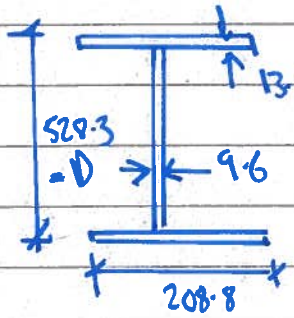
4D10 2021-22.

Qu1



$I_{xx} = 47540\ cm^4$; $I_{yy} = 2007\ cm^4$
$Z_p = 2059\ cm^3$ (x-axis).
$J = 51.5\ cm^4$; $A = 105\ cm^2$

S355 grade; mass = 82.2 kg/m.
root radius = 13 mm.

(a) $P = 0$ kN

For flange: (in compression) $c/t = [[208.8 - 9.6 - 2 \times 13]/2]/13.2$
$= 6.56$

f_y: grade

$\varepsilon = [235/355]^{1/2} = 0.814$; $9\varepsilon = 7.32 > 6.56$
class (1) from datasheets

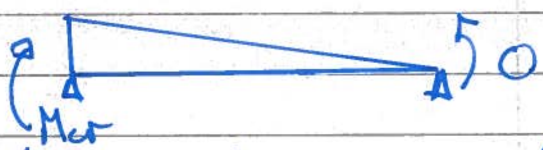
For web: (in bending) $c/t = [\{528.3 - \underbrace{2 \times 13.2}_{\text{thickness}} - \underbrace{2 \times 13}_{\text{radius}}\}/9.6]$
$= 49.57$

$72\varepsilon = 58.68 \geq 49.57$, so class (1) also

M_{cr} (critical moment in lateral torsional) $= \frac{\pi}{L}\sqrt{EI_yGJ}\left[1 + \frac{\pi^2}{L^2}\cdot\frac{E\Gamma}{GJ}\right]^{1/2}$

with $\Gamma = I_{yy}\,d^2/4 = (2007 \times 10^4) \times (528.3 - 13.2)^2/4$
$= 1.3297 \times 10^{12}\ mm^6$

$L = 6$ m (given).



$\Rightarrow \psi = 0$ from data sheet : $C_{unequal} = 0.6$

$M_{cr} = M_{LT}/C_{unequal} = M_{LT}/0.6$.

Qu1)

$$EI_{yy}GJ = \underbrace{210\times10^9}_{E(Pa)} \times \underbrace{(2007)\times10^{-8}}_{I_{yy}(m^4)} \times \underbrace{81\times10^9}_{G(Pa)} \times \underbrace{(51.5)\times10^{-8}}_{J(m^4)}$$

$$= 1.758\times10^{11}\,(Nm^2)^2.$$

$$EΓ/GJ = \frac{210}{81}\cdot\frac{1.33\times10^{12}}{51.5\times10^4} = \underline{\underline{6.70\times10^6\ mm^2}}$$

$$M_{LT} = \frac{\pi}{L}\left[EI_{yy}GJ\right]^{1/2}\left[1+\frac{\pi^2}{L^2}\cdot\frac{EΓ}{GJ}\right]^{1/2}.$$

$$= \frac{\pi}{6}\left[1.758\times10^{11}\right]^{1/2}\underbrace{\left[1+\frac{\pi^2}{6^2}\cdot(6.70\times10^6)\times10^{-6}\right]^{1/2}}_{1.68}$$

$$= \underline{\underline{369.79\,kNm}}$$

$$M_{cr} = M_{LT}/C_{unequal} = 370/0.6 = \underline{\underline{616.3\,kNm}}$$

$$M_{pl} = Z_p\cdot\sigma_y = 2059\times10^{-6}\times355\times10^6 = \underline{\underline{730.9\,kNm}}$$

$$\lambda = \sqrt{\frac{M_{pl}}{M_{cr}}} = \sqrt{\frac{731}{616}} = \underline{\underline{1.09}}.$$

h/b > 2 ⇒ use curve b and α (imperfection factor) $= \underline{\underline{0.34}}$ (if pursuing calc).

Reading up ⇒ $\chi = 0.54$ for $\lambda = 1.09$ curve b.

$$\Rightarrow M_{max} = \chi M_{pl} = 0.54\times730.9 = \underline{\underline{395\,kNm}}$$

Qu 1b) $M = 0$: flange outstand class 1 from (a).

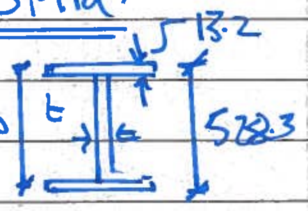
$c/t \doteq 49.6$ from (a) $> 42\varepsilon$ (of class 3)

$\Rightarrow$ need to use effective section [class 4], where a central portion of girder is removed.

First, calculate $\lambda = \sqrt{f_y/\sigma_{cr}}$, with $\sigma_{cr} = \frac{4\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$ ← central, clamped on both ed[ges]

$\sigma_{cr} = \frac{4\pi^2(210\times10^9)}{12(1-\nu^2)} \times \left(\frac{9.6}{515.5}\right)^2 = \underline{263.3\,\text{MPa}}$.   ($\nu = 0.3$)

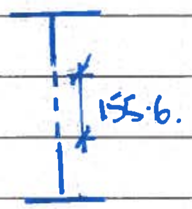
↖ b from



$\lambda = \sqrt{\frac{355}{263.3}} = 1.16$

For eff. section: $\rho = \frac{1}{\lambda}\left(1 - \frac{0.22}{\lambda}\right) = \underline{0.698 \simeq 0.7}$

Proportion removed $= (1-\rho)b = \underline{155.6}$ mm.



Area $= 105\,\text{cm}^2$ ∴ $A_{eff} = 10500 - 155.6\times9.6$
$= \underline{9006\,\text{mm}^2}$

$A_{eff}\times f_y = (9006\times10^{-6})\times355\times10^6 = \underline{3197.1\,\text{kN}}$

Over full length, buckling can proceed (x-axis)

$P_{cr} = \frac{\pi^2 E I_{xx}}{L_{cr}^2} = \frac{\pi^2(210\times10^9)\times(47540\times10^{-8})}{12^2} = \underline{6842.5\,\text{kN}}$ (1)

In y-direction, restrained at mid-length.

$P_{cr} = \frac{\pi^2 E I_{yy}}{L_{cr}^2} = \frac{\pi^2(210\times10^9)\times(2059\times10^{-8})}{6^2} = \underline{1185.4\,\text{kN}}$ (2)

P.T.O

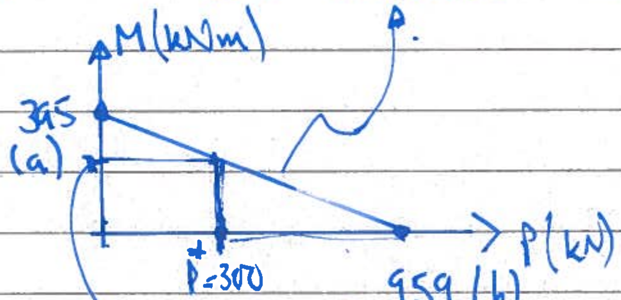
Qu 1 for ① $\Rightarrow \lambda = \sqrt{\frac{A_{eff} \cdot f_y}{P_{cr}}} = \sqrt{\frac{3197}{6842}} = 0.68.$ } a-curve

for ② $\Rightarrow \lambda = \sqrt{\frac{3197}{1185}} = 1.64.$ } b-curve

② is more critical (lower χ) $\simeq 0.3$ from reading DS1

$\Rightarrow$ Actual limit on $P = 0.3 \times 3197 = \underline{\underline{959 \text{ kN}}}.$

1c) P-M linear interaction curve.

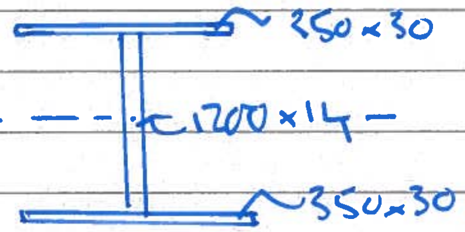


$M^* = 395 \cdot \frac{(959-300)}{959} = 271.$

$M^* = 271$ for $P^* = 300$

$\therefore (M^*, P^*) = (300, 300)$ $\underline{\underline{\text{cannot}}}$ be carried safely.

Qu2. Girder cross-section (S355)
Length (axial) = 27m



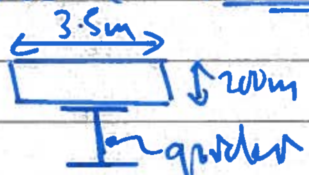
girder alone:

Area = 2×350×30 + 1200×14
= $\underline{37800\ mm^2}$

$$I_{xx} = 1200^3\times14/12 + 2\times\Big[\underbrace{350\times30^3/12}_{I_G} + \underbrace{\Big[\tfrac{1200}{2}+\tfrac{30}{2}\Big]^2\times350\times30}_{\parallel\ \text{axis.}}\Big]$$

$$= \cancel{33.018\times10^9\ mm^4} \quad = \underline{7.94\times10^9\ mm^4}$$

Length of girder = 27m ⇒ 27/4 = 6.75m : actual spacing of girders = 3.5 < 6.75 ⇒ effective slab width = $\underline{3.5m}$ = b_{eff}



~~mass~~ weight/length (girder) = $\rho A g$ N/m

$$= \underset{kg/m^3}{7800} \times \underbrace{(37800\times10^{-6})}_{A} \times \underset{m/s^2}{9.81}$$

$$= \underline{2.892\ kN/m}.$$

For the slab: 2400 × (3.5×0.2) × 9.81 = $\underline{17.16\ kN/m}$

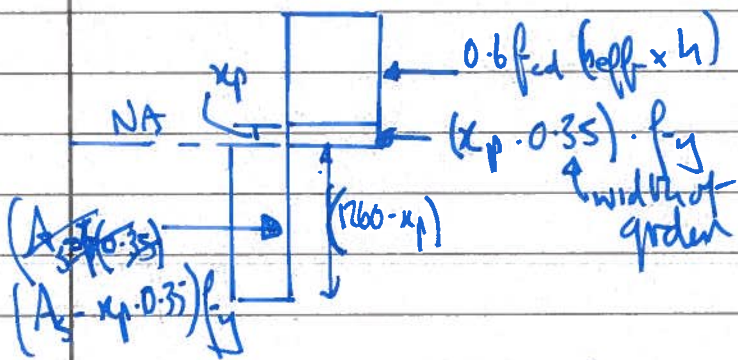
Dead load factor = 1.35 ⇒ (2.892 + 17.16) × 1.35 = $\underline{27.07\ kN/m}$
Live load factor = 1.5 ⇒ (200kN) × 1.5 = $\underline{300kN}$
(rolling point load)

Maximum moment, M_d, occurs when rolling point load is in the middle of beam, i.e.

$$M_d = 300\times\frac{\overset{\text{beam length}}{27}}{4} + \underbrace{\frac{27.07\times27^2}{8}}_{\text{distributed loading}} = \underline{4491.8\ kNm}$$

Qu 2) Assume N/A in the steel: force in concrete $= 0.6 f_{cd} \times (b_{eff} \times h)$

$$= 0.6 \times (30 \times 10^6) \times 3.5 \times 0.2 = \underline{12600\,kN}$$

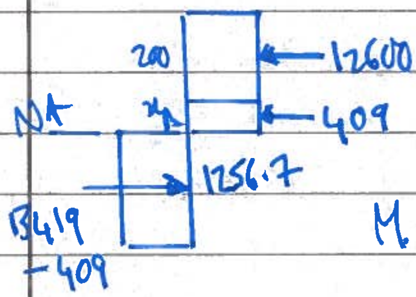


Axial eqlm:

$$0.6 f_{cd}\, h\, b_{eff} + x_p \cdot 0.35 f_y = (A_s - 0.35 x_p) f_y$$

$$\Rightarrow x_p = \left(\underbrace{A_s f_y}_{13419\,kN} - \underbrace{0.6 f_{cd}\, h\, b_{eff}}_{12600\,kN}\right) / \, 2 \times 0.35 \times \underbrace{f_y}_{355 \times 10^6}$$

$$\Rightarrow \underline{x_p = 3.3\,mm} \quad [x_p \times 0.35 \times f_y = 409\,kN].$$



Moment for force blocks:

$$M = 409 \times \underbrace{x_p}_{mm}/2 + 12600 \times [\underbrace{100 + x_p}_{mm}] + (13419 - 409) \times \underbrace{\frac{1256.7}{2}}_{mm}$$

$$= \underline{9477.5\,kNm} \quad [c.f.\ 9311\,kNm \text{ if } x_p = 0]$$

Required moment $= 4491.8\,kNm$ ∴ sufficient capacity

b) Assume a shear connector ~~height~~ diameter of 22mm

From data sheet: $P_{Rd} = (0.8 f_u \cdot \pi d^2/4)/\gamma_v$, $\gamma_v = 1.25$, $f_u = 450MPa$

$$\Rightarrow P_{Rd} = \left[0.8 \times (450 \times 10^6) \times \pi \times \frac{0.022^2}{4}\right] / 1.25 = \underline{109.5\,kN}$$

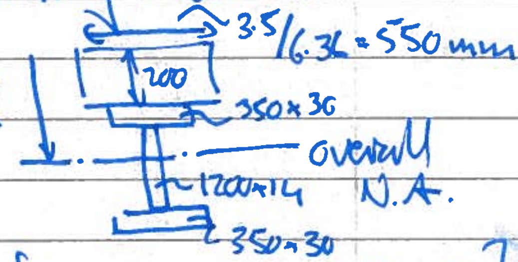
or $P_{Rd} = \dfrac{0.29 d^2 (f_{ck} \cdot E_{cm})^{1/2}}{\gamma_v} = \left\{0.29 \times 0.022^2 \times [\underbrace{30 \times 10^6}_{f_{ck}} \times \underbrace{33 \times 10^9}_{\text{modulus of concrete}}]^{1/2}\right\} \div \gamma_v$

$$= \underline{111.7\,kN}$$

Qu 2 Select lower $P_{Rd} = 109.5\,kN$. # connectors $= \dfrac{\text{axial force in conc}}{P_{Rd}}$

$$= \frac{12600}{109.5} = \underline{\underline{115}}$$

c). Need to transform section to all steel, using modular ratio $= 210/33 = 6.36$ ⇒



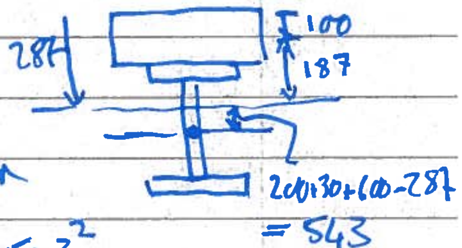
d is location of new/overall N.A.

$$\Sigma A\bar{y}_c = d \times \Sigma A : (550\times200)\times100 + 350\times30\times[\underbrace{200+15}_{\text{top flange}} + \underbrace{200+1200+30+15}_{\text{btm flange}}]$$

$$+ (1200\times14)\times[200+30+600] = d\cdot[550\times200 + 2\times350\times30 + 1200\times14]$$

$$\Rightarrow \underline{\underline{d = 286.8\,mm}}.$$

The new I_{xx} for the section is needed



$$I_{xx} = \underbrace{550\times200^3/12 + (550\times200)\times187^2}_{\text{top block}} + \underbrace{I_{\text{given}} + A\times543^2}_{7.94\times10^9 + [37800]\times543^2}.$$

$$= \underline{\underline{23.30\times10^9\,mm^4}}$$

Need to calculate stress in btm fibre, located $543 + 600 + 30 = 1173$ mm from NA.

$$\Rightarrow \sigma = \frac{M^* y}{I} = \frac{\cancel{4942\times10^3}\ (1013\times1000) \times 1.173}{(23.3\times10^9)\times(10^{-12})} = \underline{\underline{51.7\,MPa}}$$

M^* = moment when the car is 1/4 of the way across the bridge $= \underbrace{150\,kN}_{\text{reaction at support}} \times 27/4 = 1013\,kNm$

However, need to include factored load ⇒ $51.7\times1.35 = \underline{\underline{69.7\,MPa}}$

Qu2) The # cycles to failure is $N_f (\Delta\sigma)^m = K$, $m = 3$.

K in this case is $(2 \times 10^6) \times (71)^3$: $\Delta\sigma$ in MPa.

$\Rightarrow \quad 2 \times 10^6 \times 71^3 = N_f \times (69.71)^3$

$\Rightarrow \quad \underline{N_f = 2.205 \times 10^6 \text{ cycles.}}$

Qu3 (ai) - Standard checks are local buckling for flanges/web, in compression, bending and in shear.
- Elements need to be "stockier" because of higher yield limit.

(aii) To deal with local buckling "k" must be allocated numerically for each action:

$k = 4$ (internal), 0.43 (external) : compression
$k = 23.9$: bending
$k = 5.34$: shear.

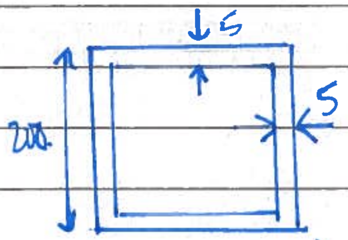
For outstand in compression: $\underbrace{\frac{0.43\,\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{c}\right)^2}_{\sigma_{cr}} > f_y$

$$\Rightarrow \frac{0.43\times\pi^2(210\times10^9)}{12(1-0.3^2)}\left(\frac{t}{c}\right)^2 > 960\times10^6 \Rightarrow \frac{t^2}{c^2} > 85 \Rightarrow \frac{c}{t} < 9.22$$

Web in bending: $\frac{23.9\,\pi^2(210\times10^9)}{12(1-\nu^2)}(t/c)^2 > 960\times10^6 \Rightarrow \frac{c}{t} < 68.74$

In shear: $\frac{5.34\,\pi^2(210\times10^9)}{12(1-\nu^2)}(t/c)^2 > \underbrace{\frac{960\times10^6}{\sqrt{3}}}_{\tau_y} \Rightarrow \frac{c}{t} < 24.67$

b)



Area $= 2\times200\times5 + 2\times190\times5 = 3900\,\text{mm}^2$

$I = \frac{200\times200^3}{12} - \frac{190\times190^3}{12} = 24.73\times10^6\,\text{mm}^4$

$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2\times(210\times10^9)\times(24.73\times10^6)\times10^{-12}}{15^2} = 227.8\,\text{kN}$

$L = 15$

Qu 3) $\sigma_{cr} = P_{cr}/A = 227.8\times10^3/[3900\times10^{-6}] = \underline{\underline{58.42\,\text{MPa}}}$

For S235: $\lambda = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{235}{58.4}} = \underline{\underline{2.00}}$ $\quad c/t = \frac{190}{5} = 38 \Rightarrow$ class 2

Use buckling curve c, as suggested.

$\Rightarrow \chi = 0.2$ (from reading up) $\Rightarrow$ actual limit on P_{cr} is $0.2\times A\times f_y = 0.2\times(3900\times10^{-6})\times(235\times10^{6})$ $= \underline{\underline{183\,\text{kN}}}$

For S960: $\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{960}} = 0.495$: $42\varepsilon = \underline{\underline{20.8}}$

c.f. $c/t = 38 > 42\varepsilon \Rightarrow$ class 4 structure: use effective X-section.

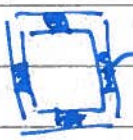
$k = 4$ (internal: compression) $\Rightarrow 4\cdot\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{c}\right)^2 = \sigma_{cr}$

$\uparrow 5/190.$

$\Rightarrow \sigma_{cr} = \underline{\underline{525\,\text{MPa}}}$

$\lambda = \sqrt{f_y/\sigma_{cr}} = \sqrt{\frac{960}{525}} = \underline{\underline{1.35}}$: $\rho = \frac{1}{\lambda}\left[1-\frac{0.22}{\lambda}\right] = \underline{\underline{0.62}}$

$\Rightarrow A_{eff} = \rho A = 0.62\times3900 = \underline{\underline{2418\,\text{mm}^2}}$ $\Leftarrow$



ρ refers to material left behind after removal

$\sigma_{cr} = P_{cr}/A_{eff} = \frac{227.8\times10^3}{2418\times10^{-6}} = \underline{\underline{94.2\,\text{MPa}}}$

$\lambda = \sqrt{f_y/\sigma_{cr}} = \sqrt{\frac{960}{94.2}} = \underline{\underline{3.19}}$, which is beyond the graphical range.

$\therefore$ for curve c, α (imp. factor) $= 0.49$

$\phi = \frac{1+\alpha(\lambda-0.2)+\lambda^2}{2} = \underline{\underline{6.32}} \Rightarrow \chi = \frac{1}{\phi+\sqrt{\phi^2-\lambda^2}}$ I.F.

Qu 3. $\chi = \dfrac{1}{6.32 + \sqrt{6.32^2 - 3.14^2}} = 0.085.$

$\Rightarrow$ actual limit of $P_{cr} = \chi A_{eff} f_y$

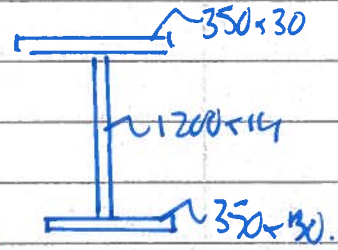
$= 0.085 \times (2418 \times 10^{-6}) \times 960 \times 10^6$

$= \underline{\underline{197.12 \text{ kN}}}$

$\therefore$ c.f. limit of 183kN for S235: a diff of 14kN!

Reason: slender column, buckling at high λ (low χ)
$\Rightarrow$ close to elastic Euler prediction in both cases: the effectiveness, or contribution, from having a high yield stress, is not realised.

Qu 4) Design moment = 3500 kNm : shear force = 1300 kN.
Need the axial stress in the top flange centroid, and at the junction of web-to-flange: therefore, determine I for the unspliced section.



$$I \simeq \frac{1200^3 \times 14}{12} + 2\times(30\times 350)\times 615^2$$
$$= \underline{\underline{9.959\times 10^9\ mm^4}}$$

Top flange stress $= \frac{My}{I} = \frac{(3500\times 10^3)\times 0.615}{(9.959\times 10^9)\times 10^{-12}} = \underline{\underline{216.14\ MPa}}$

Top junction stress $= \frac{My}{I} \sim y = 0.6 = \underline{\underline{210.9\ MPa}}$

Bolt check in shear: each M22 grade 8.8 bolt can sustain a max shear force of.

$$\frac{2 \times 0.6 \times \text{area} \times f_{ub}}{\gamma_{M2}} \quad f_{ub} \sim 800\,MPa \text{ at yield} \quad : \text{area} = \pi \times (0.011)^2$$

2: double shear; 22 mm φ; $\gamma_{M2} \sim 1.1$

$\Rightarrow$ force $= \underline{\underline{331.8\ kN}}$

Each side of junction has 8 bolts $\Rightarrow$ capacity $= \underline{\underline{2654\ kN}}$

The axial force in top flange $= \sigma_{cr} \times A = (216\times 10^6)\times 0.35\times 0.03 = \underline{\underline{2268\ kN}}$

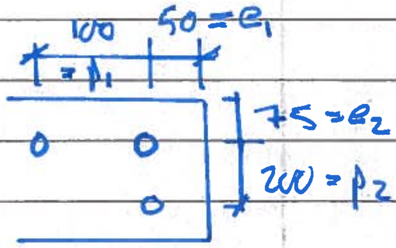
σ_{cr} = 216 MPa

$\therefore$ enough bolts for shear.

Splice plate yielding must carry axial force at yield.

$$\Rightarrow \underbrace{[0.35 + 2\times 0.15]}_{\text{hatched area}} \times 0.016 \times \underbrace{(355\times 10^6)}_{S355}$$

$$= \underline{\underline{3692\ kN}} \quad (> 2268,\ OK)$$



In bearing: use BS5.

Qn 4) Looking at top plate in plan view:

$$\alpha_d = \underbrace{\frac{e_1}{3d_0} \text{ or } \frac{p_1}{3d_0} - \frac{1}{4}}_{BS5} \qquad d_0 = \text{bolt diameter}$$

$$\Rightarrow \alpha_d = \frac{50}{3\times22} = 0.757 \text{ or } \alpha_d = \frac{100}{3\times22} - \frac{1}{4} = 1.265.$$

choose $\alpha_d = 0.757$

$k_1 = 2.8 e_2/d_0 - 1.7$ or $1.4 p_2/d_0 - 1.7$ or 2.5 (whichever is smallest)

$$k_1 = 2.8\times\frac{75}{22} - 1.7 = 7.85; \quad k_1 = 1.4\cdot\frac{200}{22} - 1.7 = 11.03.$$

$\therefore$ choose $k_1 = 2.5$

Also from BS5: $F_{b,Rd}$ (bolt capacity) $= k_1 \alpha_b \cdot f_u \cdot \phi t/\gamma_{M2}$. ($\phi$: diameter)

α_b is smallest of α_d, $\frac{f_{ub}}{f_u}$ $\left(\frac{\cancel{490}}{\cancel{800}}\right)$ or 1 $\therefore \alpha_b = \alpha_d = 0.757$

$> 1 = (800/490)$

$$\Rightarrow F_{b,Rd} = \underbrace{2}^{\text{2 bearing zones}} \times \underbrace{2.5}_{k_1} \times \underbrace{0.69}_{\alpha_b} \times \underbrace{(490\times10^6)}_{f_u} \times \underbrace{(0.022)}_{\phi} \times \underbrace{0.016}_{t \text{ of plate}} / \overbrace{1.1}^{\gamma_{M2}}$$

$$= 519\,kN$$

Again, 8 bolts per side in bearing $= 8\times519 = 4154\,kN$

(> 2268) ✓

P.T.O.

Qn 4) Local buckling of outstand 

$t = 16\,mm,\ c = 75\,mm$: use $k = 0.43$.

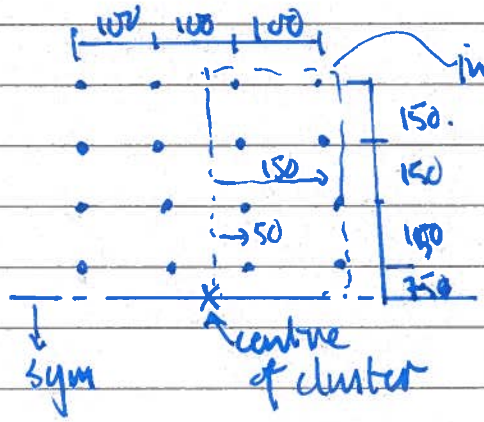
$$\sigma_{cr} = 0.43\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{c}\right)^2 = \frac{0.43\pi^2(210\times10^9)}{12(1-0.3^2)}\left(\frac{16}{75}\right)^2 = \underline{\underline{3714\,MPa}}$$

$\sigma_{cr} \gg f_u\ (490\,MPa)\ \therefore$ OK.

To deal with web connection need to find applied moment from $M = \frac{\sigma I'_{web}}{y}$, σ = top junction stress = $\underline{210.9\,MPa}$ from before

$$M = (210.9\times10^6)\times\underbrace{(0.014\times1.2^3/12)}_{I_{web}}/0.6 = \underline{\underline{705.6\,kNm}}$$

Shear force in each bolt $= r_i\,\frac{M}{\sum r_i^2}$.



in calculation of $\sum r_i^2 \times 4$ for whole plate

$$\sum r_i^2 = \left[4\times50^2 + 75^2 + \overset{75+150}{225^2} + 375^2 + 525^2\right] + \left[4\times150^2 + 75^2 + 225^2 + 375^2 + 525^2\right]$$

$$= 1045\times10^3\,mm^2.$$

$\therefore\ 4\sum r_i^2 = 4180\times10^3\,mm^2.$ $\quad F_{max} = r_{max}\cdot\frac{M}{\sum r_i^2}$

$r_{max} = 150 \rightarrow 525\uparrow$

$$\Rightarrow F_{max} = \underbrace{\left[0.15^2 + 0.525^2\right]^{1/2}}_{r_{max}} \times \frac{(705.6\times10^3)}{(4180\times10^3)\times10^{-6}} = \underline{\underline{92.17\,kN}}$$

Qu4) So, on furthest bolt $\rightarrow$ 92.2kN (for moment)

applied shear force. $\frac{1500}{16} = 93.75kN$

$\Rightarrow$ Total shear force $= \sqrt{92.2^2 + 93.75^2} = \underline{\underline{131.5kN}}$

Previous capacity determined to be 331.8kN, thus OK!

Also check bearing and local buckling of the web and splice plate.

Q1 Beam-column capacity

Part (a) was generally well answered. The most common mistakes were: (i) using the fully plastic moment in the calculations, without checking whether the beam was indeed Class 1 or 2; (ii) using the full length L=12 m in the check for lateral-torsional buckling, thus ignoring the out-of-plane restraint at mid-span; and (iii) using the wrong y-value in calculating $C_{unequal}$. Out of the two 6 m long segments to be checked for lateral-torsional buckling, the segment subject to the larger moments was critical, and thus: y = 0.5 and $C_{unequal}$ = 0.8.

In Part (b), a small minority of students did not check the class of the cross-section in compression (despite the guidance in the question) and thus failed to realize it was Class 4. In the calculation of the column slenderness, the numerator should contain $A_{eff}\,f_y$, while the Euler load in the denominator is based on the *gross* section properties. It is also not immediately clear whether minor axis buckling (L_{cr} = 6 m, smaller I) or major axis buckling (L_{cr} = 12 m, but larger I) would be critical and both needed to be checked.

Q2 Composite floor-decking design

Part (a) was generally well answered. A common (relatively minor) mistake was to use the characteristic concrete strength, f_{ck}, in the calculations, rather than the design strength, $f_{cd} = f_{ck}/1.5$. In the calculation of the applied moment, the self-weight of the composite beam was quite often ignored or calculated wrongly.

The most common mistake in Part (b) was the failure to realize that N_c/P_{Rd} results in the number of connectors over *half* the span.

In Part (c), very few students were able to correctly calculate the fatigue limit state moment as the moment at quarter-span, caused only by the (unfactored) point load, placed at quarter-span. The remainder of the question seemed much less problematic.

Q3 Buckling and compactness design

Most students realized that local buckling would become a more prominent issue in high-strength steel beams but did not always draw the conclusion that the cross-sections would have to be more ‘stocky’ (containing less slender flanges and webs) to keep them inside Class 3.
In Part (b) the most common oversight was not checking the class of the cross-section, resulting in a failure to realize that the S960 column was Class 4.

Q4 Bolted splice joint design

A small minority of students attempted this question. While the connection looked perhaps intimidating due to the sheer number of bolts, the design checks were relatively straightforward. Almost all students correctly identified the various failure modes. Identifying a realistic tear-out mechanism in the flange plates appeared to be the most common problem.