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Emnekode/*Course code*: BYG200 Emnenavn/*Course title*: Stålkonstruksjoner

Dato/*Date*: 07.12.2017 Kandidat nr./*Candidate no.*:

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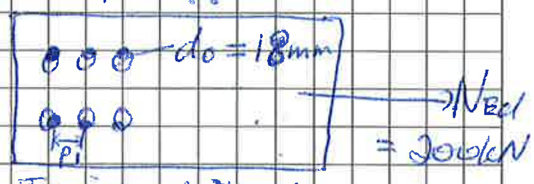
Question (1)

(a) (i) Check the suitability of the Tension plate (1):



Tension plate 1 (160mm X 10 mm)
Steel grade S275
$t = 10mm < 40mm$
$f_y = 275\ N/mm^2$ and
$f_u = 430\ N/mm^2$

- Yielding ~~check~~ of plate:

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{(160 \times 10) \times 275}{1{,}05 \times 10^3}$$

$$N_{pl,Rd} = 419{,}05\ kN > N_{Ed} = 200\ kN$$

- Local fracture near holes:

$$N_{u,Rd} = \frac{0{,}9 \cdot A_{net}\ f_u}{\gamma_{M2}}, \text{ where}$$

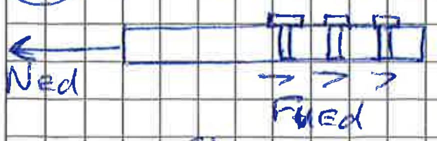
$$A_{net} = A - 2d_0t = (160 \times 10) - 2 \times 18 \times 10$$
$$= 1240\ mm^2$$

$$N_{u,Rd} = \frac{0{,}9 \times 1240 \times 430}{1{,}25 \times 10^3} = 383{,}90\ kN > N_{Ed} = 200kN$$

∴ Tension plate 1 is suitable (Can withstand the loading)

(ii) The resistance capacity of the plates is dependent on the Area. The area of Tension member 2 is larger than those to Tension member 1. Therefore, the Tension member 2 can withstand (carry) the loading.

(b) (i) Check the suitability of Bolt size:



Bolt size: M16 ; $d = 16mm$ & $d_0 = 18mm$

Bolt Class 8.8 ; $f_{yb} = 640\ N/mm^2$ & $f_{ub} = 800\ N/mm^2$

Design Shear ~~Tension~~ load ; $6 F_{v,Ed} = N_{Ed} \Rightarrow F_{v,Ed} = \frac{N_{Ed}}{6} = 33{,}33\ kN$

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Resistance capacity of bolt size:

~~[crossed out]~~ - Shear plane passes through threaded portion.

$F_{V,Ed} \leq F_{V,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$, where

From Tabel 6.1 in "STÅLKONSTRUKSJONER":

$F_{V,Rd} = F_{d,v}^{*}$ (Bolt 8.8 & through threaded portion & M16)

$= 60,3 \text{ kN}$

$F_{V,Ed} = 33,33 \text{ kN} < F_{V,Rd} = 60,3 \text{ kN}$

∴ Bolt size M16 is suitable.

(ii) Checking for bearing failure:

$F_{b,Ed} \leq F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$ (for the bolted plate.)

- Since the bearing resistance capacity of the bolts is much greater than that to the plates ~~whos~~ we just need to check the bearing capacity of the plates.

Given: $e_1 = 40mm$; $e_2 = 40mm$; $p_1 = 60mm$; $p_2 = 80mm$

(For Tension plate 1 b/c that is with the the smallest area).

- For edge bolts: α_d = Smallest of $\frac{e_1}{3d_0}$ & 1,0

$= 0,741 \text{ \& } 1,0 = 0,741$

For Inner bolts: $\alpha_d = \text{Min}\left[\frac{p_1}{3d_0} - \frac{1}{4}; 1,0\right] = [0,861; 1,0] = 0,861$

- ~~For~~ k_1 = Lesser of $\left[2,8\frac{e_2}{d_0} - 1,7; 1,4\frac{p_2}{d_0} - 1,7; 2,5\right]$

$= \text{Min}[4,52; 4,52; 2,5] = 2,5$

- α_b = Minimum of α_d, $\frac{f_{ub}}{f_u}$ or 1,0 = Min[0,741, 1,86; 1,0]
$= 0,741$

Design load of bearing for the plates:

$F_{b,Ed} = F_{v,Ed} = 33,33$ kN

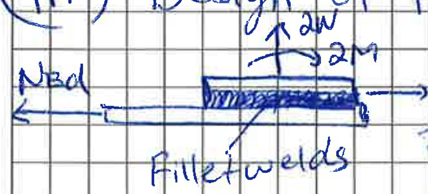
$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} = \frac{2,5 \times 0,741 \times 430 \times 16 \times 10}{1,25 \times 10^3}$

$= 101,96$ kN

Check: $F_{b,Ed} = 33,33$ kN $<$ $F_{b,Rd} = 101,96$ kN.

∴ The connection can withstand the design load without bearing failure.

(iii) Design of fillet welds:



- Throat thickness: $a = \frac{w}{\sqrt{2}} = \frac{10}{\sqrt{2}} = 7,07$
$a > 0,5\, t_p = 5$ mm > 3 mm

- Weld length: $L = ?$

- Design loads: $2M = 80 \times N_{Ed} \Rightarrow M = 40 \times 10^{-3} N_{Ed} = 8$ kNm

$2N = 0 \Rightarrow N = 0$

$2V = N_{Ed} \Rightarrow V = \frac{N_{Ed}}{2} = 100$ kN

- Elastic stresses at the throat:

From Table 6.2 in "STÅLKONSTRUKSJONER":

$\sigma_\perp = \tau_\perp = 0$; $\tau_\parallel = \frac{V}{L \cdot a} = \frac{100 \text{ kN}}{L a}$

- Checking for ULS of the elastic stresses

Check ①: $\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)} \le \frac{f_u}{\gamma_{M2} \beta_w}$

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$\sqrt{3\tau_{||}^2} = \sqrt{3}\,\frac{V}{La} \leq \frac{430}{1{,}25 \times 0{,}85}$

$L \cdot a \geq \frac{\sqrt{3} \times 100 \times 10^3 \times 1{,}25 \times 0{,}85}{430}$

$L \cdot a \geq 427{,}48$, where $a = 7{,}071 = \frac{10}{\sqrt{2}}$

$L \geq \frac{427{,}48\,mm^2}{7{,}071\,mm} = 60{,}53$

$\therefore$ $a = 7{,}071\,mm$ and $L = 65\,mm$

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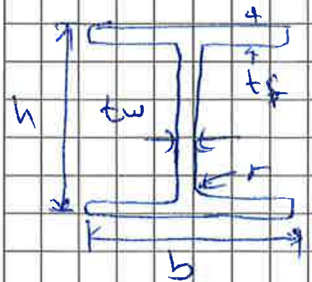
Question (2)

(a) (i) Check suitability of column if C/S is HE200B



$h = 200\,mm$, $t_f = 15\,mm$

$b = 200\,mm$, $t_w = 9\,mm$

$r = 18\,mm$

$A = 7810\,mm^2$

$i_y = 85{,}4\,mm$; $i_z = 50{,}7\,mm$

$L_{cr,y} = L_{cr,z} = 0{,}7\,L = 4{,}2\,m$

S275

$f_y = 275\,N/mm^2$

$\varepsilon = 0{,}92$

~~Welding capacity of the colum~~

~~$N_{c,Rd}$ =~~ Class classification:

Flange [Compression]:

$$\frac{C_f}{t_f \cdot \varepsilon} = \frac{0{,}5(b - t_w - 2r)}{t_f \cdot \varepsilon} = 5{,}62 < 9. \quad \text{Class (1)}$$

Web [Compression]

$$\frac{C_w}{t_w \cdot \varepsilon} = \frac{h - 2t_w - 2r}{t_w \cdot \varepsilon} = 16{,}2 < 33 \quad \text{Class (1)}.$$

∴ C/S is in Class (1) (No local buckling)

Yielding check:

$$N_{c,Rd} = N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{7810 \times 275}{1{,}05 \times 10^3} = 2045{,}5\,kN$$

$N_{c,Rd} > N_{Ed}$ ∴ Not subjected to yielding.

Buckling check:

$$N_{b,Rd} = \frac{\chi f_y A}{\gamma_{M1}} \geq N_{Ed}.$$

Find χ:

y-y axis

$$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \times \frac{1}{\lambda_1} = \frac{6000 \times 0,7}{85,4\,mm \times 93,9 \times 0,92}$$

$\bar{\lambda}_y = 0,569 > 0,2$

z-z axis

$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \times \frac{1}{\lambda_1} = \frac{4200}{50,7 \times 93,9 \times 0,92}$$

$\bar{\lambda}_z = 0,959 > 0,2$

From Table 6.2 in NS EN 1993-1-1.

Rolled sections: $\frac{h}{b} = 1 < 1,2$; $t_f = 15\,mm < 400\,mm$

Buckling curve "b"

$\alpha_y = 0,34$ (From Table 6.1)

$\phi_y = 0,5[1+\alpha_y(\bar{\lambda}_y - 0,2) + \bar{\lambda}_y^2]$

$= 0,7246$

$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^2 - \bar{\lambda}_y^2}} = 0,852$

Buckling curve "c"

$\alpha_c = 0,49$ (From Table 6.1)

$\phi_z = 0,5[1+\alpha_z(\bar{\lambda}_z - 0,2) + \bar{\lambda}_z^2]$

$= 1,146$

$\chi_z = 0,564$

χ = Lesser of χ_y & χ_z = 0,564.

∴ Buckling axis is z-z.

$$N_{b,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0,564 \times 7810 \times 275}{1,05 \times 10^3} = 1153,65\,kN.$$

$N_{b,Rd} < N_{Ed} = 1300\,kN$ ∴ The column will buckle.

∴ The column will buckle. It's not suitable

(ii) If the support end B is changed, L_{cr} will increase which will then increase $\bar{\lambda}$, χ decreases and $N_{b,Rd}$ decreases too. Same as above ($N_{b,Rd} < N_{Ed}$), the column will buckle.

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(b)

(i) * The reason for class classification of steel is

① The check whether a certain C/s will local buckle which can reduce it's resistance capacity;

② In order to choose the right C/s for the right purpose (If safety is important, as public buildings, class 1 C/s is preferable ...).

* Class classification of the C/s.

Flange [Compression]

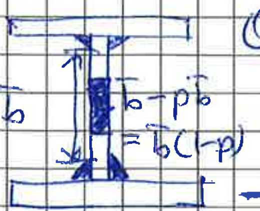
$$\frac{C_f}{t_f\,\varepsilon} = \frac{0{,}5(190-10-2\times8)}{12\times0{,}92} = 7{,}43 < 9 \quad \text{Class ①}$$

Web [Compression]

$$\frac{C_w}{t_w\,\varepsilon} = \frac{474-2\times12-2\times8}{10\times0{,}92} = 47{,}2 > 42 \quad \text{Class.}$$

∴ The C/s is in Class 4.

* The Effective cross sectional area.



Only web is in Class 4.

Find ρ:

- Stress ~~cont~~ distribution: $\psi = \frac{\sigma_2}{\sigma_1} = 1$

- $k_\sigma(\psi=1$ & Internal$) = 4{,}0$.

- $\bar{b} = C_w = 474-2\times12-2\times8 = 434$ mm and $t = 10$ mm

- $\bar{\lambda}_p = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} = \frac{434/10}{28{,}4\times0{,}92\times2} = 0{,}831 > 0{,}673$

- $\rho = \frac{\bar{\lambda}_p - 0{,}055(3+\psi)}{\bar{\lambda}_p^2} = 0{,}885 < 1{,}0$

- $A_{eff} = A - \bar{b}(1-\rho)\cdot t$, when

$A = 190 \times 12 \times 2 + (474 - 2\times 12) \times 10 = 9060 \text{ mm}^2$

$A_{eff} = 9060 - 434(1 - 0{,}885) \times 10 = 8560{,}9 \text{ mm}^2$

(ii) Design compressive resistance: $N_{c,Rd}$

$N_{c,Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M0}} = \frac{8560{,}9 \times 275}{1{,}05 \times 10^3} = 2242{,}14 \text{ kN}$

Check: $N_{c,Rd} > N_{Ed} = 1300 \text{ kN}$.

∴ The column will not yield

(iii) Design buckling resistance: $N_{b,Rd}$

$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}}$

Find χ:

y-y axis

$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \times \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} = \frac{4200 \times \sqrt{\frac{8560{,}9}{9060}}}{187{,}2 \times 0{,}92 \times 93{,}9}$ | $\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \times \frac{\sqrt{\frac{A_{eff}}{A_1}}}{\lambda_1}$, $i_z = 38{,}9 \text{ mm}$

$\bar{\lambda}_y = 0{,}282 > 0{,}2$. | $\bar{\lambda}_z = 1{,}215 > 0{,}2$.

From Table 6.2: Welded I-section; $t_f = 12 \text{ mm} < 40 \text{ mm}$

Buckling curve "b" & $\alpha_y = 0{,}34$ | Buckling curve "c" & $\alpha_z = 0{,}49$

$\phi_y = 0{,}541$ | $\phi_z = 1{,}48$

$\chi_y = 0{,}981$ | $\chi_z = 0{,}429$

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χ = Lesser of χ_y & χ_z = 0,429.

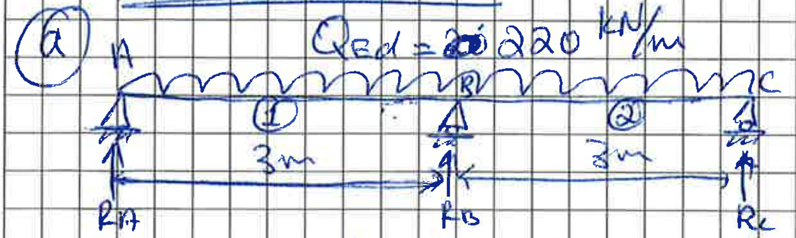
$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} = \frac{0{,}429 \times 8560{,}4 \times 275}{1{,}05 \times 10^3} = 961{,}9 \text{ kN}$$

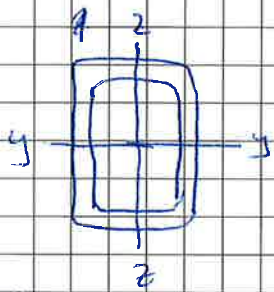
Check: $N_{b,Rd} < N_{Ed} = 1800$ kN

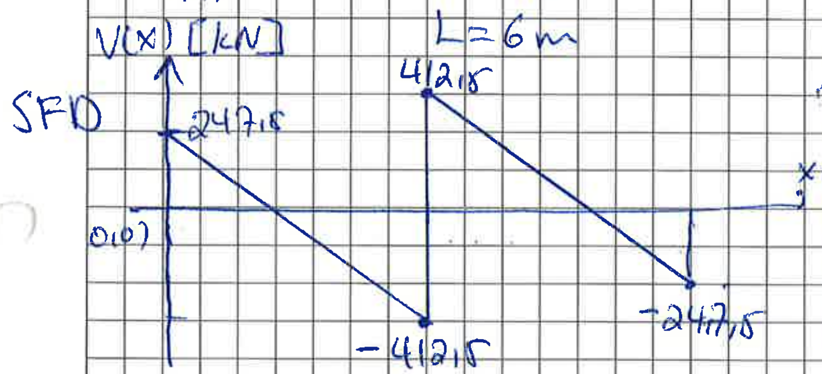
∴ The Column cannot withstand the axial load without overall flexural buckling.

Question (3)

(a)







Support Reactions. (Table 3.5)

$R_A = Q_{Ed} \frac{L}{2} \times 0{,}375 = 247{,}5$ kN

$R_B = 1{,}25\, Q_{Ed} \frac{L}{2} = 825$ kN

$R_C = Q_{Ed} L - R_A - R_B$

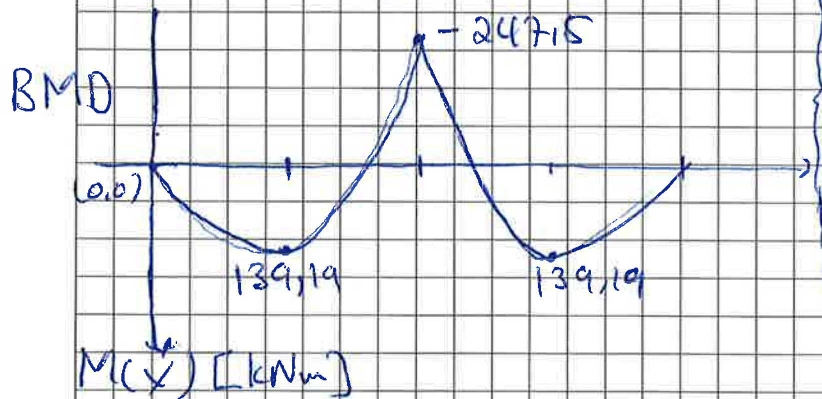
$= 247{,}5$ kN

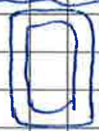
$M_1 = M_2 = 0{,}0703\, Q_{Ed} \left(\frac{L}{2}\right)^2$

$= 139{,}19$ kNm

$M_B = -0{,}125 \times Q_{Ed} \times \left(\frac{L}{2}\right)^2$

$= -247{,}5$ kNm





S355 grade

$f_y = 355$ N/mm²

$\varepsilon = 0{,}81$

(i) Design hot rolled RHS:

- Design loads: $M_{y,Ed} = 247{,}5$ kNm , $V_{Ed} = 412{,}5$ kN

$F_{b,Ed} = 412{,}5$ kN

- Trial section.

$$W_{pl,y} \geq \frac{\gamma_{M0} M_{Ed}}{f_y} = \frac{1{,}05 \times 247{,}5 \times 10^6}{355} = 732{,}04 \times 10^3 \text{ mm}^3$$

RHS 300 x 200 x 8 is considered as a trial section.

$W_{pl,y} = 775 \times 10^3$ mm³ , $A = 7650$ mm² , $i_y = 112$ mm

$i_z = 82{,}1$ mm

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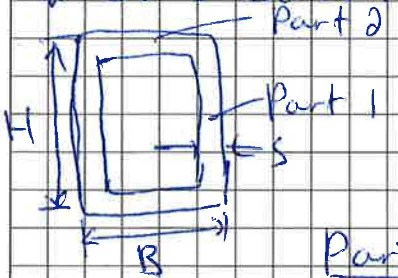
* Class classification (RHS 300 x 200 x 8)



Part ① [Bending]

$$\frac{c_1}{s\varepsilon} = \frac{H-3s}{s\cdot\varepsilon} = 42{,}6 < 72 \quad \text{Class ①}$$

Part 2 [Compression] "Internal"

$$\frac{c_2}{s\varepsilon} = \frac{B-3s}{s\cdot\varepsilon} = 27{,}2 < 33 \quad \text{Class ①}$$

∴ c/s is class ①.

* Design moment resistance

$$M_{c,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{778\times10^3 \times 355}{1{,}05\times10^6} = 262{,}02 \text{ kNm}$$

Check: $M_{c,Rd} > M_{Ed} = 247{,}5$ kNm.

* RHS is a closed c/s; ~~The~~ RHS is ~~not~~ has sufficient resistance to LTB;

$$M_{b,Rd} \approx M_{c,Rd}.$$

~~RHS 300x200x8 is a suitable c/s considering ULS.~~

* Checking for shear resistance

$$V_{c,Rd} = \frac{A_v\, f_y/\sqrt{3}}{\gamma_{M0}}, \quad \text{where } A_v = \frac{A\,h}{b+h} = 4590 \text{ mm}^2$$

$$V_{c,Rd} = \frac{4590 \times \frac{355}{\sqrt{3}}}{1{,}05\times10^3} = 895{,}97 \text{ kN}$$

Check: $V_{c,Rd} > V_{Ed} = 412{,}5$ kN

* Checking for combined effect (M+V)

$V_{Ed} < 0{,}5 \times V_{c,Rd} = 447{,}98$ kN

∴ No combined effect.

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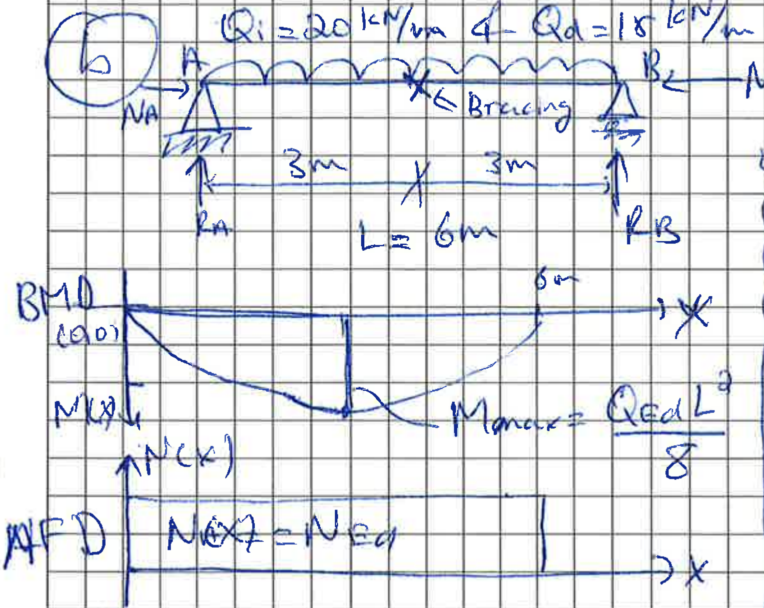
* The longer part of the c/s is in Class 1. Therefore there is sufficient bearing capacity.

∴ RHS 300×200×8 is a suitable c/s considering ULS

(ii) An I section with the same values of $M_{c,Rd}$ & $V_{c,Rd}$ as above designed RHS-section, can withstand the above loading if it ~~has~~ is safe from lateral-torsional buckling, that is if it has a buckling moment resistance ($M_{b,Rd}$) greater than $M_{Ed} = 247,5$ kNm.

(b)



Support reactions

$R_A = R_B = \frac{Q_{Ed} L}{2}$, where

$Q_{Ed} = 1,35 \times 15 + 1,5 \times 20$

$= 50,25$ kN/m

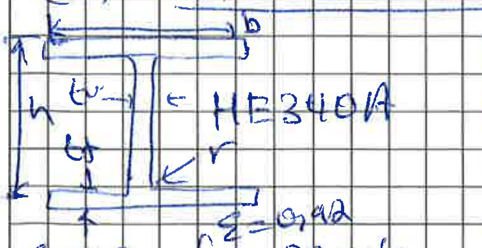
$R_A = R_B = 150,75$ kN

$\xrightarrow{+} \Sigma F = 0$; $N_A = N_{Ed} = 270$ kN

Design loads: $M_{y,Ed} = \frac{Q_{Ed} L^2}{8} = 226,13$ kNm

$V_{Ed} = 150,75$ kN, $N_{Ed} = 270$ kN

(i) Class classification:



S275; $f_y = 275 N/mm^2$

Flange [Compression]

$\frac{c_f}{t_f \cdot \varepsilon} = \frac{0,5(b - t_w - 2r)}{t_f \cdot \varepsilon} = 7,79 < 9$

∴ Class 1

Web [Compression + Bending]

$$\frac{c_w}{t_w \cdot \varepsilon} = \frac{h - 2t_f - 2r}{t_w \cdot \varepsilon} = 27{,}8 < 33$$

Lower bound of Limit of class (1). $\left[33 \le \frac{368}{13\alpha - 1} \le 72\right]$ $(0{,}5 \le \alpha \le 1)$

∴ C/s is in Class (1)

(ii) Check the beam for buckling failure

* First Check for lateral bracings:

$$\bar{\lambda}_f = \frac{k_c L_c}{i_{f,z} \lambda_1} \le \bar{\lambda}_{c0} \cdot \frac{M_{c,Rd}}{M_{y,Ed}}, \quad k_c = 0{,}87 \quad L_c = 3{,}0\,m \quad \bar{\lambda}_{c0} = 0{,}3$$

$$i_{f,z} = \sqrt{\frac{I_{eff,c}}{A_{eff,c} + \frac{1}{3} A_{eff,w,c}}} = 82{,}76\ mm$$

$$\bar{\lambda}_f = \frac{0{,}87 \times 3000}{82{,}76 \times 93{,}9 \times 0{,}92} = 0{,}365$$

$$M_{c,Rd} = \frac{2 S_y f_y}{\gamma_{M0}} = \frac{2 \times 925 \times 10^3 \times 275}{1{,}05 \times 10^6} = 484{,}5\ kNm$$

$$\bar{\lambda}_f = 0{,}365 < 0{,}3 \cdot \frac{484{,}5\ kNm}{226{,}13\ kNm} = 0{,}6428$$

∴ There is sufficient lateral bracings. No LTB. $(\chi_{LT} = 1{,}0)$.

- $N_{Rk} = A f_y = 13{,}3 \times 10^3 \times 275 \times 10^{-3} = 3657{,}5\ kN$
- $M_{y,Rk} = 2 S_y f_y = 508{,}75\ kNm$
- Find χ_y & χ_z:

$$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \times \frac{1}{\lambda_1} = \frac{6000}{144 \times 93{,}9 \times 0{,}92} = 0{,}482 > 0{,}2.$$

$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \times \frac{1}{\lambda_1} = \frac{0,5 \times 6000\,m}{74,6 \times 93,9 \times 0,92} = 0,466 > 0,2$$

Table 6.2; Rolled Section; $\frac{h}{b} = 1,1 < 1,2$, $t_f = 16,5\,mm < 100$

y-y axis "b"	z-z axis "c"
$\alpha_y = 0,34$	$\alpha_z = 0,49$
$\phi_y = 0,664$	$\phi_z = 0,674$
$\chi_y = 0,892$	$\chi_z = 0,861$

Eq 6.61 from NS EN 1993-1-1:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{m1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{m1}}} = \frac{270}{\frac{0,892 \times 3657,5}{1,05}} + 1,025 \frac{226,13}{\frac{508,75}{1,05}}$$

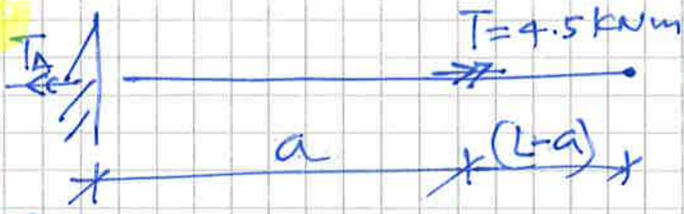
$$= 0,565 < 1,0$$

and

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{m1}}} + k_{zy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{m1}}} = 0,377 < 1,0$$

∴ HE340A is suitable. No buckling failure

Q(4)



IPE 500

S355 steel.

BC'ns

$x=0$

$\left(\begin{array}{l}\phi=0\\ \frac{d\phi}{dx}=0\end{array}\right)$

Assume $GJ_t << EC_w$ .. open c/s.

If $0 \leq x \leq a$; $\quad (T_A = T = 4.5\,kNm)$

$\sum T = 0; \quad T(x) = T_A = T = 4.5\,kNm$

$$-EC_w \frac{d^3\phi}{dx^3} = T$$

$$-EC_w \frac{d^2\phi}{dx^2} = Tx + c_1$$

$$-EC_w \frac{d\phi}{dx} = T\frac{x^2}{2} + c_1 x + c_2$$

$$-EC_w\,\phi(x) = T\frac{x^3}{6} + c_1\frac{x^2}{2} + c_2 x + c_3$$

BC'ns at $x=0$; $\frac{d\phi}{dx}=0$; $c_2 = 0$;

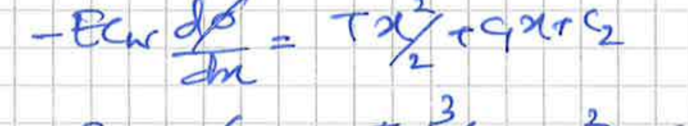
$\phi(x) = 0$; $c_3 = 0$

$$-EC_w \frac{d^2\phi}{dx^2} = Tx + c_1$$

$$-EC_w \frac{d\phi}{dx} = \frac{Tx^2}{2} + c_1 x$$

$$-EC_w\,\phi(x) = \frac{Tx^3}{6} + \frac{c_1 x^2}{2}$$

If $a < x \leq L$;



$\sum T = 0; \quad T(x) = 0$

$$-EC_w \frac{d^3\phi}{dx^3} = 0$$

$$-EC_w \frac{d^2\phi}{dx^2} = c_4$$

$$-EC_w \frac{d\phi}{dx} = c_4 x + c_5$$

$$-EC_w\,\phi(x) = c_4\frac{x^2}{2} + c_5 x + c_6$$

B.C's;

at $x = L$; $\frac{d^2\phi}{dx^2} = 0$; $C_4 = 0$

However; end boundary conditions are not sufficient to solve the eq^ns;

Therefore; Compatibility Condition should be considered

i.e; { at $x = a$ both side; }

(*) $(\sigma_w)_{\text{Left side}} = (\sigma_w)_{\text{Right side}}$

$$aT + C_1 = 0$$

$$C_1 = -aT$$

(*) Warping displacement

$$(u)_{\text{Left side}} = (u)_{\text{Right side}}$$

$$T\left(\frac{a^2}{2}\right) + C_1 a = C_5$$

$$C_5 = -Ta^2/2$$

(*) Twisting angle:

$$(\phi)_{\text{Left side}} = (\phi)_{\text{Right side}}$$

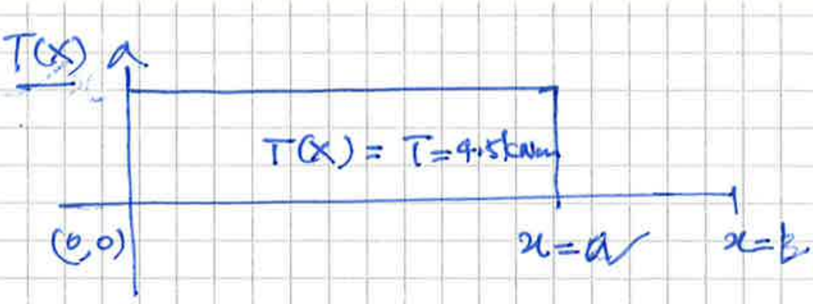
$$T\left(\frac{a^3}{6}\right) + C_1\frac{a^2}{2} = C_5 a + C_6$$

$$C_6 = \frac{Ta^3}{6} - \frac{Ta^3}{2} + \frac{Ta^3}{2} = \frac{Ta^3}{6}$$

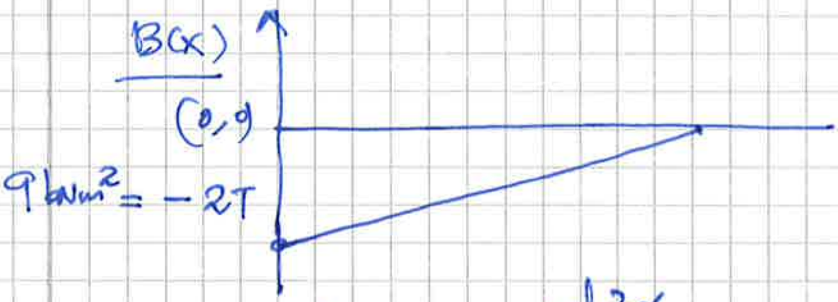
$$-EC_w\frac{d^2\phi}{dx^2} = Tx - aT = T(x-a)$$

$$\frac{d^2\phi}{dx} = \frac{-T}{EC_w}(x-a) \quad — (*)$$

TMD



$$-EC_w \frac{d^2\phi}{dx} = B(x) = T(x-a)$$



$0 \leq x \leq 2$; $\left(\frac{d^3\phi}{dx^3}\right) = \phi_{xxx} = \frac{-T}{EC_w}$

$x=0$; ~~ϕ_w~~ $B(x)_{max}$; $\left(\frac{d^2\phi}{dx^2}\right)_{x=0} = \frac{aT}{EC_w}$

IPE500

$h = 500 - t = 484 \text{ mm}$

$b = 200 \text{ mm}$

$t = t_f = 16 \text{ mm}$

$s = t_w = 10.2 \text{ mm}$

$C_w = 1249 \times 10^9 \text{ mm}^6$

$I_f = \frac{1}{12} b^3 t = \frac{1}{12} \times 200^3 \times 16 = 1.0667 \times 10^7 \text{ mm}^4$

$A_f = bt = 200 \times 16 = 3200 \text{ mm}^2$

$M_f = E I_f \frac{h}{2} \phi_{xx}$; $\sigma_w = \frac{M_f b}{2 I_f} = E \frac{h}{2} \phi_{xx} \frac{b}{2}$

$$\sigma_{w,max} = E \frac{h b}{4} (\phi_{xx})_{max}$$

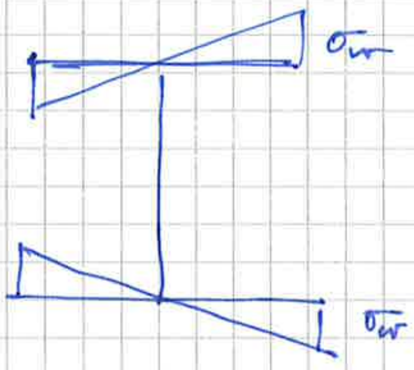
$$= \frac{Ehb}{4}\left[\frac{aT}{EC_w}\right] = \frac{aThb}{4C_w} = \frac{aThb}{4C_w}$$

$$= \frac{4.5 \times 10^6 \times 484 \times 200 \times 2000}{4 \times 1249 \times 10^9} = 174.4 \text{ N/mm}^2$$

$$V_f = EI_f \frac{h}{2} \cdot \frac{3}{2A_f} (\phi_{xxx})_{max} = EI_f \frac{h}{2} \cdot \frac{3}{2A_f} \cdot \frac{T}{EC_w}$$

$$\tau_{w,max} = 1.0662 \times 10^{7} \times \frac{484}{2} \times \frac{3}{2 \times 3200} \times \frac{4.5 \times 10^{6}}{1249 \times 10^{9}}$$

$$\tau_{w,max} = \underline{\underline{4.36}}\ N/mm^2$$



at $x = 0$; $x = L$

(ii). VLS check

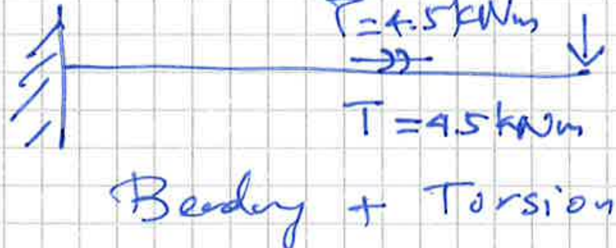
$$\sigma_{w,max} = 174.4\ N/mm^2 < \frac{f_y}{\gamma_{m0}} = \frac{355}{1.05} = 338\ N/mm^2$$

Ok//

$$\tau_{w,max} = 4.35\ N/mm^2 < \tau_y/\gamma_{m0} = f_y/\sqrt{3}\gamma_{m0} = 195\ N/mm^2$$

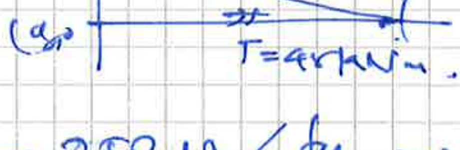
Ok//

(iii).



Bending + Torsion; $\sigma_b = \frac{M}{I} \cdot z_{max}$

$$\sigma_b \text{ (at } x=0) = \frac{50 \times 3 \times 10^{6}}{482 \times 10^{6}} \times 250 = 77.8\ N/mm^2$$



$$\sigma_{b,max} + \sigma_{w,max} = (77.8 + 174.4) = 252.18 < \frac{f_y}{\gamma_{m0}} = 338\ N/mm^2$$

Ok//.

Member is Suitable.

Skriv her. Bruk kulepenn. Ved feilskrift - bruk overstrykning. *Write here. Use a ballpoint pen. Cross out any mistakes.*

(b) (i) Fatigue life of the tension plates

* $\Delta\sigma_E = \frac{F_{max} - F_{min}}{A} = \frac{(40+30)\times 10^3}{1000 \times 10} = ~~140~~ \; 70 \; N/mm^2$

$\Delta\sigma_R = \gamma_{Ff}\,\gamma_{Mf}\,\Delta\sigma_E = 1,0 \times 1,15 \times ~~140~~ \; 70 = ~~161~~ \; 80.5 \; N/mm^2$

* Fatigue Curve

Detail category of $\Delta\sigma_C = 80 \; N/mm^2$ (From Table 8.5)

$\Delta\sigma_D = 0,737\,\Delta\sigma_C = 58,96 \; N/mm^2$

$\Delta\sigma_R > \Delta\sigma_D$ ∴ Finite fatigue life

$N_R = \left(\frac{\Delta\sigma_C}{\Delta\sigma_R}\right)^3 \times 2\times 10^6 = ~~245.33~~ \; 981.482 \times 10^3$ cycles

* Fatigue life in years $= \frac{N_R}{365 \times 40} = ~~16.8 \text{ years}~~$ 67.2 Years

(ii) ~~Bolts~~ Fillet welds: Detail category: $\Delta\sigma_C = 36^*$ (Table 8.5)

Bolts: Detail category: $\Delta\sigma_C = 50$ (Table 8.1)

(iii) (1) Reduse the maximum force (stress)

(2) Increase the dimension of the tension plates which reduses stress range.

(3). Increase the dimensions of the fillet weld.