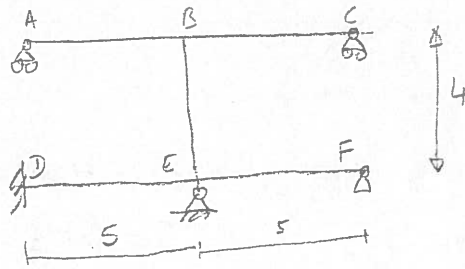


Q1 a)



Column UC 305x305x97

Beams UB 457x191x67

Inej

Steel grade irrelevant. All Young's Moduli are the same.

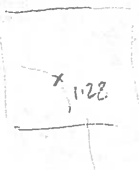
	$I_{maj}$	$L$	$I/L$
UC	22250 cm <sup>4</sup>	400 cm	55.6
UB	24380 cm <sup>4</sup>	500 cm	58.8

$$k_{top} = \frac{(EI/L)_{col}}{(EI/L)_{col} + 2(EI/L)_{beam} \times \frac{3}{4}} = \frac{55.6}{55.6 + 1.5(58.8)} = 0.387 \checkmark$$

pin ends

$$k_{bot} = \frac{(EI/L)_{col}}{(EI/L)_{col} + (EI/L)_{DE} + \frac{3}{4}(EI/L)_{EF}} = \frac{55.6}{55.6 + 1.75(58.8)} = 0.351 \checkmark$$

Check b)  
Without  
Sway  
Restraint

↑  
Both  
A and C  
on Rollers


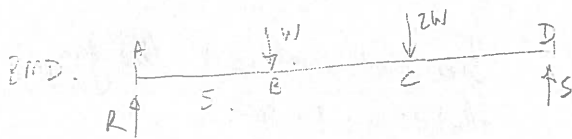
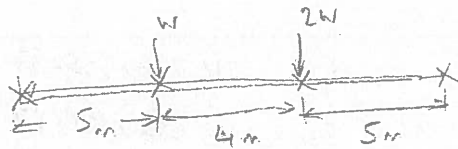
$$\begin{aligned} \text{effective length} &= 1.28 L \\ &= 1.28(4) \\ &= \underline{\underline{5.12 \text{ metres}}} \end{aligned}$$

$$\begin{aligned} P_{cr} &= \frac{\pi^2 EI}{L^2} = \frac{\pi^2 (210 \times 10^9 \text{ N/m}^2) (22.250 \times 10^{-8} \text{ m}^4)}{(5.12)^2} \\ &= \underline{\underline{17.59 \text{ MN}}} \end{aligned}$$

(40%)

b).

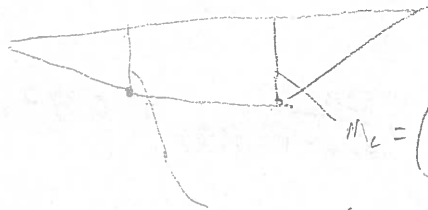
Q1(b).

 $M_A (+)$ 

$$5W + 9(2W) = 14S$$

$$S = \frac{(5+18)W}{14} = \frac{23}{14}W$$

$$R = 3W - S = \frac{19}{14}W$$



$$M_C = \left(\frac{23}{14}W\right)5 = \underline{\underline{8.214W}}$$

$$M_B = \left(\frac{19}{14}W\right)5 = \underline{\underline{6.786W}}$$

S275

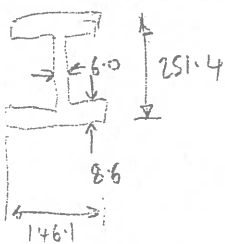
254 x 146 x 31 UB

$$I_{maj} = 4413 \times 10^{-8} m^4$$

$$I_{min} = 448 \times 10^{-8} m^4$$

$$J = 8.55 \times 10^{-8} m^4$$

$$Z_{maj, plastic} = 393 \times 10^{-6} m^3$$



Check compactness.

$$\text{flange } \lambda = \frac{(146.1 - 6.0)}{2} \sqrt{\frac{275}{355}} = 7.17 < 8 \quad \therefore \text{compact}$$

$$\begin{aligned} \text{web in bending } \lambda &= \frac{251.4 - 2(8.6)}{6} \sqrt{\frac{275}{355}} \\ &= 34.3 < 56 \quad \therefore \text{compact} \end{aligned}$$

$$\text{Calc. Plastic } M_{pl} = G_y Z_{maj} = (275 \times 10^6 N/m^2) (393 \times 10^{-6} m^3) = \underline{\underline{108.1 kNm}}$$

$$\text{Calc. Elastic } M_{cr} = \frac{\pi}{L} \sqrt{GJ E I_{min}} = \frac{\pi}{L} E \sqrt{\frac{J I_{min}}{2.6}}$$

$$\begin{aligned} \text{Assume BC in torsion} \rightarrow L &= 4m \\ &= \frac{\pi}{4} (210 \times 10^9) \left( \sqrt{\frac{8.55 \times 448}{2.6}} \right) \times 10^{-8} = \underline{\underline{63.3 kNm}} \end{aligned}$$

$$\begin{aligned} \text{Warping correction: } \sqrt{1 + \frac{\pi^2 E I}{L^2 G J}} \\ \Gamma = \frac{I_D^2}{4} = \frac{(448 \times 10^{-8}) (251.4 - 8.6)^2 \times 10^{-6}}{4} \\ = \underline{\underline{6.6 \times 10^{-8} m^6}} \end{aligned}$$

Q1 b) cont'd.

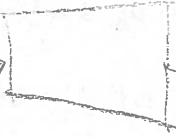
$$\sqrt{1 + \frac{\pi^2 EI}{L^2 GJ}} = \sqrt{1 + \frac{\pi^2}{4^2} \frac{210(2.6)}{210} \frac{6.6 \times 10^{-8}}{8.55 \times 10^{-8}}}$$

$$= \sqrt{1 + 1.238} = \sqrt{2.238} = 1.496$$

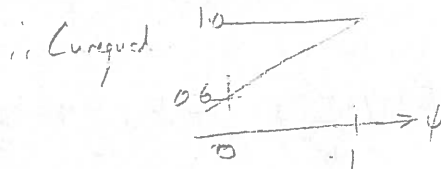
$$\therefore M_{LTB} = 1.496(63.3) = \underline{\underline{94.7 \text{ kNm}}}$$

(eff)

Curved



$$8.214 W \rightarrow \psi = \frac{6.786}{8.214} = 0.826$$



$$Curved = 0.6 + 0.4\psi$$

$$= 0.6 + 0.4(0.826)$$

$$= 0.9304$$


$$\therefore M_{elastic} = \frac{M_{LTB}}{Curved} = \frac{94.7}{0.9304} = \underline{\underline{101.8 \text{ kNm}}}$$

$$\lambda = \sqrt{\frac{I_p}{I_{EL}}} = \sqrt{\frac{M_{PL}}{M_{EL}}} = \sqrt{\frac{108.1}{101.8}} = \underline{\underline{1.03}}$$

Curve? Rolled I section,  $\frac{h}{b} = \frac{251.4}{146.1} = 1.72 < 2 \Rightarrow \text{Curve a) BS2.}$

$$\Rightarrow \chi = 0.66 + (DS1)$$

$$\therefore M_{max} = \chi M_{pe} = 0.66(108.1) = \underline{\underline{71.34 \text{ kNm}}}$$

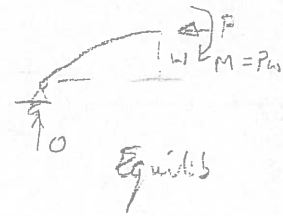
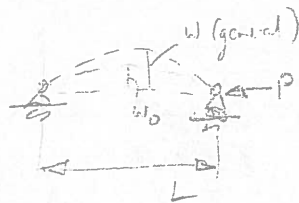


$$8.214 W = 71.34 \text{ kNm}$$

$$W = \frac{71.34}{8.214} = \underline{\underline{8.69 \text{ kN}}}$$

[60%]

Perry Robertson.



$$\left( \begin{array}{l} \text{stress at centre} = \frac{P}{A} + \frac{M_y}{I} \\ \text{at extreme fibre} \end{array} \right)$$

$$M = Pw \quad \text{equilib}$$

$$M = -EI(w - w_0)''$$

$$-EIw'' + EIw_0'' = Pw$$

$$\rightarrow \text{governing eqn} \quad EIw'' + Pw = EIw_0''$$

$$\text{let } w = a \sin \frac{\pi x}{L}$$

$$w_0 = a_0 \sin \frac{\pi x}{L}$$

$$w'' = -\frac{\pi^2}{L^2} a \sin \frac{\pi x}{L}$$

$$w_0'' = -a_0 \frac{\pi^2}{L^2} \sin \frac{\pi x}{L}$$

$$\left( EI \left( -\frac{\pi^2}{L^2} a \right) + Pa \right) \sin \frac{\pi x}{L} = \left( -a_0 \frac{\pi^2}{L^2} \right) EI$$

$$(-P_{\text{Euler}} + P)a = -a_0 P_{\text{Euler}}$$

$$P_{\text{cr}} =$$

$$a = \left( \frac{a_0}{1 - P/P_E} \right)$$

$$\text{let } \sigma = P/A \quad \sigma_E = P_E/A$$

let extreme fibre stress reach yield.

$$\text{so } \sigma_{\text{ext.fib}} = \sigma_y = \sigma + \frac{M_y}{I}$$

$$M = Pw$$

$$\frac{M}{A} = \frac{Pw}{A}$$

$$= \sigma w$$

$$\text{at centre } w = a = \left( \frac{a_0}{1 - P/P_E} \right)$$

$$\frac{M_y}{I} = \frac{M}{A} \left( \frac{A_y}{I} \right) = \frac{\sigma_0}{(1 - P/P_E)} \left( \frac{A_y}{I} \right) = \frac{\sigma_0}{1 - P/P_E} \left( \frac{a_0 y}{r^2} \right) = \left( \frac{\sigma}{1 - \sigma/\sigma_E} \right) \eta$$

$$\text{so } \sigma_y = \sigma + \frac{\sigma}{(1 - \sigma/\sigma_E)} \eta \Rightarrow (\sigma_y - \sigma) = \eta \frac{\sigma \sigma_E}{(\sigma_E - \sigma)}$$

$$(\sigma_E - \sigma)(\sigma_y - \sigma) = \sigma \sigma_E \eta$$

Q2 a) cont'd

$$(G_E - G)(G_Y - G) = \eta G G_E \quad \text{Perry Robertson.}$$

$$G_E G_Y - G G_Y - G G_E + G^2 = \eta G G_E$$

$$G^2 - G[G_Y + G_E(1+\eta)] + G_E G_Y = 0$$

$$\div G_Y^2 \quad \left(\frac{G}{G_Y}\right)^2 - \left(\frac{G}{G_Y}\right)\left(1 + \frac{G_E}{G_Y}(1+\eta)\right) + \left(\frac{G_E}{G_Y}\right) = 0$$

$$\text{Let } \frac{G}{G_Y} = \frac{1}{\beta} \quad \frac{1}{\beta^2} - \frac{1}{\beta}\left(1 + \frac{G_E}{G_Y}(1+\eta)\right) + \frac{G_E}{G_Y}$$

$$\text{Define } \lambda = \sqrt{\frac{\text{Plati}}{\text{Elati}}} = \sqrt{\frac{G_Y}{G_E}} \quad \therefore \frac{G_E}{G_Y} = \frac{1}{\lambda^2}$$

$$\frac{1}{\beta^2} - \frac{1}{\beta}\left(1 + \frac{1}{\lambda^2}(1+\eta)\right) + \frac{1}{\lambda^2}$$

$$\frac{\lambda^2}{\beta^2} - \frac{1}{\beta}(\lambda^2 + 1 + \eta) + 1 = 0$$

$$\beta^2 - \beta(\lambda^2 + 1 + \eta) + \lambda^2 = 0$$

$2\Phi$  define

$$\beta^2 - \beta 2\Phi + \lambda^2 = 0$$

$$\beta = \frac{2\Phi \pm \sqrt{4\Phi^2 - 4\lambda^2}}{2} = \Phi \pm \sqrt{\Phi^2 - \lambda^2}$$

$$\frac{G_Y}{G} = \beta \quad \therefore G = \frac{1}{\beta} G_Y = \lambda G_Y \quad \lambda = \frac{1}{\Phi \pm \sqrt{\Phi^2 - \lambda^2}}$$

Take lowest root, i.e. +ve sign  
 $G \leq D$

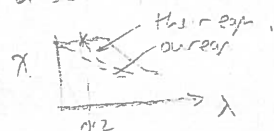
Note: no real explanation for 0.2

$$\text{We had } \Phi = \frac{\lambda^2 + 1 + \eta}{2} = \frac{\lambda^2 + 1 + \alpha\lambda}{2}$$

$$\text{and DSI has } \Phi = \frac{\lambda + 1 + \alpha(\lambda - 0.2)}{2}$$

$$\eta \equiv \alpha\lambda$$

Just a curve fit to data



Q2 b)

page 6

$$\eta = a_0 \frac{y}{r^2} = \alpha \lambda$$

$$= \left( \frac{a_0}{L} \right) \left( \frac{y}{r} \right) \left( \frac{L}{r} \right) = \alpha \lambda$$

$\uparrow$     $\uparrow$     $\uparrow$   
 bowing   shape   slenderness

$$\text{We have } \lambda = \sqrt{\frac{\text{Plastic}}{\text{Elastic}}} = \left( \frac{G_y A}{\pi^2 EI / L^2} \right)^{1/2} = \frac{1}{\pi} \left( \frac{G_y}{E} \right)^{1/2} \left( \frac{L^2}{I/A} \right)^{1/2} = \left( \frac{1}{\pi} \sqrt{\frac{G_y}{E}} \right) \left( \frac{L}{r} \right)$$

$$\text{so } \frac{L}{r} = \left( \pi \sqrt{\frac{E}{G_y}} \right) \lambda$$

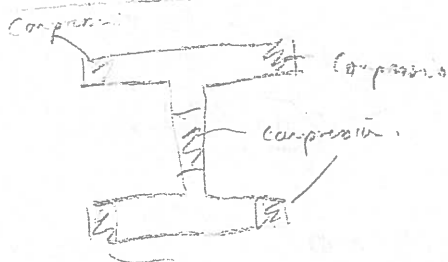
$\lambda = \text{"generalised slenderness"}$

L initial

$$= \left( \frac{a_0}{L} \right) \left( \frac{y}{r} \right) \left( \pi \sqrt{\frac{E}{G_y}} \right) \lambda = \alpha \lambda$$

$$\alpha = \left( \frac{a_0}{L} \right) \left( \frac{y}{r} \right) \left( \frac{\pi E}{\sqrt{G_y}} \right)$$

Q2 c)



rest in tension

(regions that cool first solidify and are pulled into compression as remainder cools)

Q2 d)

A theoretical difficulty is that you have two modes of deformation twist and flexure, and so you need estimate of two imperfections (initial bow and initial twist).

Q2 e)

They are basically just a curve fit to experimental data, and use the Perry-Robertson-like formula to round off the corner between pure plastic + pure elastic behaviours.

Q2 f)

Self fulfilling prophecy.  $\lambda = \sqrt{\frac{\text{Plastic}}{\text{Elastic}}}$  ; On Elastic curve  $G = G_{\text{elastic}}$

$$\chi = \frac{G}{G_{\text{plastic}}} = \frac{G_{\text{elastic}}}{G_{\text{plastic}}} = \frac{1}{\lambda^2}$$

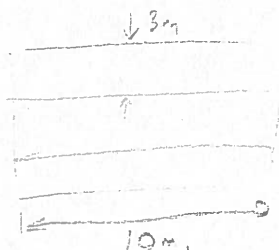
$\lambda$  is a generalised slenderness  
Not proportional to  $L$ .

Q3.0) Composite.

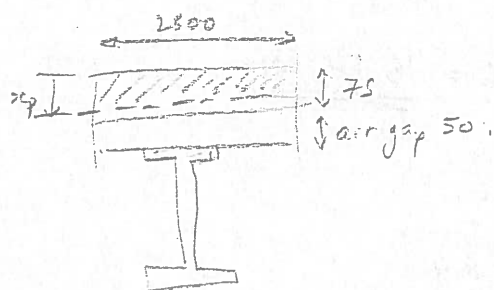


457 x 152 x 82 UB, S235.

PLAN



$$\text{Effective width} = \min(3\text{m}, \text{span} = 2.5\text{m}) = 2.5\text{m}$$



Assume neutral axis in concrete

Ax. J equilib

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

$$A_s = 105 \text{ cm}^2 = 105 \times 10^2 \text{ mm}^2$$

$$\sigma_y = 235 \text{ N/mm}^2$$

$$(105 \times 10^2)(235) = 0.6(30)(2500)x_p$$

$$x_p = \frac{(10500)(235)}{0.6(30)(2500)} = 54.8 \text{ mm}$$

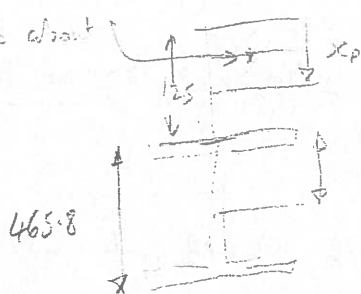
i.e. in concrete ✓.

Check compactness of beam

$$\text{flange } \frac{b}{t} \sqrt{\frac{235}{355}} = \frac{(155.3 - 10.5)}{2(18.9)} \sqrt{\frac{235}{355}} = 3.1 < 8$$

$$\text{web } \frac{b}{t} \sqrt{\frac{235}{355}} = \frac{465.8 - 2(18.9)}{10.5} \sqrt{\frac{235}{355}} = 33.1 < 56 \text{ OK loading.}$$

Strength: Moments about



$$M = A_s \sigma_y \left[ \frac{465.8}{2} + 125 - \frac{54.8}{2} \right]$$

$$= 105 \times 10^2 \text{ mm}^2 / 235 \text{ N/mm}^2 \times 329.5 \text{ mm}$$

$$= 813 \times 10^6 \text{ Nmm}$$

$$= \underline{813 \text{ kNm}} \quad \text{Strength.}$$

Q3 a) cont'd

page 8

Loads:

$$DL: \text{Concrete} = (3m)(0.1m)(2400 \text{ kg/m}^3)$$

$$= 720 \text{ kg/m}$$

$$\text{Steel} = \underline{82.1 \text{ kg/m}}$$

$$\Sigma = 802.1 \text{ kg/m}$$

$$= 7869 \text{ N/m}$$

$$= \underline{7.87 \text{ kN/m}}$$



$$2 \text{ kN/m} = \text{perm service} \rightarrow 6 \text{ kN/m} \quad (3 \text{ m width}).$$

$$\therefore DL (\text{perm}) = 7.87 + 6 = \underline{13.87 \text{ kN/m}}$$

$$\text{Total} = 1.35 DL + 1.5 LL.$$

$$= 1.35(13.87) + 1.5(3w)$$

$$= \underline{(18.72 + 4.5w)} \text{ kN/m}.$$

$$M = \frac{wL^2}{8} \quad "w" = \frac{8M}{L^2} = \frac{8(813)}{10^2} = \underline{65.0 \text{ kN/m}}$$

$$65.0 = 18.75 + 4.5w$$

$$w = \frac{65 - 18.75}{4.5} = \underline{10.2 \text{ kN/m}^2} \quad (\text{Larger, eg Library load}).$$

unfactored live load.

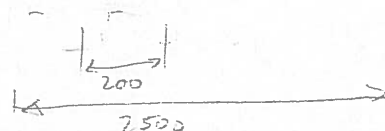
Q3 b) Shear studs.

$$\text{Total force at centre} = A_{st} \sigma_y = (105 \times 10^2)(235) = \underline{2467 \text{ kN}}$$

Shear studs in Each half span needs to carry this.

$$\text{eg. } 25 \text{ dia, } 100 \text{ mm high} \rightarrow 154 \text{ kN each} \rightarrow \text{need } \frac{2467}{154} \Rightarrow 16 \text{ of them.}$$

We have



= 12.5 troughs, so need to double up.

~~25~~ troughs

$$\text{Paired } 25 \times 100 \text{ studs} \rightarrow 154 \text{ kN} \times 2 \times 0.8 = 246 \text{ kN/pair} \rightarrow 10 \text{ req'd.}$$

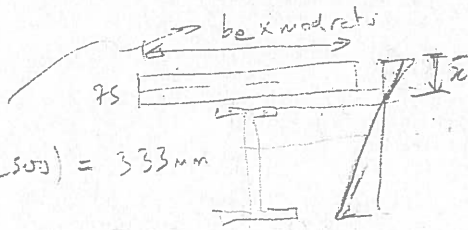
dia

and so pair in each trough.

Q3(c) Deflection.

$$w = 10.2 \text{ kN/m}^2$$

$$3 \text{ m width} \rightarrow 30.6 \text{ kN/m}$$



$$\text{modular ratio} = \frac{28 \text{ GPa}}{210 \text{ GPa}} = 0.133$$

$$(0.133)(2500) = 333 \text{ mm}$$

Assume r/A in trough region.  $x$  measured from top.

$$A\bar{x} = \int x \, dA$$

$$[(333)(75) + 10500] \bar{x} = (333)(75) \left( \frac{75}{2} \right) + 10500 \left( 125 + \frac{465.8}{2} \right)$$

$$35.48 \times 10^3 \bar{x} = 936.6 \times 10^3 + 3758 \times 10^3$$

$$\bar{x} = \frac{936.6 + 3758}{35.48} = 132.3 \text{ mm}$$

So r/A was actually in the steel, (just).

but hardly makes any difference, because lower arm of steel in compression is so small ( $\approx 7 \text{ mm}$ ) so ignore, and continue

$$I = \frac{(333)(75)^3}{12} + 333(75) \left[ 132.3 - \frac{75}{2} \right]^2 + 36540 \times 10^4 + 10500 \left[ \frac{465.8}{2} + 125 - 132.3 \right]^2$$

$$= 11.7 \times 10^6 + 224.5 \times 10^6 + 365.9 \times 10^6 + 534.4 \times 10^6$$

$$= 1.136 \times 10^9 \text{ mm}^4 = 1.136 \times 10^{-3} \text{ m}^4$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(30.6 \times 10^3)(10)^4}{384(210 \times 10^9)(1.136 \times 10^{-3})} = \frac{0.0167}{1} = 17 \text{ mm}$$

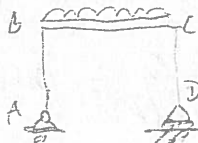
Span = 10 metres

$$\frac{\text{Span}}{\Delta} = \frac{10000}{17} = 598 \text{ ie } \frac{\text{Span}}{500}$$

so fine  $< \frac{\text{Span}}{250}$

so serviceability does not govern.

Q.4



- $\therefore$  No horizontal reaction  $\therefore$  No horizontal reaction at A.  
 $\therefore$  No moments at B and C.  
 $\therefore$  Can treat beam as simply-supported.

20 m span,  $w = 100 \text{ kN/m}$ ,  $M_{\text{centre}} = \frac{wl^2}{8} = \frac{100(20)^2}{8} = 5000 \text{ kNm}$ .

Shear at ends  $= \frac{wl}{2} = \frac{100(20)}{2} = 1000 \text{ kN}$ .

a) Compactness

$\lambda = \frac{b}{t} \sqrt{\frac{E}{355}}$  DS4  $\sqrt{\frac{235}{355}} = 0.8136$

Flanges:  $b = \frac{750 - 2(20) - 2(10)}{2} = 230 \text{ mm}$

$t = 10$

$\frac{b}{t} = 23$

$\frac{b}{t} \sqrt{\frac{235}{355}} = 18.71 < 24 \therefore$  Compact in compression

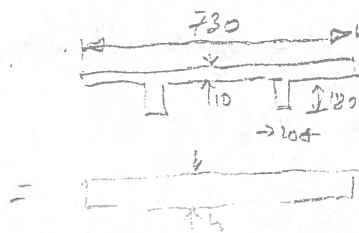
Web  $b = \frac{1500 - 2(10) - 20}{2} = 730$

$\frac{b}{t} \sqrt{\frac{235}{355}} = 59.4 > 56 \therefore$  NOT Compact in bending

$> 24$  NOT Compact in compression

Stiffness  $\frac{b}{t} \sqrt{\frac{235}{355}} = \frac{180}{20} (0.8136) = 7.32 < 8 \therefore$  Compact.

b) Smeared Section

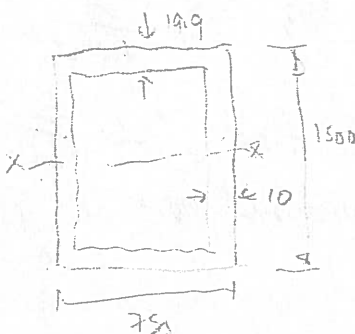


Area  $= 730(10) + 2(180)(20)$   
 $= 14500 \text{ mm}^2$

$\therefore h(730) = 14500$

$\therefore h = 19.9 \text{ mm}$

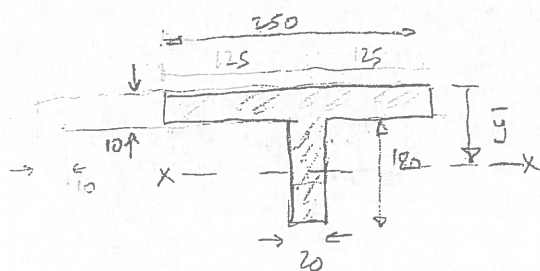
Ignore stiffeners in web as close to neutral axis



$$I_{xx} = \frac{750(1500)^3}{12} - \frac{730(1500 - 2(19.9))^3}{12}$$

$$= 21.54 \times 10^9 \text{ mm}^4$$

c)  $M = 12.50 \text{ kNm}$ ,  $\sigma = \frac{M y}{I} = \frac{(5000 \times 10^3)(0.75)}{(21.54 \times 10^{-3})} = 174 \text{ MPa}$



$$A = 250(10) + 180(20) = 6100 \text{ mm}^2$$

$$A \bar{y} = (250)(10)(5) + (180)(20)(10+90) = 372500 \text{ mm}^3$$

$$\bar{y} = \frac{372500}{6100} = 61.1 \text{ mm}$$

$$I_{xx} = \frac{(250)(10^3)}{12} + 250(10)(61.1-5)^2 + \frac{20(180^3)}{12} + 20(180)(100-61.1)^2$$

$$= 20.83 \times 10^3 + 7.868 \times 10^6 + 9.72 \times 10^6 + 5.448 \times 10^6 = 23.06 \times 10^6 \text{ mm}^4$$

Check Plastic:  $N_{pe} = \sigma_y A = 235 \text{ N/mm}^2 \times 6100 \text{ mm}^2 = 1433.5 \text{ kN}$

Check Elastic:  $N_{el} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 (210 \times 10^3 \text{ N/mm}^2)(23.06 \times 10^6 \text{ mm}^4)}{(2500)^2 \text{ mm}^2} = 7647 \text{ kN}$

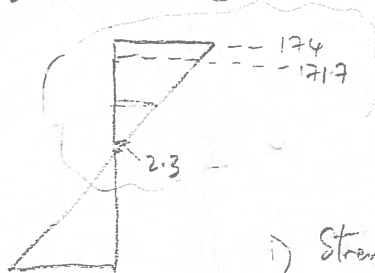
$$\lambda = \sqrt{\frac{N_{pe}}{N_{el}}} = \sqrt{\frac{1433.5}{7647}} = 0.433 \quad \text{Use curve c) welded, } \frac{b}{t_f} = 25 \leq 30$$

DS1  $\rightarrow \chi \approx 0.87$   $N_{design} = 0.87(1433.5) = 1247 \text{ kN}$

But require  $\sim 174 \text{ MPa} \times 6100 \text{ mm}^2 = 1061 \text{ kN}$   $\therefore \underline{\underline{OK}}$

d) Check Centre.

Shear stress = 0



$$\sigma_c = \frac{171.7 + 2.3}{2} = 87 \text{ MPa}$$

$$\sigma_b = 171.7 - 87 = 84.7 \text{ MPa}$$

i) Strength  $\sigma^2 \leq \sigma_y^2 - 3\tau^2 = \sigma_y^2$  ✓  
(174 < 235)

ii) Stability  $\lambda = \frac{b}{t} \sqrt{\frac{235}{355}} = 59.4$  part a).

$$\sigma_c = 87 \text{ MPa}$$

$$\lambda = 59.4 \rightarrow K_b = 0.5$$

$$\sigma_b = 84.7 \text{ MPa}$$

$$\lambda = 59.4 \rightarrow K_b \approx 1.15$$

DS4.

$$\tau = 0$$

Q4 d) cont'd.

$$\frac{(87)}{0.5(235)} + \left[ \frac{84.7}{(1.15(235))} \right]^2 + 0 < 1^2$$

$$0.74 + 0.10 = 0.84 < 1 \quad \therefore \underline{\text{OK for stability}}$$

Check ends of beam for shear.

$$\text{Shear force} = 1000 \text{ kN}$$

$$\text{Shear area} = (1506)10 \times 2 \text{ mm}^2 = 30 \times 10^3 \text{ mm}^2$$

$$\text{Shear stress} = \frac{1000 \times 10^3 \text{ N}}{30 \times 10^3 \text{ mm}^2} = \underline{\underline{33.3 \text{ MPa}}} < \frac{235}{\sqrt{3}} = 135 \text{ MPa}$$

 $\therefore \text{OK for strength + stability.}$ 

e) No improvements req'd. Could possibly increase spacing of cross-frames if desired, for economy.

**ENGINEERING TRIPOS PART IIB 2018  
COMMENTS FROM ASSESSOR'S REPORT  
MODULE 4D10, STRUCTURAL STEELWORK**

**Question 1: axial buckling/ lateral torsional capacity**

Axial buckling in the first part contended with flexibilities of the end connections using hyperbolic graphs: this was a new question feature and many candidates chose not to answer this part, which was a straightforward substitution of “EI” values into each  $k_1$  and  $k_2$  formula, in order to establish a critical effective length from the charts. Use of the standard Euler buckling formula was then sufficient for the critical buckling load. The second part on traditional lateral torsional buckling was answered well; ideally, candidates ought to have considered all sections for their proximity to critical behaviour, but many guessed the correct section alone.

**Question 2: Perry-Robertson formula and implementation**

The least favourite question but answered mostly well by all. Some did not derive the Perry-Robertson formula from first principles: almost everyone gave clear and fully discursive answers for the rest of the question, which was pleasing to see.

**Question 3: composite floor design**

The floor slab layout and supporting beams were specified, and candidates were asked to calculate the ultimate live load for both ULS and SLS design; in previous years, the load was typically specified with candidates choosing a supporting beam. These parts were executed well although some candidates forgot that only live loads mattered for the SLS assessment. The number of shear studs also had to be found: virtually no-one suggested pairing studs in each trough, when this would have reduced their overall number, but proposed solutions were nonetheless satisfactory.

**Question 4: plate girder design**

The strength and panel stability of a plate-girder portal frame was assessed. This was answered well for most but typical errors included incorrect calculation of stress resultants, in order to find the relevant stresses, and using the wrong effective width for the top flange. Some candidates also assessed critical moment and shear together in view of strength and stability: these occurred at different positions within the structure, indeed, at maximum moment, there is no shear force: separate assessments for each position have to be carried out.

K A Seffen, Second Assessor (not setter), May 2018