Structural Steel Design Project	Job No:	Sheet ***1 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

PROBLEM:

The girder showed in Fig. E1 is fully restrained against lateral buckling throughout its span. The span is 36 m and carries two concentrated loads as shown in Fig. E1. Design a plate girder.

Yield stress of steel, f_y = $250 \ N/mm^2$
Material factor for steel, γ_m = 1.15
Dead Load factor, γ_{fd} = 1.35
Imposed load factor, $\gamma_{f\ell}$ = 1.50

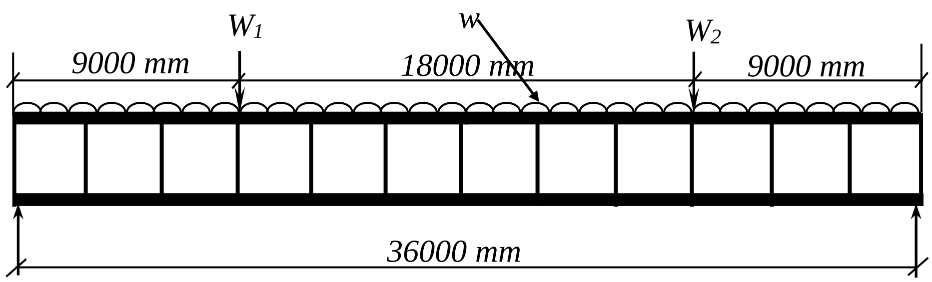

Fig. E1 Example plate girder

1.0 LOADING

<u>Dead load:</u>

Uniformly distributed load, w_d = 20 kN/ m (Including self-weight)
Concentrated load, W_{1d} = 200 kN
Concentrated load, W_{2d} = 200 kN

<u>Live load:</u>

Uniformly distributed load, w_ℓ = 35 kN/m
Concentrated load, $W_{1\ell}$ = 400 kN
Concentrated load, $W_{2\ell}$ = 400 kN

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet *2 **of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Factored Loads:

$w' = w_d * \gamma_{fd} + w_\ell * \gamma_{f\ell} = 20 * 1.35 + 35 * 1.5 = 79.5$ kN/m

$W'_1 = W_{1d} * \gamma_{fd} + W_{1\ell} * \gamma_{f\ell} = 200 * 1.35 + 400 * 1.5 = 870$ kN

$W'_2 = W_{2d} * \gamma_{fd} + W_{2\ell} * \gamma_{f\ell} = 200 * 1.35 + 400 * 1.5 = 870$ kN

2.0 BENDING MOMENT AND SHEAR FORCE

	Bending moment (kN-m)	***Shear force (kN)***
UDL effect	$\frac{w^1\ell^2}{8} = \frac{79.5*36*36}{8} = 12879$	$\frac{w^1\ell}{2} = 1431$
Concentrated load effect	$\frac{W\ell}{4} = 870*9 = 7830$	*W = 870*
TOTAL	*20709*	*2301*

The design shear forces and bending moments are shown in Fig. E2.

3.0 INITIAL SIZING OF PLATE GIRDER

Depth of the plate girder:

The recommended span/depth ratio for simply supported girder varies between 12 for short span and 20 for long span girder. Let us consider depth of the girder as 2400 mm.

$$\frac{\ell}{d} = \frac{36000}{2400} = 15.0$$

Depth of 2400 mm is acceptable.

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***3 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

(For drawing the bending moment and shear force diagrams, factored loads are considered)

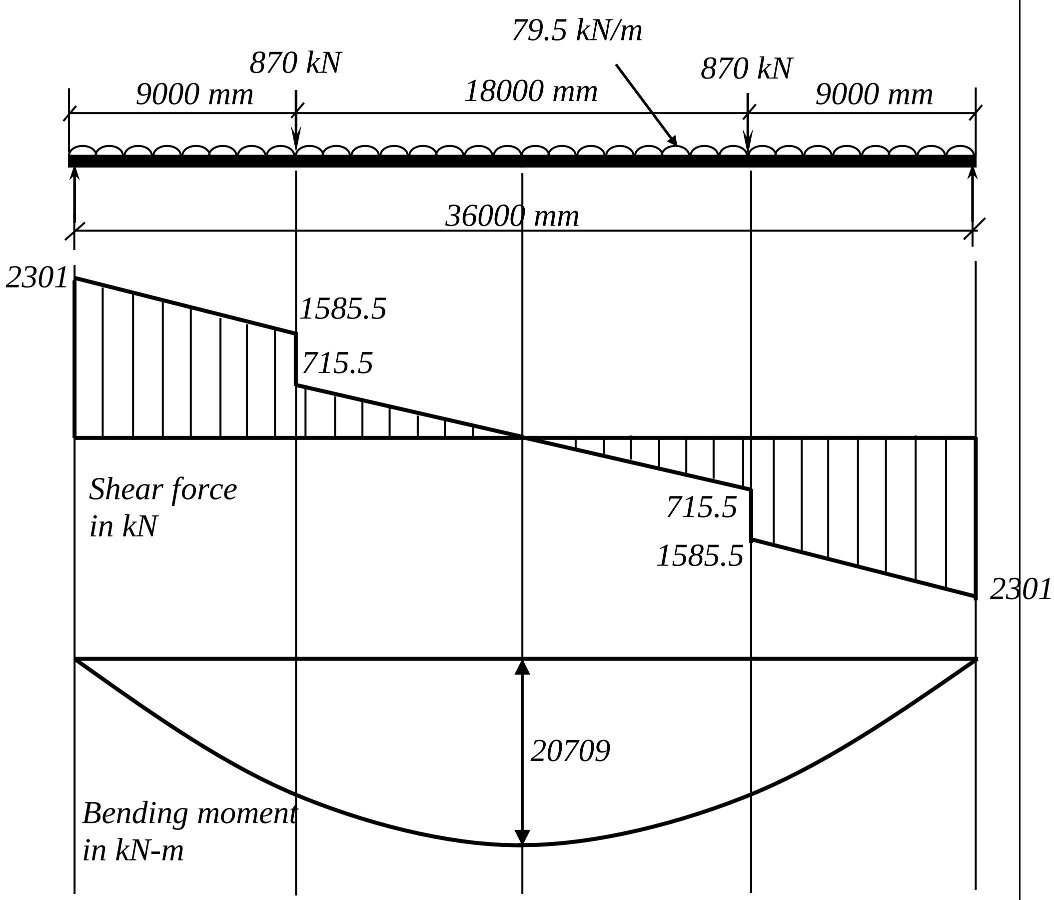

Fig. E2 Bending moment and shear force diagrams

Structural Steel Design Project Calculation Sheet		Sheet **4 of 18**	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Flange:

p_y = 250/1.15 = 217.4 N/mm^2

Single flange area,

$$A_f = \frac{M_{\max}}{d\ p_y} = \frac{20709*10^6}{2400*217.4} = 39690.7\ \ mm^2$$

By thumb rule, the flange width is assumed as 0.3 times the depth of the section. Try 720 X 60 mm, giving an area = 43200 mm^2.

Web:

Minimum web thickness for plate girder in buildings usually varies between 10 mm to 20 mm. Here, thickness is assumed as 14 mm.

Hence, web size is 2400 X 14 mm

4.0 SECTION CLASSIFICATION

Flange:

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{1/2} = \left\{\frac{250}{250}\right\}^{1/2} = 1.0$$

$$b = \frac{B-t}{2} = \frac{720-14}{2} = 353$$

$$\frac{b}{T} = \frac{353}{60} = 5.9 < 7.9\varepsilon$$

Hence, Flange is PLASTIC SECTION.

Structural Steel Design Project Calculation Sheet	Job No:	Sheet *5 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Web:

$$\frac{d}{t} = \frac{2400}{14} = 171.4 > 66.2\varepsilon$$

Hence, the web is checked for shear buckling.

5.0 CHECKS

Check for serviceability:

$$\frac{d}{250} = \frac{2400}{250} = 9.6\ mm < t$$

Since, $t > \frac{d}{250}$

Web is adequate for serviceability.

Check for flange buckling in to web:

Assuming stiffener spacing, a > 1.5 d

$$t \geq \frac{d}{294}\left(\frac{p_{yf}}{250}\right)^{1/2} = \frac{2400}{294} \times \left(\frac{217.4}{250}\right)^{1/2} = 7.6\ \ mm$$

Since, t (= 14 mm) > 7.6 mm, the web is adequate to avoid flange buckling into the web.

Check for moment carrying capacity of the flanges:

The moment is assumed to be resisted by flanges alone and the web resists shear only.

Distance between centroid of flanges, $h_s = d + T = 2400 + 60 = 2460$ *mm*

$$A_f = B * T = 720 * 60 = 43200\ mm^2$$

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet *6 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

$M_c = p_{yf} * A_f * h_s = 217.4 * 43200 * 2460 * 10^{-6} = 23103.5$ *kN-m*
> 20709 *kN-m*

Hence, the section in adequate for carrying moment and web is designed for shear.

6.0 WEB DESIGN

The stiffeners are spaced as shown in Fig. E5. The spacing of stiffeners is taken as 3000 mm. The spacing can be increased towards the centre of the span for economy.

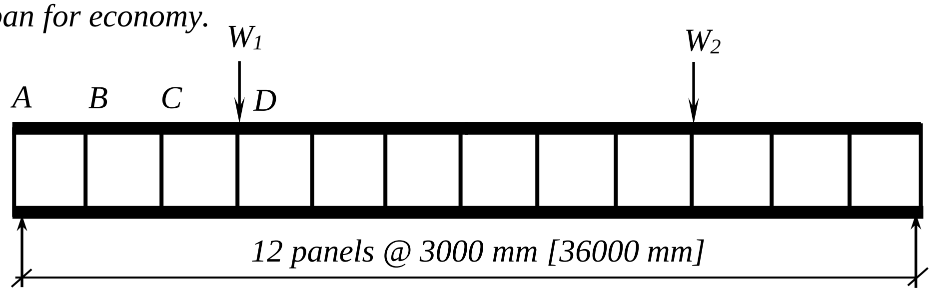

Fig.E3 Trial stiffener arrangement

Panel AB is the most critical panel (Maximum shear zone), so design checks for the web are made for panel AB only.

<u>End panel (AB) design:</u>

$d = 2400$ *mm*
$t = 14$ *mm*

$$\frac{a}{d} = \frac{3000}{2400} = 1.25$$

$$\frac{d}{t} = \frac{2400}{14} = 171.4$$

Structural Steel Design Project	*Job No:*	Sheet ***7 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Calculation of critical shear strength, q_{cr}:

Elastic critical stress, q_e (when $a/d > 1$) = $[1.0 + 0.75/(a/d)^2][1000/(d/t)]^2$

$= [1 + 0.75/(1.25)^2][1000/(171.4)]^2$

$= 50.4\ N/mm^2$

Slenderness parameter, λ_w $= [0.6(f_{yw}/\gamma_m)/q_e]^{1/2}$

$= [0.6(250/1.15)/50.4]^{1/2}$

$= 1.61 > 1.25$

Hence, Critical shear strength ($q_{cr} = q_e$) = $50.4\ N/mm^2$

$$f_v = \frac{F_{VA}}{dt} = \frac{2301*10^3}{2400*14} = 68.5\ N/mm^2$$

Since, $f_v > q_{cr}$ (68.5 > 50.4)

Panel AB is designed using tension field action.

Calculation of basic shear strength, q_b:

$$\phi_t = \frac{1.5q_{cr}}{\sqrt{1+\left(\frac{a}{d}\right)^2}} = \frac{1.5*50.4}{\sqrt{1+(1.25)^2}} = 47.2$$

$y_b = (p_{yw}^2 - 3q_{cr}^2 + \phi_t^2)^{1/2} - \phi_t = (217.4^2 - 3*50.4^2 + 47.2^2)^{1/2} - 47.2 = 157.4$

$$q_b = q_{cr} + \frac{y_b}{2\left[\frac{a}{d} + \sqrt{1+\left(\frac{a}{d}\right)^2}\right]} = 50.4 + \frac{157.4}{2\left[1.25 + \sqrt{1+(1.25)^2}\right]} = 78.0\ N/mm^2$$

Structural Steel Design Project	Job No:	Sheet ***8 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Since, $q_b > f_v$ (78.0 > 68.5)

Panel AB is safe against shear buckling.

<u>Checks for the web panel:</u>

End panel AB should also be checked as a beam (Spanning between the flanges of the girder) capable of resisting a shear force R_{tf} *and a moment* M_{tf} *due to anchor forces.*
(In the following calculations boundary stiffeners are omitted for simplicity)

<u>Check for shear capacity of the end panel:</u>

$$H_q = 0.75\, dt\ p_y \left[1 - \frac{q_{cr}}{0.6\, p_y}\right]^{1/2} \left[\frac{f_v - q_{cr}}{q_b - q_{cr}}\right]$$

$$q_{cr} = 50.4\, N/mm^2$$

$$H_q = 0.75*2400*14*217.4 \left[1 - \frac{50.4}{0.6*(250/1.15)}\right]^{1/2} \left[\frac{68.5 - 50.4}{78 - 50.4}\right] = 2814\ kN.$$

$$R_{tf} = \frac{H_q}{2} = \frac{2814}{2} = 1407\, kN$$

$A_v = t\,.a = 14 * 3000 = 42000\ mm^2$

$P_v = 0.6\, p_{yw}\, A_v = 0.6 * (250/1.15) * 42000/1000 = 5478\ kN$

Since, $R_{tf} < P_v$*, the end panel can carry the shear force.*

Structural Steel Design Project Calculation Sheet	Job No:	Sheet *9 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Check for moment capacity of end panel AB:

$$M_{tf} = \frac{H_q\,d}{10} = \frac{2814 * 2400}{10} * 10^{-3} = 675.4\ kN - m$$

$$y = \frac{a}{2} = \frac{3000}{2} = 1500$$

$$I = \frac{1}{12} t a^3 = \frac{1}{12} * 14 * 3000^3 = 3150 * 10^7\ mm^4$$

$$M_q = \frac{I}{y} p_y = \frac{3150 * 10^7}{1500} * (250/1.15) * 10^{-6} = 4565\ kN - m$$

Since, $M_{tf} < M_q$ *(675.4 < 4565)*

$\therefore$ *The end panel can carry the bending moment.*

7.0 DESIGN OF STIFFENERS

Load bearing stiffener at A:

Design should be made for compression force due to bearing and moment.

Design force due to bearing, F_b *= 2301 kN*

Force(F_m*) due to moment* M_{tf}*, is*

$$F_m = \frac{M_{tf}}{a} = \frac{675.4}{3000} * 10^3 = 225\ kN$$

Total compression = $F_c = F_b + F_m = 2301 + 225 = 2526\ kN$

Structural Steel Design Project Calculation Sheet	Job No:	Sheet *10 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Area of stiffener in contact with the flange, A:

Area (A) should be greater than $\dfrac{0.8\,F_c}{p_{ys}}$

$$\frac{0.8F_c}{p_{ys}} = \frac{0.8*2526}{217.4}*10^3 = 9295\ mm^2$$

Try stiffener of 2 flats of size 240 X 25 mm thick

Allow 15 mm to cope for web/flange weld

*A = 225 * 25 * 2 = 11250* mm^2 *>* *9295* mm^2

$\therefore$ *Bearing check is ok.*

Check for outstand:

Outstand from face of web should not be greater than 20 $t_s\varepsilon$.

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{1/2} = \left\{\frac{250}{250}\right\}^{1/2} = 1.0$$

Outstand b_s *= 240 mm <* *20* $t_s\varepsilon$ *(= 20 * 25 * 1.0 = 500)*

b_s *= 240 mm <* *13.7* $t_s\varepsilon$ *(= 13.7 * 25 * 1.0 = 342.5)*

Hence, outstand criteria is satisfied.

Structural Steel Design Project	Job No:	Sheet ***11 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Check stiffener for buckling: (*The effective stiffener section is shown in Fig. E4*)

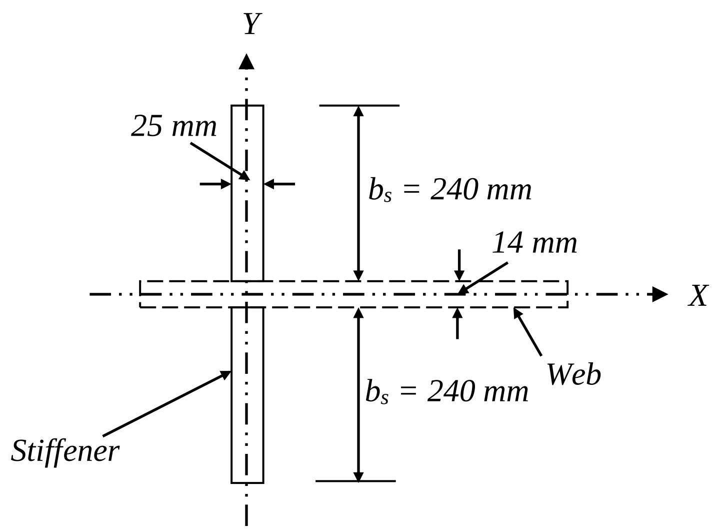

Fig. E4 End bearing stiffener

The buckling resistance due to web is neglected here for the sake of simplicity.

$$I_x = \frac{25*494^3}{12} - \frac{1}{12}*25*14^3 = 25115*10^4 \ mm^4$$

A_e = *Effective area* = $240 * 25 * 2 = 12000 \ mm^2$

$$r_x = \left[\frac{I_x}{A_e}\right]^{1/2} = \left[\frac{25115*10^4}{12000}\right]^{1/2} = 144.7 \ mm$$

Flange is restrained against rotation in the plane of stiffener, then

$l_e = 0.7\, l = 0.7 * 2400 = 1680 \ mm$

$$\lambda = \frac{l_e}{r_x} = \frac{1680}{144.7} = 11.6$$

Structural Steel Design Project	Job No:	Sheet ***12 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

For f_y = 250 N/mm^2 and λ = 11.6

σ_c = 250 N/mm^2 from table (3) of chapter on axially compressed columns

Buckling resistance of stiffener is

$P_c = \sigma_c A_e / \gamma_m = (250/1.15) * 12000 * 10^{-3} = 2609$ kN

Since $F_c < P_c$ (2526 < 2609), stiffener provided is safe against buckling.

<u>Check stiffener A as a bearing stiffener:</u>

Local capacity of the web:

Assume, stiff bearing length b_1 = 0

n_2 = 2.5 * 60 * 2 = 300 *BS 5950: Part – 1, Clause 4.5.3*

$P_{crip} = (b_1 + n_2)\ t\ p_{yw}$

$= (0 + 300) * 14 * (250/1.15) * 10^{-3} = 913$ kN

Bearing stiffener is designed for F_A

$F_A = F_c - P_{crip} = 2526 - 913 = 1613$ kN

Bearing capacity of stiffener alone

$P_A = p_{ys} * A = (250/1.15) * 12000/1000 = 2609$ kN

Since, $F_A < P_A$ (1613 < 2609)

The designed stiffener is OK in bearing.

<u>Stiffener A</u> *– Adopt 2 flats 240 mm X 25 mm thick*

Structural Steel Design Project	Job No:	Sheet *13 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Design of intermediate stiffener at B:

Stiffener at B is the most critical intermediate stiffener, hence it will be chosen for the design.

Minimum Stiffness:

$$I_s \geq 0.75 dt^3 \quad for \; a \geq d\sqrt{2}$$

$$I_s \geq \frac{0.75 dt^3}{a^3} \quad for \; a < d\sqrt{2}$$

$$d\sqrt{2} = \sqrt{2} * 2400 = 3394 \; mm$$

$$\therefore a < d\sqrt{2} \qquad (3000 < 3394)$$

Conservatively' t' is taken as actual web thickness and minimum' a' is used.

$$\frac{1.5 d^3 t^3}{a^2} = \frac{1.5 * 2400^3 * 14^3}{3000^2} = 632 * 10^4 \; mm^4$$

Try intermediate stiffener of 2 flats 90 mm X 12 mm

$$(I_s)_{Provided} = \frac{12 * 194^3}{12} - \frac{12 * 14^3}{12} = 730 * 10^4 \; mm^4$$

The section provided satisfies the minimum required stiffness.

Structural Steel Design Project	Job No:	Sheet **14 of 18**	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Check for outstand:

Outstand of the stiffener $\leq 13.7\ t_s\varepsilon$

$13.7\ t_s\varepsilon = 13.7 * 14 * 1.0 = 192$ mm

Outstand = 90 mm (90 < 192)

Hence, outstand criteria is satisfied.

Buckling check:

Stiffener force, $F_q = V - V_s$

where, V = *Total shear force*
$V_s = V_{cr}$ *of the web.*

Elastic critical stress, $q_e = 50.4\ N/mm^2$

$V_{cr} = q_{cr}\, d\, t = 50.4 * 2400 * 14 * 10^{-3} = 1693$ kN

Shear force at B, $V_B = 2301 - [(2301 - 1585.5)*(3000/9000)] = 2062.5$ **kN**

Stiffener force, $F_q = [2062.5 - 1693] = 369.5$ kN

Structural Steel Design Project Calculation Sheet	Job No:	Sheet *15 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Buckling resistance of intermediate stiffener at B:

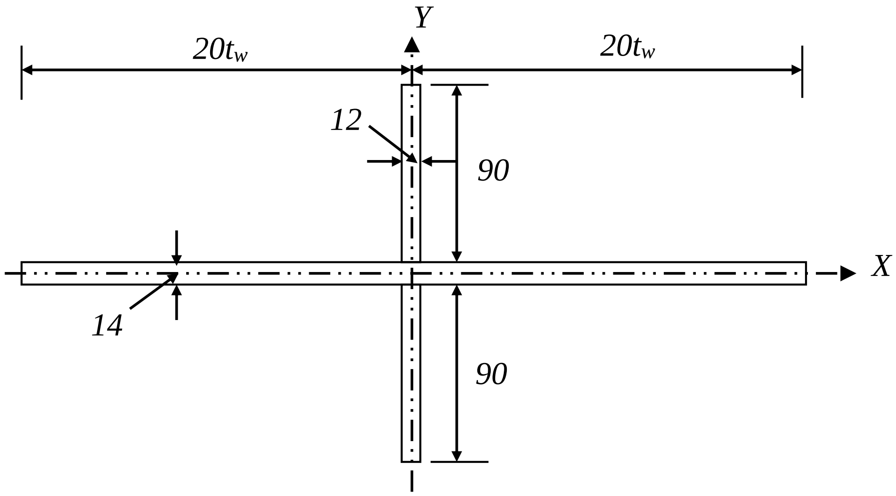

Fig.E5 Effective section

20 t_w *= 20 * 14 = 280 mm*

$$I_x = \frac{1}{12} * 12 * 194^3 + \frac{560*14^3}{12} - \frac{12*14^3}{12} = 743*10^4 \ mm^4$$

*A = 180 * 12 + 560 * 14 = 10000* mm^2

$$r_x = \left[\frac{743*10^4}{10000}\right]^{1/2} = 27.3$$

l_e *= 0.7 * 2400 = 1680*

$$\lambda = \frac{l_e}{r_x} = \frac{1680}{27.3} = 61.5$$

For f_y *= 250 N/mm²* *and* λ *= 61.5*

From table3 of chapter on axially compressed columns,

$\sigma_c = 182.3$ *N/mm²*

Structural Steel Design Project	Job No:	Sheet ***16 of 18***	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
Calculation Sheet		Checked by *PU*	Date *25-04-00*

Buckling resistance $= (182.3/1.15) * 10000 * 10^{-3} = 1585\ kN$

$F_q <$ *Buckling resistance.* $(369.5 < 1585)$

Hence, intermediate stiffener is adequate

Intermediate stiffener at B **-** *Adopt 2 flats 90 mm X 12 mm*

Intermediate Stiffener at D (Stiffener subjected to external load):

Try intermediate stiffener 2 flats 90 mm X 12 mm thick

It satisfies the minimum stiffness requirement as in case of stiffener at B.

Buckling check:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \le 1$$

$F_q = V - V_s$ $\qquad$ $V = 1585.5\ kN$

$V_s = V_{cr} = q_{cr}\, d\, t = 50.4 * 2400 * 14 * 10^{-3} = 1693\ kN$

F_q *is negative and so we can take* $F_q - F_x = 0$

$M_s = 0$

$\mathbf{F_x = 870\ kN}$

Structural Steel Design Project Calculation Sheet	Job No:	Sheet *17 of 18*	Rev
	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

Buckling resistance of load carrying stiffener at D:

(Calculation is similar to stiffener at B)

Buckling resistance, $P_x = (182.3/1.15) * 10000 * 10^{-3} = 1585$ *kN*

$F_x / P_x = 870/1585 = 0.55 < 1.0$

Hence, stiffener at D is OK against buckling

Stiffener at D - *Adopt flats 90 mm X 12 mm thick*

Web check between stiffeners:

$f_{ed} \le p_{ed}$

$f_{ed} = w^1 / t = 79.5/14 = 5.7\ N/mm^2$

when compression flange is restrained against rotation relative to the web

$$p_{ed} = \left[2.75 + \frac{2}{\left(a/d\right)^2}\right]\frac{E}{\left(d/t\right)^2} = \left[2.75 + \frac{2}{\left(\frac{3000}{2400}\right)^2}\right]\frac{200000}{\left(\frac{2400}{14}\right)^2}$$

$$= \frac{3.79*20000}{26406} = 27.4\ N/mm^2$$

Since, $f_{ed} < p_{ed}$ *[5.7 < 27.4], the web is OK for all panels.*

<table>
<tr><td rowspan="5">Structural Steel Design Project

Calculation Sheet</td><td>Job No:</td><td>Sheet 18 of 18</td><td>Rev</td></tr>
<tr><td colspan="3">Job Title: PLATE GIRDER</td></tr>
<tr><td colspan="3">Worked Example - 1</td></tr>
<tr><td rowspan="2"></td><td>Made by SSSR</td><td>Date 15-04-00</td></tr>
<tr><td>Checked by PU</td><td>Date 25-04-00</td></tr>
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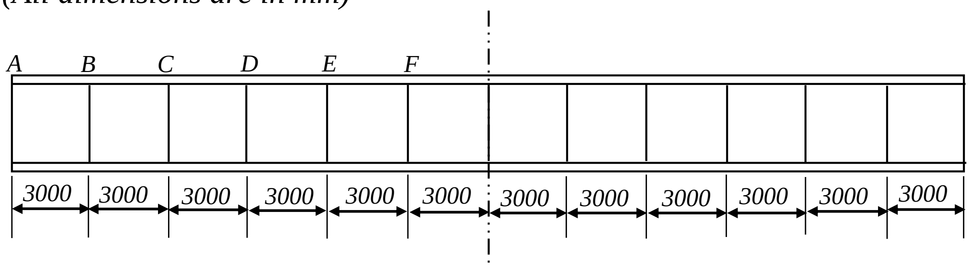
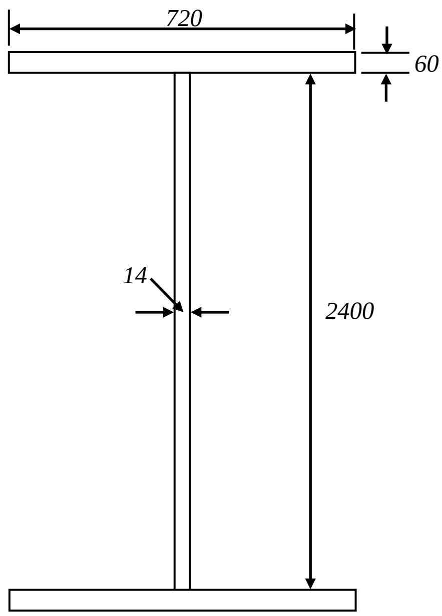
8.0 FINAL GIRDER

(All dimensions are in mm)



(a) Longitudinal section of plate girder



(b) Flange and web proportions

Fig. E6 Final girder