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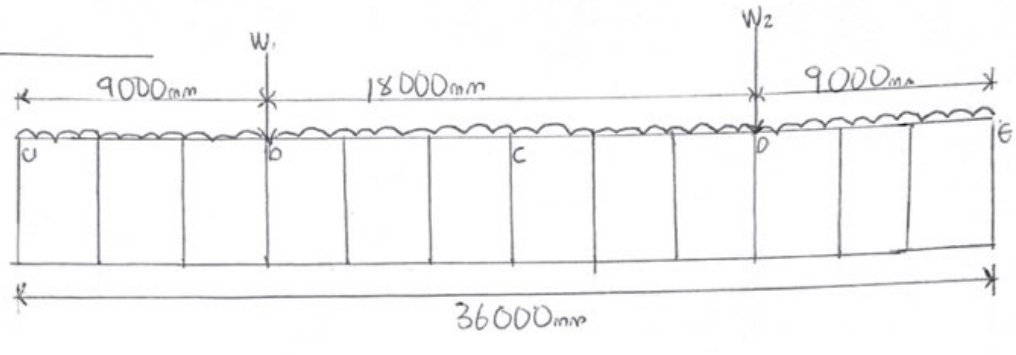
17/ENG03/024

Civil Engineering

CVE 407 (Structural Design II) Ass.

Question



The girder showed in fig. 6I is fully restrained against Lateral buckling throughout its span. The span is 36m and carries 2 concentrated loads as shown in fig. 6I. Design a plate girder.

Yield stress of steel, $f_y = 250N/mm^2$

factor for steel $= y_m = 1.15$

Dead load factor, $y_{fd} = 1.35$

Imposed load factor $= y_f = 1.50$

Dead load

Uniformly distributed load, $wd = 20kN/m$

Concentrated load, $w1d = 200kN$

Concentrated load, $w2d = 200kN$

Live load

Uniformly distributed load, $wl = 35kN/m$

Concentrated load, $w1L = 400kN$

Concentrated load, $w2L = 400kN$

Solution

Design load = $1.35 G_k + 1.5 Q_k$

For UDL

$G_k = 20$

$Q_k = 35$

$\therefore$ Design load for UDL $= 20 \times 1.35 + 35 \times 1.5$
$= 79.5$ kN/m

For Concentrated load 1 =

$G_k = 200$

$Q_k = 400$

$\therefore$ Design load $= 200 \times 1.35 + 400 \times 1.5 = 870$ kN

For Concentrated load 2:

$G_k = 200$

$Q_k = 400$

$\therefore$ Design load $= 200 \times 1.35 + 400 \times 1.5 = 870$ kN

Reactions

$M_G = ((79.5 \times 36) \times 18) + 870 \times 9 + 870 \times 27 \div 36$

$= 2862(18) + 7830 + 23490 \div 36$

$M_E = \frac{82836}{36}$

$M_E = 2301$ kN

$M_A = (870 + 870 + 2862) - 2301$

$M_A = 2301$ kN

Calculating Shear force

1) $A = 2301 \text{ kN}$

$B' = 2301 - 79.5 \times 9 = 1585.5 \text{ kN}$

$B = 2301 - \{(79.5 \times 9) - 870\} = 715.5 \text{ kN}$

$C = 715.5 - 79.5 \times 9 = 0$

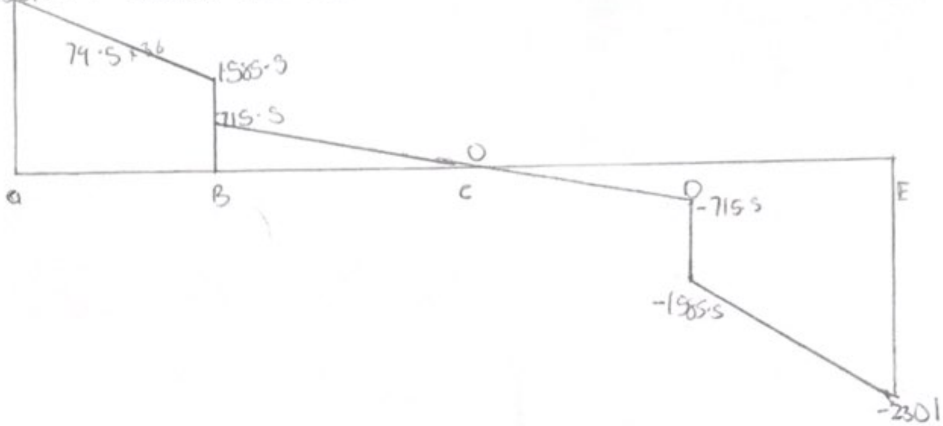
$D' = 0 - 79.5 \times 9 = -715.5 \text{ kN}$

$D = -715.5 - 870 = -1585.5 \text{ kN}$

$E' = -1585 - 79.5 \times 9 = -2301 \text{ kN}$

$E = -2301 + 2301 = 0$

2301 kN = maximum shear force.



Maximum Bending Moment.

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

$= 20709 \text{ kN/m}^2$

Girder Section

$D(\text{depth of girder}) = \frac{\text{Span}}{12} = \frac{36000}{12} = 3000\text{mm}$

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

Single flange area

$A_F = \frac{M_{max}}{d \cdot p_y} = \frac{20709 \times 10^6}{3000 \times 217.4} = 31752.5\text{mm}^2$

thickness of Flange = 60mm (Assumed)

Breadth of girder = $3000 \times 0.3 = 900$

Using Area as $900mm \times 60mm = 54\,000 mm^2$

Web thickness = 13mm

Web size = 3000×13 mm

=

Section Classification

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

$$b = \frac{B-t}{2} = \frac{900-13}{2} = 443.5$$

$$\frac{b}{t} = \frac{443.5}{60} = 7.39 < 7.9\varepsilon$$

Flange is plastic

Web:-

$$\frac{d}{t} = \frac{3000}{13} = 230.8$$

Check for Web serviceability

$$\frac{d}{250} = \frac{3000}{250} = 12mm < t$$

∴ check is Ok

Check for flange buckling in the Web:-

Stiffener spacing $a > 1.5d$

$$t \geq \frac{d}{294}\left(\frac{p_y}{250}\right)^{0.5} = \frac{3000}{294}\left(\frac{217.4}{250}\right)^{0.5} = 8.95mm$$

$t = 13mm > 7.6mm$

∴ check is Ok

Moment Capacity

$BT(d+T) \cdot P_y$

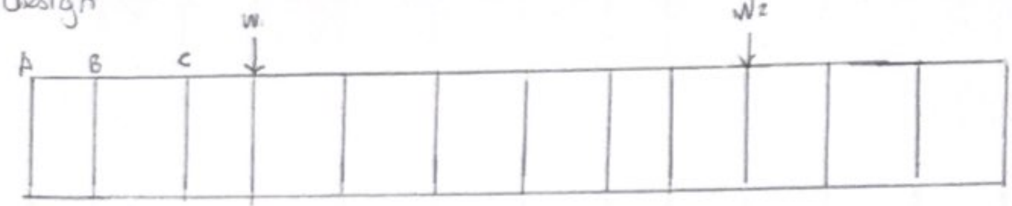
$$= \frac{900 \times 60 (3000+60) \cdot 217.4}{10^6} = \cancel{35923.1kNm}$$

35923.21 kNm

$> M_{max}$

∴ check is Ok

Web design



Design for end panel (AB)

$d = 3000\,mm$
$t = 13\,mm$

$\frac{a}{d} = \frac{3000}{3000} = 1$

$\frac{d}{t} = \frac{3000}{13} = 230.8$

Shear Strength

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

Elastic stress, q_e (when $\frac{a}{d} = 1$) $= \left(1.0 \cdot \frac{0.75}{(a/d)^2}\right)\left(1000/(d/t)\right)^2$

$= \left(1 + \frac{0.75}{(1)^2}\right)\left(1000/(230.8)\right)^2$
$\doteq 32.9\,N/mm^2$

Slenderness parameter $\lambda_w = \left(0.6\left(\frac{f_{yw}}{\sqrt{3}}\right)/q_e\right)^{0.5}$

$= 1.99 > 1$

$q_{cr} = q_e = 32.9\,N/mm^2$

$F_v = \frac{f_{vA}}{dt} = \frac{2301 \times 10^3}{3000 \times 13}$

$= 59\,N/mm^2$

$F_v > q_{cr}$

Design for panel AB using tension field action

Basic shear strength (q_b):-

$$Q_t = \frac{1.5 q_{cr}}{\sqrt{1+\left(\frac{a}{d}\right)^2}} = \frac{1.5 \times 32.9}{\sqrt{1+1}} = 34.9$$

$$y_b = \left(p_{yw}^2 - 3q_{cr}^2 + Q_t^2\right)^{1/2} - Q_t =$$

$$\left(217.4^2 - 3(32.9)^2 + 34.9^2\right)^{1/2} - 34.9 = 177.8$$

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

$$q_b = 32.9 + \frac{177.8}{2[1) + \sqrt{1+(1)^2}}$$

$$\equiv 84.98 N/mm^2$$

$q_b > f_v$

$\therefore$ AB is Ok against buckling

Web panel checks.

Checks for shear capacity of end panel

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

$$H_q = 0.75 \times 3000 \times 217.4 \left(\frac{1-32.9}{0.6 \times 217.4}\right)^{0.5} \left(\frac{59 - 32.9}{84.98 - 32.9}\right)$$

$H_q = 275.6 kN$

$R_{tf} = \frac{H_q}{2} = \frac{2756}{2} = 1378 \text{ kN}$

$A_v = t \times a = 13 \times 3000 = 39000 \text{ mm}^2$

$P_v = 0.6 p_{yw} A_v$

$P_v = 0.6 \times 217.4 \times 39$

$P_v = 5087.16 \text{ kN}$

$R_{tf} < P_v$

$\therefore$ Section is Ok

<u>Moment capacity of end panel check</u>

$M_{tf} = \frac{H_q d}{10} = \frac{2756 \times 3000}{10} \times 10^{-3} = 826.8 \text{ kN/m}$

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

$I = \frac{1}{12} ba^3 = \frac{1}{12} \times 13 \times 3000^3 = 2925 \times 10^7 \text{ mm}^4$

$M_q = \frac{I}{y} p_y = \frac{2925 \times 10^7}{1500} \times (217.4 \times 10^{-6})$.

$M_q = 4239 \text{ kN/m}$

$M_{tf} < M_q$

$\therefore$ Section AB is Ok

Design of Stiffeners

Load bearing of stiffener A:

Max shear force = 2301 kN

Force (f_m) due to moment M_{tf} = $f_m = \frac{M_{tf}}{a} = \frac{826.8}{3000} \times 10^3$

$= 275.6$ kN

Total compression = $f_c = f_b + f_m = 2301 + 275.6 = 2576.6$ kN

Area of stiffener in contact with flange

$$\text{Area (A)} > \frac{0.8 f_c}{P_y}$$

$$= \frac{0.8 \times 2576.6}{217.4} \times 10^3 = 9481.5\,mm^2$$

Use stiffener size of 2 flats of size 240 x 25m thick

Allow 15mm to cope for weld /flange weld

$A = 225 \times 25 \times 2 = 11250\,mm^2 > 9481.5\,m^2$

∴ Bearing check is Ok.

Check for outstand :-

$\varepsilon = 1.04$

Outstands: $b_s = 240\,mm < 20 t_s \varepsilon = 20 \times 25 \times 1.04 = 520$

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

Criteria is satisfied.

Checking stiffner for buckling

$$I_x = \frac{25 \times 445^3}{12} - \frac{1}{12} \times 25 \times 13^3 = 24963 \times 10^4 \, mm^4$$

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

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

Since flange is restrained against rotation

$l_e = 0.7L = 0.7 \times 2400 = 1680 mm$

$$L = \frac{l_e}{r_x} = \frac{1680}{144.2} = 11.65$$

Buckling resistance of stiffner

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

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

Stiffner is safe against buckling

Checking stiffner A as a bearing stiffner

Load capacity of web :-

Assume stiff bearing length b_1 = 0

$n_2 = 2.5 \times 60 \times 2 = 300$

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

$= (0 + 300) \times 13 \times (217.4) \times 10^{-3} = 848 kN$

Bearing stiffener for F_A

$F_A = F_c - P_{crip} = 2526 - 848 = 1678\,kN$

Bearing capacity of stiffener alone

$P_a = P_{ys} \times A = 217.4 \times 12000 \times 10^{-3} = 2609\,kN$

$F_A < P_a$

The designed stiffener is ok in bearing.

Stiffener A: 2 flats 240mm x 25mm thick

Design for intermediate stiffener at B

Minimum stiffeness

$I_s \geq 0.75\,d t^3$ for $a \geq d\sqrt{2}$

$I_s \geq \frac{1.50\,d^3 t^3}{a^2}$ for $a < d\sqrt{2}$

$d\sqrt{2} = 3394\,mm$

$\therefore a < d\sqrt{2}$

$I_s = \frac{1.5 \times 2400^3 \times 13^3}{3000^2} = 506 \times 10^4\,mm^4$

Try 90mm x 12m of 2 flats

(I_s) provided $= \frac{12 \times 193^3}{12} - \frac{12 \times 13^3}{12}$

Section is Ok

Check for outstand

Outstand of the stiffner $\leqslant 13.7 t_s \varepsilon$

$13.7 t_s \varepsilon = 13.7 \times 13 \times 1.04 = 185.224$

Outstand = 90mm $(90 < 185.224)$

Section is ok

Buckling check

Stiffener force . $f_q = V - V_s$

V = shear force

V_s = V_{cr} of web

where

$V_{cr} = q_{cr}\, d t = 32.9 \times 3000 \times 13 \times 10^{-3} = 1283.1 kN$

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

Stiffener force, $f_q = 2062.5 - 1283.1 = 779.4$

Buckling resistance at intermediate stiffener at B

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

$I_x = \frac{1}{12} \times 12 \times 193^3 + \frac{520 \times 13^3}{12} - \frac{12 \times 13^3}{12} = 728 \times 10^{2} mm^4$

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

$L_e = 0.7 \times 3000 = 1680$

$L = \frac{2100}{27} = 77.8$

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

Intermediate stiffener at D (stiffener subjected to external load)

Use 2 flats 90mm x 12mm thick

Buckling check:-

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

$f_q = V - V_s \quad V = 1585.5kN$

$V_s - V_{cr} = q_{cr}dt = 32.9\times 3000\times 10^{-3}\times 13 = 1283.1kN$

$F_q = 231.42$

$M_s = 0$

$F_x = 870kN$

$$\frac{231.42 - 870}{250} + \frac{870}{15855} \leq 1$$

$= 2.0 \leq 1$

$\therefore$ Section is OK

Use flats 90mm x 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 web:

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

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

$f_{ed} < P_{ed}$

Web is ok