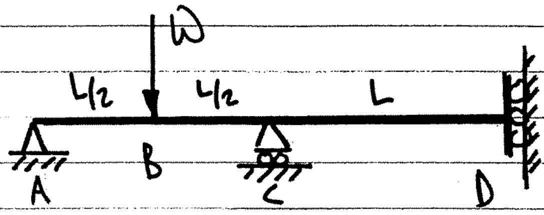
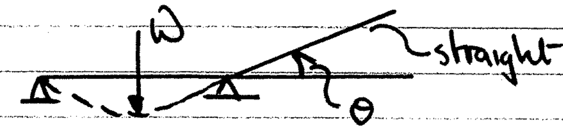
Qu 1 4D10/2003

a)



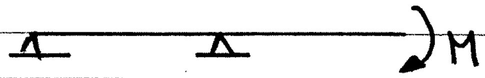
Moment @ D is the redundancy: tackle by deflection coefficients (struct. data bk).

Remove M.

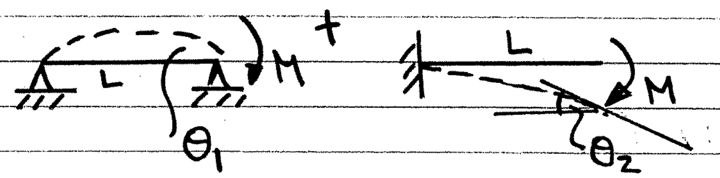


$$\boxed{\theta = \frac{WL^2}{16EI}}$$

Restore M.



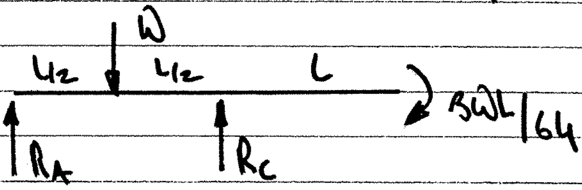
=



$$\theta_1 + \theta_2 = \frac{ML}{3EI} + \frac{ML}{EI} \Rightarrow \boxed{\theta_1 + \theta_2 = \frac{4}{3}\frac{ML}{EI}}$$

By compatibility; $\theta = \theta_1 + \theta_2 \Rightarrow \frac{WL}{16} = \frac{4}{3}M \Rightarrow \boxed{M = \frac{3WL}{64}}$

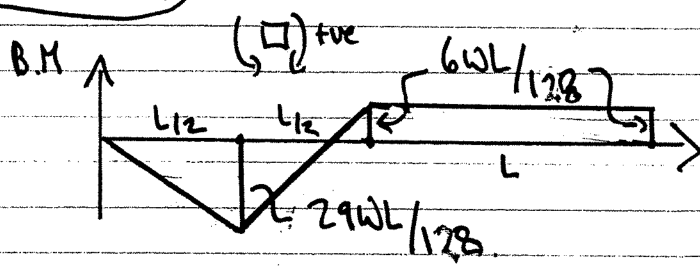
Reactions:



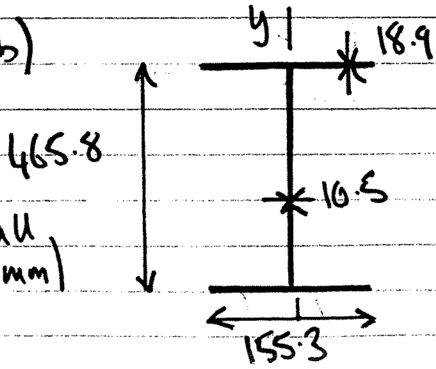
$R\uparrow$. $R_A + R_C - W = 0$: $M \circlearrowright A \Rightarrow LR_C - \frac{WL}{2} - \frac{3WL}{64} = 0$

$\Rightarrow \boxed{R_C = 35W/64,\ R_A = 29W/64}$

Bending moment diagram:



b)



457×152×82 UB. (struct data book).

$I_{yy} = 1185\,cm^4$, $J = 89.2\,cm^4$, $Z_p = 1811\,cm^3$

$G = 81\,GPa$, $E = 205\,GPa$. $\sigma_y = 275\,MPa$

Total span = 20m $\Rightarrow$ $\boxed{L = 10m}$ P.T.O.

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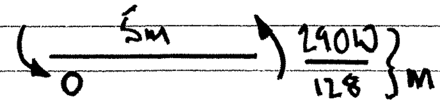
Tackle by DS2: there are two critical span lengths $L = 5\,m$ (AB, BC) and $L = 10\,m$ (CD). Must evaluate

$$M_E = \left[M_1^2 + M_2^2\right]^{1/2} \text{ with } M_1 = \frac{\pi}{L}\left[GJEI_{yy}\right]^{1/2}, \quad M_2 = \frac{\pi^2}{L^2} ED \underbrace{I_{yy}/2}_{\sim I_{flange}}$$

$$D = 465.8 - \underbrace{18.9}_{\text{flange thickness}} = 446.9 \text{ mm (depth to mid-flanges).}$$

	$L = 5\,m$	$L = 10\,m$
M_1 (kNm)	263.2	131.6
M_2 (kNm)	214.3	53.6
M_E (kNm)	339.4	142.1
$M_p = Z_p \sigma_y$ (kNm)	498.0	498.0.
$\lambda_{LT} = 75\sqrt{M_p/M_E}$	90.8	140.4
$\bar{M}_c$ (DS2)	~ 0.53	~ 0.25

~ to level of accuracy of measurement.

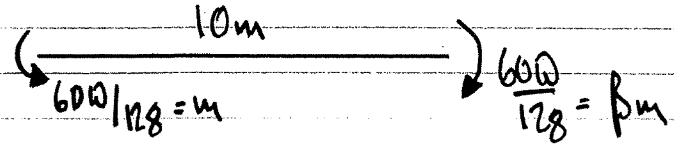
AB) 

[N.B. bending mom = $\frac{290WL}{128}$ ~ 10m, not 5!]

$\beta = 0 \Rightarrow M_u$ (equiv.-unif. mom) $= 0.6\,m$ (DS2) $\Rightarrow M_u = \frac{174W}{128}$

Strength check: $m \leq M_p \Rightarrow \frac{290W}{128} \leq M_p \Rightarrow \underline{W \leq 219.8\,kN}$

Stability check: $M_u \leq M_c \Rightarrow \frac{174W}{128} \leq \underbrace{0.53M_p}_{M_c \text{ for } L=5m \text{ above}} \Rightarrow \underline{W \leq 194.2\,kN}$

CD) 

$\beta = 1 \Rightarrow M_u = m$ (DS2)

$M_u = 60W/128$

Strength check: $m \leq M_p \Rightarrow \frac{60W}{128} \leq M_p \Rightarrow \underline{W \leq 1062.4\,kN}$

Stability check: $M_u \leq M_c \Rightarrow \frac{60W}{128} \leq \underbrace{0.25M_p}_{M_c \text{ for } L=10 \text{ above}} \Rightarrow \underline{W \leq 265.6\,kN}$

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BC) $\overset{5\text{m}}{\longleftrightarrow}$ $\frac{2900W}{128} = m$, $\frac{60W}{128} = \beta m$ $\Rightarrow -\frac{60W}{128} = \beta \cdot \frac{2900W}{128} \Rightarrow \beta = -\frac{6}{29}$

$\Rightarrow$ $\beta = -0.21$ [-ve sign required since both moments in same sense, opposite to convention in DS2]

DS2 $\Rightarrow$ $M_u = [0.6 + 0.4\beta]m = 0.516m$

Strength check: $m \leq M_p \Rightarrow 2900W/128 \leq M_p \Rightarrow \underline{W \leq 219.8\text{kN}}$

Stability check: $M_u \leq M_c \Rightarrow 0.51 . \frac{2900W}{128} \leq 6.53M_p \Rightarrow \underline{W \leq 228.4\text{kN}}$

M_c for $l = 5$.

Summary: member	W (strength)	W (stability)	
AB	219.8	194.2*	kN
BC	219.8	228.4	kN
CD	1062.4	265.6	kN

*lowest W $\Rightarrow$ $W_{MAX} = 194.2$ kN governed by stability of AB

Shear check: no S.F. in CD (constant moment); the greatest change in bending moment is in BC, where

Shear force, $V_1 = \left[\frac{(29+6).W \times 10}{128}\right] / 5 = \underline{106.2\text{kN}}$

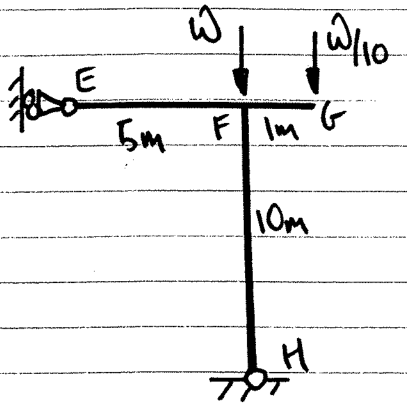
$V_c = A_{web} \times \underbrace{\tau_y}_{\sigma_y/\sqrt{3}} = \underbrace{(0.465 \times 0.0105)}_{A_{web},\ m^2} \times \underbrace{275 \times 10^6}_{\sigma_y,\ Pa} = \underline{775.2\text{kN}}$

Thus, $V \ll V_c/2$ [The condition for using M_p in the above analysis]

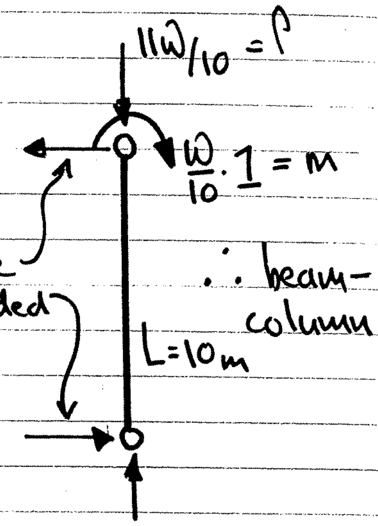
KAS.

Qu 2 4D10/2003

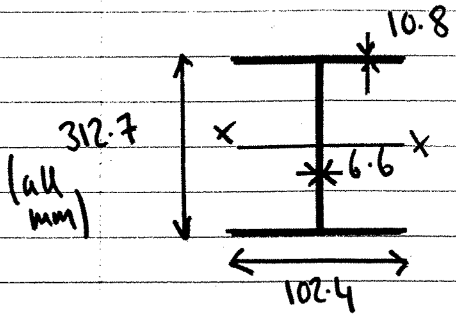
a) Bookwork: imperfection, residual stresses (hot-rolling / welding), section shape effects (y/r effects).

b) 

System is STATICALLY DETERMINATE ⇒



∴ beam-column



$305 \times 102 \times 33$ UB ; major axis bending

struct data book $\left\{ \underline{r_{xx} = 12.5\,cm}, \; \underline{Z_p = 481\,cm^3}, \; \underline{A = 41.8\,cm^2} \right.$

$\underline{\sigma_y = 355\,MPa}, \; \underline{y = \frac{312.7}{2}\,mm}$ (extreme fibre dist).

Compactness: $\lambda_{flange} = \overbrace{\left(\frac{102.4 - 6.6}{2}\right)}^{\text{outstand width}} / 10.8 = \underline{4.4}$ (<8, OK).

$\lambda_{web} = \frac{\overbrace{(312.7 - 2\times 10.8)}^{\text{inside web depth}}}{6.6} = \underline{44.1}$ (<56, OK to use CDC)

P_p (squash load) $= A\sigma_y = \underbrace{41.8\times 10^{-4}}_{m^2} \times \underbrace{355\times 10^6}_{\sigma_y} = \underline{1484\,kN}$

$M_p = Z_p\sigma_y = \underbrace{481\times 10^{-6}}_{m^3} \times 355\times 10^6 = \underline{170.8\,kNm}$

Given $W = 750\,kN$ ⇒ $\underline{\text{axial load } P = 825\,kN}$, $\underline{m = 75\,kNm}$

$r/y = 125/\left(\frac{312.7}{2}\right) = \underline{0.80} > 0.7$ ⇒ $\underline{\text{curve A, BS1}}$ (hot-rolled)

$\sigma/\sigma_c = P/P_p = 825/1484 = 0.56$ ⇒ $\underline{\lambda \simeq 91}$

For CDC, $\lambda = \frac{L_c}{r}\sqrt{\sigma_y/355}$ ⇒ $91 \times 0.125 = \underline{L_c = 11.375\,m}$

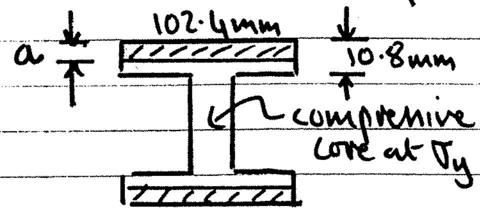
P.T.O.

Qu 2 4D10/2003

b) contd) $\Rightarrow L/L_c = 10/11.375 = \underline{\underline{0.88}}$: choose 0b major axis bending curve in DS3. (CDC method).

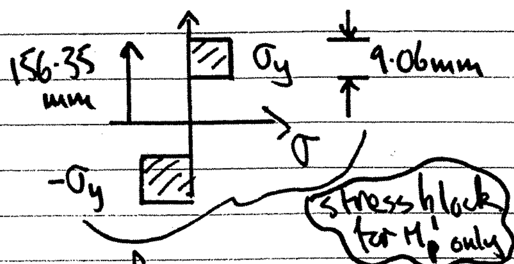
N.B. $\beta = 0$, since lower end is pinned $\Rightarrow$ $\underline{\underline{M_c/M_p' = 0.27}}$

Need estimate of M_p' : assume comp. core extends into flange.



$\underbrace{(P_p - P)}_{\text{axial force capacity lost to } M_p'}$ = shaded area = $2 \times (0.1024 \times a) \times \sigma_y$

$\Rightarrow \underline{a = 9.06\text{mm}}$ (<10.8, OK)



$$M_p' = 2 \times \overbrace{\left[0.1564 - \frac{0.0091}{2}\right]}^{\text{lever arm}} \times \sigma_y \times \overbrace{(0.1024 \times 0.0091)}^{\text{area}}$$

$\underline{M_p' = 100.5\text{kNm}}$.

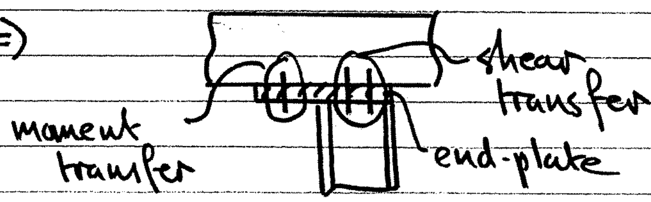
Recall $M_c = 0.27 M_p'$ $\Rightarrow$ $\underline{M_c = 27.1\text{ kNm}}$: HOWEVER, $m < M_c$ for safety.

but $m = 75\text{kNm}, > M_c \Rightarrow$ FAILS

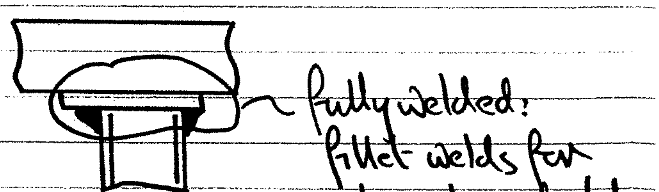
Cannot check shear with CDC (no need with failure)

c) Joint detail : full beam-to-column at F ;

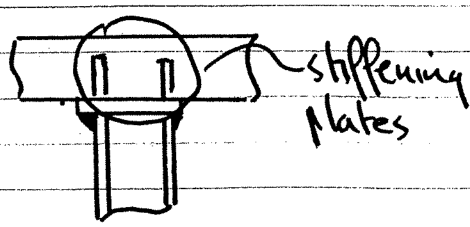
(i) bolted end-plate $\Rightarrow$

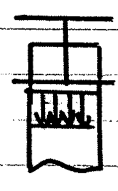


(ii) welded end-plate $\Rightarrow$



for a uniform bending stress transfer

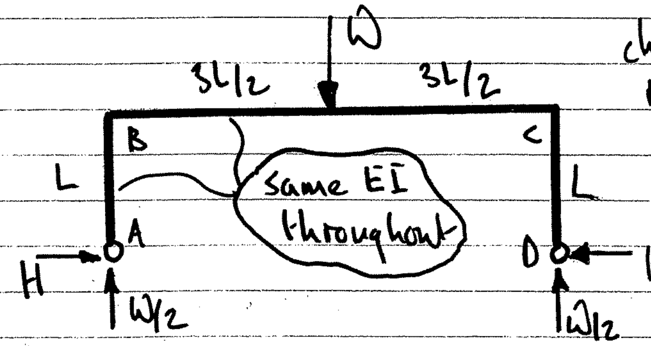




KMB.

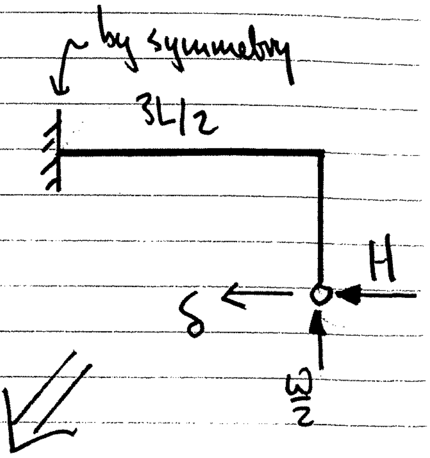
Qu 3 4D10/2003

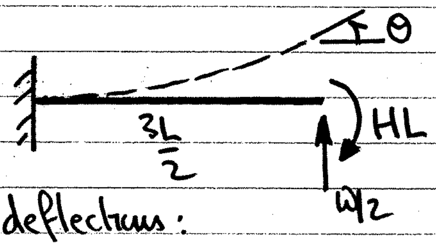
a)



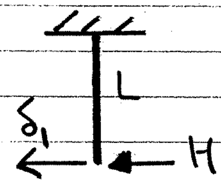
choose H as redundancy

$\Rightarrow$





+



Use data book deflections:

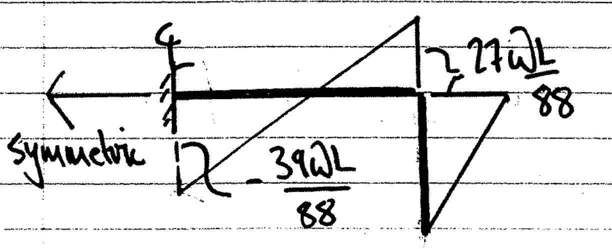
$$\theta = \left(\frac{W}{2}\right)\left(\frac{3L}{2}\right)^2 \Big/ 2EI - HL\left(\frac{3L}{2}\right)\Big/EI$$

$$\Rightarrow \quad \theta = \frac{9WL^2}{16EI} - \frac{3HL^2}{2EI} \quad ; \quad \delta_1 = \frac{HL^3}{3EI}$$

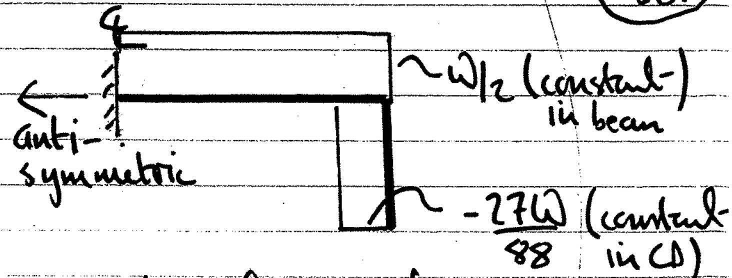
$$\delta(\text{total}) = \delta_1 - \theta L \Rightarrow \frac{H}{3} = \frac{9W}{16} - \frac{3H}{2} \Rightarrow H = \frac{27}{88}W$$

bending moment @ $C = 27WL/88$

bending moment @ midspan, M: $M(C) + \frac{3WL}{2 \cdot 2} - HL = 0 \Rightarrow M = -\frac{39WL}{88}$



bending moment (M +ve tension on outside of beam)



shear force

b) Before tackling question, note that for the web: $\sigma_y = 275\text{MPa}$

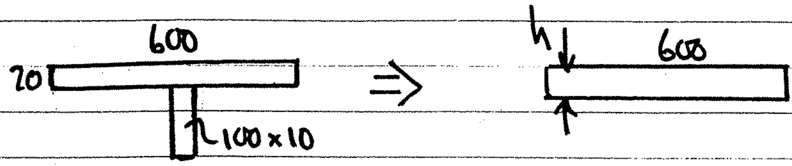
$$\lambda_{web} = b/t\sqrt{\sigma_y/355} = \frac{800}{20}\sqrt{\frac{275}{355}} = 35.2 \ (< 56 \text{ for bending})$$

∴ O.K. to use M_p, if required.

Qu3 4D10/2003

b)(contd) Need to find I_{xx}, Z_e and A for cross-section : estimate via use of SMEARED section:

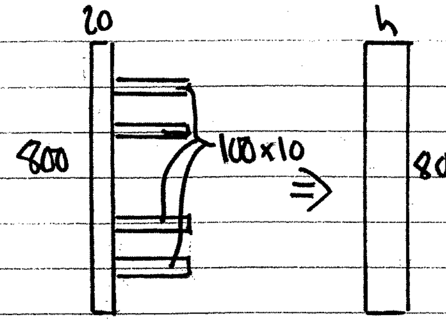
top flange)



$600h = 600 \times 20 + 100 \times 10$

$\boxed{h = 21.67\text{ mm}}$

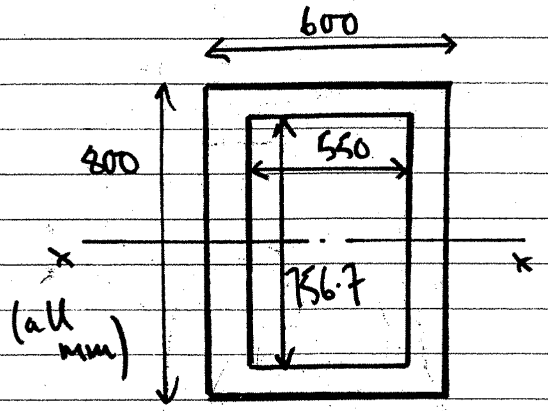
web)



$800h = 800 \times 20 + 4 \times [100 \times 10]$

$\Rightarrow \boxed{h = 25\text{ mm}}$

Smeared section:



I_{xx} : consider "$1/12\, bd^3$" approach

$$I_{xx} = 1/12\left[600 \times 800^3 - 550 \times 756.7^3\right] = \underline{5741 \times 10^6\text{ mm}^4}$$

$$A = 600 \times 800 - 550 \times 756.7 = \underline{63815\text{ mm}^2}$$

$$Z_e = I_{xx}/y_{max} = \frac{5741 \times 10^6}{400} = \underline{1.435 \times 10^7\text{ mm}^3}$$

Loads: need stresses in centre of beam (where bending moment is max.).

$$\sigma = \frac{|M|}{Z_e} + \frac{P}{A} \quad : \quad |M| = \frac{39WL}{88} \quad P = \frac{27W}{88}$$

$W = 100\text{ kN}$, $L = 6\text{ m}$ $\Rightarrow$ σ_{MAX} (compressive) $= \dfrac{39 \times 100 \times 10^3 \times 6}{88 \times (1.435 \times 10^{4})} + \dfrac{27 \times 100 \times 10^3}{88 \times 63815 \times 10^{-6}}$

$$\Rightarrow \boxed{\sigma_{MAX} = \underbrace{18.52}_{\text{bending}} + \underbrace{0.48}_{\text{compression}}\ \text{MPa}} \quad (\sim 19\text{ MPa})$$

b.i) For top flange, $\lambda = \dfrac{600}{20}\sqrt{\dfrac{275}{355}} = 26.4$ (> 24 for compression)

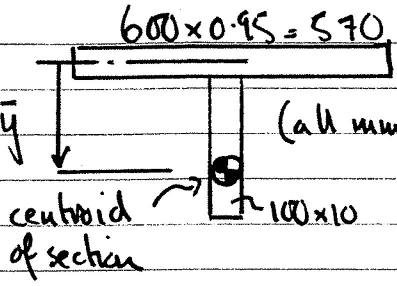
∴ NOT COMPACT, must use effective width $b_e = K_c \cdot b$

P.T.O.

Qu 3 4D10/2003

K_c is given by DS4 for $\lambda = 26.4$ as $\sim \underline{0.95}$

b.i.) (contd) Thus, effective T-section for compression flange is:



$$A_{T\text{-sect}} = 570 \times 20 + 100 \times 10 = \underline{12400\,\text{mm}^2}$$

$$\Rightarrow 12400\,\bar{y} = (100 \times 10) \times (10 + 50)$$

$$\Rightarrow \underline{\bar{y} = 4.84\,\text{mm}} \text{ from centroid of top flange}$$

$$I_{T\text{-sect}} = \tfrac{1}{12} \times 570 \times 20^3 + (570 \times 20) \times 4.84^2 + \tfrac{1}{12} \times 100^3 \times 10 + (100 \times 10) \times [50 + 5.16]^2$$

$$\underline{I_{T\text{-sect}} = 4.523 \times 10^6\,\text{mm}^4} \Rightarrow r_{T\text{-sect}} = \sqrt{\frac{I_{T\text{-sect}}}{A_{T\text{-sect}}}} = \underline{19.10\,\text{mm}}$$

Thus $(r/y)_{T\text{-sect}} = \dfrac{19.10}{105.16} = 0.18 \; (< 0.45) \Rightarrow$ curve C (welded) } DS1.

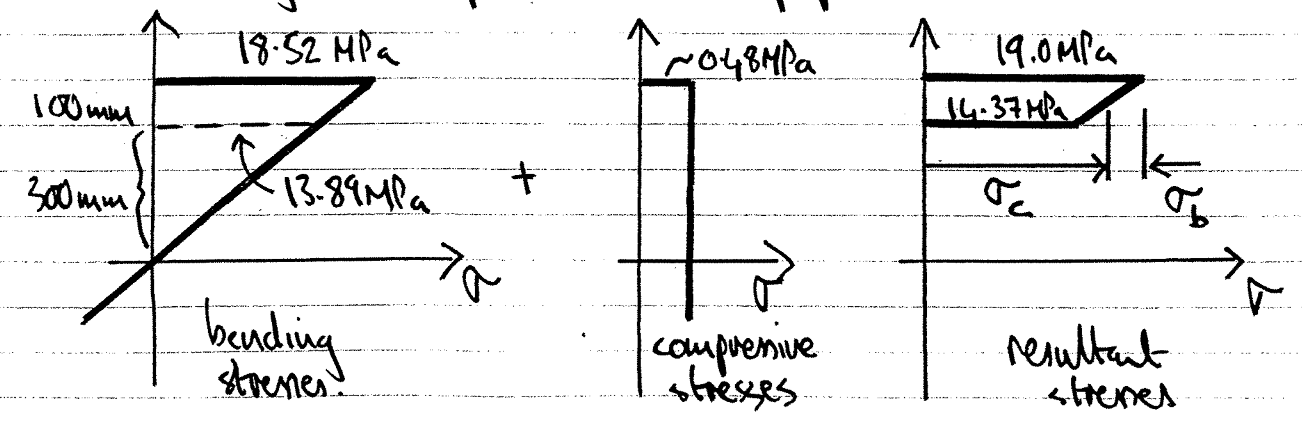
The cross-frame spacing is 1 m $\Rightarrow$ $\lambda = \dfrac{\overbrace{1000}^{L}}{r_T}\sqrt{\dfrac{275}{375}} = \underline{46.1}$

$$\Rightarrow \bar{\sigma}_c\,(\text{DS1}) \simeq 0.8 \Rightarrow \sigma_c = 0.8 \times \underbrace{275}_{\sigma_y} = \underline{220\,\text{MPa}}$$

This value is $\gg \sigma_{max}$ (19 MPa), so top flange is OK in compression.

It is unlikely that panel buckling etc. take place, but check is required.

b.ii) Most heavily-stressed panel is the top panel:



Qu 3 4D10/2003

b.ii) contd) $\sigma_c = \dfrac{19.0+14.37}{2} = \underline{16.7\text{ MPa}}$, $\sigma_b = \dfrac{19.0-14.37}{2} = \underline{2.3\text{ MPa}}$

For $\lambda_{web} = 35$, from before, DS4 gives

$$K_c \sim 0.78, \quad K_b \sim 1.21, \quad K_q = 1$$

(N.B. for K_q, $\phi = \dfrac{1000 \text{ cm length}}{100 \text{ cm height}} \Big\}$ panel $= 10$ (> 3, use $\phi > 3$ curve).

Strength check $\left(\dfrac{\sigma_{max}}{\sigma_y}\right)^2 + \dfrac{3\tau^2}{\sigma_y} \leq 1$

However, $\tau = 0$, since shear force $V = 0$ (max b.m.)

$\Rightarrow \left(\dfrac{19}{275}\right)^2 + 0^2 < 1$; strength OK

Stability check $\dfrac{\sigma_c}{\sigma_{cc}} + \left(\dfrac{\sigma_b}{\sigma_{bc}}\right)^2 + \left(\dfrac{\tau}{\tau_c}\right)^2 \leq 1$

$\sigma_{cc} = K_c\sigma_y$, $\sigma_{bc} = K_b\sigma_y$, $\tau_c = K_q\tau_y$

$$\frac{16.7}{0.78\times 275} + \left(\frac{2.3}{1.21\times 275}\right)^2 + 0^2 = 0.08 \ll 1$$

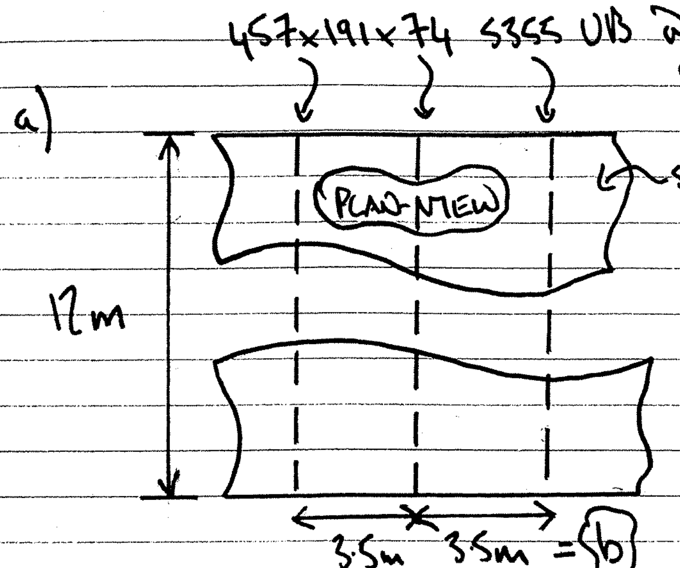
stability OK.

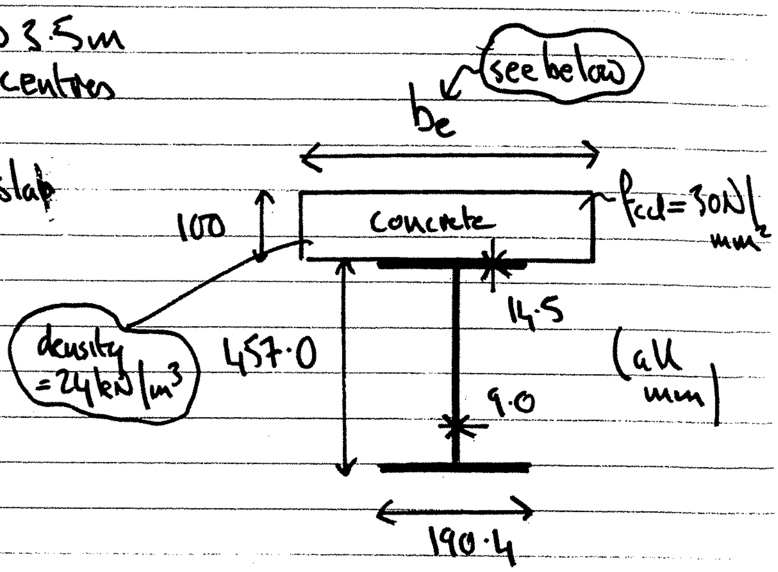
Top panel is adequate : no need to check elsewhere (not asked).

KAS.

Qu 4 4D10/2005

a)





From struct. data book (UB): $A_s = 94.6\text{cm}^2$, mass/length $= 74.3\text{kg}$
$I_{xx} = 33320\ \text{cm}^4$

Check for compaction: $\lambda_{flange} = \left(\dfrac{190.4-9.0}{2}\right)/14.5 = \underline{6.25}$ (<8, OK)

$\lambda_{web} = \left(\dfrac{457.0-14.5}{2}\right)/9.0 = \underline{49.2}$ (<56, OK)

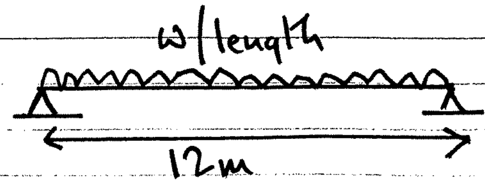
Effective concrete span: smallest of b (3.4m) or span/4 (12/4 = 3m) $\Rightarrow b_e = 3\text{m}$

Load intensities:
slab wt $24 \times 0.1 \times 3.5 = \underline{8.4\ \text{kN/m}}$
UB self-wt $= \underline{0.743\ \text{kN/m}}$
services $= 0.5 \times 3.5 = \underline{1.75\ \text{kN/m}}$
permanent loads

imposed load: $6 \times 3.5 = \underline{21\ \text{kN/m}}$

Total permanent load $= 10.9\ \text{kN/m}$; imposed load $21\ \text{kN/m}$
$\times$ 1.4 (factor: given) $\qquad \times 1.6$
$15.26 \quad + \quad 33.6$

$= \underline{48.9\ \text{kN/m}}$ (= w)



Max b.m $= wL^2/8 = \dfrac{48.9 \times 10^3 \times 12^2}{8}$

$\underline{\text{Max b.m.} = 880.2\ \text{kNm}}$ P.T.O.

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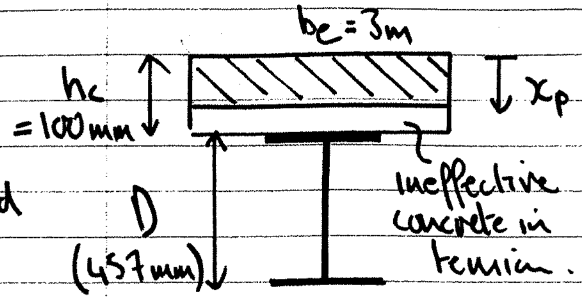
)contd



Assume N.A. in concrete, as shown.

Axial eqlm $\Rightarrow 0.6 f_{cd} b_e x_p = A_s \sigma_y$

$\Rightarrow 0.6 \times (30 \times 10^6) \times 3 \times x_p = (94.6 \times 10^{-4}) \times (355 \times 10^6)$

$\Rightarrow x_p = 0.0622m = \underline{\underline{62.2mm}}$ ($< h_c$, OK)

From notes (eqlm of stress block for above), the design moment is

$$M_d = A_s \sigma_y \left[\frac{D}{2} + h_c - x_p/2 \right]$$

$$= (94.6 \times 10^{-4}) \times (355 \times 10^6) \left[\frac{0.457}{2} + 0.1 - \frac{0.0622}{2} \right] = \underline{998.8 kNm}$$

Thus, $M_d >$ actual M_{max} (880.2kNm)

b) Studs : $P_d = 47kN$ from DS6 ; axial force in concrete $= A_s \sigma_y$

$= (94.6 \times 10^{-4}) \times (355 \times 10^6) = \underline{3358.3kN}$

From notes: $\left\{ \frac{\text{change in axial force between edge and Max}}{P_d} \right\} = \frac{A_s \sigma_y}{P_d} \leq$ no. of studs.

($\frac{1}{2}$ floor span)

$\Rightarrow$ no. studs $> \frac{3358.3}{47} = 71.5 \Rightarrow \underline{\underline{144 \text{ studs}}}$ (full span)

Spacing $= \frac{12}{144} =$ 83.3mm ($>$ 13mm, ϕ of stud, OK).

c) Imposed load short term application :

(concrete) $E_c = 28kN/mm^2$; $E_s/E_c = \frac{205}{28} = 7.3$ (modular ratio)

$\therefore$ use a transformed section (all steel)

P.T.O.

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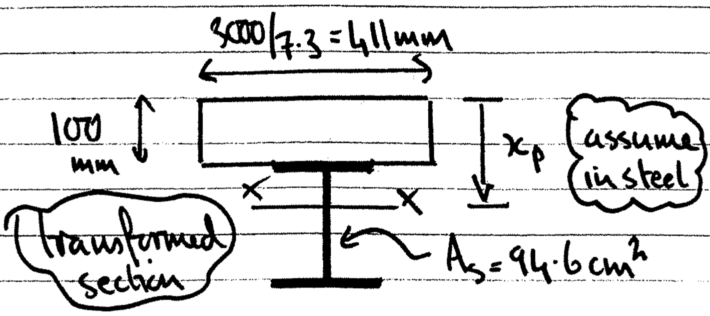
1 contd



Axial eqlm: $\left[\underbrace{(411\times100)}_{\text{transformed concrete}} + \underbrace{94.6\times10^2}_{A_s}\right] x_p = \left[(411\times100)\times50 + (94.6\times10^2)\times\left(\frac{457}{2}+100\right)\right]$

$\Rightarrow$ $x_p = 102.1\,mm$; just inside steel

Find I_{xx} for section

$$I_{xx}\ (mm^4) = \frac{1}{12}\times411\times100^3 + (411\times100)\times(102.1-50)^2 + \underbrace{33320\times10^4}_{I_{xx}\ UB} + (94.6\times10^2)\times\left(\frac{457}{2}-2.1\right)^2$$

$$I_{xx} = 963.9\times10^6\,mm^4$$

actual imposed load

$$\delta = \frac{5wL^4}{384\,E\,I_{xx}} = \frac{5\times(21\times10^3)\times12^4}{384\times(205\times10^9)\times(963.9\times10^{-6})} = \underline{28.7\,mm}$$

steel

Span/250 = $\underline{48\,mm}$ (> 28.7mm, actual defln, therefore OK)

KAS.