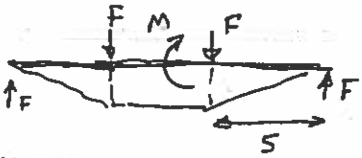
410 STRUCTURAL STEELWORK. 2017 (FAM)

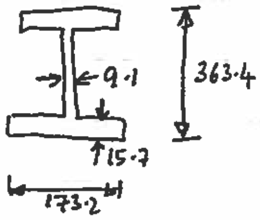
1 (a)



Moment $M = 5F$ Shear $= F$ in end spans.

Centre span critical.

Beam $356 \times 171 \times 67$ UB, Grade S275



Major $I_{xx} = 19460 \times 10^{-8}\,m^4$

Minor $I_{yy} = 1362 \times 10^{-8}\,m^4$

Major $Z_p = 1211 \times 10^{-6}\,m^3$

$J = 55.7 \times 10^{-8}\,m^4$

$A = 85.5 \times 10^{-4}\,m^2$

Compactness check: (DS4)

Ext. plate in compression (flange) $\lambda = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} = \frac{(173.2-9.1)}{2\,(15.7)}\sqrt{\frac{275}{355}} = 4.6 < 8 \therefore$ ok ✓

Int. plate in compression (web) } $\lambda = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} = \frac{363.4-2(15.7)}{9.1}\sqrt{\frac{275}{355}} = 32.1$

" " " bending (web) }

$32.1 > 24$ $\therefore$ NOT COMPACT IN COMPRESSION

$32.1 < 56$ $\therefore$ Compact in bending.

Calculate the plastic: $M_{pl,major} = Z_p \sigma_y = (1211 \times 10^{-6}\,m^3)(275 \times 10^6)\,N/m^2$

$= \underline{333\ kNm}$

Calculate elastic: $M_{LT} = \frac{\pi}{L}\sqrt{GJEI_{min}}\left(1 + \frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}\right)^{1/2}$

Basic moment $= \frac{\pi}{5}\sqrt{\left(\frac{210 \times 10^9}{2.6}\right)(55.7 \times 10^{-8})(210 \times 10^9)(1362 \times 10^{-8})}$

$= \frac{\pi}{5}(210 \times 10)\sqrt{\frac{(55.7)(1362)}{2.6}} = \underline{\underline{225.4\ kNm}}$

$\Gamma = \frac{ID^2}{4} = \frac{(1362 \times 10^{-8})(363.4 - 15.7)^2 \times 10^{-6}}{4} = 0.4116 \times 10^{-6}\,m^6$

$= 41.2 \times 10^{-8}\,m^6$

$1 + \frac{\pi^2}{L^2}\frac{E\Gamma}{GJ} = 1 + \frac{\pi^2}{5^2}(2.6)\frac{41.2 \times 10^{-8}}{55.7 \times 10^{-8}} = 1 + 0.7586 = 1.7586$

$\sqrt{\ } = 1.326$

$M_{LT,BASIC} = 225.4\ kNm \times 1.326 = \underline{\underline{299\ kNm}}$

4D1
Q1(a) Cont'd

$C_{unequal}$, worst case $= 1.0$

$$M_{LT} = \frac{M_{LT\ BASIC}}{C_{unequal}} = \underline{\underline{299\ kNm}}$$

Sausage: $\lambda = \sqrt{\frac{Plastic}{Elastic}} = \sqrt{\frac{333}{299}} = 1.055$

Which curve? $\frac{h}{b} = \frac{363.4}{173.2} = \cancel{\ \ } 2.1 \ \cancel{\ \ } > 2 \rightarrow$ Curve (b) (DS3 for LTB)

Curve (b) DS1, for $\lambda = 1.055 \Rightarrow \chi \simeq 0.575$

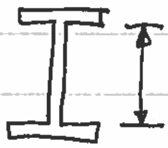
$\therefore \quad M_{design} = \chi M_{pl} = 0.575(333) = \underline{\underline{191.5\ kNm}}$

$M_{applied} = 5F \quad \therefore \quad F_{max} = \frac{191.5}{5} = \underline{\underline{38.3\ kN}}$

b) ~~Beam~~ Column: Web not compact for axial, so ignore part of web

$\lambda = 32.1$ from part a) $> 24 \Rightarrow K_c \simeq 0.8$ (DS4)

$\therefore$ Ignore $(1-K_c) = 0.2$ of the web



$363.4 - 2(15.7) = 332$, so remove $0.2(332) \times 9.1 = 604\ mm^2$

$\therefore A_{eff} = 8550\ mm^2 - 604\ mm^2 = \underline{79.46 \times 10^{-4}\ m^2}$

Calculate the plastic: $N_{pl} = (79.5 \times 10^{-4}\ m^2) \times (275 \times 10^6\ N/m^2)$

$= \underline{2186\ kN}$

Calc elastic: $N_{elastic,\ minor} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2}{5^2}(210 \times 10^9)(1362 \times 10^{-8}) = \underline{\underline{1129\ kNm}}$

$N_{elastic,\ major} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2}{15^2}(210 \times 10^9)(19460 \times 10^{-8}) = \underline{\underline{1793\ kNm}}$

$\lambda_{min} = \sqrt{\frac{Plastic}{Elastic}} = \sqrt{\frac{2186}{1129}} = 1.39$

$\lambda_{maj} = \sqrt{\frac{Plastic}{Elastic}} = \sqrt{\frac{2186}{1793}} = 1.10$

Which curve (DS2) $\frac{h}{b} = 2.1 > 1.2$, $t_f < 40mm$, z-z ~~y~~, S275 $\rightarrow$ $\underline{\underline{\text{Curve (b)}}}$

$\chi_{min} = 0.38 \qquad \chi_{maj} = 0.54$

4D10
Q1(b)
cont'd.

$\therefore\ N_{design} = 0.38(2186) = 831\text{ kN}$ minor
$\quad\quad\quad\quad\ \ 0.54(2186) = 1180\text{ kN}$ major

So $P_{crit} = \underline{\underline{831\text{ kN}}}$, failure by buckling in combination of minor axis flexural buckling, and yielding.
~~lateral torsional buckling~~

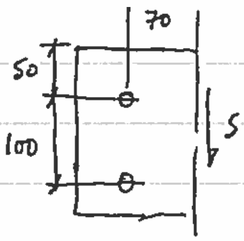
(also part 1(a) – check shear:

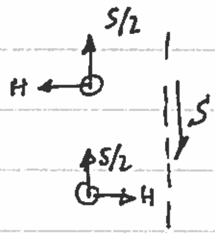
$$F_{max} = 38.3\text{ kN}$$

$$\tau = \frac{38.3\times10^3\text{ N}}{(9.1)(332)\text{ mm}^2} = 12.7\text{ N/mm}^2 < \frac{275}{\sqrt{3}}\ \checkmark \text{ Von Mises}$$
)

Q2 a)

4D10
2017.

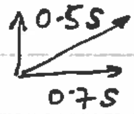




$M = 70 \times S = 100H$

$\therefore \quad H = 0.7S$.

$\therefore$ Any bolt is critical.



$= 0.86\,S$ resultant.

Check bolts

M16, 8.8 $\rightarrow$ 55 kN shear (DSS a) = 110 kN double shear

$\therefore$ need $0.86\,S < 110\text{ kN} \Rightarrow S < \underline{\underline{128\text{ kN}}}$ ①

Check plate in bearing $= \underset{\text{N/mm}^2}{2(450)}\underset{\text{mm}}{(10)}\underset{\text{mm}}{(16)} = \underline{\underline{144\text{ kN}}}$ (= 288kN double shear (as two plates)) ②

$\therefore$ ~~need 2 0.86~~ $\therefore$ Not critical.

Check web in bearing $= 2(275)(9.1)(16) = 80\text{ kN}$ single shear ③

$\therefore$ need $0.86\,S < 80\text{ kN} \Rightarrow \underline{\underline{S = 94\text{kN}}}$ $\underline{\underline{\text{CRITICAL}}}$

$\therefore \quad S_{ULS} = 94\text{ kN}$. web in bearing ③.

b). $M_{section,pl,maj} = \underline{\underline{333\text{ kNm}}}$ (see Q1(c)). ①

Need to subtract bolt holes.

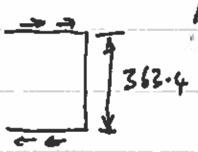
Each is $(16\text{ mm} \times 15.7\text{ mm}) \times 2$ of them $\times (363.4 - 15.7)$ mm lever arm

$= 174.7 \times 10^3\text{ mm}^3 = 174.7 \times 10^{-6}\text{ m}^3$

$\therefore \quad Z_{pl,maj} = (1211 - 174.7) \times 10^{-6}\text{ m}^3$

$M_{maj} = (1036 \times 10^{-6})(275 \times 10^6) = \underline{\underline{284\text{ kNm}}}$ ②.

Bolt shear



M16 x 8.8 $\rightarrow$ $\underline{55\text{ kN}}$ (DS5a) single shear

x 4 of them

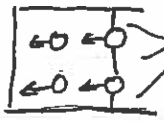
x 0.3634 m lever arm

$= \underline{\underline{80\text{ kNm}}}$ ③

Splice plate bearing $= 2\sigma_y dt = \underline{\underline{144\text{kN}}}$ (see above) $> 55\text{kN}$ $\therefore$ not critical ④

Flange bearing $= 2(275)(16)(15.7) = 138\text{ kN} > 55\text{ kN}$ $\therefore$ not critical ⑤.

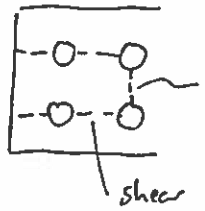
Splice plate tension (or compression)



area $= (160-32)(10) = 1280\,mm^2 \times 450\,N/mm^2$

$= 576\,kN \gg 4 \times 55\,kN$ bolts

$\therefore$ NOT CRITICAL (6)

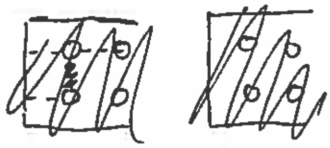


$(80-16)(10) \times 450 = 288\,kN$

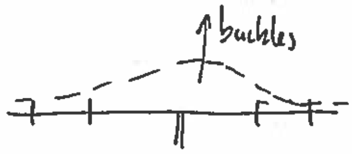
+ shear

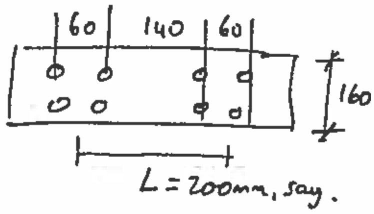
(already greater than $4 \times 55 = 220\,kN$ bolts)

$\therefore$ NOT CRITICAL (7)



Splice plate compression.





~~Use plastic~~ ~~Use~~ Treat as a column.

Calculate plastic: $N_{pl} = (160)(10)(450) = \underline{\underline{720\,kN}}$

Calc. elastic: $N_{el} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2}{(0.2)^2}(210 \times 10^9)(1.33 \times 10^{-8}) = \underline{\underline{690\,kN}}$

$I = \frac{bd^3}{12} = \frac{(0.16)(0.01)^3}{12} = 1.33 \times 10^{-8}\,m^4$

$\lambda = \sqrt{\frac{\text{Plastic}}{\text{Elastic}}} \rightarrow \sqrt{\frac{720}{690}} = 1.021 \rightarrow \chi = 0.45$ worst case (curve d)

$\rightarrow N = 0.45(720) = \underline{\underline{324\,kN}}$

$\times$ lever arm $= 0.363 \times 324 = \underline{117.6\,kNm}$ (8)

NOT CRITICAL

$\therefore$ Critical case is (3), shear in bolts $\rightarrow$ $\underline{\underline{80\,kNm}}$

Q3
4D10
2017

a) Check compactness.

Flange: $b = 200 - 20 = 180$
$t = 10$

$\frac{b}{t}\sqrt{\frac{\cancel{235}\ 450}{355}} = \cancel{18.8}\ 20.3 < 24$ ∴ Compact in Compression

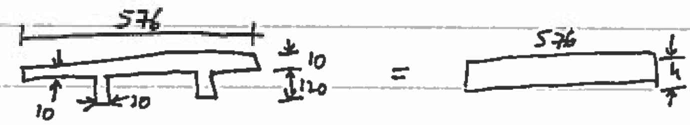
Web: $\frac{b}{t} = \frac{\cancel{1000}\ \cancel{500}\ \cancel{820}\ 600 - 20}{10} = \frac{\cancel{480}\ 5}{10}$

$\frac{b}{t}\sqrt{\frac{\cancel{235}\ 450}{355}} = 65.3 > 56$ ∴ NOT Compact in bending or compression

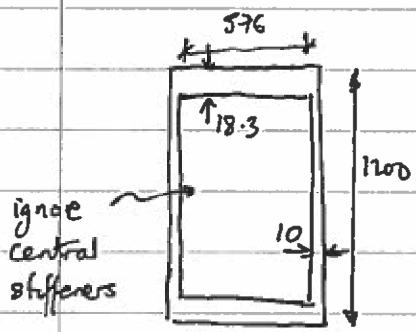
Stiffeners $\frac{b}{t} = \frac{120}{20}$

$\frac{b}{t}\sqrt{\frac{450}{355}} = 6.75 < 8$ ∴ Compact.

b) Smeared section



$h = \frac{(576)(10) + 2(120)(20)}{576} = \underline{18.3\text{mm}}$



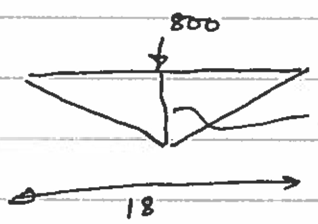
$I_{maj} = \frac{2(576)(18.3)^3}{12} + 2(576)(18.3)\left(600 - \frac{18.3}{2}\right)^2$

$+ \frac{2(10)(1200)^3}{12}$

$= 588.3 \times 10^3 + 7.36 \times 10^9 + 2.88 \times 10^9$

$= \underline{\underline{10.24 \times 10^9\ \text{mm}^4}}$

c)

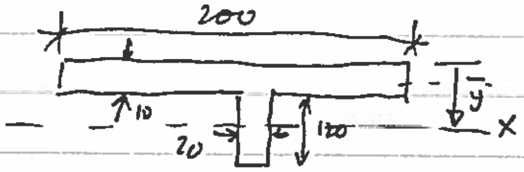


$\frac{WL}{4} = \frac{(800)(18)}{4} = \underline{3600\ \text{kNm}}$

$\sigma = \frac{My}{I} = \frac{(3600 \times 10^3)(0.6)}{10.24 \times 10^{-3}} = \underline{\underline{211\ \text{MPa}}}$

OK

3 c) cont'd



$40-5-5 = 35.5$

$70-40.5 = 29.5$

$A = (200)(10) + (120)(20) = 4400 \text{ mm}^2$

$A\bar{y} = (200)(10)5 + (120)(20)(70) = 178{,}000 \text{ mm}^3$

$\bar{y} = \dfrac{178{,}000}{4{,}400} = \underline{\underline{40.5 \text{ mm}}}$

$I_{xx} = \dfrac{(200)(10)^3}{12} + (200)(10)(35.5)^2 + \dfrac{20(120)^3}{12} + 20(120)(29.5)^2$

$= 16{,}667 + 2.52 \times 10^6 + 2.88 \times 10^6 + 2.089 \times 10^6$

$= \underline{7.51 \times 10^6} \text{ mm}^4$

Calc. plastic: $N_{pl} = 450 \text{ N/mm}^2 \times 4400 \text{ mm}^2 = \underline{1980 \text{ kN}}$

Calc. Elastic. $N_{elastic} = \dfrac{\pi^2 EI}{L^2} = \dfrac{\pi^2 (210 \times 10^3 \text{ N/mm}^2)(7.51 \times 10^6 \text{ mm}^4)}{(3000)^2 \text{ mm}^2}$

$= \underline{1729 \text{ kN}}$

$\lambda = \sqrt{\dfrac{N_{pl}}{N_{el}}} = \sqrt{\dfrac{1980}{1729}} = 1.07.$ Use curve c) (Welded)

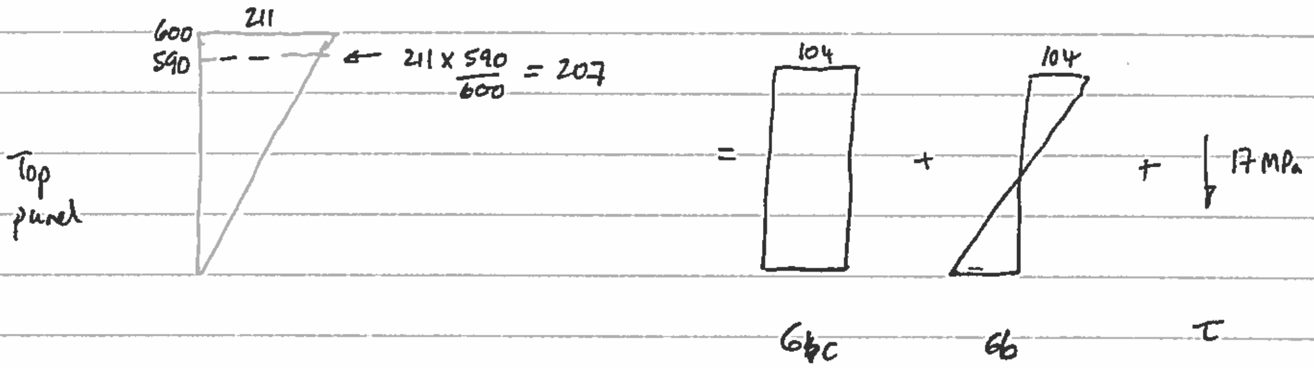
$\rightarrow \chi = 0.51$ (DS1)

$N_{design} = 0.51\,(1980) = \underline{\underline{1010 \text{ kN}}}$

but require $(4400 \text{ mm}^2) \times 211 \text{ N/mm}^2 = \underline{\underline{928 \text{ kN}}}$ $\therefore$ $\underline{\underline{\text{OK}}}$

3 d) Shear force $= \dfrac{W}{2} = 400$ kN

Shear stress $\tau = \dfrac{400 \times 10^3}{2(10)(1180)} = \underline{\underline{16.9 \text{ MPa}}}$.



Strength $\quad \sigma^2 \le \sigma_y^2 - 3\tau^2 \quad$ DS4

$\sqrt{\sigma^2 + 3\tau^2} < \sigma_y$?

$\sqrt{(211)^2 + 3(16.9)^2} = 213 < 450$ ✓ OK

Stability : $\lambda = 65.3$ (part a)

$\sigma_c = 104$ MPa $\qquad \lambda = 65.3 \rightarrow K_c = 0.42$

$\sigma_b = 104$ MPa $\qquad \lambda = 65.3 \rightarrow K_b = 1.1$

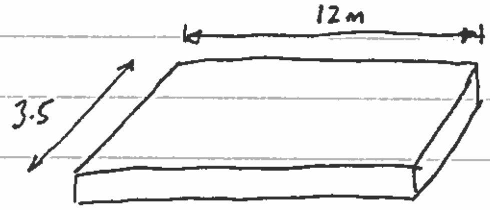
$\phi = 3/0.5 \ge 3, \; \lambda = 65.3, \qquad K_q = 0.73$

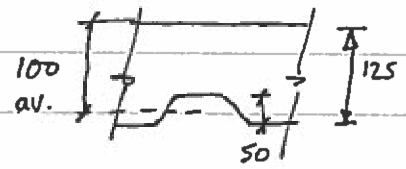
$$\frac{104}{(0.42)(450)} + \left(\frac{104}{1.1(450)}\right)^2 + \left(\frac{16.9}{0.73(450/\sqrt{3})}\right)^2$$

$$0.55 + 0.04 + 0.01 = 0.6 < 1$$

$\therefore$ STABILITY OK.

4D10 Q4.
2017





a) Loads: Concrete $= 3.5\text{m} \times 0.1\text{m} \times (2.4 \times 9.81)\text{kN/m}^3 = 8.24\text{ kN/m}$

Steel $= 82.2 \times 9.81 = 806\text{ N/m} = 0.81\text{ kN/m}$

$\underline{9.05\text{ kN/m}}$ Dead

Permanent Services $3\text{kN/m}^2 \times 3.5\text{m} = 10.5\text{ kN/m}$ Permanent

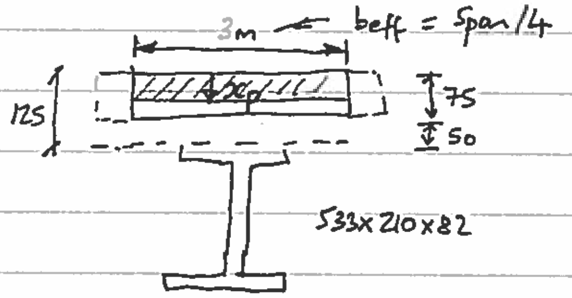
$\underline{19.55\text{ kN/m}}$ Perm + Dead.

Live Load $7\text{kN/m}^2 \times 3.5\text{m} = \underline{24.5\text{ kN/m}}$

Total factored $= 1.35(19.55) + 1.5(24.5) = \underline{63.1\text{ kN/m}}$

$$M_{max} = \frac{wl^2}{8} = \frac{(63.1)(12)^2}{8} = \underline{1137\text{ kNm}}$$

STRENGTH.



Assume N/A in concrete at depth x_p below top of slab

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

$A_s = 105\text{ cm}^2 = 105 \times 10^2\text{ mm}^2$, $\sigma_y = 355\text{ MPa}$

flanges $\frac{b}{t}\sqrt{1} = \frac{(208.8 - 9.6)}{2(13.2)} = 7.5 < 8$ OK

web $\frac{b}{t}\sqrt{1} = \frac{528.3 - 2(13.2)}{9.6} = 52.2 < 56$ OK for bending

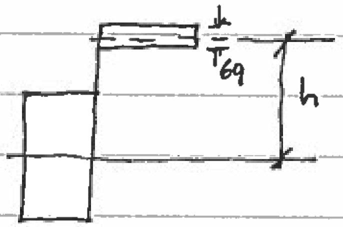
> 24 Not compact for compression (but OK because in tension.

4D10
Q4 a)
Cont'd

$$x_p = \frac{(10500 \text{ mm}^2)(355 \text{ N/mm}^2)}{0.6\,(30\,\text{N/mm}^2)(3000\text{ mm})} = \underline{\underline{69\text{ mm}}} < 75\text{ mm}$$

(3000 mm $\uparrow$ b_{eff})

$\therefore$ In concrete ✓.



$\therefore M = (A_s \sigma_y)\, h$

$A_s \sigma_y = (10500)(355) = 3727.5$ kN

$h = \frac{528.3}{2} + 125 - \frac{69}{2} = 354.7$ mm $= 0.355$ m

$\therefore \quad M_{strength} = 3727.5 \times 0.355 = \underline{\underline{1322\text{ kNm}}}$

$> \underline{1137\text{ kNm}}$ req'd

$\therefore$ ADEQUATE

b) shear studs $\quad$ 100 mm x 25 mm $\rightarrow$ 154 kN each, DS6

Axial force – 3728 kN $\rightarrow$ 24.2 studs per half span, say 25

$\rightarrow$ $\underline{\underline{50\text{ studs total}}}$.

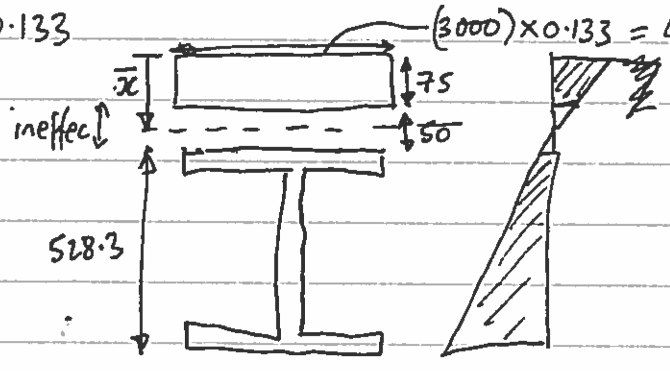
Spacing $= \frac{12\text{ m}}{50} = \underline{\underline{0.24\text{ m}}}$ $\quad$ Troughs 200 mm, so put one stud in each trough $\rightarrow$ 60 studs total.

c) Short term deflection, under live load only

Load $= 7$ kN/m$^2 \times 3.5$ m $= 24.5$ kN/m

EI: Modular ratio $\quad E_c = 28$ GPa (short term, DS6)

$\frac{28}{210} = 0.133$



Assume N/A in gap.

Q4c $\quad A\bar{x} = \int x \, dA$ $\qquad$ from top surface

$$\left[(400 \times 75) + 10{,}500\right]\bar{x} = 400 \times 75 \times \frac{75}{2} + 10{,}500\left[125 + \frac{528.3}{2}\right]$$

$$= 1{,}125 \times 10^3 + 4{,}086 \times 10^3$$

$$(40.5 \times 10^3)\,\bar{x} = 5{,}211 \times 10^3 \text{ mm}^3$$

$$\bar{x} = 128.7 \text{ mm} = 129 \text{ mm}$$

Greater than gap. $\therefore$ N/A in steel.

$$I = \frac{(400)(75)^3}{12} + (400)(75)\left(129 - \frac{75}{2}\right)^2 + 47540 \times 10^4 + 10{,}500\, h^2$$

$$h = \frac{528.3}{2} - (4 \text{mm}) = 260 \text{ mm}$$

$$= 14.1 \times 10^6 + 251.2 \times 10^6 + 475.4 \times 10^6 + 709.8 \times 10^6$$

$$= 1450 \times 10^6 \text{ mm}^4 = 1450 \times 10^{-6} \text{ m}^4$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(24.5 \times 10^3)(12)^4}{384(210 \times 10^9)(1450 \times 10^{-6})}$$

$$= 21.7 \text{ mm}$$

$$\frac{\text{Span}}{250} = \frac{12}{250} = 48 \text{ mm}, \qquad 21.7 \text{mm} < 48 \text{mm} \quad \therefore \text{OK}$$

ENGINEERING TRIPOS PART IIB 2017
COMMENTS FROM ASSESSOR'S REPORT
MODULE 4D10, STRUCTURAL STEELWORK

Question 1: lateral torsional/axial buckling capacity
The buckling limits of a simply supported beam were determined for lateral torsional and axial behaviour. Common mistakes were not reading the design curves accurately enough or not checking the shear capacity of the cross-section; in part (b) some candidates did not perform an effective area reduction at the start of the calculation, and some assumed that minor axis buckling always governed *i.e.* they did not perform a major axis buckling check.

Question 2: splice plate joint capacity
This type of question does not appear regularly on past papers, as reflected in the one attempt. Candidates were asked to assess the joint capacity of a splice connection, in order to determine ultimate limits for its bending moment and shear capabilities. It was a straightforward, if detailed question, but very poorly answered in this one case.

Question 3: plate girder panel buckling
The local buckling capacities of individual web panels in a plate girder section and its compressive limit for the top flange were assessed. An almost universal mistake was assuming that the webs should be "smeared" for calculating the cross-sectional bending stiffness: but because the only web stiffeners lay on the axis of bending, this was not essential (nor indeed would smearing apply at all in this context). Some solutions did not extract the correct width of "T-beam" for the compressive assessment, and approximately half of the candidates could not estimate the web shear stresses: either they forgot there were two webs or choose the web length between stiffeners instead of the whole length in order to calculate the shear stress area.

Question 4: composite floor capacity
The ultimate moment capacity was calculated alongside the number of shear studs for composite action whilst ensuring the cross-section was stiff enough. Very few errors were present although some candidates did not enforce the spacing requirement for shear studs or mis-calculated the total loading on floor.

K A Seffen, Second Assessor (not setter), May 2017