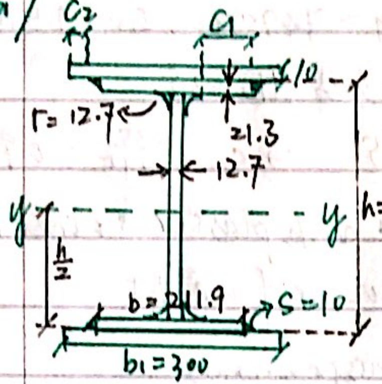
CV3012
S2 1617
April/May 2017

1.



A beam under in-plane bending about y-y axis.

(a) Beam flange outstand c_1 (Compression)

$$\frac{c_1}{t\varepsilon} = \frac{(211.9 - 12.7 - 2\times 12.7)/2}{(10+21.3)\times 0.92} = 3.02 < 9$$

$\Rightarrow$ Class 1 #

533×210 UB122

$S275 \Rightarrow \varepsilon = \sqrt{\frac{235}{275}} = 0.92$

Compound plate outstand c_2 (Compression)

$$\frac{c_2}{t\varepsilon} = \frac{(300-211.9-2\times 10)/2}{10\times 0.92} = 3.70 < 9$$

$\Rightarrow$ Class 1 #

(b) Web internal part (Bending)

$$\frac{c}{t\varepsilon} = \frac{544.5 - 2\times 21.3 - 2\times 12.7}{12.7\times 0.92} = 40.8 < 72$$

$\Rightarrow$ Class 1 #

(c) Plastic modulus $W_{pl,y}$

$I_y' = I_y + A(d_y)^2$

$$I_{y,total} = I_{y,I} + I_{y(plates)}$$
$$= 76000\times 10^4 + 2\times\left[\frac{1}{12}300\cdot 10^3 + 300\cdot 10\cdot\left(\frac{544.5}{2}+\frac{10}{2}\right)^2\right]$$
$$= 76000\times 10^4 + 46125\times 10^4$$
$$\approx 1.2\times 10^9 \text{ mm}^4$$

$$W_{pl,y} = \frac{I_{y,total}}{y_{max}} = \frac{1.2\times 10^9}{\frac{544.5}{2}+10} = 4.3\times 10^6 \text{ mm}^3 \quad \#$$

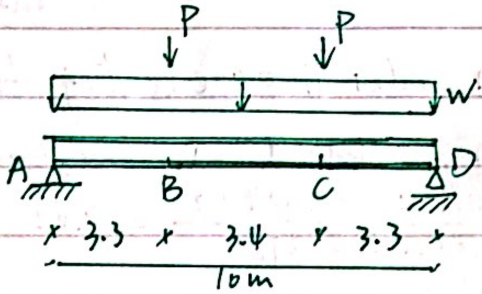
$A = 303\,cm^2$ $h = 635.8$ $b = 311.4$ $t_w = 18.4$ $t_f = 31.4$

$r = 16.5.$ $C_f/t_f = 4.14$, $C_w/t_w = 29.3$

$I_y = 209000\,cm^4$, $I_z = 15800\,cm^4$

$W_{pl,y} = 7490\,cm^3$, $I_w = 14.5\,dm^6$, $I_T = 785\,cm^4$

2.

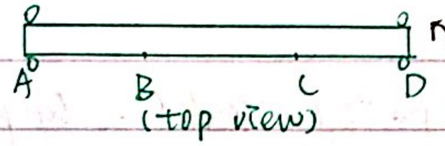


610×305 UB 238

S275 $\Rightarrow \varepsilon = \sqrt{\frac{235}{275}} = 0.92$

Beam with plaster & other brittle finish

neglect self weight

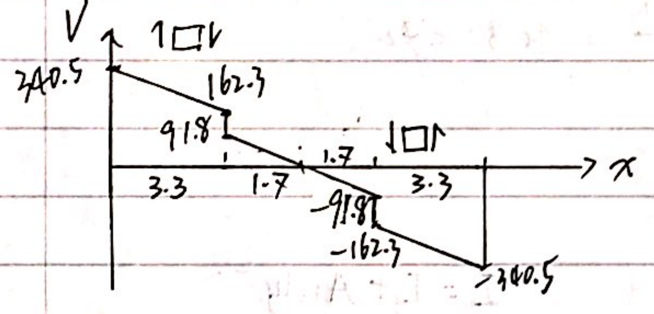


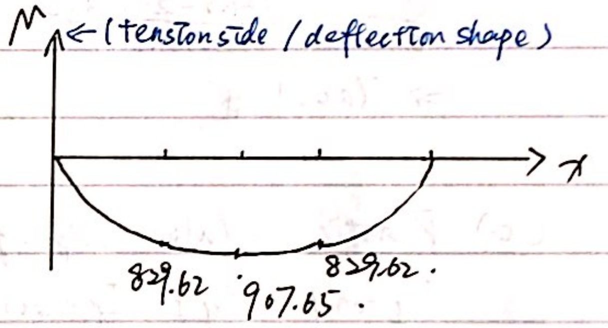
lateral restraints (e.g. forks).

Ultimate Limit State Design:

Factored actions: $P = 1.35 \times 30 + 1.50 \times 20 = 70.5\,KN$

$W = 1.35 \times 20 + 1.35 \times 20 = 54\,KN/m$





$V_{Ed} = 340.5\,KN$ $M_{Ed} = 907.65\,KN \cdot m$.

Cross-section check: web (bending): $\frac{C_w}{t_w \varepsilon} = \frac{29.3}{0.92} = 31.8 < 72 \Rightarrow$ Class 1

flange (bending): $\frac{C_f}{t_f \varepsilon} = \frac{4.14}{0.92} = 4.5 < 9 \Rightarrow$ Class 1

Shear [EN1993-1-1 6.2.6]

Rolled I section; load parallel to web

$A_v = A - 2bt_f + (t_w + 2r)t_f \geq \eta h_w t_w = 1.0 \times (635.8 - 2\times31.4 - 2\times16.5) \times 18.4 = 9936.$ (SG Annex)

$= 303 \times 10^2 - 2 \times 311.4 \times 31.4 + (18.4 + 2\times16.5) \times 31.4$

$= 12358\,mm^2 > 9936\,cm^2.$

$V_{pl,Rd} = \frac{A_v (f_y/\sqrt{3})}{\gamma_{M0}} = \frac{12358 \times 275/\sqrt{3}}{1.0} = 1962\,KN > 340.5\,KN$ Sufficient #

(SG Annex)

[EN1993-1-1 6.2.6(6)] Unstiffened section: $\frac{h_w}{t_w} = \frac{635.8 - 2\times31.4 - 2\times16.5}{18.4}$

$= 29.3$

$72\frac{\varepsilon}{\eta} = 72 \cdot \frac{0.92}{1.0} = 66.24$ ← conservative

Hence, shear buckling resistance doesn't need to be taken into account.

Bending and shear [EN1993-1-1 6.2.8]

$V_{Ed} = 340.5\,KN < 981\,KN = \frac{1}{2} V_{pl,Rd}$

Hence, the effect of shear on moment resistance may be neglected.

Bending moment [EN1993-1-1 6.2.5]

Class 1 cross-section

$M_{c,Rd} = M_{pl,Rd} = \dfrac{W_{pl} \cdot f_y}{\gamma_{m0}} = \dfrac{7490 \times 10^3 \times 275}{\underset{\uparrow}{1.0}} = 2059.8\,KN \cdot m$

SG Annex

$> 907.65\,KN \cdot m$

sufficient #

Lateral torsional buckling

$L_{cr} = L = 10m$

[EN1999-1-1 Annex I.1.1(1)] gives guidelines for finding critical moment M_{cr}.

$$M_{cr} = \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{L^2 G \cdot I_t}{\pi^2 E \cdot I_z} + \frac{I_w}{I_z}}$$

$$= \frac{\pi^2 \cdot 210{,}000 \cdot 15800 \times 10^4}{10{,}000^2} \cdot \sqrt{\frac{10{,}000^2 \cdot 81000 \cdot 785 \times 10^4}{\pi^2 \cdot 210{,}000 \cdot 15800 \times 10^4} + \frac{14.5 \times 10^{12}}{15800 \times 10^4}}$$

$= 1751\,KN \cdot m$

[EN1993-1-1 6.3.2.2]

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{7490 \times 10^3 \times 275}{1751 \times 10^6}} = 1.08$$

Imperfection factor:

rolled I section: $\dfrac{h}{b} = \dfrac{635.8}{311.4} = 2.04 > 2 \Rightarrow$ buckling curve b

$\alpha_{LT} = 0.34$

$$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2]$$

$= 0.5 [1 + 0.34 (1.08 - 0.2) + 1.08^2]$

$= 1.2328$.

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}}$$

$$= \frac{1}{1.2328 + \sqrt{1.2328^2 - 1.08^2}}$$

$$= 0.547 < 1.0 .$$

~~Ab~~.

[EN 1993-1-1 6.3.2.1]

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}}$$

$$= 0.547 \times 7490 \times 10^3 \cdot \frac{275}{1.0}$$ ← SG Annex

$$= 1127 \text{ KN}\cdot\text{m} . > 907.65 \text{ KN}\cdot\text{m} \Rightarrow \text{won't buckle} \ \#$$

Serviceability Limit State Design.

Factored actions: $P = 0 \times 30 + 1.0 \times 20 = 20 \text{ KN}$

$W = 0 \times 20 + 1.0 \times 20 = 20 \text{ KN/m}$

For deflection at the mid length of the beam, superposition of loads.

. P: $d_P = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$ ← from the shear, moment, deflection table or similar

$$= \frac{20 \times 10^3 \times 10{,}000^3}{48 \times 210{,}000 \times 20900 \times 10^4}\left[\frac{3 \times 3.3}{10} - 4\left(\frac{3.3}{10}\right)^3\right]$$

$$= 8.03 \text{ mm}.$$

$$W: d_W = \frac{5}{384}\frac{WL^3}{EI}$$

$$= \frac{5}{384} \cdot \frac{20 \frac{10^3 N}{10^3 mm} \times 10{,}000^3}{210{,}000 \times 20900 \times 10^4 \frac{N}{mm^2} \times mm^4}$$

$$= 5.93 \times 10^{-3} \text{ mm}.$$

$$d_{total} = 2d_P + d_W = 2 \times 8.03 + 5.93 \times 10^{-3} = 16.06 \text{ mm} \ \#$$

S275 $\varepsilon = 0.92$ $I = 13300\ cm^4$ $i = 9.5\ cm$ $A = 147\ cm^2$

3. 250×250×16 SHS $c/t = 12.6$ $W_{pl} = 1280\ cm^3$

Cross-sectional check

Internal part (compression): $\frac{c}{t \cdot \varepsilon} = \frac{12.6}{0.92} = 13.7 < 33 \Rightarrow$ Class 1

(bending): $\frac{c}{t \cdot \varepsilon} = 13.7 < 72 \Rightarrow$ Class 1

(a) Action (Designed)

$C_y = B_y = \frac{1}{2} \cdot 80 \cdot 3.2 = 128\ KN$

$N_{Ed,1} = B_y = 128\ KN$ $N_{Ed,2} = 2300\ KN$

$N_{Ed} = 128 + 2300 = 2428\ KN$

$M_{y,Ed} = N_{Ed,1} \times (100 + \frac{h}{2})$

$= 128 \times (100 + \frac{250}{2})$

$= 28.8\ KN \cdot m$

(b) Effective length $L_{cr} = 0.7L = 0.7 \times 6 = 4.2m$ (fixed + pinned)

Flexural buckling

[EN1993-1-1 6.3.1]

(Eq 6.50) $\bar{\lambda} = \frac{L_{cr} \times 1}{i \times \lambda_1} = \frac{L_{cr}}{i \cdot 93.9\varepsilon} = \frac{4.2 \times 10^3}{9.5 \times 10 \times 93.9 \times 0.92} = 0.5117$

(Table 6.2) Hollow section + hot finished + S275 $\Rightarrow$ buckling curve a

(Table 6.1) Imperfection factor $\alpha = 0.21$

(Eq. 6.49) $\phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2]$

$= 0.6636$

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = 0.92 < 1.0$

(Eq. 6.47) Class 1 $N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0.92 \times 147 \times 10^2 \times 275}{1.0 \text{ (SG Annex)}} = 3721\ KN$

$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2428}{3721} = 0.6525 < 1.0$

Lateral torsional buckling

[EN1993-1-1 6.3.2.1 (2)] square & hollow sections are not susceptible to LTB.

(Eq. 6.13) $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = \frac{1280 \times 10^3 \times 275}{1.0 \text{ (SG Annex)}} = 352 \text{ kN·m}$

$\frac{M_{Ed}}{M_{b,Rd}} = \frac{M_{Ed}}{M_{c,Rd}} = \frac{28.8}{352} = 0.0818 < 1.0$.

Simplified method for the buckling check for combined bending + axial compression.

$\frac{N_{Ed}}{N_{b,min,Rd}} + 1.0 \frac{M_{y,Ed}}{M_{b,Rd}} + 1.5 \frac{M_{z,Ed}}{M_{c,z,Rd}}$

$= \frac{N_{Ed}}{N_{b,Rd}} + 1.0 \frac{M_{y,Ed}}{M_{c,Rd}} + 0$

$= 0.6525 + 1.0 \times 0.0818$

$= 0.7343 < 1.0$. $\Rightarrow$ The section is satisfactory.

(c) $L_{cr} = L = 6$ m. (pinned-pinned)

Same procedure as part (b)

$\bar{\lambda} = 0.7311 \quad \phi = 0.823 \quad \chi = 0.8326 < 1.0$

$N_{b,Rd} = 3366$ kN

$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2428}{3366} = 0.7213 < 1.0$, $\frac{M_{y,Ed}}{M_{b,Rd}} = 0.0818$ (unchanged)

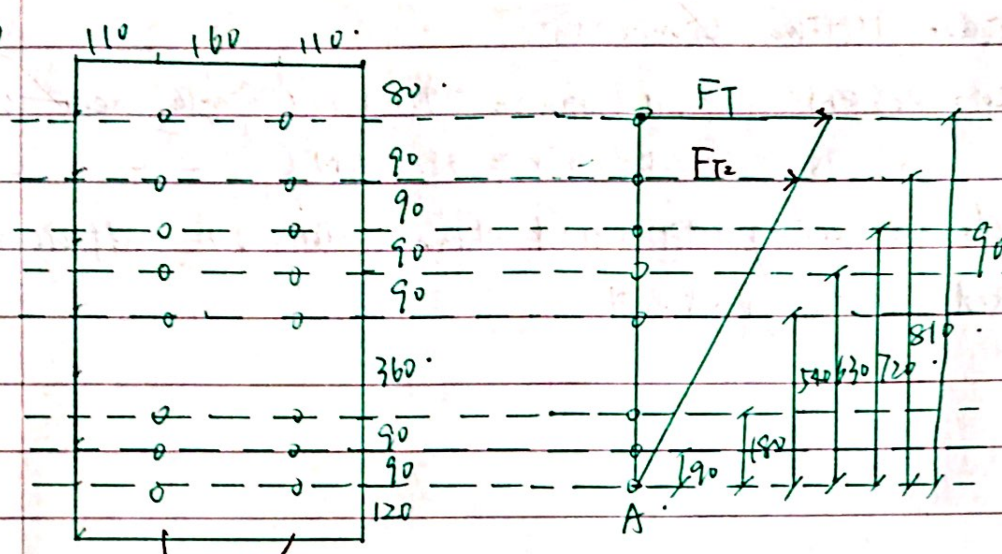
$\frac{N_{Ed}}{N_{b,Rd}} + 1.0 \frac{M_{y,Ed}}{M_{b,Rd}}$

$= 0.7213 + 0.0818$

$= 0.8031 < 1.0$ $\Rightarrow$ adequate.

4 $M_{Ed} = 1200\ kN \cdot m$ $\quad \underset{\downarrow}{\text{shear}}\ F_{Ed} = 380\ kN$

(a)



Similar triangle

$\frac{F_{T2}}{F_T} = \frac{y_2}{y_1}$

$F_{T2} = \frac{y_2}{y_1} F_T$

$M_2 = F_{T2} \times y_2 = F_T \cdot \frac{y_2^2}{y_1}$

$\boxed{\text{Tension}}$ $M_R = 2 \times (F_T \times \frac{y_1^2}{y_1} + F_T \times \frac{y_2^2}{y_1} + \cdots +)$

$= \frac{2 F_T \Sigma y^2}{y_1}$

$= 2F_T \frac{900^2 + 810^2 + 720^2 + 630^2 + 540^2 + 180^2 + 90^2}{900}$

$= 2 \times F_T \times 3015$

$= 6030\ F_T$

Let $M_R = M_{Ed} = 1200\ kN \cdot m$

$\Rightarrow F_T = \frac{M_{Ed}}{6030} = \frac{1200 \times 10^3}{6030} = 199.005\ kN$ #

$\boxed{\text{Shear}}$ $F_S = \frac{F_{Ed}}{\underset{\substack{\uparrow \\ \text{No. of bolts}}}{8 \times 2}} = \frac{380}{16} = 23.75\ kN$ #

(b) M24 Class 8.8 non-preloaded S275

[BS-EN-1993-1-8:2005]

$A_s = 353\ mm^2$ $\quad F_{t,Rd} = 203\ kN$ $\quad F_{v,Rd} = 136\ kN$ (Single)

On one bolt: ∴ maximum tension at the top bolt (one of the two bolts at the top).

$\boxed{\text{Tension}}$ $F_{t,Rd} = 203\ kN > \frac{199.005}{2} = 99.5\ kN$. $\Rightarrow$ OK.

single bolt

$\boxed{\text{Shear}}$ $F_{v,Rd} = 136\ kN > 23.75\ kN$ $\Rightarrow$ OK.

$\boxed{\text{Combined}}$ $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 F_{t,Rd}} = \frac{23.75}{136} + \frac{99.5}{1.4 \times 203} = 0.524 < 1.0 \Rightarrow$ OK

(c) M24 Class 8.8 preloaded.

Consider Ultimate Limit State.

$199.05 < F_{t,Rd} = 203$ kN $F_{v,Rd,max} = 79.1$ kN (Single shear $\mu = 0.5$)

$23.75 < F_{v,Rd,min} = 31.6$ kN (- - - - $\mu = 0.2$)

The design bending ~~actions~~ and shear actions are sufficiently resisted at the joint. #

* If you have any questions regarding the solution, do not hesitate to contact me @ liuchang0626@gmail.com. All the best for your exams !

Chang :)