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18/Eng03/059
Assignment Steel Structural design

The girder Showed in Fig E1 is fully restrained against lateral buckling throughout it's span the span is 36m and carries two concentrated loads as Shown in fig E1 - Design a Plate girder
Yeild Stress of Steel, $f_y = 250 N/mm^2$
material factor of Steel $\gamma_m = 1.15$
Dead load factor $\gamma_{fd} = 1.35$
Imposed load factor $\gamma_{f\lambda} = 1.50$

Dead load:
Uniformly distributed load $w_d = 20kN/m$ (including Self-weight)
Concentrated load $W_{1d} = 200kN$
Concentrated load $W_{2d} = 200kN$
Life load.
Uniformly distributed load $w_\lambda = 35kN/m$
Concentrated load $W_{1\lambda} = 400kN$
Concentrated load $W_{2\lambda} = 400kN$

Solution

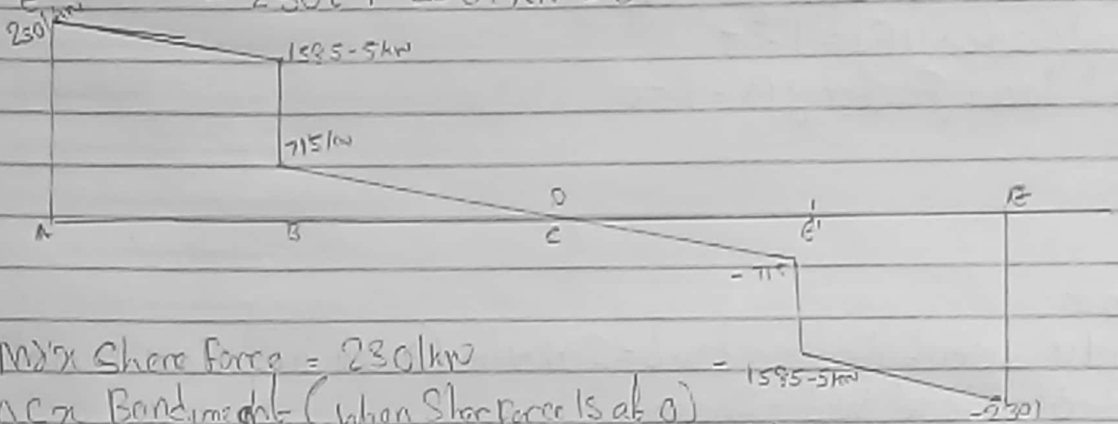
Using the load factor
$1.35G_k + 1.5Q_k$
For UDL $= 20 \times 1.35 + 1.5 \times 35 = 79.5 kN/m$
For conc Load 1 $= 200 \times 1.35 + 400 \times 1.5 = 870kN$
For conc load 2 $= 200 \times 1.35 + 400 \times 1.5 = 870kN$

S.F and B.M

Calculating Reactions
moment about a = 0
$0 = (870 \times 900) + (79.5 \times 36000 \times 18000) + (870 \times 27000) - 36000$
$E = 2301 kN$
$A = (79.5 \times 36) + 870 + 870 - 2301$
$A = 2301 kN$

Calculating Shear Force

@ A = 2301 kN
@ B' 2301 − 79.5×9 = 1585.5 kN
@ B 2301 − (79.5×9) − 870 = 715.5 kN
@ C 715.5 − 79.5 (9) = 0
@ D' 0 − 79.5×9 = −715.5 kN
@ D −715.5 − 870 = 1585.5 kN
@ E' −1585.5 − (79.5×9) = −2301 kN
@ E −2301 + 2301 kN = 0



Max Shear Force = 2301 kN
Max Bending moment (when Shear force is at 0)
= 2301×18) − (79.5×$\frac{18^2}{2}$) − 870×9)
= 20709 kNm²

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

Flange
$P_y = \frac{250}{1.15} = 217.4 N/mm^2$

Single Flange Area
$A_f = \frac{M_{max}}{d \cdot P_y} = \frac{20709 \times 10^6}{2400 \times 217.4} = 39690.7 mm^2$

Assuming thickness of flange as 60 mm
Breadth of girder = 2400 × 0.3 = 720
using 720 mm × 60 mm = 43200 mm² (Area)
web:- using 13 as thinness of web =
web size = 2400 × 13 mm
Section Classification
Flange:-

$C = \left(\frac{275}{Py}\right)^{0.5} = \left(\frac{275}{250}\right)^{0.5} = 1.04$

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

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

Flange is plastic//

Web

$d/t = \frac{2600}{13} = 184.6 > 636$

Checking for Shear buckling

Checks for web

Serviceability

$\frac{d}{250} - \frac{2400}{250} = 9.6mm < 6$

Check is ok

Check for flange buckling in to web:

Assuming Stiffness spancing $a > 1.5d$

$t \geq \frac{d}{294}\left(\frac{Py}{250}\right)^{1/2} = \frac{2400}{294}\left(\frac{217.4}{250}\right)^{0.5} = 7.6mm$

$t = 13m > 7.6mm$

Check is Ok//

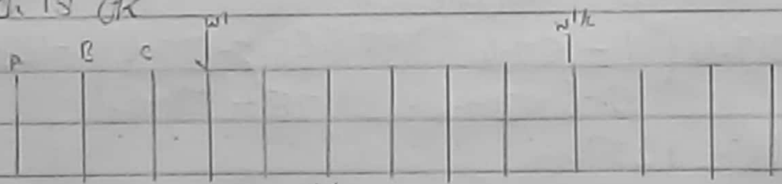
Moment Capacity

$= BT(d+t)Py$

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

Check is ok

web design



12 Panels Divided equally = 3000mm Per Pach

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

$= (1 + (0.75/1.25)^2)(1000/(184.6))^2$

$= 43.4 N/mm^2$

Slenderness Parameter $\lambda_w = (0.6\, Py/\gamma_m / q_c)^{0.5}$

$(0.6 (250/1.15) / 50.6)^{0.5}$

$= 1.61 > 1.25$

$q_{cr} = q_c = 43.4 N/mm^2$

$$f_v = \frac{F_v}{dt} = \frac{2301\times10^3}{2400\times14} = 68.5\,N/mm^2$$

$f_v > q_{cr}$

Design End panel AB using Tension Field action

Basic Shear Strength (q_b) :-

$$\phi_t = \frac{1.5 q_{cr}}{\sqrt{1+(a/d)^2}} = \frac{1.5\times50.4}{\sqrt{1+(1.25)^2}} = 47.2$$

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

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

$$q_b = q_{cr} + \frac{4b}{2\left(a/d + \sqrt{1+\left(\frac{a}{b}\right)^2}\right)}$$

$$= 50.4 + \frac{157.4}{2(1.25+\sqrt{1+(1.25)^2})} = 78\,N/mm^2$$

$q_b > f_v$

AB is ok against Buckling

Web Panel checks

Checks for Shear Capacity of end panel

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

$$H_q = 0.75\times2400\times217.4\left(1-\frac{50.4}{0.6\times217.4}\right)^{0.5}\left(\frac{68.5-50.4}{78-50.4}\right) = 2814\,kN$$

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

$A_v = t\times a = 13\times3000 = 39000\,mm^2$

$P_v = 0.6p_{yw}\,A_v = 0.6\times217.4\times39 = 508.716\,kN$

$R_{tf} < P_v$

section is ok

Check for moment capacity of end Panel

$$M_{tf} = \frac{H_q\,d}{10} = \frac{2814\times24000}{10}\times10^{-3} = 675.4\,kNm$$

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

$$I = \frac{1}{12}ta^3 = \frac{1}{12}\times14\times3000^3 = 3150\times10^7\,mm^4$$

$$M_q = \frac{I}{y}f_y = \frac{3150\times10^7}{1500}\times(217.4\times10^{-6}) = 4565\,kN/m$$

$M_{LF} < M_q$

Section AB is ok

Design of Stiffeness

Load bearing of Stiffeness A

max Shear force = 231 kN

Force (F_m) due to moment M_{LF} - $F_m = \frac{M_{LF}}{a} = \frac{675 \cdot 4 \times 10^3}{3000} = 225$ kN

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

Area of Stiffnes is contact with flange A.

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

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

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

Bearing Check is ok

Check for Out Stand

$\varepsilon = 1.04$

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

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

Criterma is Satisfied

Checking Stiffness for buckling

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

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

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

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

$F_c < P_c$ (2526 < 2609)

Stiffner is safe against Buckling.

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

Stiffener A = 2 Plates 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 $a < d\sqrt{2}$
$d\sqrt{2} = 3394 m$
$\therefore a < d\sqrt{2}$
$I_s = \frac{1.5^2 \times 2400^3 \times 13^3}{3000^2} = 506 \times 10^4 mm^4$
Try 90 mm x 12 mm of 2 Plate
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 $\leq 13.7 t_s \varepsilon$
$13.7 t_s \varepsilon = 13.7 \times 13 \times 104 = 185.224$
Outstand = 90mm $(90 < 185.224)$
Section is ok
Buckling Check
Stiffener force $F_q = V - V_s$
V = Shear force
V_s = V_{cr} of web

Where

$V_{cr} = q_{cr} d t = 43.4 \times 2400 \times 13 \times 10^{-3} = 1354.08$ kN

Shear Force at B, $V_B = 2301 - (2301 - 1585.5 \times 3000/4000) = 2062.5$ 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

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

$\frac{F_q - F_x}{F_q} + \frac{F_x}{P_x} + \frac{M_s}{M_f} \leq 1$

$F_q = V - V_s = V = 1585.5$ 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

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

$= 2.0 \leq 1$

Section is ok

Use Flats 90mm × 12mm thick

Web Check between Stiffeners

$F_{ed} \leq P_{ed}$

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

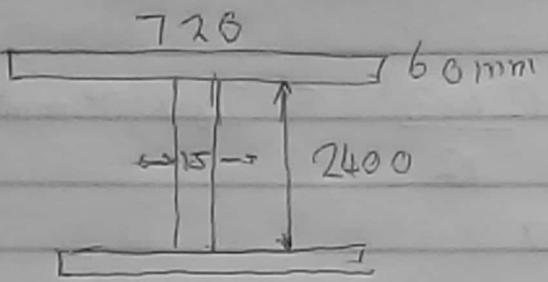
When Compression Flange is restrained against rotation relative to the web

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

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

$f_{cd} < f_{cd}$

Web is ok



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