ENGINEERING TRIPOS PART IIB 2021

MODULE 4D10, STRUCTURAL STEELWORK

EXAMINATION CRIB AND COMMENTS

Question 1 4D10 2020-2021

254x146x37 UB

$b_f = 146.4$ mm

$t_f = 10.9$ mm
$h = 256$ mm

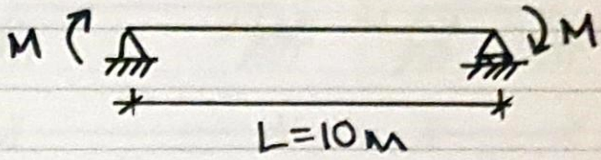
$\left.\begin{array}{l} t_f = 10.9 \text{ mm} \\ h = 256 \text{ mm} \end{array}\right\} \begin{array}{l} D = 256 - 10.9 \\ D = 245.1 \text{ mm.} \end{array}$

$t_w = 6.3$ mm.

$Z_{pl,major} = 483\ cm^3$

$I_{YY(minor)} = 571\ cm^4$

$J = 15.3\ cm^4$



$\sigma_y = 355$ MPa

$E = 205$ GPa

$G = 81$ GPa

(a) Section resistance : compact check → class 1.

"CALCULATE THE PLASTIC" : $M_{PLASTIC} = \sigma_y Z_{PL,MAJ}$

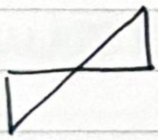
$= 355 \times 483 \times 10^3$

$= 171.5$ kNm.

"CALCULATE THE ELASTIC" : $M_{LT} = \frac{\pi}{L}\sqrt{E I_{MINOR} G J}\sqrt{1 + \frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}}$

$\boxed{\text{from DS3}}$

$= 40.4$ kNm



$\varphi = -1$

$\therefore C_{unequal} = 0.4 \Rightarrow M_{CR} = \frac{M_{LT}}{0.4} = 101$ kNm

$M_{ELASTIC}$.

"ELASTO-PLASTIC INTERACTION": $\bar{\lambda}_{LT} = \sqrt{\frac{M_{PLASTIC}}{M_{ELASTIC}}} = 1.30$

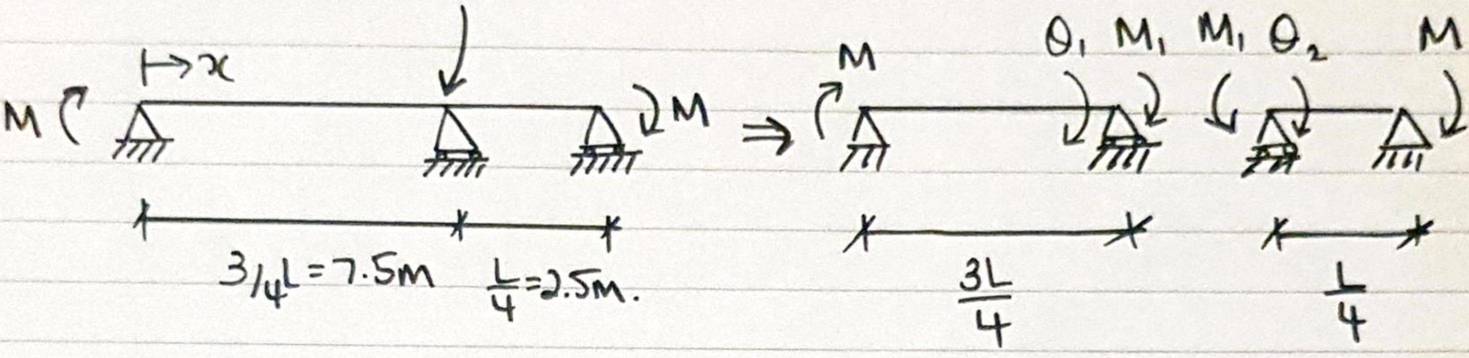
$h/b = 256/146.4 = 1.75 > 1.2$

$\Rightarrow$ use curve (a) $\boxed{\text{DS2}}$

$\chi = 0.48$

$M_{max} = 171.5 \times 0.48 = \underline{82.3 \text{ kNm.}}$

(b) support at $x = 3/4L$



From structures databook : $\theta_1 = \frac{-M\left(\frac{3L}{4}\right)}{3EI} \cdot \frac{1}{2} + \frac{M_1\left(\frac{3L}{4}\right)}{3EI}$

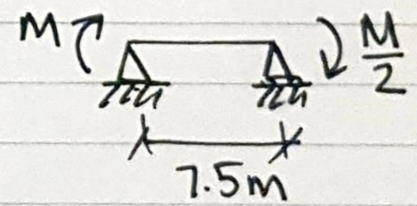
$$\theta_2 = \frac{-M_1\left(\frac{L}{4}\right)}{3EI} - \frac{M\left(\frac{L}{4}\right)}{3EI} \cdot \frac{1}{2}$$

Compatibility $\Rightarrow \theta_1 = \theta_2$

$$\frac{-ML}{8EI} + \frac{M_1 L}{4EI} = \frac{-M_1 L}{12EI} - \frac{ML}{24EI}$$

$$\frac{M_1}{3} = \frac{M}{6}$$

$$M_1 = \tfrac{1}{2}M$$

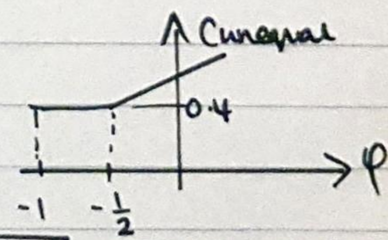


"CALCULATE THE PLASTIC" : same as previous case $\Rightarrow M_{PL} = 171.5\,kNm$

"CALCULATE THE ELASTIC" : Now $\varphi = -\frac{1}{2}$

Cunequal = 0.4

L = 7.5m.



$M_{LT} = \frac{\pi}{L}\sqrt{EI_{MINOR}GJ}\sqrt{1+\frac{\pi^2 E\Gamma}{L^2 GJ}}$ $\boxed{DS3}$

$M_{LT} = 56.4\,kNm$

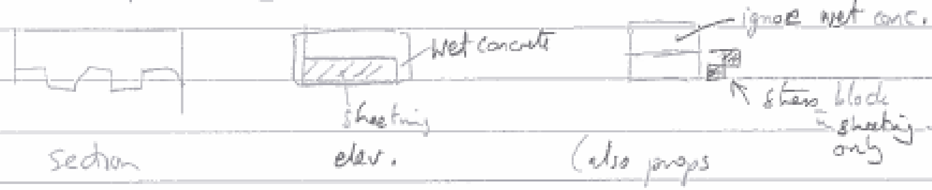
$M_{ce} = M_{LT}/0.4 = 141\,kNm$

"ELASTO-PLASTIC" : $\bar{\lambda}_{LT} = \sqrt{\frac{M_{PLASTIC}}{M_{ELASTIC}}} = 1.10 \Rightarrow \chi = 0.6$

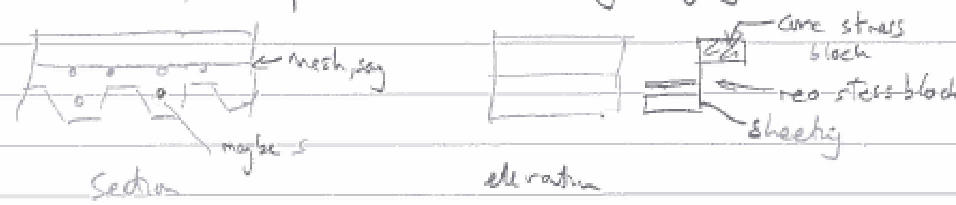
$M_{max} = 102.9\,kNm$

Factor of increase = 1.25 (or 25%)

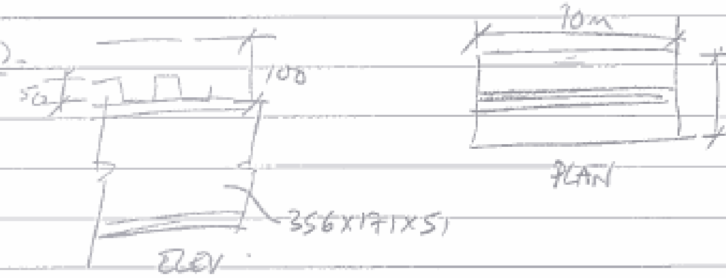
Q2(a) Load case 1, during construction. Only sheeting has strength.



Load case 2, after construction → eg carrying library load



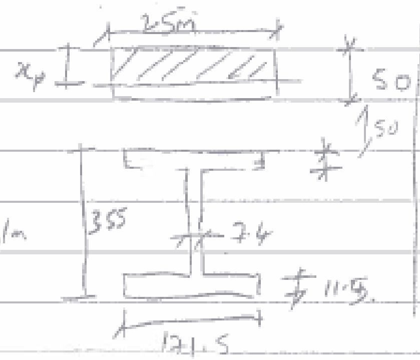
Q2(b).



$b_e = \min\left(\frac{\text{span}}{4}, \text{width}\right)$

$= \min(2.5, 3)$

$b_e = \underline{\underline{2.5\,m}}$



Area = $\underline{\underline{6490\ mm^2}}$

$0.6 f_{cd}(b)d = 2250\,kN.$

$A_{st}\sigma_y = (6490)(275) = 1785\,kN$

∴ N/A in concrete

$0.6 f_{cd}\, b\, x_p = A_{st}\sigma_y$

$x_p = \frac{1785 \times 10^3}{0.6(30)(2500)}$

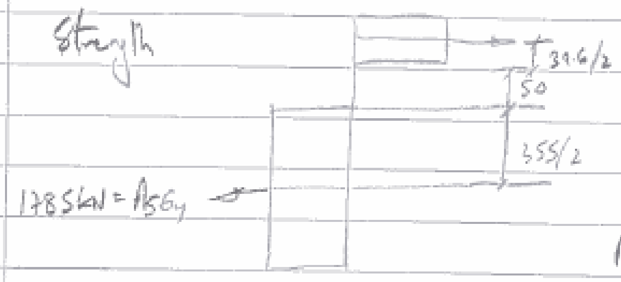
$= \underline{\underline{39.6\ mm}}$

Check compactness → flange $= \frac{171.5 - 7.4}{2(11.5)} = 7.13$ OK < 8.

web $= \frac{355 - 2(11.5)}{7.4} = 44.9 < 56$ OK in bending (but in tension anyway, mostly)

Q2 b) cont'd.

Strength



$$M = \underset{kN}{(1785)}\left(\frac{355 + 39.6}{2} + 50\right)\frac{1}{1000}$$

$= \underline{441 \text{ kNm}}$ strength.

Loads = 51 kg/m beam = 51 × 9.81 = 0.500 kN/m

Concrete = 0.075 m × 3m × 2400 kg/m³ × 9.81 = 5.3 kN/m

TOT = 5.8 kN/m DL.

Load = 1.35(5.8) + 1.5(wb) = W.

$M = \frac{WL^2}{8}$ $\quad W = \frac{8(441)}{100} = 35.3$ kN/m

$1.5wb = W - 1.35(5.8)$

$wb = \frac{35.3 - 1.35(5.8)}{1.5} = \underline{18.3 \text{ kN/m}}$

$w = 18.3$ $\quad = \underline{6.1 \text{ kPa}}$ (~ Library)

$b \to 3$ $\quad$ floor load.

Check shear. $S = \frac{WL}{2} = 35.3 \times 5 = \underline{176.5 \text{ kN}}$

$A_{net} = [355 - 2(11.5)](7.4) = \underline{2457 \text{ mm}^2}$

$\tau = \frac{176.5 \times 10^3}{2457} = \underline{71.8 \text{ MPa}}$ cf $\frac{\sigma_y}{\sqrt{3}} = \frac{275}{\sqrt{3}} = \underline{159 \text{ MPa}}$

∴ OK

Force at centre $= A_{st}\sigma_y = (1785\ kN)$

13 mm dia studs
65 mm high } 47 kN/stud (DSG)
(others don't fit)

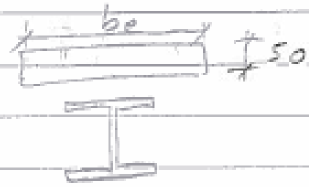
$\longrightarrow$ 38 studs req'd each half span.
(76 total span).

Number of troughs each half span $= \dfrac{5000}{200} = 25$ troughs ea

$\therefore$ Need 2 in each trough :–
50 troughs,
but at 80% of load $\rightarrow$
equivalent to 40 studs at full strength

So need 2 studs (13 x 65) in each trough.
Only just works

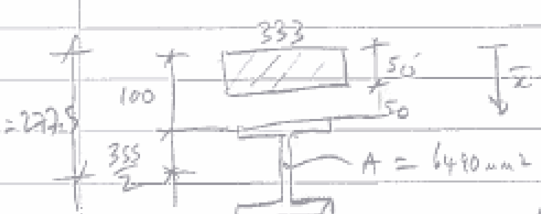
Q2(c) Deflection.



Unfactored LL = 6.1 kPa
(from earlier)
$w_{unfactored} = 18.3\ kN/m$

$b_e \times$ modular ratio $= 2.5 \left(\dfrac{28\ GPa}{210\ GPa}\right)$ ← short term

$= \underline{0.333\ m.}$



$A_{conc} = 16,667\ mm^2$
$A_{TOT} = 16,667 + 6490 = 23157\ mm^2$

$A_{TOT}\bar{x} = A_{conc}(25) + A_{st}(277.5)$

$\bar{x} = \dfrac{16667(25) + 6490(277.5)}{23157} = \underline{95.8\ mm}$
(in trough region).

Q2 contd.

$$I = \frac{(333)(50)^3}{12} + (333)(50)(95.8-25)^2$$

$$+ 14140 \times 10^4 \text{ mm}^4 + (6490)(277.5)^2$$

$$= 3.47 \times 10^6 + 83.46 \times 10^6$$

$$+ 141.4 \times 10^6 + 499.8 \times 10^6 = 728 \times 10^6 \text{ mm}^4$$

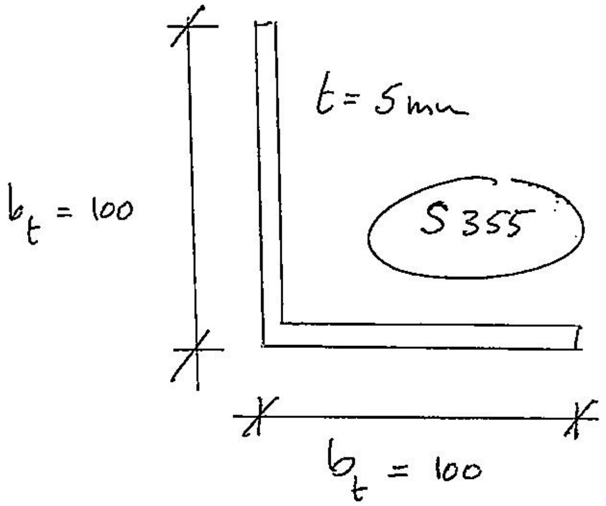
$$\Delta = \frac{5wL^4}{384EI} \quad \text{data book} = \frac{5(183 \times 10^3)(10^4)}{384(210 \times 10^9)(728 \times 10^{-6})} \quad \frac{\text{N/m} \quad \text{m}^4}{\text{N/m}^2 \quad \text{m}^4}$$

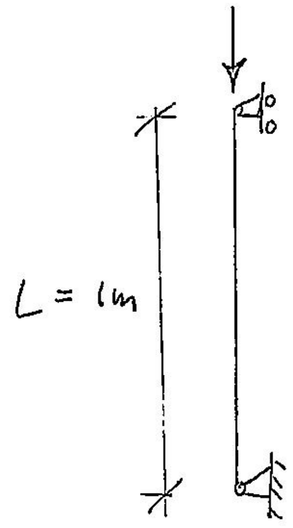
$$= 0.0156 \text{ m} = \underline{\underline{16 \text{ mm}}}$$

$$\frac{\text{Span}}{150} = \frac{10000}{150} = \underline{\underline{66 \text{ mm}}} \quad \therefore$$

Ours is $\sim \frac{\text{span}}{625}$ $\therefore$ fine for deflections.

Q3





Class?

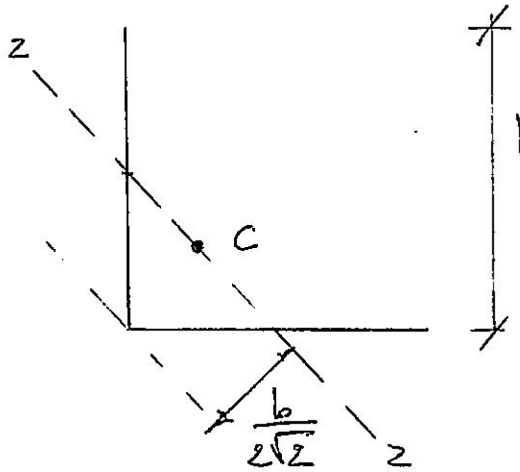
Class 3 cut-off : $\frac{c}{t} \overset{?}{\leq} 14\varepsilon$ $\qquad \varepsilon = 0.81$

$14\varepsilon = 11.34$

$$\frac{c}{t} = \frac{100-5}{5} = 19 > 11.34$$

$\rightarrow$ Class 4

Gross-section properties



$$b = 100 - \frac{5}{2} = 97.5 \text{ mm}$$

$$A = 2bt = 975 \text{ mm}^2$$

$$I_{zz} = 2\frac{(\sqrt{2}t)}{12}\left(\frac{b}{\sqrt{2}}\right)^3 = \frac{tb^3}{12}$$

$$= 386 \times 10^3 \text{ mm}^4$$

Effective cross-section (compression)

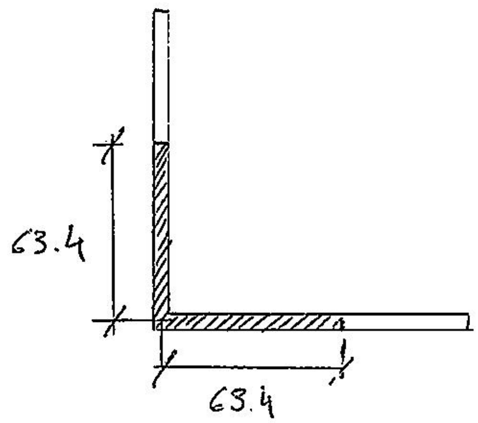
$$\sigma_{cr} = \underbrace{0.43}_{\text{outstand element}} \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{5}{97.5}\right)^2 = 204 \text{ MPa}$$

(EN 1993-1-5)

$$\lambda = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{355}{204}} = 1.32$$

$$\rho = \frac{1}{\lambda}\left(1 - \frac{0.188}{\lambda}\right) = 0.65$$

$$b_{eff} = \rho\, b = 63.4 \text{ mm}$$



Concentric column strength

$$N_{Euler} = \frac{\pi^2 E I_{zz}}{L^2} = \pi^2 (200\,000)(386) / 1000^2$$

$$= 762 \text{ kN}$$

$$\lambda_c = \sqrt{\frac{A_{eff} \cdot f_y}{N_{Euler}}} = \sqrt{\frac{(2)(63.4)(5)(355)}{(762)(10^3)}} = 0.30$$

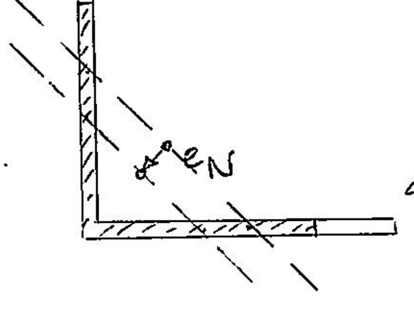
Buckling curve $\underline{b}$: $\alpha = 0.34$ (EN 1993-1-1)

$$\phi = \frac{1}{2}\left[1 + 0.34\,(0.3 - 0.2) + 0.3^2\right] = 0.56$$

$$\chi = \frac{1}{0.56 + \sqrt{0.56^2 - 0.30^2}} = 0.964$$

$$N_{b,Rd} = \chi\, A_{eff} \cdot \frac{f_y}{\gamma_{M0}} = (0.964)(225) = 217 \text{ kN}$$

Shift of the effective centroid

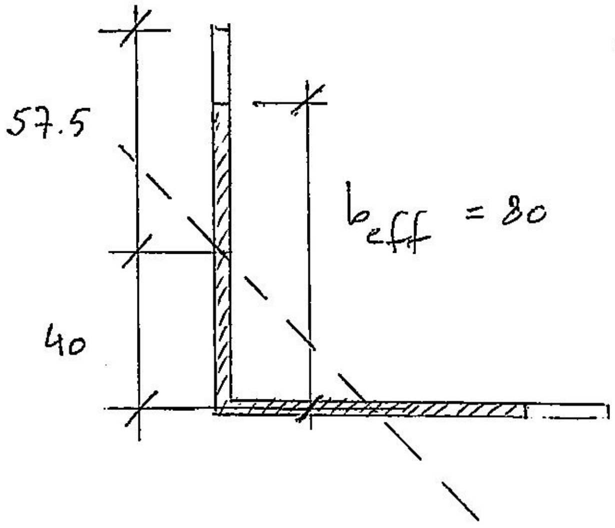


$$e_N = \frac{97.5}{2\sqrt{2}} - \frac{63.4}{2\sqrt{2}} = 12.06 \text{ mm}$$

tips are subjected to additional compression as a result of the bending moment caused by the shift of the effective centroid

Effective cross-section in bending

In a first iteration,
assume $b_{eff} = 80$ mm



$$\psi = -\frac{40}{57.5} = -0.7$$

$$k = 0.57 - 0.21\psi + 0.07\psi^2 = 0.75$$

$$\sigma_{cr} = k \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{5}{97.5}\right)^2 = 356 \text{ MPa}$$

$$\lambda = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{355}{356}} = 1.0$$

$$\rho = \frac{1}{\lambda}\left(1 - \frac{0.188}{\lambda}\right) = 0.812$$

$$\rightarrow b_{eff} = 40 + (0.812)(57.5) = 86.7 \text{ mm}$$

... close enough

If one more iteration is carried out:

$$\psi = -\frac{43.35}{54.15} = -0.8$$

$k = 0.78$

$\sigma_{cr} = 372$ MPa

$\lambda = 0.98$

$\rho = 0.83$ $\qquad b_{eff} = 43.35 + (0.83)(54.15)$

$= 88.1$ mm

W_{eff} ?

$$I_{zz,eff} = \frac{(5)(88.1)^3}{12} = 285\ (10)^3\ mm^4$$

$$W_{zz,eff} = \frac{I_{zz,eff}}{b_{eff}/2\sqrt{2}} = \frac{285\,050}{31.2} = 9150\ mm^3$$

$$M_{c,Rd} = W_{zz,eff} \cdot \frac{f_y}{\gamma_{M0}} = 3.25\ kNm$$

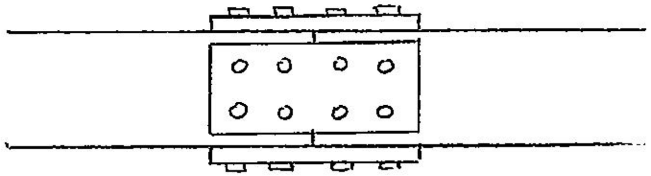
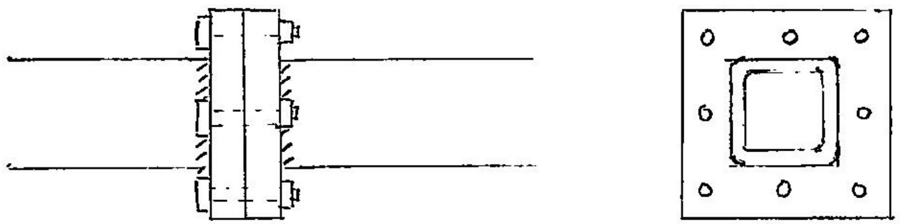
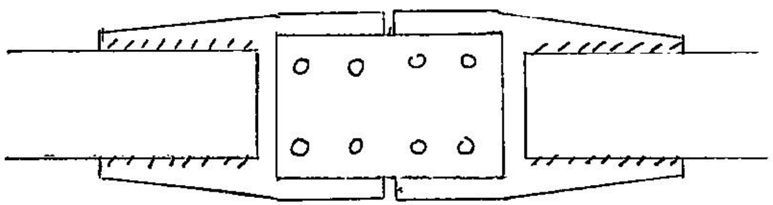
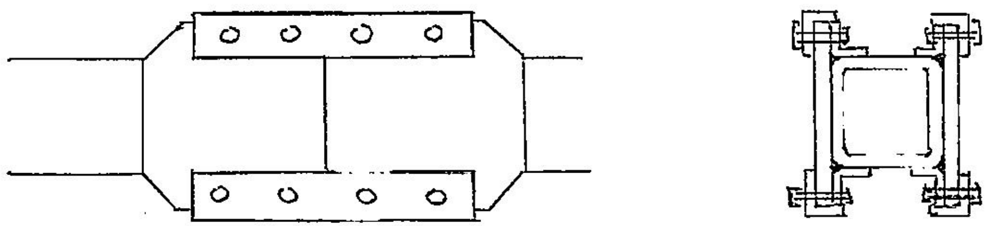
Ultimate capacity

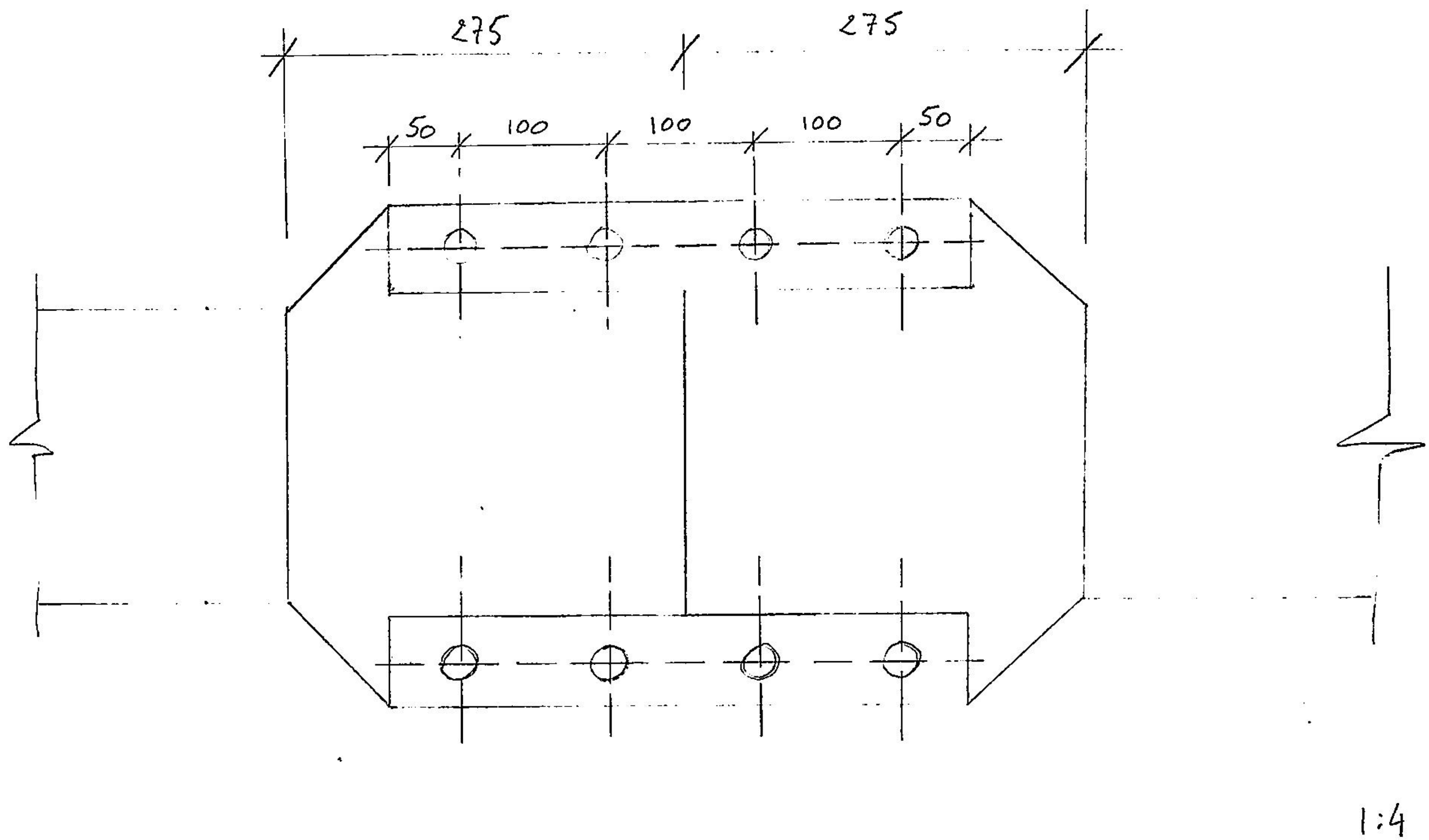
$$\frac{N^*}{\chi A_{eff} \cdot f_y} + \frac{N^* \cdot e_N}{W_{eff,zz} \cdot f_y} = 1.0$$

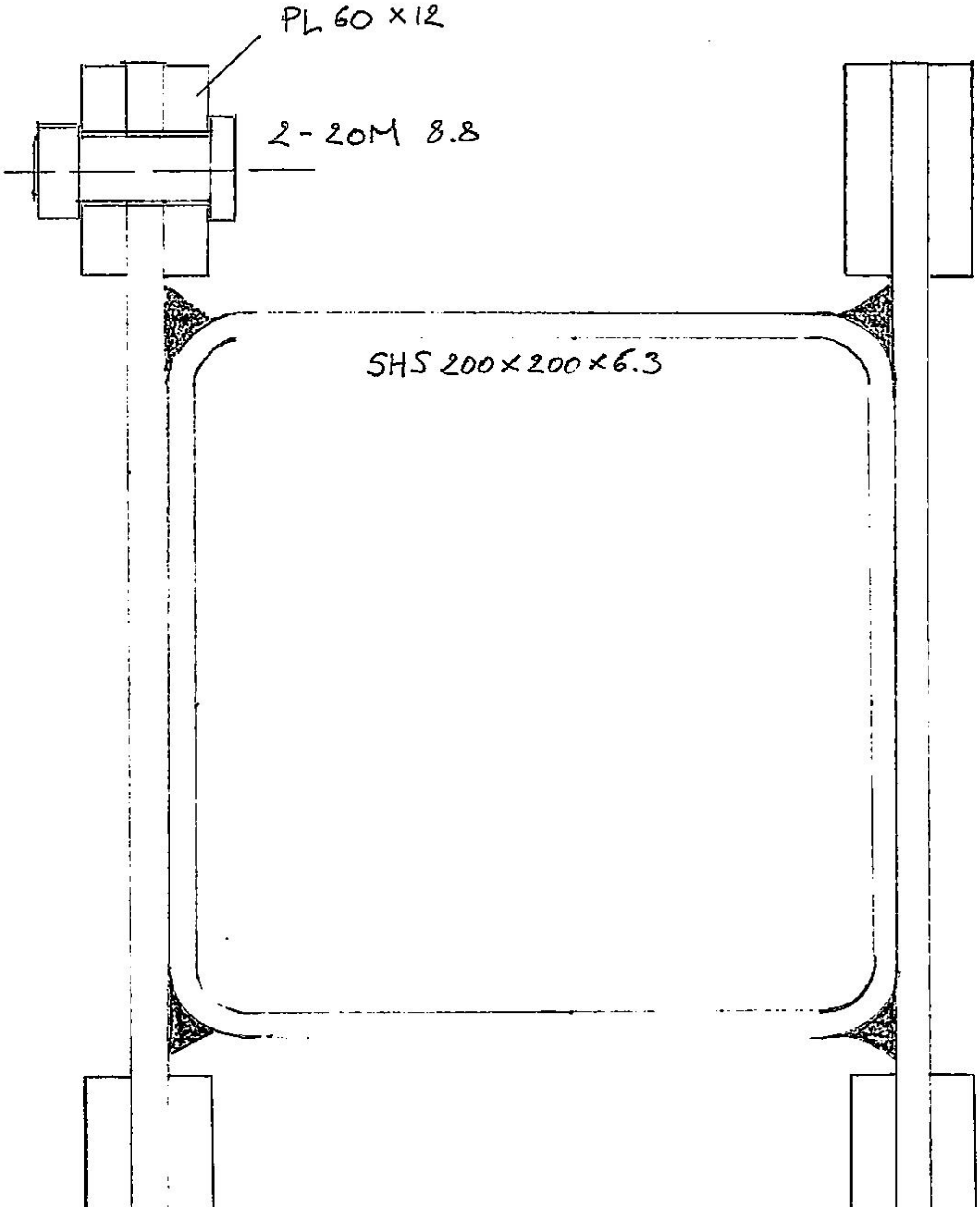
$$N^* = 1.0 / \left(\frac{1}{\chi A_{eff} \cdot f_y} + \frac{e_N}{W_{eff,zz} \cdot f_y} \right)$$

$$\boxed{N^* = 120\ kN}$$

Q4

Solution	Comments	Multiplier
	Impossible to install bolts.	0.5
	Prying action under tension	0.75
	Compromises out-of-plane stability of chord, but Ok in tension.	0.9
	'Standard' industry solution (Packer & Henderson)	1.0


275
275
50
100
100
100
50
1:4



1:2

Weld

$$\sqrt{3}\,\tau \leq \frac{f_u}{\beta_w \gamma_{M2}} \Rightarrow \sqrt{3}\left(\frac{1250\text{ kN}}{4}\right) / a.L_{eff} \leq \frac{490\text{ MPa}}{(0.9)(1.1)}$$

$$\Rightarrow a.\,L_{eff} \geq 1473\text{ mm}^2$$

Take 8 mm fillet $\Rightarrow a = (0.7)(8) = 5.6$ mm

$\rightarrow L_{eff} = 260$ mm

$L_{weld} = 260 + 2a \approx 275$ mm   'long' weld ? No

Bolts

$$F_{req.} = \frac{1250\text{ kN}}{8} = 156\text{ kN}$$

Take 20M grade 8.8 $\rightarrow A_s = 245\text{ mm}^2$ (conservative)

$$F_{v,Rd} = 0.6\,(245)\,\frac{640}{1.1} \times 2 = 171\text{ kN} \quad \text{ok}$$

double shear

Splice plates (8 plates $\Rightarrow$ take 156 kN each)

Net section fracture:

$$(0.9)\,A_{net}\,\frac{(490)}{(1.1)} \geq 156\text{ kN} \rightarrow A_{net} \geq 389\text{ mm}^2$$

$$A_g = 389 + \underbrace{(22)}_{d_0}\underbrace{(12)}_{t} = 653\text{ mm}^2 \rightarrow PL\ 60 \times 12$$

Gross section yielding:

$$(720)(355) = 256\text{ kN} \quad \text{ok}$$

Bolt bearing:

Take $e_1 = 50$ mm, $e_2 = 30$ mm, $p_1 = 100$ mm

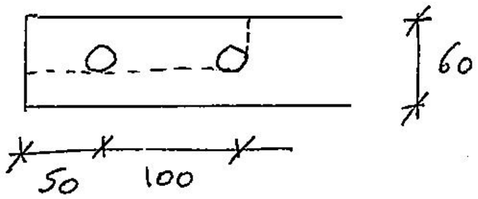
$$\alpha_d = \min\left(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}\right) = \min\left(\frac{50}{66}, \frac{100}{66} - 0.25\right) = 0.75 \Rightarrow \alpha_b = 0.75$$

$$k_1 = \min\left(2.8\,\frac{e_2}{d_0} - 1.7,\ 2.5\right) = 2.12$$

$$F_{b,Rd} = (0.75)(2.12)(490)(20)(12)/_{1.1} = 170\ kN > \frac{156}{2}\ kN$$

ok

Block tear-out :



$$A_{nt} = (30)(12) = 360\ mm^2$$

$$A_{nV} = (150)(12) = 1800\ mm^2$$

$$F_{eff,Rd} = \frac{(360)(490)}{1.1} + \frac{(1800)(355)}{\sqrt{3}} = 529\ kN \gg 156\ kN$$

ok

Side plates

Net section fracture :

$$(0.9)\underbrace{[340 - 2(22)]}_{A_{net}}\, t\ \frac{(490)}{1.1} \geqslant \frac{1250}{2}\ kN$$

$$\Rightarrow \quad t \geqslant 5.3\ mm$$

Gross-section yielding :

$$(340)\, t\ (355) \geqslant \frac{1250}{2}\ kN \quad \Rightarrow \quad t \geqslant 5.2\ mm$$

Bearing :

$$\left.\begin{array}{l} e_1 = 50\ mm \\ e_2 = 30\ mm \\ p_1 = 100\ mm \end{array}\right\} \quad \begin{array}{l} \alpha_b = 0.75 \\ k_1 = 2.12 \end{array} \quad \text{(see above)}$$

$$F_{b,Rd} = (0.75)(2.12)(490)(20)\, t/_{1.1} \geqslant \frac{1250}{8}\ kN$$

$$\Rightarrow t \geqslant 11.0\ mm$$

$\rightarrow$ PL 340 x 12

Block tear-out

Same situation as above

$$F_{eff,Rd} = 529\ kN > \frac{1250}{4}\ kN = 313\ kN$$

ok

ENGINEERING TRIPOS PART IIB 2021
COMMENTS FROM ASSESSORS REPORT
MODULE 4D10, STRUCTURAL STEELWORK

Examination, Question 1: lateral torsional buckling capacity
Candidates were presented with a familiar buckling question, with a novel second part requiring reassessment of the buckling capacity when further restraint is included. All candidates fielded solutions which were, in general, very detailed, with some candidates achieving full marks.

Examination, Question 2: composite floor design
Candidates were required to verify the performance of a heavy-duty composite floor section using profiled steel decking and cast concrete supported by commercial beams. Atypically, candidates were asked to calculate the limits of live-loading (when it is normally specified). This was a popular question answered well. The most common mistake, however, was failing to account for the influence of the construction process on the deflections; in unpropped construction, the self-weight acts on the beams alone whilst the live-loading acts on the composite section. Most candidates furnished an adequate serviceability check of deflections and designed for the correct number of shear connectors.

Examination, Question 3: interactive axial buckling
Candidates were asked to determine the axial and local buckling capacities of a right-angled, "equal leg" cross-section. This question was a reinterpretation of the interaction equation approach to axial/flexural bucking effects from previous years, and was tackled by most candidates. Most of them found the effective area for compressive behaviour alone. Some failed to identify correctly the minor axis of bending of cross-section, and drew it parallel to one of the angle side-lengths; some candidates did not register that flexural buckling was possible and mistakenly substituted the cross-sectional compressive capacity into the interaction equation.

Examination, Question 4: tension splice joint design
Candidates were asked to design, from scratch, a splice joint for a standard commercial square section in tension. There were only three solutions, possibly because of the "open-ended" design nature or because it was the last question on the paper (when 3 from 4 solutions were required). One solution was poorly detailed and unrealistic, but the other two incorporated proper design checks and some innovative features.