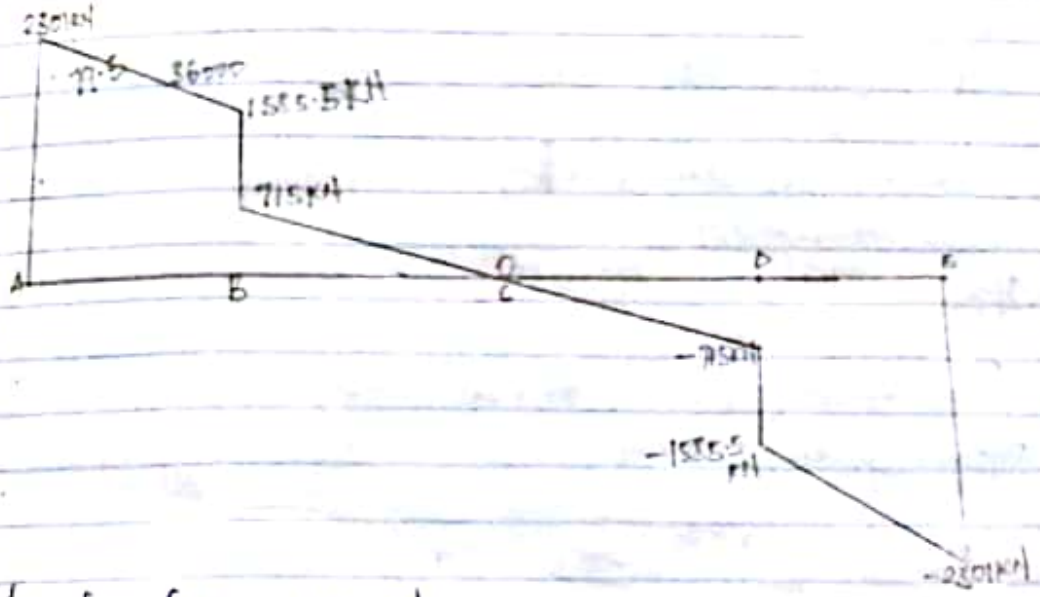
17/SC14/013
CIVIL ENGR.
IRIHANE, DANIEL TELUWASE

Using load factors 1.35Gk + 1.5Qk
UDL = 20×1.35 + 1.5×35 = 79.5kN/m
Point load (i) = 200×1.35 + 400×1.5 = 870kN
(ii) = 200×1.35 + 400×1.5 = 870kN

SF & BM
moment about (A) = 0
0 = (870×7) + (79.5×36)/2 × 18) + (870×2) − 36E
E = 2301kN ... A + ((79.5×36) + 870 + 870) − 2301
A = 2301kN

Shear Force
@A = 2301kN B' = 2301 − (79.5×7) = 1585.5kN
@B = 1585.5 − 870 = 715.5kN C = 715.5 − (79.5×7) = 0
@D = 0 − (79.5×7) = −715.5kN D = −715.5 − 870 = −1585.5kN
@E' = −1585.5 − (79.5×7) = −2301kN @E = −2301 + 2301 = 0



Max shear force = 2301kN
Max BM = (2301 × 18) − (79.5 × 15²/2) − (870×7) = 20707 kN/m²

Girder Section
D = span/15/10/12 using 15 = 36000/15
→ 2400mm

flang Py = 250/1.15 = 217.4N/mm²

Single flange area
Af = Mmax/d·fy = 20707×10⁶/(2400×217.4) = ~~3467~~ 39670.7mm²

Assume thickness of d = 60mm
Breadth of girder = 2400×0.3 = 720
Using 720mm × 60mm = 43200mm² (Area)
Web: Using 13 as thickness of web
Web size = 2400×15mm

Section
flange:-

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

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

$$\frac{b}{t} = \frac{353.5}{60} = 5.87 < 7.9\varepsilon$$

flange is plastic

Web

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

Checking for shear buckling
Web is Serviceability

$$d/250 = 2400/250 = 9.6mm < t$$

check is OK

flange ~~buffing~~ buckling into web (check)
stiffness spacing, a, > 1.5d

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

t = 13mm > 7.6mm

check is OK

Moment Capacity

$$BT(d+T)Py$$

$$= \frac{720\times60(2400+60)\cdot217.4}{10^6} = 23103.5kN/m > M_{max}$$

Check is OK

Web Design

End Panel (AB) design

$d = 2400 \quad t = 15mm \qquad a/d = \frac{3000}{2400} = 1.25$

$d/t = \frac{2400}{15} = 184.6$

Shear Strength

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

Elastic Stress, q_e (when $a/d > 1$) $= \left(1 + \frac{0.75}{(a/d)^2}\right)\left(1000(d/t)\right)^2$

$= \left(1 + \frac{0.75}{(1.25)^2}\right)(1000/184.6)^2 = 43.4 N/mm^2$

Slenderness $= \left(0.6\left(\frac{f_{yw}}{\gamma_m}\right)/q_e\right)^{0.5}$

$= \left(0.6\left(\frac{250}{1.15}\right)/50.4\right)^{0.5} = 1.61 > 1.25$

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

$R_v = \frac{f_{vA}}{dt} = \frac{2301 \times 10^3}{2400 \times 4} = 68.5 N/mm^2$

$R_v > q_{cr}$

Tension Field Action Design

Using Panel AB

Basic shear strength (q_b)

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

$y_b = \left(P_{yw}^2 - 3q_{cr}^2 + q_t^2\right)^{0.5} - q_t$

$= \left(217.4^2 - 3(50.4)^2 + 47.2^2\right)^{0.5} - 47.2 = 157.4$

$q_b = q_{cr} + \frac{y_b}{2\left(\frac{a}{d} + \sqrt{1 + \left(\frac{a}{d}\right)^2}\right)} = 50.4 + \frac{157.4}{2\left(1.25 + \sqrt{1 + (1.25)^2}\right)} = 78 N/mm^2$

$q_b > R_v$

AB is OK against buckling

* Web Panel Checks

* Checks for shear capacity of end panel

$H_q = 0.75 d_t \cdot P_y \left(1 - \frac{\tau_{cr}}{0.6 P_y}\right)^{0.5} \left(\frac{\tau_b - \tau_{cr}}{\tau_b - \tau_{cr}}\right)$

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

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

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

$R_v = 0.6 P_y \cdot A_v = 0.6 \times 217.4 \times 31 = 5087.16\,kN$

$R_H < R_v$

* Moment Capacity Checks

$M_H = \frac{H_q d}{10} = \frac{2814 \times 2400}{10} \times 10^{-3} = 675.4\,kN/m$

$y = d/2 = 3000/2 = 1500$

$I = 1/12\, t d^3 = 1/12 \times 14 \times 3000^3 = 3150 \times 10^7\,mm^4$

$M_q = \frac{I}{y} P_y = \frac{3150 \times 10^7}{1500} \times (217.4 \times 10^{-6}) = 456.5\,kN/m$

$M_H < M_q$

Section AB is OK

Design of Stiffners

Load Bearing On Stiffner A

max SF = 2301 kN

force (F_m) due to moment M_H = $F_m = \frac{M_H}{d} = \frac{675.4}{3000} \times 10^3 = 225\,kN$

* Total Compression = $F_c = F_b + F_m = 2301 + 225 = 2526\,kN$

* Area of Stiffner in contact with the flange ; A

$A > \frac{0.8 F_c}{P_y}$

$\Rightarrow \frac{0.8 \times 2526}{217.4} \times 10^3 = 9275\,mm^2$

Try stiffner size of 2 flats of size 240x25mm thick

Allowing 15mm to for weld

$A = 225 \times 25 \times 2 = 11250\,mm^2 > 9275\,mm^2$

Bearing check is OK

* Check for ~~outstand~~ outstands

$\varepsilon = 1.04$

outstands b/s = 240mm < $20t\varepsilon = (20 \times 25 \times 1.04) = 520$

b/s = 240mm < $13.7t\varepsilon = 13.7 \times 2.5 \times 1.04 = 356.2$

Satisfied

* Checking Stiffners for buckling

$I_x = \frac{25 \times 473^3}{12} - 1/12 \times 25 \times 13^3 = 24968 \times 10^4 \, mm^4$

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

$r_x = \left(\frac{I_x}{A_e}\right)^{0.5} = \left(\frac{24968 \times 10^4}{12000}\right)^{0.5} = 144.2mm$

Since flange is restrained against rotation

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

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

* Buckling Resistance ~~for~~ of stiffner

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

$R_c < P_c$ (2526 < 2609)

Stiffners is safe against buckling

* Checking Stiffner A as a ~~bearing~~ bearing stiffner

Load capacity of Web

Assume stiff bearing length $b_1 = 0$

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

$P_{cap} = (b_1 + n_2) t_w p_{yw}$

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

* Bearing Stiffner for F_A

$F_A = F_c - P_{cap} = 2526 - 848 = 1678KN$

Bearing Capacity of Stiffner alone

$P_a = p_y \times A = 217.4 \times 12000 \times 10^{-3} = 2609KN$

$F_A < P_a$ (1678 < 2609)

Designed stiffner is OK in bearing

Stiffner A = 2flats 240mm X 25mm thick

* Design for Intermediate stiffner at B

min. stiffness

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

$I_s \geq \frac{1.5 d t^3}{a^2}$ for $a < d\sqrt{2}$

$d\sqrt{2} = 3394mm$

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

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

Try 70mm x 12mm Of 2 flats

(Ie) Provided $= \frac{12 \times 173^3}{12} - \frac{12 \times 13^3}{12} = 730 \times 10^4 mm^4$

Section is OK

* Check for Outstand

Outstand of stiffner $\leq 13.7 k\varepsilon$

$13.7 k\varepsilon = 13.7 \times 6 \times 1.04 = 155.224$

Outstand = 90mm (90 < 155.224)

Section is OK

* Buckling Check

Stiffner force. $f_q = V - V_s$

V = shear force

$V_s = V_{cr}$ at web

$V_{cr} = \tau_{cr} dt = 43.4 + 2400 \times 13 \times 10^{-3} = 1354.08 kN$

Shear force at B, $V_B = 2301 - ((2301 - 1555.5) \times \frac{3000}{4000}) = 2062.5 kN$

Stiffner force $f_q = 2062.5 - 1354.08 = 708.42$

* Buckling Resistance at intermediate stiffner at B

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

$I_x = 1/12 \times 12 \times 173^3 + \frac{520 \times 13^3}{12} - \frac{12 \times 13^3}{12} = 728 \times 10^4 mm^4$

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

$l_e = 0.7 \times 2400 = 1680$

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

$= (182.3/1.15) \times 10000 \times 10^{-3} = 1555 kN$

* Intermediate Stiffener at D (stiffener subjected to external load)

Use 2 flats 110mm × 12mm thick

Buckling check :-

$$\frac{f_q}{P_q} + \frac{f_x}{P_x} + \frac{M_s}{M_{ys}} \leq 1$$

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

$V_s = V_{cr} = \tau_{cr}\,dt = 45.4 \times 2400 \times 14 \times 10^{-3} = 1354.08\,kN$

$f_q = 231.42$

$M_s = 0 \quad f_x = 870\,kN$

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

$-2.0 \leq 1 \quad \therefore$ Section is OK

Use flats 70mm × 12mm thick

Web check between stiffeners

$f_{ed} \leq P_{ed}$

$f_{ed} = W/t = \frac{775}{13} = 6\,N/mm^2$

When the compression flange is restrained against rotation relative to the web

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

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

$f_{ed} < P_{ed} \therefore$ Section is OK