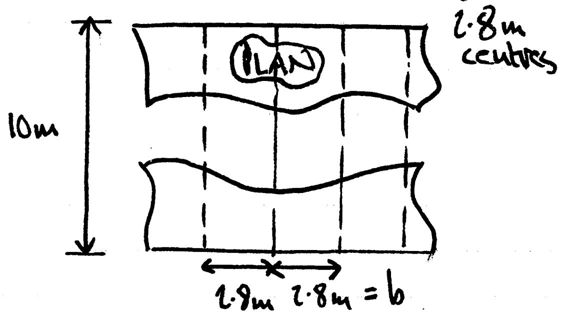
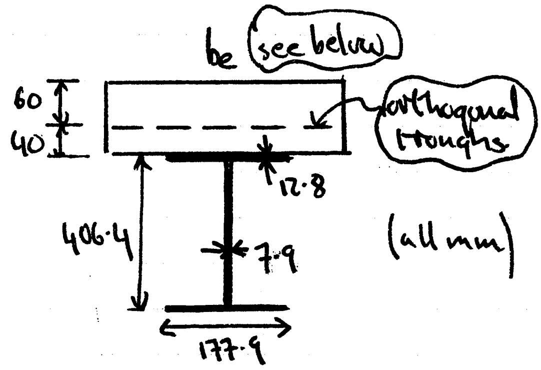
4D10

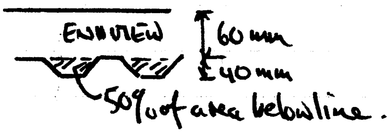
(1)

Qu 1   4/10/2004

a) 406×178×60 S275 UB







$f_{cu} = 30N/mm^2$; concrete density = $2400 kg/m^3$

From struct. data book (UB): $A_s = 76.5cm^2$; mass/m = 60.1kg ; $I_{xx} = 21600cm^4$

Check for compactness: $\lambda_{flange} = \left(\frac{177.9 - 7.9}{2}\right)/12.8 = 6.64$ (< 8, OK).

$\lambda_{web} = \frac{406.4 - 2\times12.8}{7.9} = 48.2$ (< 56, OK).

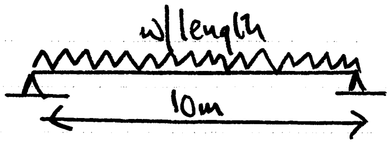
Effective concrete span: smallest of b (2.8m) or span/4 (10/4 = 2.5m) ⇒ $b_e = 2.5m$

Load intensities: slab ↳ $24\times(0.06 + 0.04/2)\times2.8$ = 5.38kN/m
UB selfwt = 0.59 kN/m
services – 3.0×2.8 = 8.4 kN/m.
} permanent loads

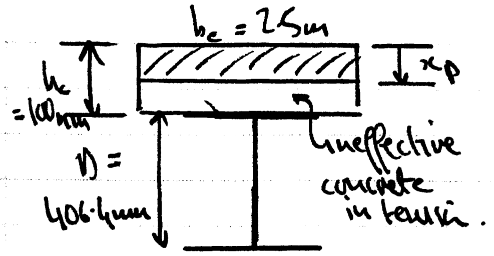
imposed load = 5.5×2.8 = 15.4kN/m

Total permanent load = 14.4kN/m × 1.4 (factor, given) = 20.12 + ; imposed load 15.4 × 1.6 = 24.64

= 44.8kN/m = w



Max b.m = $\frac{wl^2}{8} = \frac{44.8\times10^2}{8}$ = 560kNm



Assume N.A. in concrete at deep x_p in solid part of slab.

P.T.O.

Qu 1 4/10/2004

Axial eqlm: $A_s\sigma_y = 0.6 f_{cd} . b_e . x_p \Rightarrow (76.5\times10^{-4})\times(275\times10^6)$
$= 0.6\times(30\times10^6)\times2.5\times x_p$
$\Rightarrow$ $\underline{x_p = 46.8\text{mm}}$ (<60mm, deep of solid slab, OK).

From the lecture notes, eqlm of stress block for previous section has

design moment $M_d = A_s\sigma_y\left[\frac{D}{2}+h_c-x_p/2\right] = [76.5\times10^{-4}]\times[275\times10^6]\times\left[\frac{0.406}{2}+0.1-\frac{0.0468}{2}\right]$

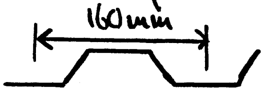
$= \underline{588.2\text{kNm}}$; Thus, $M_d >$ actual M_{MAX} (560kNm)

b) Studs. $P_d = 47\text{kN}$ from BS6 ; axial force in concrete $= A_s\sigma_y$
$= (76.5\times10^{-4})\times[275\times10^6] \simeq \underline{2103.4\text{kN}}$

From notes: no of studs $\geq 2\times A_s\sigma_y/P_d = \frac{2\times2103.4\times10^3}{47\times10^3} = 89.5$.

$\Rightarrow$ no. of studs (even, whole) = $\underline{90}$

Spacing $= \frac{10\text{m}}{90} = 111$mm ; but this cannot be achieved in practice since troughs interfere with placement. Therefore, need to place studs in PAIRS in troughs. But the nett strength is reduced by 80% (from notes) $\Rightarrow$ $\underline{114}$ studs (90/80%) required.



Thus, 57 pairs @ 10m/72 = 175 mm spacing, at most to be effective. Hence, can place more pairs to meet space constraints ∴
10m/160mm = $\underline{\underline{\text{63 pairs effective}}}$.

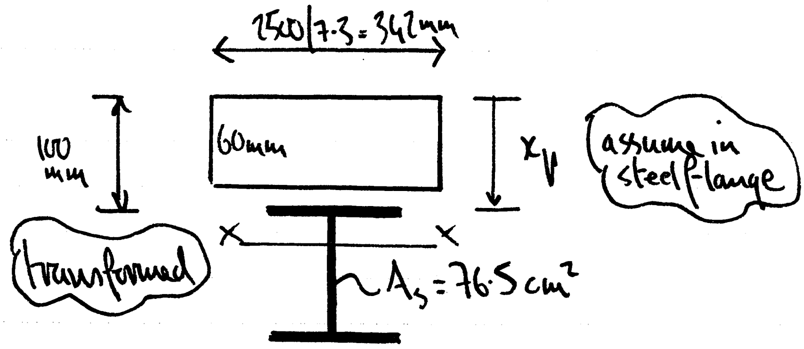
c). Imposed short term load application:

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

Use a transformed section (all steel).

P.T.O

Qu 1 4D10/2004



Axial eqlm: $x_p \cdot \left[\underbrace{342\times60}_{\text{conc}} + 76.5\times10^2\right] = \left[(342\times60)\times30 + (76.5\times10^2)\times\left(100+\frac{406}{2}\right)\right]$

$\Rightarrow$ $x_p = 104.1$ mm ; just inside flange (of thickness 12.8).

Find I_{xx} for section:

$$\underset{(mm^4)}{I_{xx}} = \tfrac{1}{12}\left[342\times60^3\right] + (342\times60)\times(104.1-30)^2 + \overbrace{21600\times10^4}^{I_G\ UB} + (76.5\times10^2)\times\left(\frac{406}{2}-4.1\right)^2$$

$$I_{xx} = 637.6\times10^6\ mm^4$$

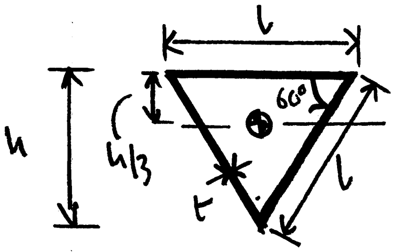
$$\delta = \frac{5\overset{\text{actual load}}{w}L^4}{384\underset{\text{steel}}{E}I_{xx}} = \frac{5\times(15.4\times10^3)\times10^4}{384\times205\times10^9\times(637.6\times10^{-6})} = \underline{\underline{15.3\ mm}}$$

Span/250 = $\underline{40\ mm}$ (> 15.3mm, actual defl'n, therefore OK).

KAS.

Qu 2 4D10 / 2004

Before tackling question, need to establish some overall properties of the section. Assume plate thickness << length.

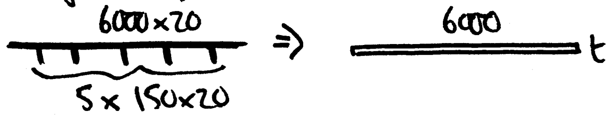


$h = \frac{\sqrt{3}}{2} l$: $I_{xx} = \underbrace{lt \times \left(\frac{h}{3}\right)^2}_{//\text{axis}} + \overbrace{2}^{\text{webs}}\left[\underbrace{lt \times \left(\frac{h}{6}\right)^2}_{//\text{axis}} + \frac{t' h^3}{12}\right]^{*}$

* IA trick:
t' is horizontal width of web $= t . 2/\sqrt{3}$

$\Rightarrow I_{xx} = lt . \frac{1}{9} l^2 . \frac{3}{4} + 2\left[lt \times \frac{1}{12} . \frac{3}{4} l^2 + \frac{1}{12} . \frac{2t}{\sqrt{3}} . \frac{\sqrt{3} . 3 l^3}{8}\right] \left(= \frac{tl^3}{4}\right)$

For a given plate, must use SMEARED section.



$\Rightarrow 6000t = 6000 \times 20 + 5 \times 150 \times 20$

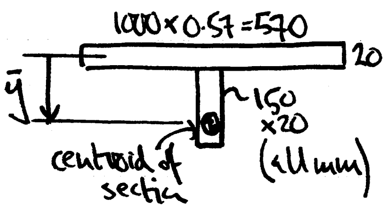
$\Rightarrow \underline{t = 22.5\,mm}$

$\therefore I_{xx} = \frac{22.5 \times 6000^3}{4} = \underline{1.215\,m^4}$; $A = 3 \times 6 \times 0.0225 = \underline{0.405\,m^2}$; $h = 3\sqrt{3}\,m$

a) Top flange: $\lambda_{flange} = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} = \frac{1000}{20}\sqrt{1} = 50 \ (>24)$ $\therefore$ not compact for compression.

Use effective width $b_e = K_c b$ (DS4) ; $\underline{K_c = 0.57}$ for $\lambda = 50$. ($\sigma_y = 355\,MPa$)

Thus, effective T-section for compression flange is



$A_{T\text{-}sect} = 570 \times 20 + 150 \times 20 = \underline{14400\,mm^2}$

$14400\,\bar{y} = (150 \times 20) \times (10 + 75) \Rightarrow \underline{\bar{y} = 17.7\,mm}$

$I_{T\text{-}sect} = \frac{1}{12} \times 570 \times 20^3 + (570 \times 20) \times 17.7^2 + \frac{1}{12} \times 150^3 \times 20 + (150 \times 20) \times (75 + 10 - 17.7)^2$

$= \underline{23.16 \times 10^6\,mm^4}$ $\quad r_{T\text{-}sect} = \sqrt{\frac{I}{A}}\Big|_{T\text{-}sect} = \underline{40.1\,mm}$

Thus $(r/y)_{T\text{-}sect} = \frac{40.1}{(150 - 17.7)} = \underline{0.28} \Rightarrow$ (Curve (c), DS1)

The cross-frame spacing is every 5 m $\Rightarrow \lambda = \frac{5000}{40.1}\sqrt{1} = \underline{125}$

$\Rightarrow \bar{\sigma}_c\,(DS1) \simeq 0.29 \Rightarrow \underline{\sigma_c = 103\,MPa}$.

The maximum stress at the top flange must be less than σ_c.

P.T.O.

Qu 2 4D10/2004

Two load cases:

	S (kN)	M (kNm)	P (kN)
(1)	10000	60000	0
(2)	0	20000	30000.

compression in top flange

Use $\sigma = M\underbrace{y}_{h/3}/I + P/A$

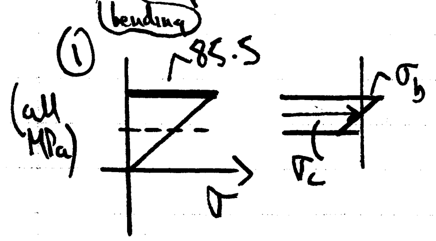
(1) $\sigma = 60\times10^6 \times \sqrt{3}/1.215 + 0 = \underline{85.5\text{ MPa}}$, OK.

(2) $\sigma = 20\times10^6\times\sqrt{3}/1.215 + 30\times10^6/0.405 = \underline{28.5} + \underline{74.1}\text{ MPa} = \underline{102.6\text{ MPa, OK.}}$

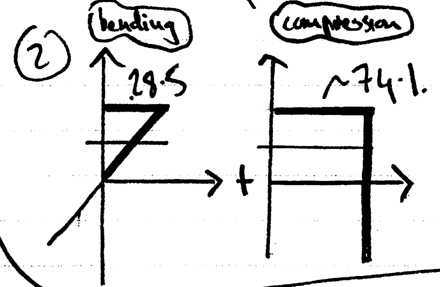
For (1), OK, but for (2), the load is JUST CARRIED (c.f. $\sigma_c = 103$ MPa).

b) In each web, the shear force // to it must have a vertical component equal to, in total, the actual shear force. The most heavily stressed panel in each case is the top: note, there are only two panels above the centroid of section.

(1) (all MPa)



(2)



$\sigma_c = 74.1 + \frac{3}{4}\times28.5 = \underline{95.5\text{ MPa}}$

$\sigma_b = \frac{1}{4}\times28.5 = \underline{7.1\text{ MPa}}$

$\left.\begin{array}{l}\sigma_c = 3/4\times85.5 = \underline{64\text{ MPa}}\\ \sigma_b = 1/4\times85.5 = \underline{21.4\text{ MPa}}\end{array}\right\}$ Also $\tau = \dfrac{S/(\sqrt{3}/2)}{2A_{web}} = \dfrac{10\times10^6\times2/\sqrt{3}}{2\times0.02\times6} = \underline{48.1\text{ MPa}}$.

$\lambda_{web} = \frac{1000}{20}\sqrt{1} = \underline{50} \Rightarrow$ BS4 given: $K_c \doteq 0.57$; $K_b \doteq 1.17$; $K_q \doteq 0.88$.

[N.B. for K_q, $\phi = \dfrac{5000 \leftarrow \text{length}}{1000 \leftarrow \text{ht}} = 5$ ($>3 \Rightarrow$ use $\phi > 3$ curve)].

Must perform @ (1) & (2) { (i) strength checks; (ii) stability checks

$\underbrace{\left(\frac{\sigma}{\sigma_y}\right)^2}_{max} + \left(\frac{\tau}{\underbrace{\tau_y}_{\sigma_y/\sqrt{3}}}\right)^2 \le 1$

$\frac{\sigma_c}{\underbrace{\sigma_{cc}}_{K_c\sigma_y}} + \left(\frac{\sigma_b}{\underbrace{\sigma_{bc}}_{K_b\sigma_y}}\right)^2 + \left(\frac{\tau}{\underbrace{\tau_c}_{K_q\cdot\tau_y}}\right)^2 \le 1$

P.T.O.

Qu 2 4D10/2004

(1): $\sigma_{max} = 85.5\,MPa$; $\tau = 48.1\,MPa$, $\sigma_c = 64\,MPa$, $\sigma_b = 21.4\,MPa$

strength: $\left(\frac{85.5}{355}\right)^2 + \left(\frac{48.1}{355/\sqrt{3}}\right)^2 = 0.11 \ll 1$, OK.

stability: $\frac{64}{0.57 \times 355} + \left(\frac{21.4}{1.17 \times 355}\right)^2 + \left(\frac{48.1}{0.88 \times \frac{355}{\sqrt{3}}}\right)^2 = 0.4 < 1$, OK.

(2): $\sigma_{max} = 102.6\,MPa$, $\tau = 0$, $\sigma_c = 102.6\,MPa$, $\sigma_b = 7.1\,MPa$.

strength: $\left(\frac{102.6}{355}\right)^2 + \left(\frac{0}{355/\sqrt{3}}\right)^2 < 1$, OK.

stability: $\frac{95.5}{0.57 \times 355} + \left(\frac{7.1}{1.17 \times 355}\right)^2 + \left(\frac{0}{\tau_c}\right)^2 = 0.47 < 1$, OK.

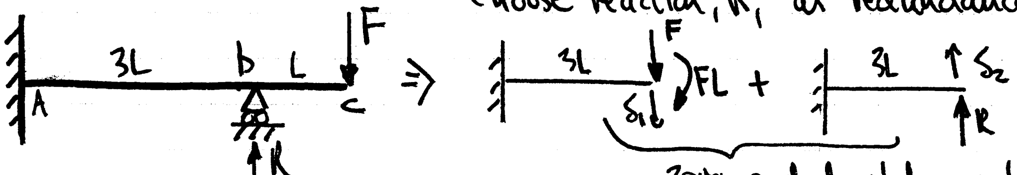
$\therefore$ Top panel is adequate: no need to check anywhere else (not asked).

N.B., in btm flange $\sigma_{max} \simeq 170\,MPa$ (tensile) $\ll \sigma_y$.

KAS.

Qu3 4D10/2004.

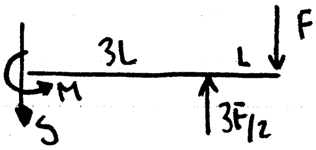
a) Choose reaction, R, as redundancy



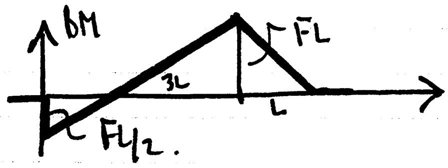
zero end displmt overall

From Struct data book: $\delta_1 = \frac{FL.9L^2}{2EI} + \frac{F(3L)^3}{3EI} = \frac{FL^3}{EI}\left(\frac{9}{2}+9\right)$

$\delta_2 = R.(3L)^3/3EI = \frac{RL^3}{EI}.9$ ∴ $\delta_1 = \delta_2 \Rightarrow \underline{\frac{3F}{2} = R}$.



$M + \frac{3F}{2} \times 3L - F.4L = 0 \Rightarrow$ $M = -FL/2$, $S = F/2$ in dirns as shown



203 × 102 × 23 S275 UB: struct data book gives

I_{yy} (minor) = 164 cm⁴ ; J = 7.02 cm⁴ ; Z_p = 234 cm³ ;

G = 81 GPa, E = 205 GPa, σ_y = 275 MPa. L = 2m (given).

Tackle by DS2: there are two critical spans, AB & BC, of different lengths. Must evaluate

$M_E = (M_1^2 + M_2^2)^{1/2}$ with $M_1 = \frac{\pi}{L}\left[GJEI_{yy}\right]^{1/2}$; $M_2 = \frac{\pi^2}{L^2} ED \frac{I_{yy}}{2}$ ~ I_{flange}.

Here, $D = \underbrace{203.2}_{\text{depth}} - \underbrace{9.3}_{\text{flange thickness}} \Rightarrow \underline{D = 193.9\text{mm}}$

	AB (L = 6m)	BC (L = 2m).
M_1 (kNm)	22.90	68.68
M_2 (kNm)	4.94	80.42
M_E (kNm)	24.58	105.76.
$M_p = Z_p\sigma_y$ (kNm)	64.35	64.35.
$\lambda_{LT} = 75\sqrt{M_p/M_E}$	~121	~58
$\bar{M}_c$ (DS2)	0.32.	0.87

to level of accuracy of measurement.

P.T.O.

Qu3 4D10/2004

AB) 3L = 6m; $\frac{FL}{2} = F$ (βm); FL = 2F (= m, DS2)

$\beta = -\frac{1}{2}$ (from DS2 $\Rightarrow M_u = 0.4m$

$\Rightarrow M_u = 0.8F$

For strength: $m \leq M_p \Rightarrow 2F \leq M_p \Rightarrow$ <u>$F \leq 32.2$ kN</u>

For stability: $M_u \leq M_c \Rightarrow 0.8F \leq 0.32 M_p \Rightarrow$ <u>$F \leq 25.7$ kN</u>

BC. 2F (m); 0 (βm) $\Rightarrow \beta = 0$; $M_u = 0.6m$ from DS2.

Strength: $m \leq M_p \Rightarrow 2F \leq M_p \Rightarrow$ as above, <u>$F \leq 32.2$ kN</u>

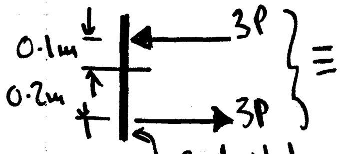
stability: $M_u \leq M_c \Rightarrow 0.6 \times 2F \leq 0.87 M_p \Rightarrow$ <u>$F \leq 46.7$ kN</u>

<u>Summary:</u> (max values in kN)

member	F (strength)	F (stability)
AB	32.2	25.7*
BC	32.2	46.7

* lowest F $\rightarrow$ ($F_{MAX} = 25.7$ kN), governed by stability of AB.

c) By lower bound theorem, the moment at the base, has to be carried by equal and opposite tensions in each line of bolts (equal to, say, P).



$\equiv M = \frac{FL}{2}$ $\therefore$ by equivalence $\frac{FL}{2} = 3P\,[0.1 + 0.2]$ (P: bolt force)

$\Rightarrow$ <u>$P = 1.11F$</u>. (critical for tension bolts on btm).

Assume, by lower bound theorem, all of the shear force is carried by just <u>ONE</u> of the tension bolts.

On this bolt, there is 1.11F in tension = <u>28.5 kN</u>

F/2 in shear = <u>12.9 kN</u>.

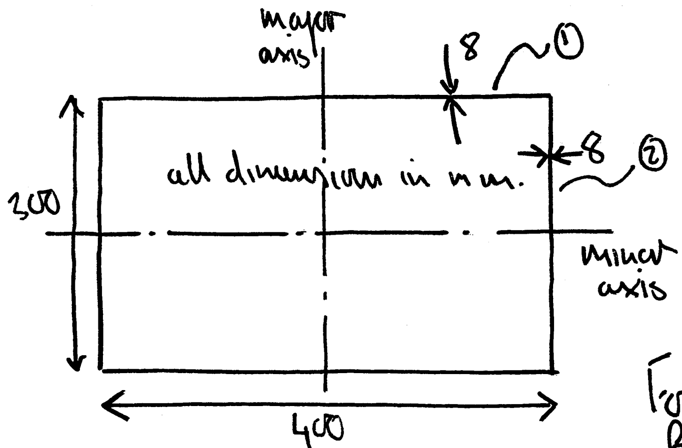
Using DS5, grade 4.6 bolts, interaction diagram suggests use of <u>M20 (20 mm ϕ)</u>: may even get away with M16.

If the moment is only carried by a pair of bolts $\Rightarrow$ tension is $3.33F \triangleq 85$ kN $\Rightarrow$ <u>M30 bolts</u>! this is the worst case scenario.

KAS.

(1).

Qu 4 4D10/2004

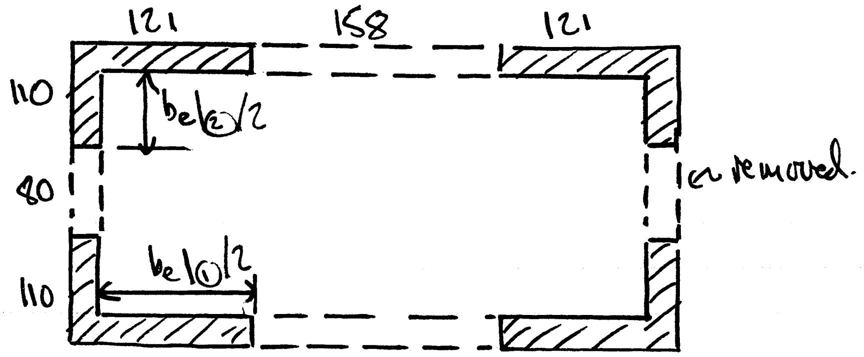


$$\lambda_{(1)} = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} = \frac{400-2\times8}{8}\sqrt{\frac{355}{355}} = \underline{48}$$

$$\lambda_{(2)} = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} = \frac{300-16}{8}\sqrt{1} = \underline{35.5}$$

For (1), DS4 gives $\underline{K_c = 0.59}$
for (2), " gives $\underline{K_c = 0.72}$.

$$b_e|_{(1)} = K_c \times b = 0.59 \times (400-16) = \underline{226}$$
$$b_e|_{(2)} = K_c \times b = 0.72 \times (300-16) = \underline{204}.$$



$$A_{eff} = (400\times300) - (384\times284) - 2(158\times8) - 2(80\times8) = \underline{7136\ mm^4}$$

$$I_{major} = \tfrac{1}{12}\left[300\times400^3 - 284\times384^3\right] - 2\left[\tfrac{1}{12}\times8\times158^3\right] - 2\left[80\times8\times196^2 + 80\times8^3/12\right]$$
$$= \underline{2.07\times10^8\ mm^4}$$

$$I_{minor} = \tfrac{1}{12}\left[400\times300^3 - 384\times284^3\right] - 2\times\left[\tfrac{1}{12}\times8\times80^3\right] - 2\left[158\times8\times146^2 + \frac{158\times8^3}{12}\right]$$
$$= \underline{1.12\times10^8\ mm^4}$$

In both cases, $r = \sqrt{I/A_{eff}}$ $\Rightarrow$ $r_{major} = \underline{170.3\ mm}$
$r_{minor} = \underline{125.3\ mm}$

For both cases, $r/y > 0.7$ $(0.85, 0.84$ resp$)$.
0.852 0.835

extreme fibre

$\Rightarrow$ use curve (b) DS1 (welded column)

P.T.O.

Qu 4 4/10/2004

$L_E = kL$, where k depends on supports ($L = 12\,m$, given).
Major axis, pin-pinned ; $k = 1$; minor (fixed-fixed) $k = 0.7$

$\therefore$ for major $\lambda = \frac{L_E}{r}\sqrt{\frac{\sigma_y}{355}} = \frac{12}{0.17}\sqrt{1} = 71$
for minor $\lambda = \frac{L_E}{r}\sqrt{\frac{\sigma_y}{355}} = \frac{12 \times 0.7}{0.125}\sqrt{1} = 67$

$\therefore$ Major axis most critical with $\bar{\sigma}_c = 0.66$ via DS1
$\bar{\sigma}_c = P_c/P_p$ or $A_{eff} \times \sigma_y \Rightarrow P_c = 0.66 \times 7136 \times 10^{-6} \times 355 \times 10^6$
$\Rightarrow P_c = 1672.0\,kN$

$\therefore$ Column can carry 1000 kN.
c) Factor of increase of (b) over 1000 kN = 67%.

KAS.