@ A = 2301 kN

@ B' $2301 - 79.5 \times 9 = 1585.5$ kN

@ B $2301 - (79.5 \times 9) - 870 = 715.5$ kN

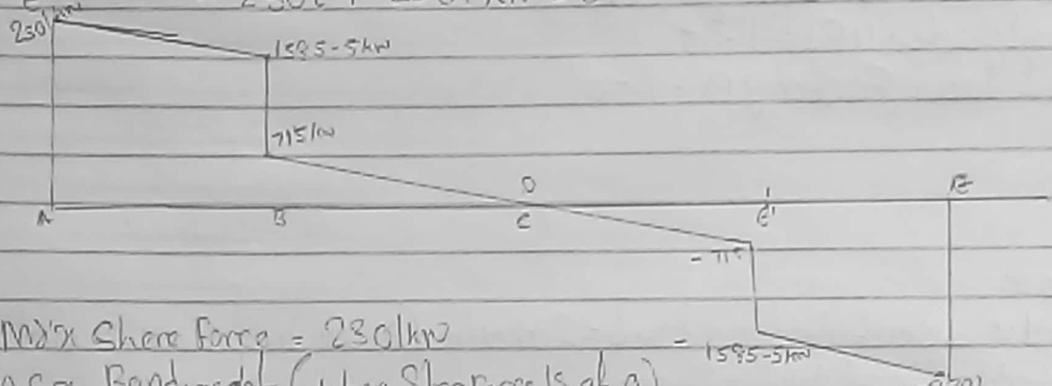
@ C $715.5 - 79.5(9) = 0$

@ D' $0 - 79.5 \times 9 = -715.5$ kN

@ D $-715.5 - 870 = 1585.5$ kN

@ E' $-1585.5 - (79.5 \times 9) = -2301$ kN

@ E $-2301 + 2301$ kN $= 0$



Max Shear Force = 2301 kN

Max Bending moment (when Shear force is at 0)

$= 2301 \times 18) - (79.5 \times 18^2/2) - 870 \times 9)$

$= 20709$ kN/m²

Girder Section

Depth of Girder $= D = \frac{Span}{15/10/12}$ using 15 $= \frac{36000}{15} = 2400$ mm

Flange

$P_y = \frac{250}{1.15} = 217.4$ N/mm²

Single Flange Area

$A_f = \frac{M_{max}}{d \cdot P_y} = \frac{20709 \times 10^6}{2400 \times 217.4} = 39690.7$ mm²

Assuming thickness of flange as 60 mm

Breadth of girder $= 2400 \times 0.3 = 720$

Using 720 mm × 60 mm = 43200 mm² (Area)

Web:- Using 13 as thickness of web =

web size = 2400 × 13 mm

Section Classification

Flange:-

$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5} = \left(\frac{275}{250}\right)^{0.5} = 1.04$

$b = \frac{B-t}{2} = \frac{720-13}{2} = 353.5$

$\frac{b}{t} = \frac{353.5}{60} = 5.89 < 7.9\varepsilon$

Flange is plastic//

web

$d/t = \frac{2400}{13} = 184.6 > 63\varepsilon$

Checking for Shear buckling

Checks for web

Serviceability

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

Check is ok

Check for flange buckling in to web:-

Assuming Stiffness spancing $a > 1.5d$

$t \geq \frac{d}{294}\left(\frac{p_y}{250}\right)^{1/2} = \frac{2400}{294}\left(\frac{217.6}{250}\right)^{0.5} = 7.6mm$

$t = 13m > 7.6mm$

Check is ok//

Moment capacity

$= BT(d+T)p_y$

$= 720 \times 60(2400+60) \frac{217.4}{10^6} = 23103.5\,kNm > m_{max}$

Check is ok

web design

12 Panels Divided equally = 3000mm Per Pack

End Panel (AB) design

$d = 2400$

$t = 13mm$

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

$d/t = \frac{2400}{13} = 184.6$

Shear Strength

$V_{cr} = q_{cr} \cdot dt$

elastic stress q_e (when $a/d > 1$) $= \left(1.0 + \frac{0.75}{(a/d)^2}\right)\left(\frac{1000}{(d/t)}\right)^2$

$= \left(1 + \left(\frac{0.75}{1.25}\right)^2\right)\left(1000/(184.6)\right)^2$

$= 43.4\,N/mm^2$

Slenderness Parameter $\lambda_w = \left(0.6\,p_y/q_e\right)^{0.5}$

$\left(0.6\left(\frac{250}{1.15}\right)/56.6\right)^{0.5}$

$= 1.61 > 1.25$

$q_{cr} = q_e = 43.4\,N/mm^2$

$F_v = F_{vc} = \frac{2301 \times 10^3}{dt} = \frac{}{2400 \times 14} = 68.5 \, N/mm^2$

$F_v > q_{cr}$

Design for panel AB using Tension field action

Basic Shear Strength (q_b) :-

$$f_t = \frac{1.5 q_{cr}}{\sqrt{1+(a/d)^2}} = \frac{1.5 \times 50.4}{\sqrt{1+(1.25)^2}} = 47.2$$

$$q_b = (p_{yw}^2 - 3q_{cr}^2 + f_t^2)^{1/2} - f_t$$

$$\therefore = (217.4^2 - 3(50.4)^2 + 47.2^2)^{1/2} - 47.2 = 157.4$$

$$q_b = q_{cr} + \frac{q_b}{2\left(a/d + \sqrt{1+\left(\frac{a}{b}\right)^2}\right)}$$

$$= 50.4 + \frac{157.4}{2\left(1.25 + \sqrt{1+(1.25)^2}\right)} = 78.5 \, N/mm^2$$

$q_b > F_v$

AB is ok against buckling

Web panel checks

Checks for Shear Capacity of end panel

$$H_q = 0.75\, d\, t\, f_y \left(\frac{1 - q_{cr}}{0.6 p_y}\right)^{0.5} \left(\frac{F_v - q_{cr}}{q_b - q_{cr}}\right)$$

$$H_q = 0.75 \times 2400 \times 217.4 \left(\frac{1 - 50.4}{0.6 \times 217.4}\right)^{0.5} \left(\frac{68.5 - 50.4}{78 - 50.4}\right) = 2814 \, kN$$

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

$A_v = t \times a = 13 \times 3000 = 39000 \, mm^2$

$P_v = 0.6 p_{yw} A_v = 0.6 \times 217.4 \times 39 = 508.716 \, kN$

$R_{tf} < P_v$

Section is Ok

Check for moment capacity of end Panel

$$M_{tf} = \frac{H_q d}{10} = \frac{2814 \times 24000}{10} \times 10^{-3} = 675.4 \, kNm$$

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

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

$$M_q = \frac{I}{y} f_y = \frac{3150 \times 10^7}{1500} \times (217.4 \times 10^{-6}) = 4565 \, kN/m$$

$M_{kf} < M_q$

Section AB is Ok

Design of Stiffeners

Load bearing of Stiffeners A

max Shear force = 231 kN

Force (Fm) due to moment M_{kf} - $F_m = \frac{M_{kf}}{a} = \frac{675.4 \times 10^3}{3000} = 225$ kN

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

Area of Stiffeners is contact with flange A.

Area ~~of Stif~~ (A) should be greater than $\frac{0.8 F_c}{f_{ys}}$

$= \frac{0.8 \times 2526}{217.4} \times 10^3 = 9295 \text{mm}^2$

Try Stiffeners size of 2 Plates of size 240x25 mm thick Allowing 15mm to cope for weld/flange weld $A = 225 \times 25 \times 2 = 11250 \text{mm}^2 > 9295 \text{mm}^2$

Bearing Check is Ok

Check for Out Stand

$\varepsilon = 1.04$

Outstands be 240 mm $< 20 t_s \varepsilon$ $(20 \times 25 \times 1.04 = 520$

$b_s = 240 \text{mm} < 13.7 t_s \varepsilon = 13.7 \times 25 \times 1.04 = 356.2$

Criterion is Satisfied

Checking Stiffener for buckling

$I_x = \frac{25 \times 493^3}{12} - \frac{1}{12} \times 25 \times 13^3 = 24963 \times 10^4 \text{mm}^4$

A_c = Effective area $= 240 \times 25 \times 2 = 12000 \text{mm}^2$

$r_x = \left(\frac{I_x}{A_c}\right)^{0.5} = \left(\frac{24963 \times 10^4}{12000}\right)^{0.5} = 144.2$ mm

Since flange is restrained against rotation

$L_e = 0.7 l = 0.7 \times 2400 = 1680$ mm

$\lambda = \frac{L_e}{r_x} = \frac{1680}{144.2} = 11.65$

Buckling resistance of Stiffeners

$P_c = \frac{f_y A_e}{\gamma_m} = \frac{250 \times 12000 \times 10^{-3}}{1.15} = 2609$ kN

$F_c < P_c$ $(2526 < 2609)$

Stiffener is safe against Buckling.

Checking Stiffener A as a bearing Stiffener
Load Capacity of web
Assume Stiff bearing length $b_1 = 0$
$n_a = 2.5 \times 60 \times 2 = 300$
$P_{crip} = (b_1 + n_2) t\, p_{yw}$
$= (0 + 300) + 13 \times (217.4) \times 10^{-3} = 848kN$
Bearing Stiffener for FA
$F_A = F_C - P_{crip} = 2526 - 848 = 1678kN$
Bearing Capacity of Stiffener alone
$P_a = P_{ys} \times A \; 217.4 \times 12000 \times 10^{-3} = 2609kN$
$F_A < P_a \quad (1678 < 2609)$
The designed Stiffener is ok in bearing

Stiffener A = 2 Flats 240 mm x 25 mm thick
Design for Intermediate Stiffener at B
minimum Stiffener
$I_s \geq 0.75 d t^3$ for $a \geq d\sqrt{2}$
$I_s \geq \frac{1.5 d t^3}{a^2}$ for $a < d\sqrt{2}$
$d\sqrt{2} = 3394m$
$\therefore a < d\sqrt{2}$
$I_s = \frac{1.5^2 \times 2400^3 \times 13^3}{3000^2} = 506 \times 10^4 mm^4$
Try 90 mm x 12 mm of 2 Flats
I_a provided $= \frac{12 \times 193^3}{12} - \frac{12 \times 13^3}{12} = 730 \times 10^4 mm^4$
Section is ok
Check for Outstand
Outstand of the Stiffener $\leq 13.7 t_s \varepsilon$
$13.7 t_s \varepsilon = 13.7 \times 13 \times 1.04 = 185.224$
Outstand = 90mm $(90 < 185.224)$
Section is Ok
Buckling Check
Stiffener force $F_q = V - V_s$
V = Shear force
V_s = Vcr of web

Where

$V_{cr} = q_{cr} d t = 43.6 \times 2400 \times 13 \times 10^{-3} = 1354.08\,kN$

Shear Force at B, $V_B = 2301 - (2301 - 1585.5 \times 3000/9000) = 2062.5\,kN$

Stiffener force $F_q = 2062.5 - 1354.08 = 708.42$

Buckling Resistance of Intermediate Stiffener at B

$20 t_w = 20 \times 13 = 260\,mm$

$I_x = 1/12 \times 12 \times 193^3 + \frac{530 \times 12^3}{12} - \frac{12 \times 13^3}{12} = 728 \times 10^4\,mm^4$

$r_x = \left(\frac{728 \times 10^4}{10000}\right)^{0.5} = 27$

$L_e = 0.7 \times 2400 = 1680$

$\lambda = \frac{L_e}{r_x} = \frac{1680}{27} = 62.2$

$= (182.3/1.15) \times 10000 \times 10^{-3} = 1585.5\,kN$

Intermediate Stiffener at D (Stiffener loaded to external load)

Use 2 Flats 90mm × 12mm thick

Buckling Checks:

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

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

$V_s = V_{cr} = q_{cr} d t = 43.4 \times 2400 \times 14 \times 10^{-3}$

$F_q = 231.42$

$M_s = 0$

$F_x = 870\,kN$

$\frac{231.42 - 870}{250} + \frac{870}{1585} < 1$

$- 2.0 \le 1$

Section ok

Use Flats 90mm × 12 mm thick

Web Check between Stiffeners

$f_{ed} \le P_{ed}$

$f_{ed} = w/t = \frac{79.5}{13} = 6.1\,N/mm^2$

When Compression Flange is restrained against rotation relative to the web

$P_{ed} = \left(2.75 + \frac{2}{(a/d)^2}\right) \frac{E}{(d/t)^2}$

$$\Rightarrow \left(2.75 + \frac{2}{\left(\frac{3000}{2400}\right)^2}\right) \frac{200000}{\left(2400/13\right)^2} = 23.6\ N/mm^2$$

$f_{ad} < P_{ad}$

web is ok