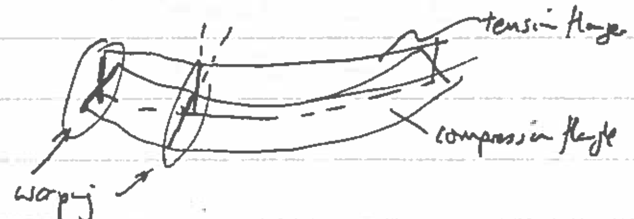
2015.

4D10 Q1.

1 a). Lateral torsional buckling of an I-beam inevitably involves warping of the section, which the section will try to resist. The warping arises because the tension + compression flanges deflect by different amounts, and so therefore warp.



(i.e. plane sections not remaining plane).

The differential curvatures of the flanges imply differential shear stresses which create a torque resisting the deflection. So, it is appropriate to take this resistance (this warping restraint) into account when calculating LTB.

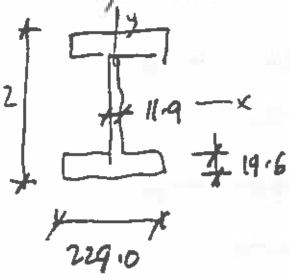
[20%]

b) One of the main difficulties is that Perry-Robertson assumes a form for the imperfection, and so if you apply this to Lateral-Torsional Buckling, you need to assume shapes + magnitudes for two independent imperfections the lateral bow and the initial twist. This complicates the mathematics considerably, and is also difficult to calibrate against real data. Hence EC3 uses Perry-Robertson-like approach merely as a sort of curve fit to LTB buckling tests.

[20%]

4D10 . Q1.

c). S275, 610 x 229 UB 125.



maj $I_{xx} = 98610 \times 10^{-8}\ m^4$

min $I_{yy} = 3932 \times 10^{-8}\ m^4$

maj $Z_{px} = 3676 \times 10^{-6}\ m^3$

min $Z_{py} = 535 \times 10^{-6}\ m^3$

$J = 154 \times 10^{-8}\ m^4$

$A = 159 \times 10^{-4}\ m^2$

Check compactness. DS4

– external plate in compression 

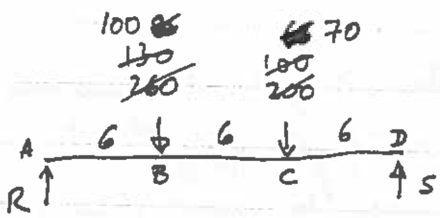
$$\lambda = \frac{b}{t}\sqrt{\frac{275}{355}} = \frac{(229-11.9)}{2(19.6)}\sqrt{\frac{275}{355}} = 4.87 \le 8 \quad \therefore OK$$

– internal plate in bending (no axial load) 

$$\lambda = \frac{b}{t}\sqrt{\frac{275}{355}} = \frac{[612.2-2(19.6)]}{11.9}\sqrt{\frac{275}{355}}$$

$$= 42.4 \le 56 \quad \therefore OK$$

BMD.



$\overset{\curvearrowright}{M_A +}$: $\cancel{260}\,100(6) + \cancel{200}\,70(12) = S(18) \rightarrow S = \cancel{120}\ 80$ kN

$R + S = \cancel{460}\ 170 \rightarrow R = \cancel{240}\ 90$ kN.

$\therefore\ M_B = 6R = 6(\cancel{240}\ 90) = \cancel{1440}\ 540$ kNm

$M_C = 6S = 6(\cancel{220}\ 80) = \cancel{1320}\ 480$ kNm.

Half.
~~440~~
~~720~~
720
~~660~~.

Calculate the plastic

$$M_{pl\ maj} = \sigma_y Z_{px} = 275 \times 10^6\ N/m^2 \times 3676 \times 10^{-6}\ m^3$$

$$= 1011\ kNm. \qquad \therefore \text{OK. cf } 720 \text{ kNm.}$$

Calculate elastic.

Q1

(c) Cont'd

$$M_{LT,Basic} = \frac{\pi}{L}\sqrt{EIGJ}\left(1+\frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}\right)^{1/2}$$

$$\frac{\pi}{6}\sqrt{210\times10^9 . 3932\times10^{-8} . 81\times10^9 . 154\times10^{-8}} \quad Nm.$$

$$= \frac{\pi}{6} . 10 . \sqrt{210(3932)(81)(154)} = \frac{\pi}{6}(10)\ 101.5 \quad kNm.$$

$$= \underline{\underline{531.4\ kNm}}.$$

$$\cancel{I_w}\ \Gamma = \frac{I D^2}{4} = \frac{(392\times10^{-8})[(612.2-19.6)/1000]^2}{4}\ m^6.$$

$$= 345\times10^{-8}\ m^6.$$

$$1+\frac{\pi^2}{L^2}\frac{E\Gamma}{GJ} = 1+\frac{\pi^2}{6^2}\frac{210}{81}\frac{345\cancel{\times10^{-8}}}{54\cancel{\times10^{-8}}} = 1+1.593$$

$$= 2.593.$$

$$\sqrt{1+\frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}} = 1.6104.$$

$$\therefore M_{LT}\ basic = 531.4\ kNm \times 1.6104 = \underline{855.7\ kNm}.$$

$C_{unequal} = ?$ Central section is worst case.

$$\psi = \frac{\cancel{660}}{\cancel{720}} = \cancel{0.9167} \quad \frac{480}{540} = 0.889$$

$$C_{unequal} = 0.6+0.4(\cancel{0.9167}\ 0.889) = \cancel{0.9667}.\ 0.9556$$

$$M_{cr} = \frac{M_{LT}}{C_{unequal}} = \frac{855.7}{\cancel{0.9667}\ 0.9556} = \frac{\cancel{885.2}}{\underline{\underline{895}}}\ \frac{\cancel{kNm}}{kNm}$$

Savage. Plover $\lambda = \sqrt{\frac{Plastic}{Elastic}} = \sqrt{\frac{1011}{\cancel{885.2}\ 895}} = \cancel{1.069}\ 1.0625$

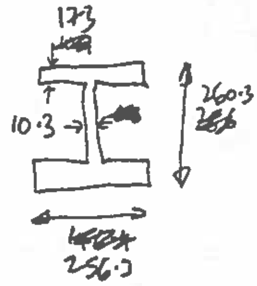
60%

Which curve, LTB, $\frac{h}{b} = \frac{612.2}{229.0} = 2.67 > 2 \quad \therefore$ Curve(b).

From graph, DS1: $\chi \simeq 0.55$ curve(b) @ $\lambda = 1.0625$

Q2.

254 x ~~[illegible]~~ 254 UC ~~[illegible]~~ 89



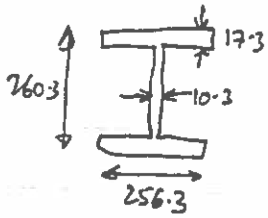
Maj:	I_{xx}	14270 $\times 10^{-8}$ m⁴
Min:	I_{yy}	4857 $\times 10^{-8}$ m⁴
Maj:	$Z_{pl\,x}$	1224 $\times 10^{-6}$ m³
Min.	$Z_{pl\,y}$	575 $\times 10^{-6}$ m³
	J	102 $\times 10^{-8}$ m⁴
	A	113 $\times 10^{-4}$ m².

Check compactness.

~~Internal plate in compression:~~  ~~$\lambda = \frac{b}{t}\sqrt{\frac{f_y}{355}} = \frac{256-2(10.9)}{6.3}\sqrt{\frac{275}{355}} = 32.7 > 24$!~~

254 x 254 x 89 UC.

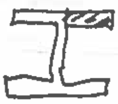
a)



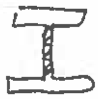
Internal compression.

$$\lambda = \frac{260.3 - 2(17.3)}{10.3}\sqrt{\frac{275}{355}} = 19.2 < 24 \quad \text{OK.}$$

External compression



$$\lambda = \frac{(256.3 - 10.3)}{2(17.3)}\sqrt{\frac{275}{355}} = 6.26 < 8 \quad \text{OK.}$$

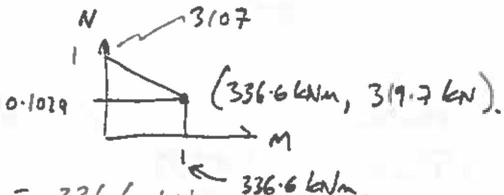
Internal, bending  $\lambda = 19.2 < 56$ again OK.

[10%]

b) Half web fraction. $A_{web} = [260.3 - 2(17.3)] \times (10.3) = 2{,}325 \text{ mm}^2 = 23.25 \text{ cm}^2$

$$a = \frac{A_{web}}{A_{TOT}} = \frac{23.25}{113} = 0.2057$$

$$\frac{a}{2} = 0.1029.$$



$M_{pl} = 275 \times 10^6 \text{ N/m}^2 \times 1224 \times 10^{-6} \text{ m}^3 = 336.6$ kNm.

$N_{pl} = 3107$ kN.

$a/2\ N_{pl} = 0.1029\,(3107) = 319.7$ kN

(10%)

Q2. Axial.

Plastic: $N_{pl} = \sigma_y A = 275\times10^6\ N/m^2 \times 113\times10^{-4} m^2 = 3107\ kN.$

Elastic: Major Euler $= \dfrac{\pi^2 EI}{L_{eff}^2} = \dfrac{\pi^2 (210\times10^9)(14270\times10^{-8})}{8^2} = 4621\ kN$

Minor Euler $= \dfrac{\pi^2 EI}{L_{eff}^2} = \dfrac{\pi^2 (210\times10^9)(4857\times10^{-8})}{4^2} = 6291\ kN.$

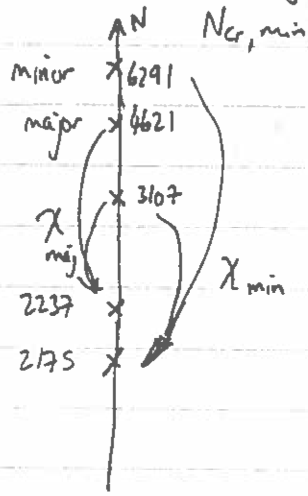
$\lambda_{maj} = \sqrt{\dfrac{\text{Plastic}}{\text{Elastic}}} = \sqrt{\dfrac{3107}{4621}} = 0.82$

$\lambda_{min} = \sqrt{\dfrac{\text{Plastic}}{\text{Elastic}}} = \sqrt{\dfrac{3107}{6291}} = 0.703$

Curve?	t_f	Axis		Curve	χ
$h/b = \frac{260.3}{256.3} = 1.01 < 1.2$	$17.3 < 100$	y-y	S275	b)	0.72
		z-z		c)	0.7

$N_{cr,maj} = 0.72(3107) \approx 2237\ kN$

$N_{cr,min} = 0.70(3107) \approx 2175.\ kN.$



Q2. <u>Moment.</u>

$$\cancel{M_{LT\,BASIC} = \frac{\pi}{L}\sqrt{EI}}$$

Plastic: $M_{pl\,major} = 336.6$ kNm.

Elastic. $M_{LT\,BASIC} = \frac{\pi}{L}\sqrt{EIGJ}\left(1 + \frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}\right)^{1/2}$

$$M_{LT,0} = \frac{\pi}{4}\sqrt{210\times10^9(4857\times10^{-8})(81\times10^9)(102\times10^{-8})}$$

$$= \frac{\pi}{4}(10)\sqrt{210(4857)(81)(102)} = \frac{\pi}{4}(10)(91.8) \text{ kNm}.$$

$$= \underline{721 \text{ kNm}}$$

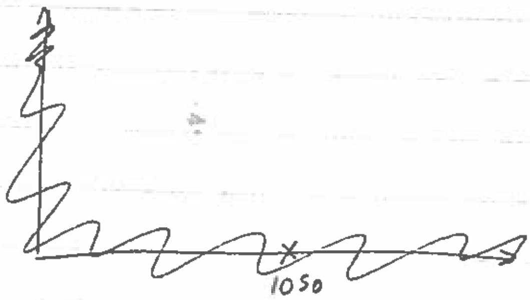
$$\Gamma = \frac{ID^2}{4} = \frac{(4857\times10^{-8})}{4}\left(\frac{260.3-17.3}{1000}\right)^2 = \underline{\underline{71.7 \times10^{-8}\ m^6}}$$

$$\left(1+\frac{\pi^2 E\Gamma}{L^2 GJ}\right)^{1/2} = \left(1 + \frac{\pi^2}{4^2}\frac{210}{81}\frac{71.7}{102}\right)^{1/2} = (1+1.1242)^{1/2}$$

$$= \underline{1.457}$$

$M_{LT,BASIC} = 721$ kNm $\times\ 1.457 = 1050$ kNm.

$C_{unequal} = 1$ (E_J + opp. end moments).

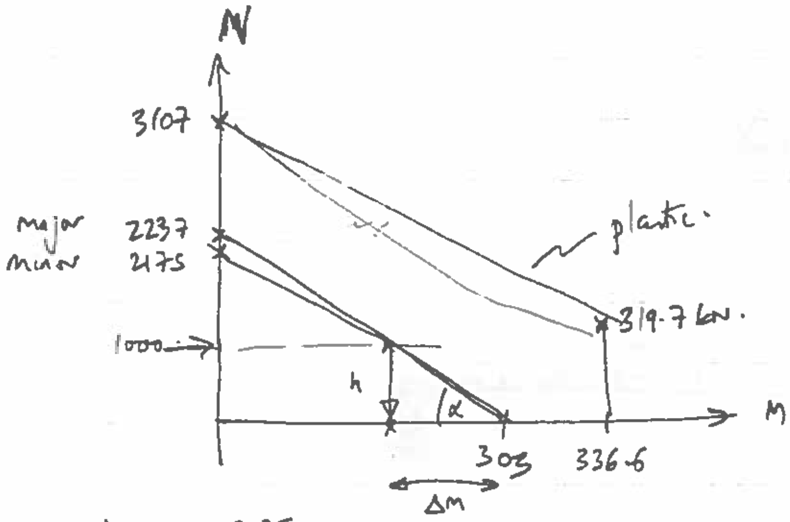


$$\lambda = \sqrt{\frac{\text{plastic}}{\text{Elastic}}} = \sqrt{\frac{336.6}{1050}} = 0.566.$$

Curve? $\frac{h}{b} \approx 1 \rightarrow$ Curve (a). $\rightarrow \chi = 0.9$

$$\therefore M_{design} = \chi M_{pl} = 0.9(336.6) = \underline{\underline{303 \text{ kNm}}}$$

Q2 d).



[60%]

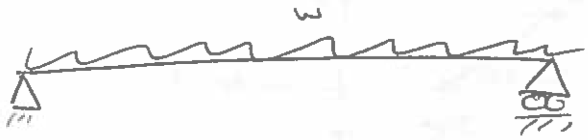
$\tan\alpha = \frac{2175}{303}$.

$\frac{h}{\Delta M} = \tan\alpha$ $\qquad \Delta M = \frac{h}{\tan\alpha} = \frac{1000}{2175}(303) = 139.3$

$\therefore M_{design} = 303 - 139.3 = \underline{\underline{163.7 \text{ kNm}}}$
(when $N = 1000$)

[20%]

QUESTION 3 (continued)



M_{max} occurs at mid-span

$$M_{max} = \frac{wl^2}{8}$$

structures databook §4.5.4

$$= 18.6\,w \text{ kN-m if } w \text{ in kN}$$

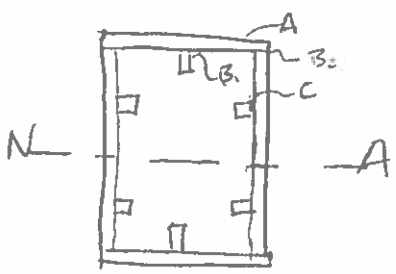
$$w = 80\text{ kN/m} \Rightarrow M_{max} = 1488.4 \text{ kN-m}$$

Calculate the second moment of Area, I

Ignore the stiffeners

$$I = \frac{1}{12}\left((500)(800)^3 - \underbrace{(500-2(10))}_{=480}\underbrace{(800-2(10))^3}_{=780}\right)$$

$$= 2.351 \times 10^9 \text{ mm}^4$$



At A: $y = 400$ mm ;

B: $y = 390$ mm ;

C: $y = 200$ mm ;

Q3

$$\sigma = \frac{My}{I}$$

struct. Databook §4.4.1

$M_{max} = 1488.4$ kN-m $= 1488.4 \times 10^6$ Nmm

$I = 2.351 \times 10^9$ mm^4

$\sigma_{max} = 0.633\, y$

At A: $y = 400$mm $\Rightarrow \sigma_{max} = 253$ MPa

B: $y = 390$ mm $\Rightarrow \sigma_{max} = 247$ MPa

C: $y = 200$ mm $\Rightarrow \sigma_{max} = 127$ MPa

Shear stress, τ

Shear force, $S = \frac{wL}{2}$ at ends; 0 in centre

Databook §4.3

$= 488$ kN

$$\tau = \frac{S}{A}$$

~~Check compactness~~

$A = \cancel{(800)(200)} = (780)(10)(2)$ webs

$= \cancel{25,600 \text{ mm}^2}$ 15,600

$\tau = \cancel{19 \text{ N/mm}^2}$ 31 N/mm^2 ends

0 centre

5 K

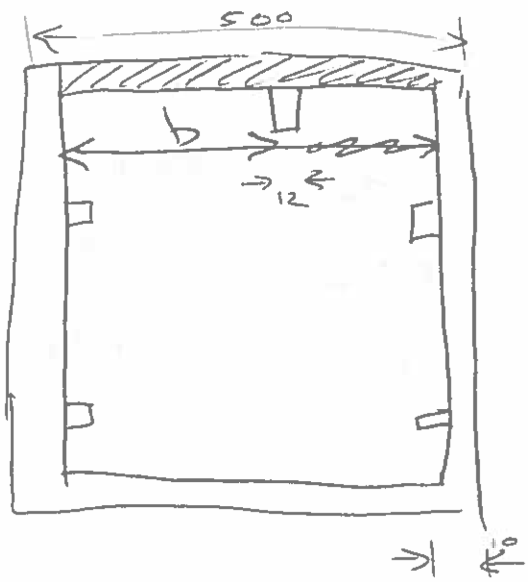
Q3

Check compactness

~~Bottom~~ Top flange in compression

$$\lambda = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$$

$b = 240$ mm



$$b = \left(\frac{500 \cancel{-12}}{2}\right) - 10$$

$= 240$ mm

$t = 10$ mm

$\therefore \lambda_{flange} = \cancel{23.4}\ 24 < 56$ Datasheet Part b

$\therefore$ compact section for bending (internal plate)

and $\lambda < 25$, compact ✓

6

Q3

Check the web for compactness.

Whole web

$$\lambda_{web} = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$$

$b = 780mm, \; t = 10mm$

$\lambda_{web} = 78 \; > 56$ — Datasheet 4, Part b

$\therefore$ need stiffeners

Check the stiffeners

$$\lambda_{stiff} = \frac{80}{12}\sqrt{\frac{\sigma_y}{355}}$$

$= 6.7 < 8$ (external plate) — Datasheet 1 Part b

$\therefore$ stiffeners are compact

Note: the students are not reqd. to check web or stiffener compactness and is included here only for completeness to show that it is a reasonable question.

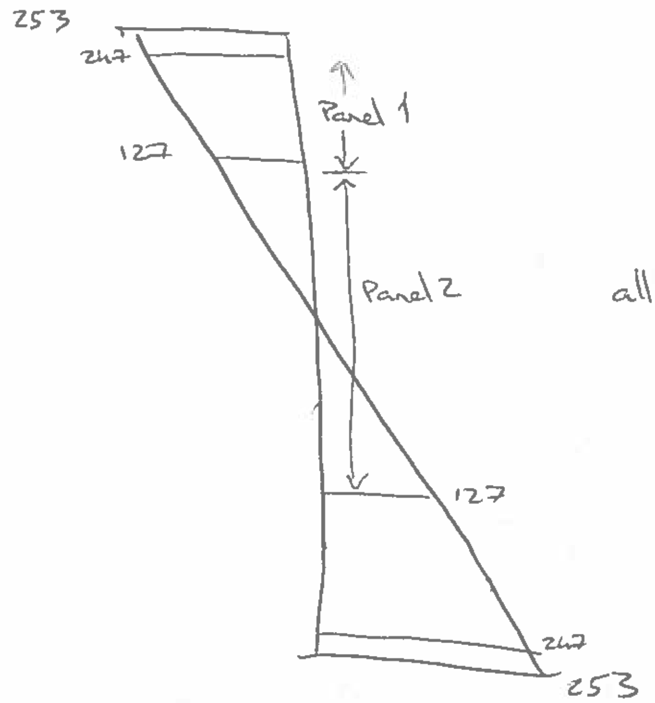
Q3

Stress checks

Flange

$\sigma_A = 253\,MPa < 355\,MPa$

Web panels



all stresses in N/mm^2

Check panel strength at B_2

$\sigma_{allow} \leq \sqrt{\sigma_y^2 - 3\tau^2}$ Datasheet 4 Note 2

$\sigma \leq \sigma_{allow}$

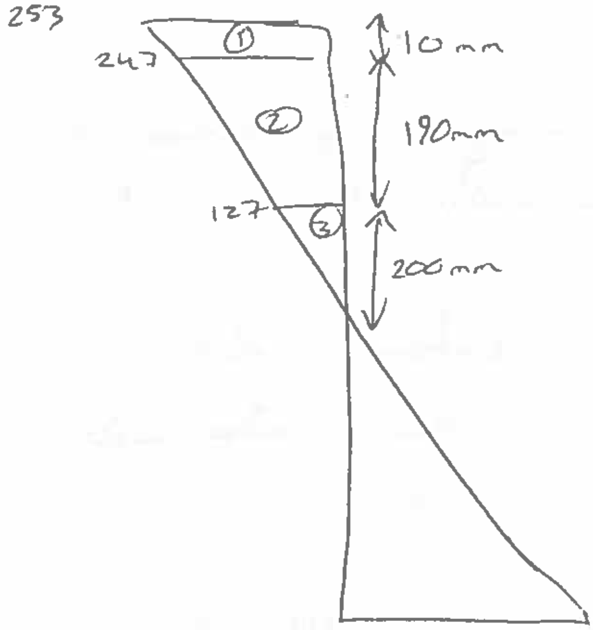
~~σ_B~~

$\sigma_{allow} = 351\ N/mm^2$

$\sigma < \sigma_{allow}$

Q3

Stress Check



Check panel 2 ("panel" 1 is the flange)

$$\lambda = \frac{190}{10}\sqrt{\frac{\sigma_y}{355}}$$

$$= 19$$

$\therefore$ compact

check panel 3

$$\lambda = \frac{200}{10}\sqrt{\frac{\sigma_y}{355}}$$

$$= 20$$

$\therefore$ compact

[70%]

Panels are compact $\Rightarrow$ use full capacity

$\sigma < \sigma_y$ for panels 2, 3

(9)

Q3

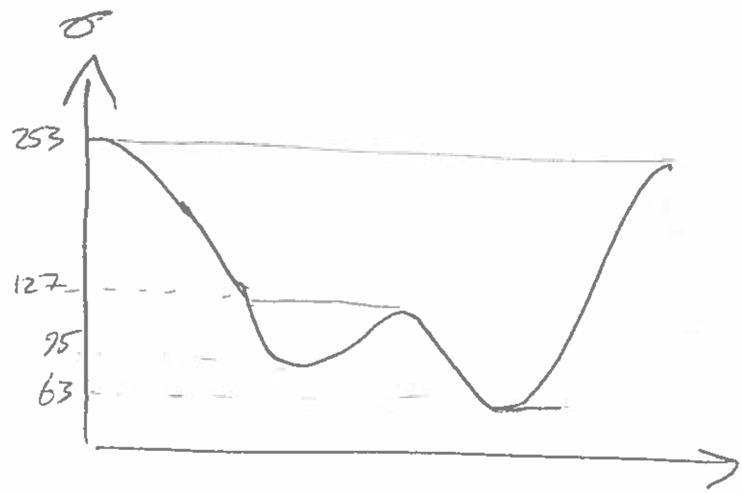
Solution to part (b)

Subtlety The quickest way to get stresses is to scale from case with $w = 80$ kN/m.

The max stress in a stiffener weld is at the flange. The stiffeners attached to the web have a lower load.

$w = 80$ kN/m ;	$\sigma_{max} =$	253 MPa	from earlier
$w = 40$ kN/m ;	"	127 "	by scaling
$w = 30$ kN/m ;	"	95 "	" "
$w = 20$ kN/m ;	"	63 "	" "

Reservoir method



One ~~cycle~~ stress range of $253 - 63 = 190$ N/mm^2
One ~~cycle~~ stress range of $127 - 95 = 32$ N/mm^2
] per cycle

Q3.

Fatigue Damage

$N \sigma_r^m = K_2$

$K_2 = 1.52 \times 10^{12}$
$m = 3.0$] for Class D — Datasheet 5

$\sigma_r = 190\ N/mm^2$
$N = 221{,}607$ cycles

$\sigma_r = 32\ N/mm^2$
$N = 46{,}386{,}719$ cycles

Number of cycles for loading

Use Miners Rule

$$\frac{1}{221{,}607} + \frac{1}{46{,}386{,}719} = \frac{1}{N_{Total}}$$ Datasheet 5 Note 3

$N_{Total} = 220{,}553$ cycles
(approx ¼ million)

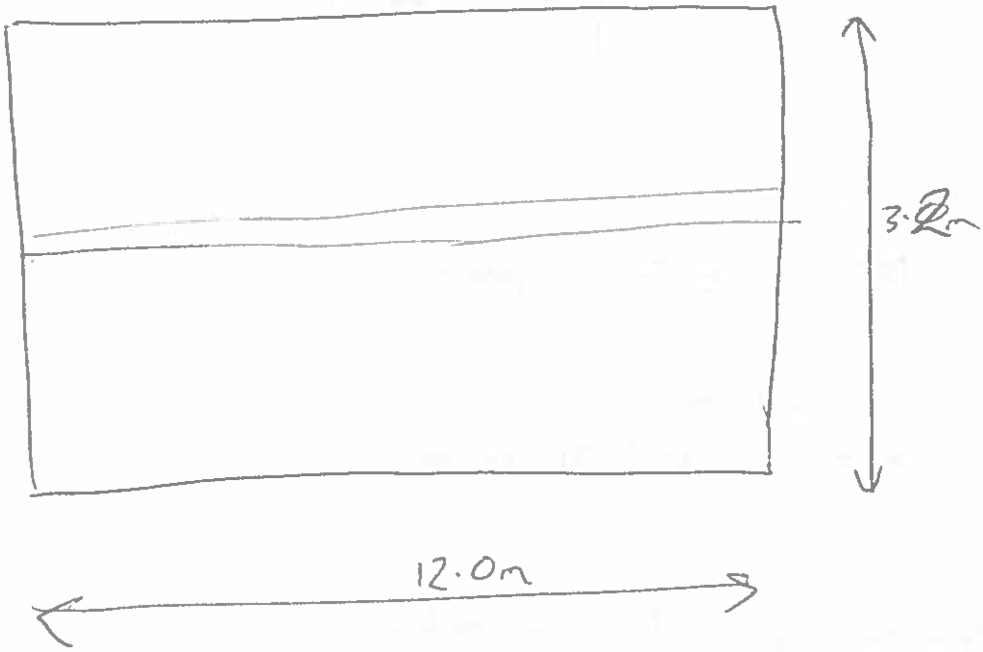
Note that the high-stress range dominates and that the low-stress part hardly matters

[30%]

11 12

QUESTION 4 (continued)

Solution



$$b_e = \min\left(3.2,\ 12.0/4\right)$$

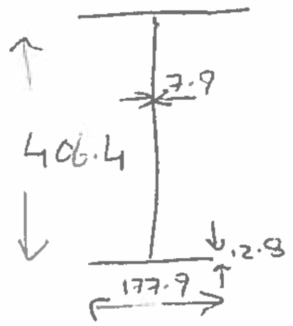
span

$$= 3.0\text{m}$$

effective slab width, b_e < distance between beams

QUESTION 4 (continued)

Solution (continued)



$A_{steel} = 76.5\, cm^2$

$mass = 60.1\, kg/m$

$I_{major} = 21600\, cm^4$

Compactness check

$$\lambda_{flange} = \left(\frac{177.9 - 7.9}{2}\right)\left(\frac{1}{12.8}\right)\sqrt{\frac{355}{355}}$$

$$= 6.64 \quad (< 8 \therefore ok)$$

$$\lambda_{web} = \left(\frac{406.4 - (2 \times 12.8)}{7.9}\right)\sqrt{\frac{355}{355}}$$

$$= 48.2 \quad (< 56 \therefore ok)$$

QUESTION 4 (continued)

Solution (continued)

Loads:

$$\text{slab} = \overset{kg/m^3}{2400} \times (\overset{m}{0.05} + \overset{m}{\tfrac{0.05}{2}}) \times 3.2\,m \times \overset{g}{9.81} \times \overset{\text{convert to kN}}{10^{-3}}$$

$$= 5.65 \text{ kN/m}$$

$$\text{beam} = 60.1 \times 9.81 \times 10^{-3}$$
$$= 0.59 \text{ kN/m}$$

$$\text{services} = 2.0 \times 3.2$$
$$= 6.40 \text{ kN/m}$$

$$\text{imposed} = 5.0 \times 3.2$$
$$= 16.0 \text{ kN/m}$$

$$\text{Permanent load} = 5.65 + 0.59 + 6.40$$
$$= 12.64 \text{ kN/m}$$

$$\text{factored perm.} = 1.35 \times 12.64$$
$$= 17.06 \text{ kN/m}$$

$$\text{Imposed load} = 16.0 \text{ kN/m}$$

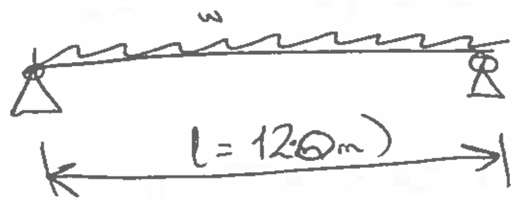
$$\text{factored imposed} = 16.0 \times 1.5$$
$$= 24.0 \text{ kN/m}$$

$$\text{total design load (factored)} = 17.06 + 24.0$$
$$= \cancel{47.4}\ 41.1 \text{ kN/m}$$

QUESTION 4 (continued)

Solution (continued)

Applied ~~Design~~ moment. Beam is simply supported

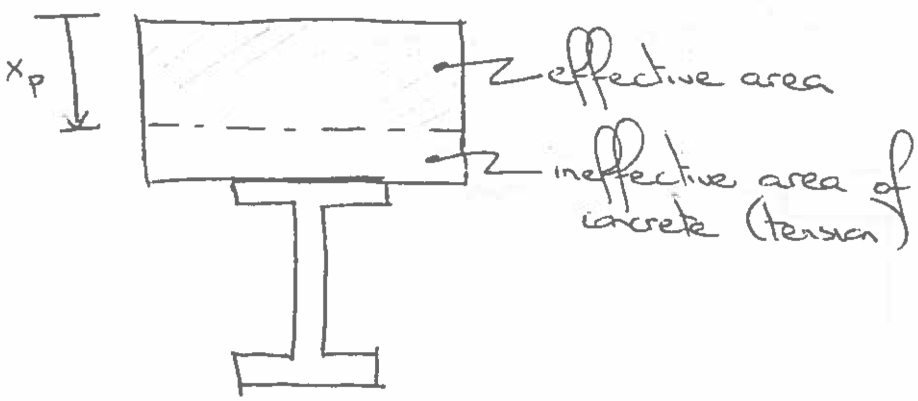


$$M_{app} = \frac{wl^2}{8}$$

$$= \frac{(41.1)(12.0)^2}{8}$$

$$= 739.8 \text{ kN-m}$$

Assume that the neutral axis is in the concrete. Use x_p to denote this distance as measured from the top of the slab.



QUESTION 4 (continued)

Solution (continued)

Axial equilibrium

note use of be

$$A_s \sigma_y = \overbrace{0.6 f_{cd}}^{\text{allowable}} b_e \, x_p$$

tensile force in steel beam — compressive force in concrete

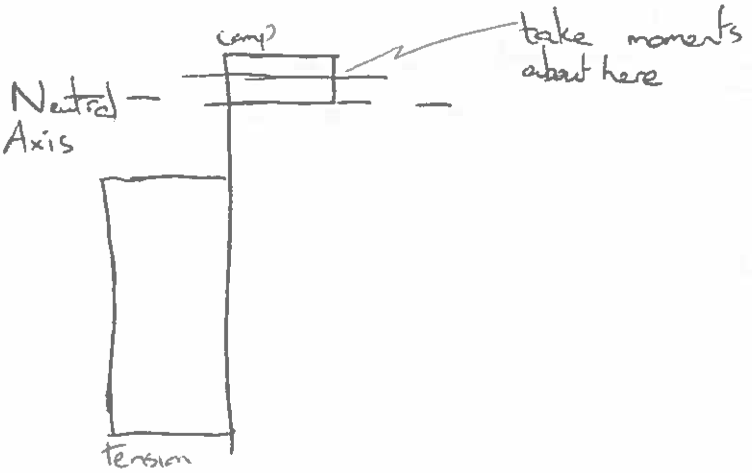
$$(76.5\times10^{-4})(355\times10^{6}) = 0.6(30\times10^{6})(3.0)(x_p)$$

$$x_p = \frac{(76.5\times10^{-4})(355\times\cancel{10^{6}})}{0.6\,(30\times\cancel{10^{6}})(3.0)}$$

$$= 0.050$$

$x_p = 50\text{mm}$ — just on the edge of trough.

Take moments about the ~~neutral axis~~ centre of concrete effective are to find the design allowable moment. (i.e. moment due to force in concrete is zero).



QUESTION 4 (continued)

Solution (continued)

$$M_{allow} = A_s \sigma_y \left(\frac{D}{2} + h_c - \frac{x_p}{2} \right)$$

$$= (76.5\times10^{-4})(355\times10^{6}) \underbrace{\left[\frac{0.406}{2} + 0.1 - \frac{0.050}{2} \right]}_{= 0.28}$$

$$= 755.0 \text{ kN.m}$$

$$M_{allow} > M_{app}$$

The beam can support the loads. [50%]

(b) Shear studs

65mm × 13mm studs

$P_d = 47$ kN per stud from DS6 (for $f_{cd} = 30$ MPa)

Axial force.

Max is $A_s \sigma_y = (76.5\times10^{-4})(355\times10^{6})$

$= 2716$ kN

QUESTION 4 (continued)

Solution (continued)

Required number of studs N_{stud}

$$N_{stud} \geq 2 \frac{A_s \sigma_y}{P_d} = 115.6$$

Require a minimum of 116 studs as it must be an integer.

$$\text{Spacing} = \frac{12.0}{116}$$

$$= 0.103 \text{ m} \quad (103\text{mm})$$

Trough spacing is 150mm running orthogonal
∴ cannot place studs sufficiently closely if in a line (used singly)

Use pairs of studs. Note that studs have only 80% capacity if used in pairs (see Note ~~4~~ to Section ~~4~~(a) in DS6.)

$$N_{stud} \geq 2 \frac{A_s \sigma_y}{0.8 P_d}$$

$$\geq 144.5 \text{ studs}$$

say 73 pairs (146 studs)

QUESTION 4 (continued)

Solution (continued)

$$\text{Spacing} = \frac{12.0}{73}$$

$$= 0.164 \quad (= 164\,\text{mm})$$

Trough spacing = 150 mm centre-to-centre

∴ Put a pair of studs in each trough

$$\text{No. troughs} = \frac{12.0}{0.150}$$

$$= 80 \text{ pairs}$$

more than sufficient (73 pairs reqd.) [20%]

(c) Deflection under short-term load

Need to use an "elastic modulus" for the concrete, E_c.

E_c = 28 GPa for short term (from DS6)

QUESTION 4 (continued)

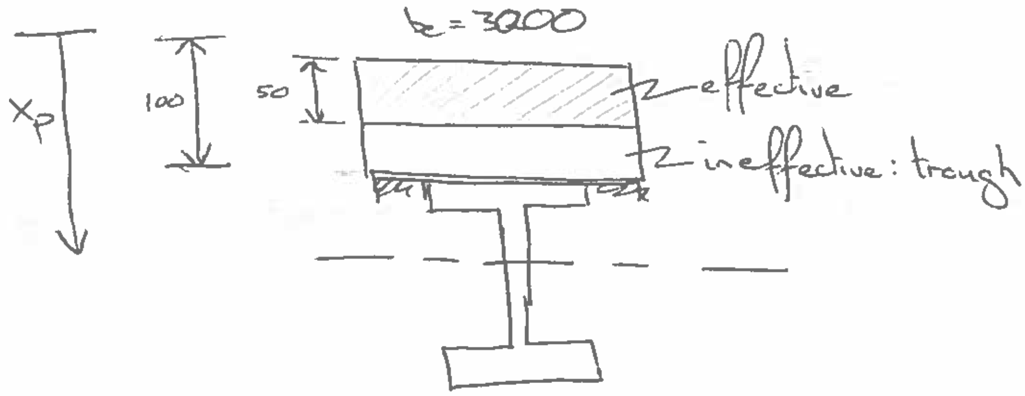
Solution (continued)

Modular ratio of steel to concrete

$$\frac{E_s}{E_c} = \frac{210}{28} = 7.5$$

Transfer to an equivalent steel section.

First, find the position of the neutral axis of the transformed section. Begin by assuming that the neutral axis is in the beam at a distance x_p below the top of the floor slab.



$$x_p\Big[\underbrace{\left(\left(\tfrac{3000}{7.5}\right)50\right)}_{\text{concrete}}\cancel{100} + \underbrace{7650 \times 10^{2}}_{\text{steel}}\Big]$$

$$= \left[\left(\frac{3000}{7.5}\right)(50)\left(\frac{50}{2}\right) + (76.5\times10^{2})\left(100 + \frac{466.4}{2}\right)\right]$$

$$= \frac{\overset{500{,}000}{\cancel{533{,}333}} + 2319{,}480}{\underset{27{,}650}{\cancel{28{,}983}}}$$

QUESTION 4 (continued)

Solution (continued)

$$X_p = \cancel{\ldots} \; 102.0 \text{ mm}$$

The neutral axis is just inside the top flange of the beam.

Calculate the second moment of area for the transformed section. (Units are mm in the following calc.)

$$I_{xx} = \frac{1}{12}\left(\frac{3000}{7.5}(50)^3\right) + \underbrace{\left(\frac{3000}{7.5}\right)(50)\left(102.0 - \left(\frac{50}{2}\right)\right)^2}_{\text{parallel axis theorem for (transformed) concrete}}$$

$$+ \underbrace{21{,}600 \times 10^4}_{I_{beam}} + \underbrace{(76.5 \times 10^2)\left(\frac{406.4}{2} - 2.0\right)^2}_{\text{parallel axis theorem for beam}}$$

NA inside steel

$$= \underbrace{4.167 \times 10^6 + 118.58 \times 10^6}_{\text{transformed concrete}} + \underbrace{216 \times 10^6 + 309.4 \times 10^6}_{\text{steel beam}}$$

$$= 648.1 \times 10^6 \text{ mm}^4 \quad (= 648.1 \times 10^{-6} \text{ m}^4)$$

Deflection, Δ for a simply supported beam

$$\Delta = \frac{5 w l^4}{384 EI}$$

Question 4 (continued)

Solution (continued)

For "E" here need to use E_s as the section has been "transformed" into the equivalent steel section.

$$\Delta = \frac{5wl^4}{384 E_s I_{xx}}$$

Use the unfactored load as it is a serviceability limit state.

$$= \frac{5(16.0\times10^3)(12.0)^4}{384(210\times10^9)(648.1\times10^{-6})}$$

be careful with units

$$= 0.032 \text{ m}$$

$$= 32\text{mm}$$

Allowable deflection, $\Delta_{allow} = \frac{\text{span}}{250}$

$$\Delta_{allow} = \frac{12.0}{250}$$

$$= 0.048\text{m}$$

$$= 48\text{mm}$$

[30%]

$\Delta_{allow} > \Delta$

$\therefore$ deflection okay