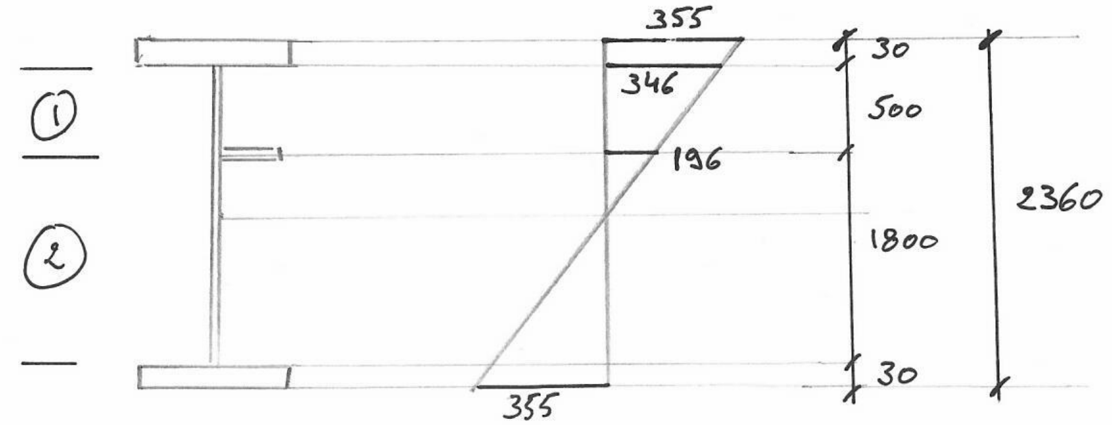
Q1a



Flange

$$\frac{c}{t} = \frac{(500-16)/2}{30} = 8.07 < 10\varepsilon = 8.1 \rightarrow \text{Class } ②$$

Web

Panel ①

$$\psi = \frac{196}{346} = 0.57$$

$$k = \frac{8.2}{1.05+0.57} = 5.07$$

$$\sigma_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 = 939 \text{ MPa}$$

$$\lambda = \sqrt{\frac{f_y}{\sigma_{cr}}} = 0.61$$

Panel ②

$$\psi = -\frac{1}{0.57} = -1.77$$

$$k = 5.98(1-\psi)^2 = 45.7$$

$$\sigma_{cr} = 653 \text{ MPa}$$

$$\lambda = 0.73$$

$$\lambda_{limit} = 0.5 + \sqrt{0.085 - 0.055\psi}$$

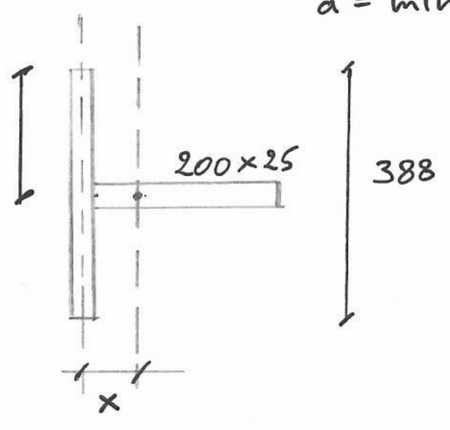
Panel ①: $\lambda < \lambda_{limit} = 0.73$

Panel ②: $\lambda < \lambda_{limit} = 0.93$

→ The girder is fully effective against local buckling

Q1b

$$d = \min(250, 15\varepsilon t) = 194 \text{ mm}$$



Class? $\frac{c}{t} = \frac{200}{25} = 8 < 10\varepsilon = 8.1$

→ Class ②

$$x = \frac{(200)(25)(108)}{(200)(25) + (388)(16)} = 48.2 \text{ mm}$$

$$A = 11208 \text{ mm}^2$$

$$I = (388)(16)(48.2)^2 + \frac{(25)(200)^3}{12} + (200)(25)(59.8)^2 = 4.90\,(10^7) \text{ mm}^4$$

$$\alpha_c = 0.49 + \frac{0.09}{i/e}$$

$$e = \max(108 - 48.2, 48.2) = 59.8 \text{ mm}$$

$$i = \sqrt{\frac{I}{A}} = 120.15 \text{ mm}$$

$\Big\}$ $\alpha_c = 0.53$

$$\text{required } \chi = \frac{196}{355} = 0.55$$

Fig 6.4 databook $\rightarrow$ $\lambda \approx 0.9$

$$\lambda = \sqrt{\frac{A.f_y}{\pi^2 EI}}\, L = 0.9 \quad \rightarrow \quad L = 4.4 \text{ m}$$

Q1c

$$I = \frac{(16)(2300)^3}{12} + 2(500)(30)\left(\frac{2300}{2} + \frac{30}{2}\right)^2 = 5.69\,(10^{10}) \text{ mm}^4$$

$$W_{el} = I/(2360/2) = 4.83\,(10^7) \text{ mm}^3$$

$$M_{c,Rd} = W_{el} \cdot f_y / \gamma_{M0} = 17\,130 \text{ kNm}$$

Q2a

$A = (16)(2300) + 2(30)(500) = 66\,800 \text{ mm}^2$

$g_k = 5.24 \text{ kN/m}$

$M_{sw} = \dfrac{(5.24)(50)^2}{8} = 1639 \text{ kNm}$

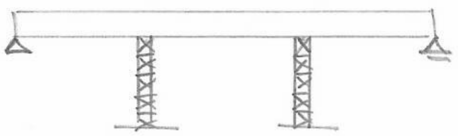
$I_{zz} = \dfrac{(500)^3(30)}{12}(2) = 6.25\,(10^8) \text{ mm}^4$

$J = \sum \dfrac{bt^3}{3} = 9.20\,(10^6) \text{ mm}^4$

$C_w = I_{zz} \cdot \dfrac{d^2}{4} = 8.48\,(10^{14}) \text{ mm}^4$ $\qquad L_{cr} = 50 \text{ m}$

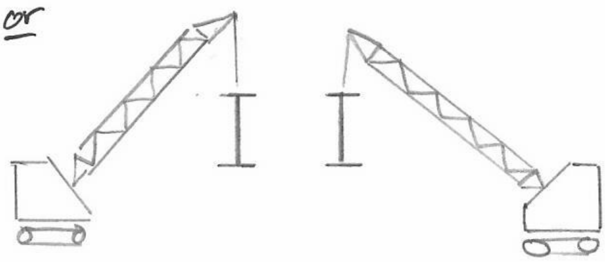
$M_{cr} = \dfrac{\pi^2 E I_{zz}}{L_{cr}^2} \sqrt{\dfrac{C_w}{I_{zz}} + \dfrac{L_{cr}^2}{\pi^2} \dfrac{GJ}{EI_{zz}}} = 824 \text{ kNm} \ll M_{sw}$

fail



Scaffolding towers to reduce s.w. moments

or



Use 2 cranes and connect cross-bracing in this position.

Q2b

$L_{cr} = 7.14 \text{ m}$

$M_{cr} = 28\,495 \text{ kNm}$

buckling curve (d) $\alpha = 0.76$

$\lambda = \sqrt{\dfrac{M_y}{M_{cr}}} = \sqrt{\dfrac{17\,130}{28\,495}} = 0.775$

$\phi = 0.5(1 + 0.76(\lambda - 0.2) + \lambda^2) = 1.02$

$\chi = 0.59$

$M_{b,Rd} = 0.59\, M_y = 10192 \text{ kNm}$

$g_k = \underbrace{5.24 \text{ kN/m}}_{\text{girder}} + \underbrace{21}_{\text{deck}} \text{ kN/m} = 26.24 \text{ kN/m}$

$M_{Ed} = (1.35)(26.24)(50)^2/8 = 11\,072$ kNm

slight fail

Q2c

$b_{eff} = \min\left(\frac{L}{4} + b_0,\ 3.5\,m\right) = 3.5\,m$

$N_c = 0.85\, f_{cd}\, b_{eff} \cdot h = 17\,354$ kN $\left(f_{cd} = f_{ck}/1.5 = \frac{35}{1.5} = 23.3\text{ MPa}\right)$

$N_s = A_s \cdot f_y = 23\,714$ kN

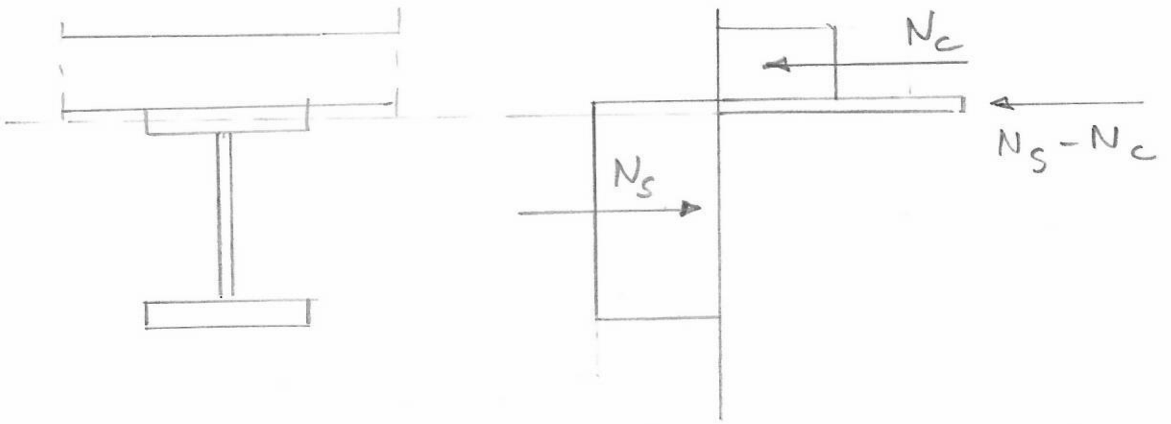
→ N.A. in the beam

Assume N.A. in top flange

→ $2 b_f (x - h) f_y = N_s - N_c$ → $x = 268$ mm $< 250 + 30$

ok

$M_{Rd} = N_s\left(\frac{2360}{2} + \frac{250}{2}\right) - (N_s - N_c)\left(\frac{x}{2}\right) = 30\,095$ kNm



Q3a.

<u>ULS</u>

- Cross-sectional bending capacity
- Shear capacity of the web (shear buckling / yielding)
- Static capacity of the shear studs
- Local buckling of the web (if N.A. in web)
- Bearing capacity over the supports (web crippling / crushing)

<u>SLS</u>

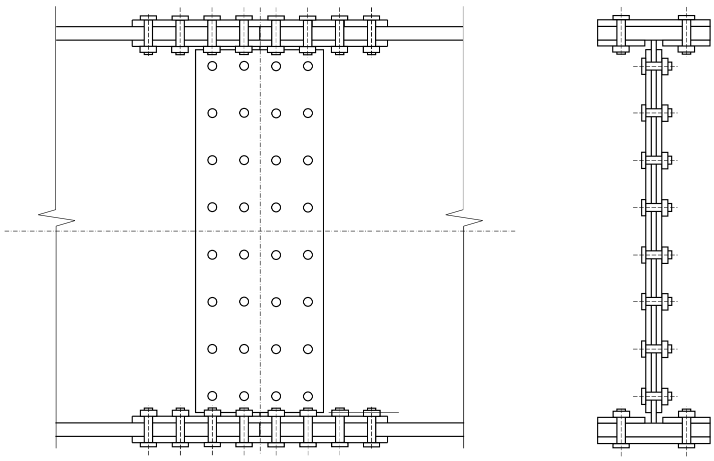
- Deflections/vibrations

<u>FLS</u>

- Fatigue of the tension flange
- Fatigue of the web (tension/shear)
- Fatigue of stiffener-to-web weld (if in tension)
- Fatigue of the web-to-flange weld (shear)
- Fatigue of the shear studs (shear)

Q3b.

- Bolt shear failure
- Bolt bearing failure in girder
- Bolt bearing failure in splice plates
- Net section failure of splice plates
  - In tension for the flange splices
  - In shear for the web splices
- Plastic shear of the net girder web
- Block tear-out of the splice plates
- Gross section yielding of the splice plates
  - In tension for the flange splices
  - In shear + bending for the web splices
- Local buckling of the splice plate in the compression flange splice



Q4

$q_k = 24\ kN/m \rightarrow q_d = (1.5)(24) = 36\ kN/m$

$g_k = 16\ kN/m \rightarrow g_d = (1.35)(16) = 21.6\ kN/m$

$57.6\ kN/m$

$V_d = \frac{(57.6)(5)}{2} = 144\ kN$

Bolt Shear

$M20 \rightarrow A_s = 245\ mm^2 \qquad A = 314\ mm^2$

$F_{V,Rd} = 0.6 A_s f_{ub} / \gamma_{M2} = (0.6)(245)(400)/1.1 = 53.5\ kN$

$\rightarrow$ 3 bolts

Bearing Take $e_1 = e_2 = 50\ mm$, $p_1 = 100\ mm$

$e_1 > 1.2 d_0$
$e_2 > 1.2 d_0$
$p_1 > 2.2 d_0$

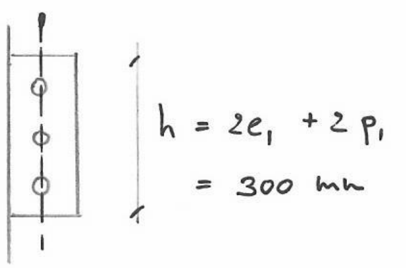
$\alpha_d = \min\left(\frac{50}{66}, \frac{100}{66} - 0.25\right) = 0.76$

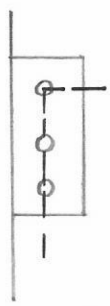
$k_1 = \min\left(2.8\left(\frac{50}{22}\right) - 1.7, 2.5\right) = 2.5$

$\alpha_b = \min\left(\alpha_d, \frac{400}{490}, 1\right) = 0.76$

$F_{b,Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} \geq \frac{V_d}{3} \rightarrow t \geq 3\ mm$

Fracture





$F_{f,1} = \frac{f_y}{\sqrt{3}} (h - 3d_0) t / \gamma_{M0} \geq V_d$

$\rightarrow t \geq 4\ mm$

$F_{f,2} = \left[0.5 \frac{f_u (50-11)}{1.1} + \frac{f_y}{\sqrt{3}} \frac{(250-55)}{1.0}\right] t \geq V_d$

$\rightarrow t \geq 3\ mm$

Flag plate : M + V

$M_d = V_d (0.05) = 7.2\ kNm$

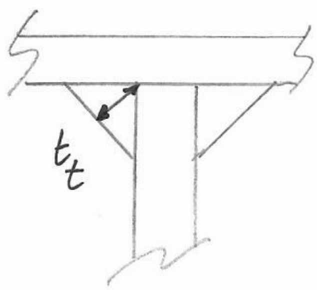
for $t = 4$ mm $\quad W_{el} = \frac{(4)(300)^2}{6} = 60\,000 \text{ mm}^3$

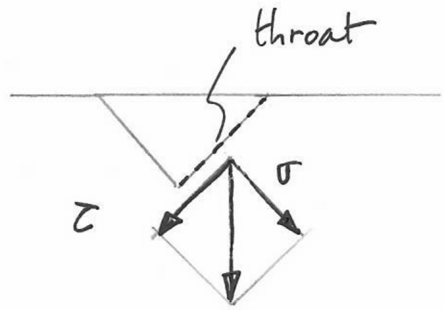
$c/t < 100 \rightarrow$ Class ③

$\sigma = \frac{M_d}{W_{el}} = 120$ MPa $\qquad \tau = \frac{V_d}{t\,h} = 120$ MPa

$\sqrt{\sigma^2 + 3\tau^2} = 240 \text{ MPa} < 355/1.0$ $\qquad$ Ok

Weld





$2\sqrt{2}\,\tau\, t_t \frac{h_{eff}^2}{6} = M_d$

$\tau_v = \frac{V_d}{2\, t_t \cdot h_{eff}}$ $\qquad \tau = \sigma = \frac{6 M_d}{2\sqrt{2}\, t_t\, h_{eff}^2}$

Try leg length of 3 mm $\rightarrow t_t = 2.12$ mm

$h_{eff} = 300 - (2)(2.12) = 295.8$ mm

$\tau = \sigma = 82$ MPa $\qquad \tau_v = 115$ MPa

$\sqrt{\sigma^2 + 3(\tau^2 + \tau_v^2)} < \frac{490}{(0.9)(1.1)}$ $\qquad \sigma \leq (0.9)\frac{490}{1.1}$

258 < 490 Ok $\qquad$ 82 < 400 Ok