plates of bolted girder should not exceed the 70% of total area of flange.

- The extension of cover plates beyond the theoretical cut-off point must be attached to girder by bolts weld or slip critical connection
- In case of welded plate girder, the flange plate can be reduced in size where moment is lesser. The reduced plate must separately satisfy the $b_f / 2t_f$ ratio limits.

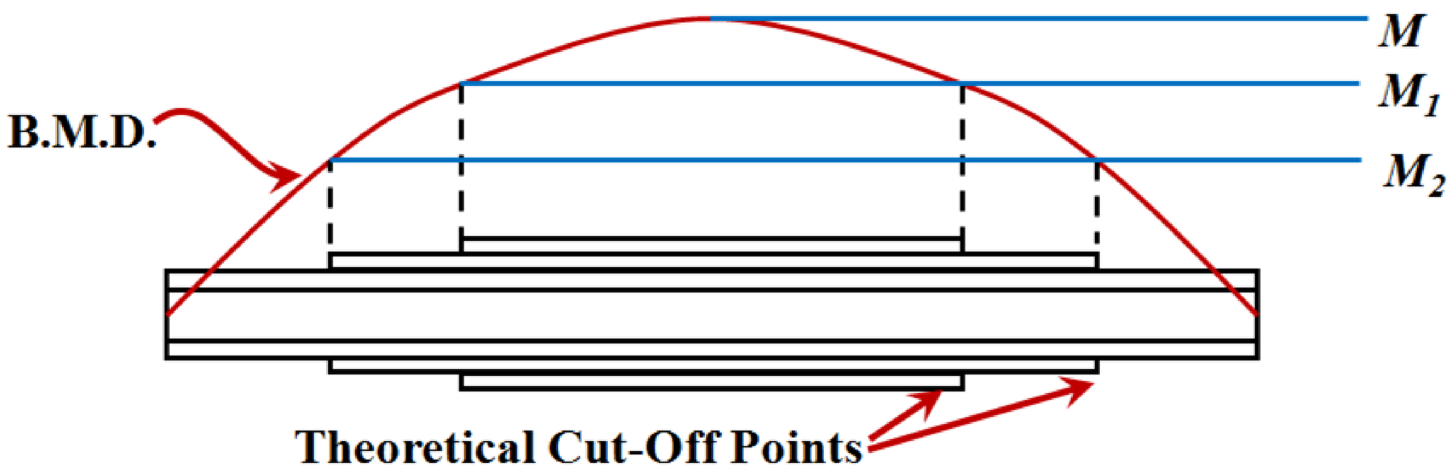


Figure 6.10. Graphical Location of Theoretical cut-off locations

M	=	full moment capacity
M_1	=	moment capacity after 1st reduction in flange area
M_2	=	moment capacity after 2nd reduction in flange area

Example 6.1:

Proportion a plate girder with a span of 20m to support a factored UDL (w_u) of 70 kN/m in addition to self-load and two factored concentrated loads (P_u) of 750 kN each located 7m from the roller and hinge supports at both ends. The compression flange is laterally supported at ends and at the points of concentrated loads. The girder is unsheltered.

Solution:

Self Weight, S.F. and B.M. Diagrams

$$(M_u)_{\max} = 750 \times 7 + \frac{70 \times 20^2}{8} = 8750, kN - m$$

Self weight $\approx$ 0.013 $(M_u)^{0.67}$ = 5.7 *kN/m*

Factored self weight = 1.2 $\times$ 5.7 = 6.9 *kN/m*

Total factored UDL = 70 + 6.9 = 76.9 *kN/m*

$(M_u)_{max}$ = 9095 *kN-m* : $(V_u)_{max}$ = 1519 *kN*

a) Depth Of Web:

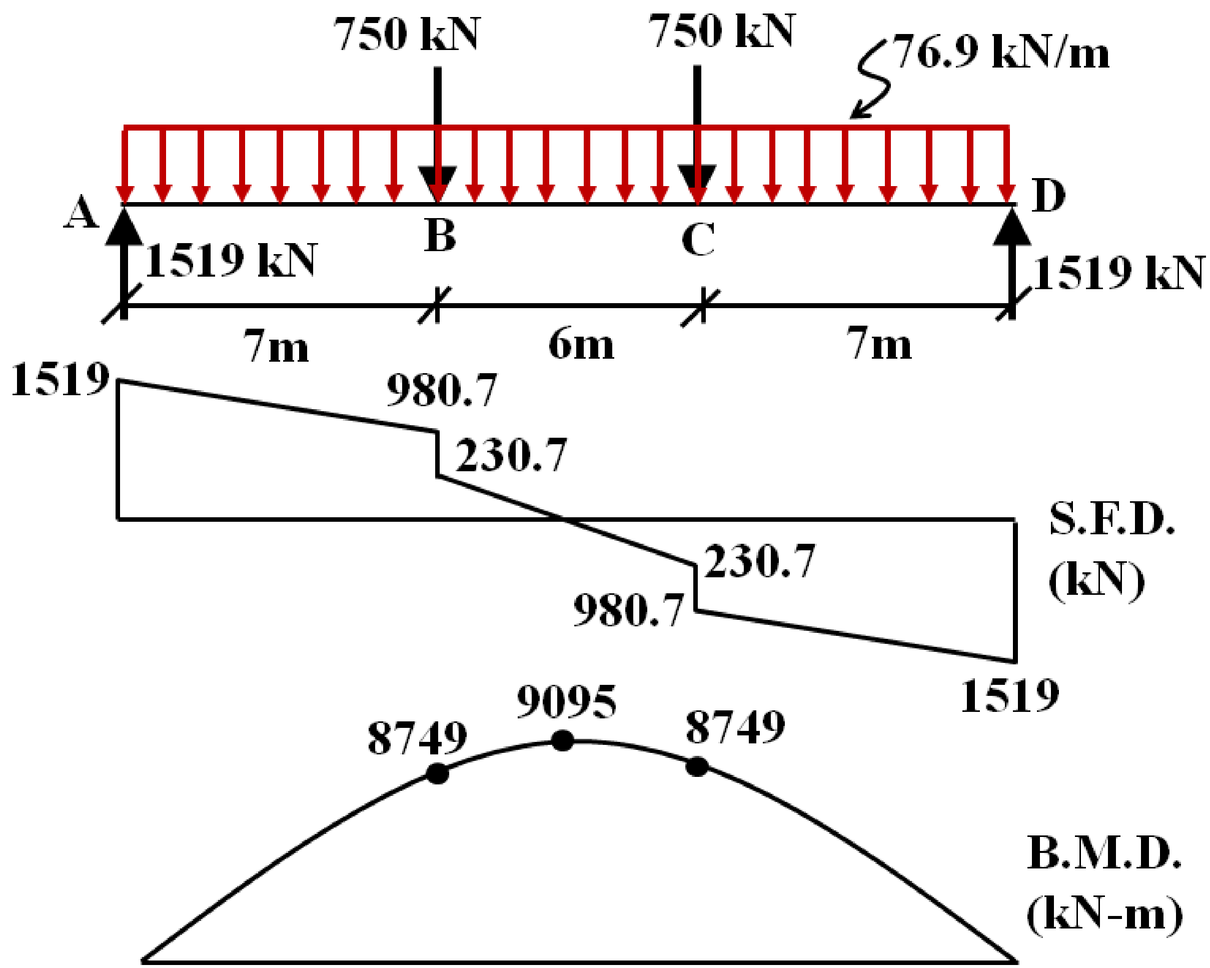
$$h = 1.1(M_u)^{\frac{1}{3}}$$

$$h = 1.1(9095 \times 10^6)^{\frac{1}{3}}$$

h= 2296 mm

Say h= 2300

(multiples of 25 mm)



b) *Thickness of Web with Intermediate Stiffeners:*

1. $(t_w)_{min}$ = 10 mm for corrosion control of unsheltered girders.

2. $(t_w)_{min} = h/320 = 2300 / 320 = 7.19$ mm

3. $(t_w)_{min} = \dfrac{V_u \times 1000}{0.6 \times \phi_v \times 0.6 \times F_y . h} = \dfrac{V_u \times 1000}{0.6 \times 0.9 \times 0.6 \times 250 \times h}$

$$(t_w)_{\min} = 12.4 \frac{V_u}{h} = 12.4 \times \frac{1519}{2300} = 8.2mm$$

t_w = maximum out of the above values rounded according to the available sizes.

$\therefore \quad t_w = 10$ mm

$$(t_w)_{\max} = \frac{h}{161.2} = \frac{2300}{161.5} = 14.27mm > t_w \qquad \textit{(OK)}$$

Trial Size Of Web = 2300×10 mm

c) Flange Dimensions:

$$A_f \approx 0.0049\frac{M_u}{h} - \frac{A_w}{6} = 0.0049\frac{9095\times10^6}{2300} - \frac{2300\times10}{6}$$

A_f = 15,543 mm^2

Check If

$$L_b > \left(387 - \frac{345}{C_b}\right)\sqrt{(A_f)_{req}}$$

$$L_b = 6000\,mm > \left(387 - \frac{345}{1}\right)\sqrt{15543} = 5236\,mm$$

(NOT OK)

L_b = 6m is for the central Portion where Moment is maximum

$$A_f = \frac{A_{f.req}}{C_b(1.11 - .0026L_b/\sqrt{A_{f.req}}} = \frac{15{,}543}{1(1.11 - .0026\times6000/\sqrt{15{,}543}}$$

A_f = 15,943 mm^2

$$b_f = \sqrt{2\lambda_p (A_f)_{req}} = \sqrt{2 \times 10.7 \times 15543} = 577$$

say b_f = 600 mm (multiples of 50 mm)

Check If

$$b_f = 600 mm > \frac{h}{6} = \frac{2300}{6} = 384 mm$$ *(OK)*

$$t_f = \frac{A_f}{b_f} = \frac{15543}{600} = 25.9 mm$$ *say 28 mm*

$$\lambda = \frac{b_f}{2t_f} = \frac{600}{2 \times 28} = 10.71 \leq \lambda_r = 19$$ ***(OK)***

Two Trial Flanges: 600×28 mm Plates

Calculation of r_t

$$r_t = \sqrt{\frac{t_f . b_f^3 / 12}{A_f + A_w / 6}} = \sqrt{\frac{28 \times (600)^3 / 12}{28 \times 600 + (2300 \times 10) / 6}} = 156 mm$$

Moment of Inertia and Section Modulus

$$I_x = \frac{A_f}{2}\left(h + t_f\right)^2 + \frac{t_w \times h^3}{12}$$

$$I_x = \frac{600 \times 28}{2}\left(2300 + 28\right)^2 + \frac{10 \times 2300^3}{12} = 5{,}566{,}367 \times 10^4\, mm^4$$

$$S_{xt} = S_{xc} = \frac{2 \times I_x}{h + 2t_f} = \frac{2(5{,}566{,}367 \times 10^4)}{2300 + 2 \times 28} = 47{,}253 \times 10^3\, mm^3$$

Critical Compression Flange Stress (Fcr)

LTB – Portion AB and CD

M_x = $1519\, x - 76.9\, x^2 / 2$

M_{max}	=	8749.0	M_A	=	2540.5
M_B	=	4845.5	M_C	=	6915.0

$$C_b = \frac{12.5\, M_{\max}}{2.5\, M_{\max} + 3\, M_A + 4\, M_B + 3\, M_C}$$

$$C_b = \frac{12.5 \times 8749.0}{2.5(8749.0) + 3(2540.5) + 4(4845.5) + 3(6915.0)} = 1.57$$

L_b = 7.00 m

L_p = 0.0311 r_t = 4.85 m,

L_r = 0.1062 r_t = 16.57 m

$L_p = 4.85 < L_b = 7 \leq L_r = 16.57 \Rightarrow$ Inelastic Buckling

$$F_{cr} = C_b F_y \left[1 - 0.3 \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y$$

$$F_{cr} = 1.57 \times 250 \left[1 - 0.3 \left(\frac{7.00 - 4.85}{16.57 - 4.85} \right) \right] = 370.9 > F_y = 250$$

$F_{cr} = F_y = 250$ MPa

Note: Full flange may be extended into these segment

LTB – Portion BC

$M_x = 8749 + 230.7x - 79.6x^2/2$

M_{max}	=	9095.0	M_A	=	9008.5
M_B	=	9095.0	M_C	=	9008.0

$$C_b = \frac{12.5 \times 9095}{2.5 \times 9095 + 3 \times 9008.5 + 4 \times 9095 + 3 \times 9008.5} = 1.0$$

L_b = 6.00 m , L_p = 4.85 m , L_r = 16.57 m

$L_p = 4.85 < L_b = 6.0 \leq L_r = 16.57 \Rightarrow$ **Inelastic Buckling**

$$F_{cr} = C_b F_y \left[1 - 0.3\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] = 1 \times 250\left[1 - 0.3\left(\frac{6 - 4.85}{16.57 - 4.85}\right)\right] = 242.6$$

$F_{cr} = 242.6 \quad \leq \quad F_y = 250$ MPa

Flange Local Buckling (FLB)

$\lambda = 10.71$, $\lambda_p = 10.8$,

$\lambda_r = 19$

$\lambda < \lambda_p$ $\therefore F_{cr} = 250\ MPa$

Plate Girder Bending Strength Reduction Factor (R_{pg}):

$$a_w = \frac{h\,t_w}{(b_f t_f)_{comp.\,flange}} = \frac{2300 \times 10}{600 \times 28} = 1.37$$

$a_w < 10$ (OK)

$$R_{pg} = 1 - \frac{a_w}{1200 + 300\,a_w}\left(\frac{h}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0$$

$$R_{pg} = 1 - \frac{1.37}{1200 + 300 \times 1.37}\left(\frac{2300}{10} - 5.7\sqrt{\frac{2 \times 10^5}{250}}\right) = 0.942$$

Check For Strength:

$$\phi_b M_n = \phi_b S_{xc} R_{pg} F_{cr} / 10^6$$

$$= 0.9 \times 47,253 \times 10^3 \times 0.942 \times 242.6 / 10^6$$

$$= 9718.8 \text{ kN-m} > (M_u)_{max} = 9095 \text{ kN-m} \qquad \text{(OK)}$$

<u>Note:</u> *Up to 15% difference on safe side is OK, otherwise, revise by reducing flange size. Less than 1% difference may be allowed on unsafe side, otherwise, revise by selecting larger flanges.*

Theoretical Location where Half Flange Area May be Curtailed

<u>Note:</u> *Reduced flange area is expected only in end panels for this simply supported girder.*

<u>Half Flange Size</u>

Reduced A_f = (600 x 28) / 2 = 8,400 mm^2

Let $b_f \approx h/5 = 500$ mm (larger than $h/6$)

$$t_f = \frac{8,400}{500} \approx 16.8$$ *say 18 mm*

$$\lambda = \frac{500}{2 \times 18} = 13.9$$ $\lambda < \lambda_r = 19$ ***(OK)***

Trial Reduced Flange Size = 500×18 mm

Calculation of r_t (Reduced Flange)

$$r_t = \sqrt{\frac{t_f . b_f^{\,3}/12}{A_f + A_w/6}} = \sqrt{\frac{18 \times (500)^3/12}{18 \times 500 + (2300 \times 10)/6}} = 121 mm$$

Calculation Of F_{cr} (Reduced Flange)

LTB

Note: Only the exterior unbraced length has the reduced flange for some part of its length.

L_b = 6.00 m , $C_b = 1.57$

L_p = $0.0311 r_t$ = 0.0311x121= 3.76 m

L_r = $0.1062 r_t$ = 0.1062x121 =12.85 m

$L_p = 3.76 < L_b = 7.0 \le L_r = 12.85 \Rightarrow$ **Inelastic Buckling**

$$F_{cr} = C_b F_y \left[1 - 0.3 \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] = 1.57 \times 250 \left[1 - 0.3 \left(\frac{7 - 3.76}{12.85 - 3.76} \right) \right] = 350.53$$

$F_{cr} = 350.53 \le F_y = 250\ Mpa$

Thus, $F_{cr} = F_y = 250\ MPa$

Flange Local Buckling (FLB)

$\lambda = 13.9$, $\lambda_p = 10.8$,

$\lambda_r = 19$

$\lambda_p < \lambda < \lambda_r \Rightarrow$ **Inelastic Buckling**

$$F_{cr} = F_y\left[1-0.3\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right)\right] = 250\left[1-0.3\left(\frac{13.9-10.7}{19-10.7}\right)\right] = 221.08$$

$F_{cr} = 221.08 \quad \leq \quad F_y = 250$ MPa

Determination of R_{pg} (Reduced Flange):

$$a_w = \frac{h\,t_w}{(b_f t_f)_{comp.\ flange}} = \frac{2300 \times 10}{500 \times 18} = 2.56$$

$a_w < 10$	*(OK)*

$$R_{pg} = 1-\frac{a_w}{1200+300\,a_w}\left(\frac{h}{t_w}-5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0$$

$$R_{pg} = 1-\frac{2.56}{1200+300 \times 2.56}\left(\frac{2300}{10}-5.7\sqrt{\frac{2\times 10^5}{250}}\right) = 0.91$$

Moment of Inertia and Section Modulus

$$I_x = \frac{A_f}{2}\left(h + t_f\right)^2 + \frac{t_w \times h^3}{12}$$

$$I_x = \frac{500 \times 18}{2}\left(2300 + 18\right)^2 + \frac{10 \times 2300^3}{12}$$

$$I_x = 3{,}431{,}823 \times 10^4\, mm^4$$

$$S_{xt} = S_{xc} = \frac{2 \times I_x}{h + 2t_f} = \frac{2(3{,}431{,}823 \times 10^4)}{2300 + 2 \times 18} = 29{,}382 \times 10^3\, mm^3$$

Check For Strength:

$$\phi_b M_n = \phi_b S_{xc} R_{pg} F_{cr} / 10^6$$

$$= 0.9 \times 23{,}382 \times 10^3 \times 0.91 \times 221.08 / 10^6$$

$$= 5320 \text{ kN-m}$$

Distance (x) Of Theoretical Cut-Off Point From End

Assuming that cut-off point lies within the end panel,

$$5320 = 1519x - 76.9x^2/2$$

$$38.45\,x^2 - 1519\,x + 5320 = 0$$

$$x = \frac{1519 \pm 1220.3}{2 \times 38.45} = 3.9m$$

<u>At $x = 3.9$m</u> (M_u = 5320 kN-m, V_u = 1219.1 kN)

Flange-To-Web Weld

At the End (Calculate shear Stress at web-flange Junction)

$$Q = A_f\left(\frac{h+t_f}{2}\right) = 500 \times 18\left(\frac{2300+18}{2}\right) = 14{,}431 \times 10^3, mm^3$$

$$I_x = 3{,}431{,}823 \times 10^4 \text{ mm}^4$$

$$V_u = 1519 \text{ kN}$$

$$Shear - Flow = \frac{V_u Q}{I_x} = \frac{1519 \times 10{,}431 \times 10^3}{3{,}431{,}823 \times 10^4} = 0.462$$

Shear Flow = 0.462 kN/mm for welds on both faces

Shear Flow = 0.231 kN/mm for weld on one side

Weld Size

t_{p1} = thickness of web = 10 mm

t_{p2} = 18 or 28 mm , $(t_w)_{min}$ = 5 mm

$(t_w)_{max} = t_{p1} - 2$ = 8 mm , $(t_w)_{opt}$ = 8 mm

$\therefore t_w = 8$ mm

Weld Value Using E 70 Electrode

R_w= smaller of

1) $0.75 \times 8 \times 0.707 \times 1 \times 0.6 \times 495/1000 = 1.26\ kN/mm$
2) $0.75 \times 10 \times 1 \times 0.6 \times 400/1000 = 1.8\ kN/mm$

$\therefore R_w$ = 1.26 kN/mm

Note:

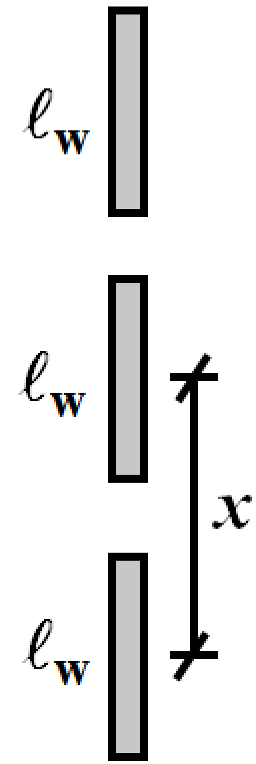
R_w is significantly greater than the calculated shear flow. This means that intermittent weld is to be used in place of continuous weld.

$(l_w)_{min} = 4t_w = 32$ mm

Let, $l_w = 50$ mm

x = *c/c* spacing of these welds

Average weld strength per unit length $= \dfrac{\ell_w \times R_w}{x}$ = required strength per unit length



$$x = \frac{l_w \times R_w}{q_v} = \frac{50 \times 1.26}{0.231} = 272 mm$$

Use 8×50 welds 250 mm c/c

Fig. 6.12. Intermittent Flange-To-Web Weld.