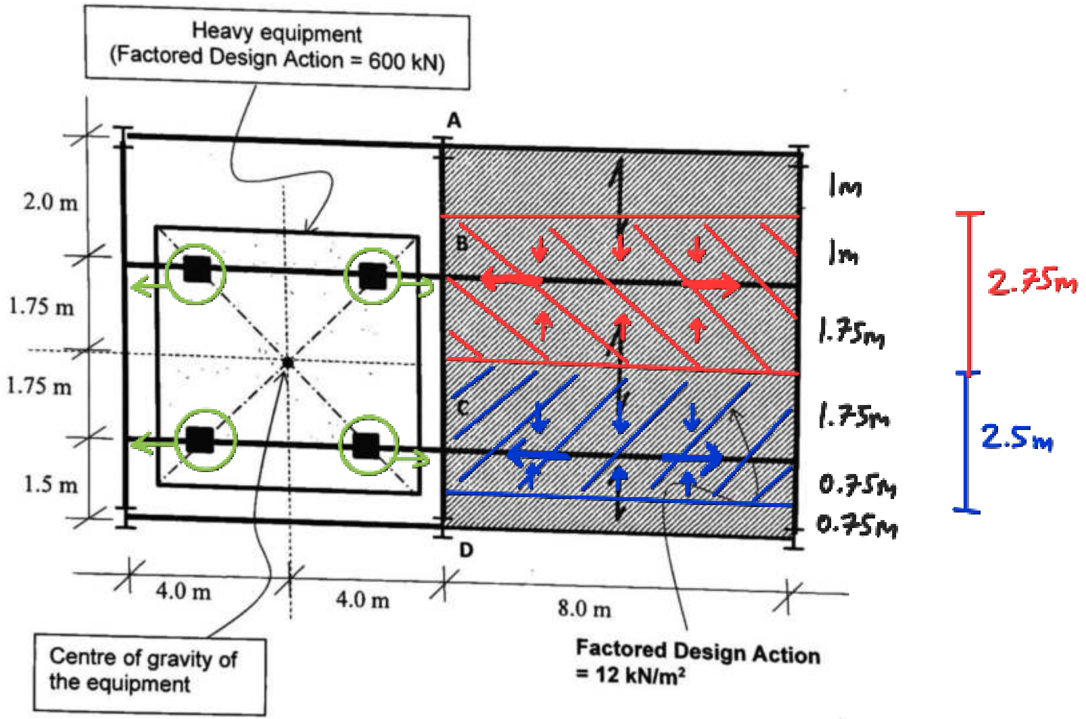
CV3012 2021/22 S2
by Renardi M.
@pardirenardi

1. Figure Q1 shows a steel platform supporting an equipment and a working platform. The equipment is sitting on 4 number of stumps. When fully loaded, total factored design action for the equipment is estimated at 600 kN, and factored design action for working platform is given as 12 kN/m². You can use table and data provided in the Appendix to assist your calculations.

(a) Ignoring self-weight of the structural framing, calculate and draw the design shear force and bending moment diagram for beam ABCD.

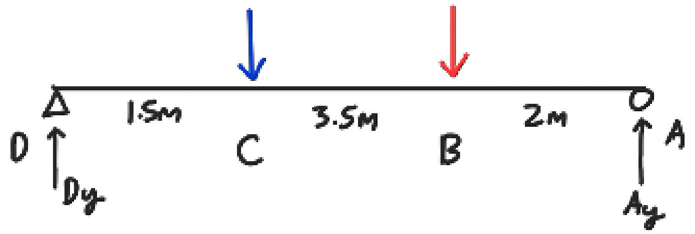


PLAN VIEW

(a)

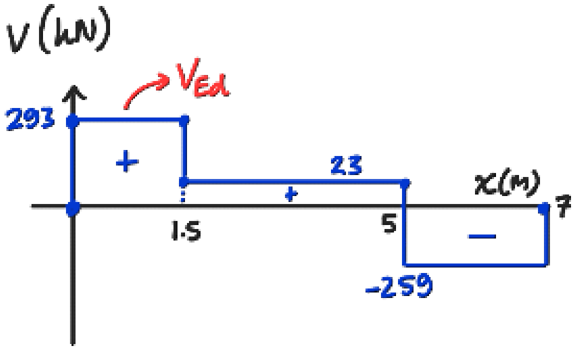
$$\frac{2.5 \times 8 \times 12}{2} + \frac{600}{4} = 270 \text{ kN}$$

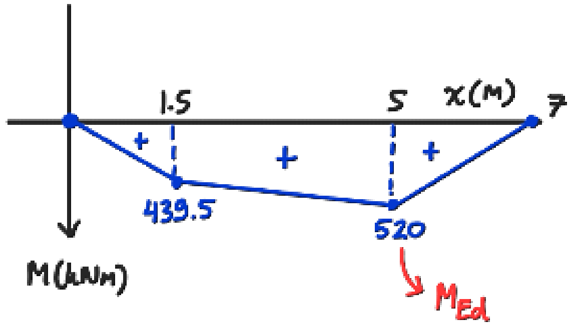
$$\frac{2.75 \times 8 \times 12}{2} + \frac{600}{4} = 282 \text{ kN}$$



$$\circlearrowleft + \Sigma M_0 = -270(1.5) - 282(5) + A_y(7) = 0$$
$$\Rightarrow A_y = 259 \text{ kN}$$

$$+\uparrow \Sigma F_y = D_y - 270 - 282 + A_y = 0$$
$$\Rightarrow D_y = 293 \text{ kN}$$





For LTB just need to check span CB.

(b) From UB sections and other information listed in the Appendix, assuming steel grade of S355, choose one section to perform design check for the Lateral Torsional Buckling (LTB) resistance in accordance with EC3. ~~Other design checks are not required.~~ You can assume that lateral restraints are provided at point A, B, C & D.

* This PYP solution will demonstrate all the checks.

(10 Marks)

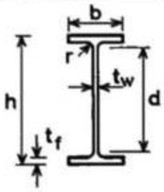
(c) Regardless of the design check results in Q1(b) above, assuming design check on LTB failed by a margin of 10%, propose two possible solutions to improve the LTB resistance for beam ABCD, and provide sketches for your proposed solutions. No calculation is required for the proposed solutions. Assumed that the platform columns, main beams and secondary beams have been erected, but the equipment and platform deck are not yet installed.

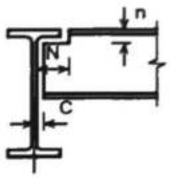
Section Properties Table for Q1 and Q2

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance® UKB





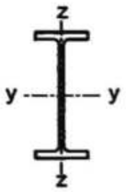
Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area	
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne
		h	b	t_w	t_f	r	d	c_f / t_f	c_w / t_w	C	N	n		
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2
610x178x82 +	81.8	598.6	177.9	10.0	12.8	12.7	547.6	5.57	54.8	7	94	26	1.87	22.9
533x210x82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	6.58	49.6	7	110	26	1.85	22.5
457x191x82	82.0	460.0	191.3	9.9	16.0	10.2	407.6	5.03	41.2	7	102	28	1.65	20.1

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance® UKB



Properties

Section Designation	Second Moment of Area		Radius of Gyration		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	U	X	I_w	I_T	A
	cm^4	cm^4	cm	cm	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
610x178x82 +	55900	1210	23.2	3.40	1870	136	2190	218	0.843	48.5	1.04	48.8	104
533x210x82	47500	2010	21.3	4.38	1800	192	2060	300	0.863	41.6	1.33	51.5	105
457x191x82	37100	1870	18.8	4.23	1610	196	1830	304	0.879	30.8	0.922	69.2	104

(b) $f_y = 355 \text{ N/mm}^2 \quad V_{Ed} = 293 \text{ kN} \quad M_{Ed} = 520 \text{ kNm}$

Choosing the beam...

$$\frac{M_{Ed}}{f_y} = \frac{520}{355} \times 10^3 \text{ cm}^3 = 1465 \text{ cm}^3$$

Any beam out of the three selections may be used as their $W_{pl,y} > 1465 \text{ cm}^3$. For this example, UB 533 × 210 × 82 will be used.

$h = 528.3$ mm
$b = 208.8$ mm
$t_w = 9.6$ mm
$t_f = 13.2$ mm
$r = 12.7$ mm

$c_f/t_f = 6.58$
$c_w/t_w = 49.6$
$I_y = 47500 \text{ cm}^4$
$I_z = 2010 \text{ cm}^4$
$W_{pl,y} = 2060 \text{ cm}^3$
$W_{pl,z} = 300 \text{ cm}^3$

$I_w = 1.33 \text{ dm}^6$
$I_T = 51.5 \text{ cm}^4$
$A = 105 \text{ cm}^2$

Cross - section Classification

$$\varepsilon = \sqrt{\frac{235}{355}} = 0.81$$

Flange : (pure compression) $\quad \frac{c_f}{t_f} = 6.58 < 9\varepsilon = 7.29 \Rightarrow$ Flange is Class 1

Web : (pure bending) $\quad \frac{c_w}{t_w} = 49.6 < 72\varepsilon = 58.32 \Rightarrow$ Web is Class 1

∴ Section is overall Class 1

Shear Buckling Check

$$\frac{h_w}{t_w} = \frac{528.3 - 2(13.2)}{9.6} = 52.3 < 72\frac{\varepsilon}{\eta} = 72\frac{(0.81)}{1.0} = 58.3$$ ✓ok!

Plain Shear Check

$$A_v = 105 \times 10^2 - 2(208.8)(13.2) + (9.6 + 2(12.7))(13.2)$$
$$= 5450 \text{ mm}^2 \geq \eta h_w t_w = (1.0)[528.3 - 2(13.2)](9.6) = 4818 \text{ mm}^2$$

$$V_{pl,Rd} = \frac{(5450 \times 10^{-6})\left(\frac{355 \times 10^6}{\sqrt{3}}\right)}{1.0} \times 10^{-3} \text{ kN} = 1117 \text{ kN} > V_{Ed} = 293 \text{ kN}$$

✓ok!

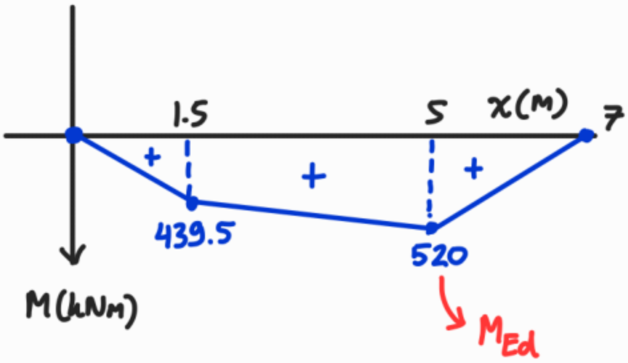
Bending Resistance Check

$V_{Ed} = 293 \text{ kN} < 0.5 V_{pl,Rd} = 559 \text{ kN} \Rightarrow$ Low Shear Case

$$\text{Class 1} \longrightarrow M_{pl,Rd} = \frac{(2060 \times 10^{-6}) \times (355 \times 10^{6})}{1.0} \times 10^{-3} \text{ kNm}$$

$$= 731 \text{ kNm} > M_{Ed} = 520 \text{ kNm} \quad \checkmark \text{ok!}$$

LTB Check



For LTB just need to check span CB.

① Longest unrestrained span

② Biggest M

- $L = 3.5$ m
- Finding C_1:

$$\psi = \frac{439.5}{520} = 0.845$$

Note:
ψ must be $-1 \leq \psi \leq 1$.
That's why it's $\frac{439.5}{520}$ not $\frac{520}{439.5}$

$$C_1 = 1.75 - 1.05(0.845) + 0.3(0.845)^2$$
$$= 1.077 \leq 2.3$$

$h = 528.3$ mm
$b = 208.8$ mm
$t_w = 9.6$ mm
$t_f = 13.2$ mm
$r = 12.7$ mm

$c_f/t_f = 6.58$
$c_w/t_w = 49.6$
$I_y = 47500 \text{ cm}^4$
$I_z = 2010 \text{ cm}^4$
$W_{pl,y} = 2060 \text{ cm}^3$
$W_{pl,z} = 300 \text{ cm}^3$

$I_w = 1.33 \text{ dm}^6$
$I_T = 51.5 \text{ cm}^4$
$A = 105 \text{ cm}^2$

- $$M_{cr} = (1.077)\frac{\pi^2(210 \times 10^{9})(2010 \times \cancel{10^{-8}})}{(3.5)^2}\sqrt{\frac{1.33 \times 10^{-6}}{2010 \times 10^{-8}} + \frac{(3.5)^2(81 \times \cancel{10^{9}})(51.5 \times \cancel{10^{-8}})}{\pi^2(210 \times \cancel{10^{9}})(2010 \times \cancel{10^{-8}})}} \times 10^{-3} \text{ kNm}$$

$$= 1025.8 \text{ kNm}$$

- $\bar{\lambda}_{LT} = \sqrt{\dfrac{(2060 \times 10^{-6})(355 \times 10^{6})}{1025.8 \times 10^{3}}} = 0.844$

- $\dfrac{h}{b} = \dfrac{528.3}{208.8} = 2.53 > 2 \Rightarrow \text{Curve } ⓑ \Rightarrow \alpha_{LT} = 0.34$

- $\phi_{LT} = 0.5\left[1 + 0.34(0.844 - 0.2) + 0.844^{2}\right] = 0.966$

- $\chi_{LT} = \dfrac{1}{0.966 + \sqrt{0.966^{2} - 0.844^{2}}} = 0.696$

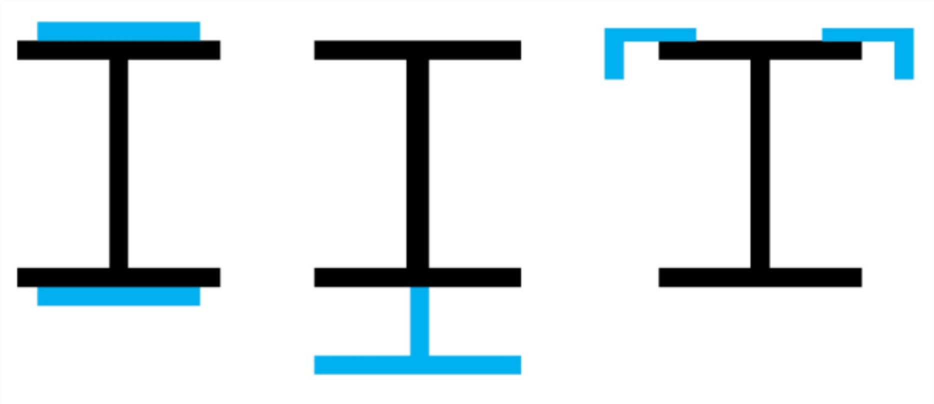
- $M_{b,Rd} = 0.696 \dfrac{(2060 \times 10^{-6})(355 \times 10^{6})}{1.0} \times 10^{-3}$

  $= 509 \text{ kNm} < M_{Ed} = 520 \text{ kNm}$ ✗ NOT OK !

Deflection Check

Can't do deflection check as only ULS design load was given (no SLS load specified).

(c)



(increase the second moment of area, I_y)

CV3012 21-22 Sem 2 #2 (Web buckling)

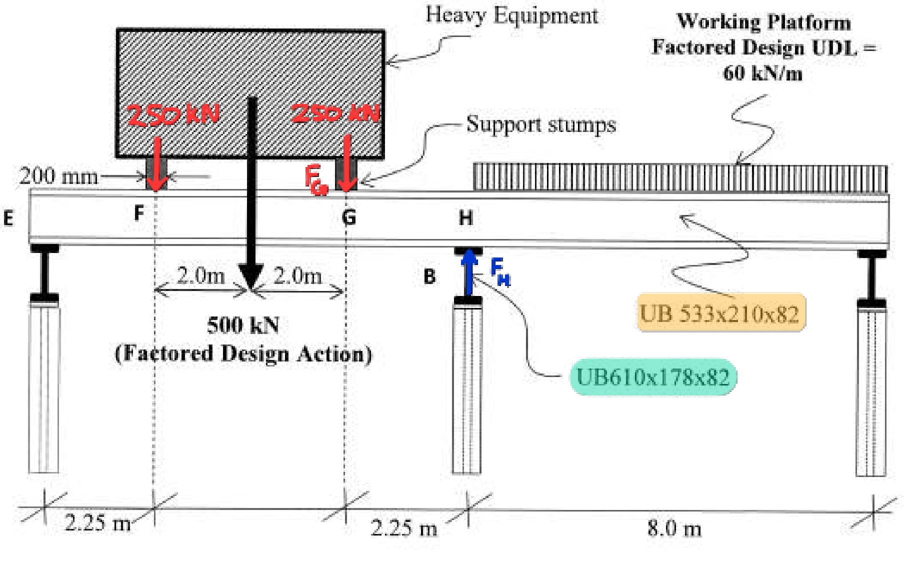
2. Figure Q2 shows a cross section of another equipment support platform. Stumps supporting the equipment has a cross section of 200 mm x 200 mm beam sizes and factored design actions are shown in Figure Q2. Perform the following design checks.

   (a) Using steel grade of S355, check the adequacy of the beam web resistance to transverse force at **locations G & H** in accordance with EN1993-1-5 guidelines. Verification for combined resistance is not required.

   (22 Marks)

   (b) Regardless of the design check result in Q2(a), suggest a practical solution to reduce the risk of web bearing and buckling failure in Beam EFGH. Provide a sketch of your suggested solution. Calculation is not required.

   (3 Marks)



Cross section of an equipment support platform

Figure Q2

② (a) Point G

- $F_{Ed,G} = \frac{500}{2} = 250$ kN , $f_y = 355$ N/mm²

- Type (a), no web stiffeners → $a = \infty$, $\frac{h_w}{a} = 0$

  $k_F = 6 + 2(0)^2 = 6$

- $F_{cr} = 0.9(6)(210\,000)\frac{(9.6)^3}{528.3-2(13.2)} \times 10^{-3}$ kN $= 1999$ kN

- $m_1 = \frac{\cancel{355} \times 208.8}{\cancel{355} \times 9.6} = 21.75$

  $m_2 = 0.02\left(\frac{528.3-2(13.2)}{13.2}\right)^2 = 28.91$

- $l_y = 200 + 2(13.2)\left(1 + \sqrt{21.75 + 28.91}\right) = 414.30$ mm

  ↳ S_s is based on force's point of contact

- $\bar{\lambda}_F = \sqrt{\dfrac{(414.3)(9.6)(355)}{1999 \times 10^3}} = 0.84 > 0.5$ (m_2 ok!)

- $\chi_F = \dfrac{0.5}{0.84} = 0.595 \leq 1.0$

- $L_{eff} = 0.595 \times 414.30 = 246.6$ mm

- $F_{Rd} = \dfrac{355 \times 246.6 \times 9.6}{1.0} \times 10^{-3}$ kN $= 840.4$ kN //

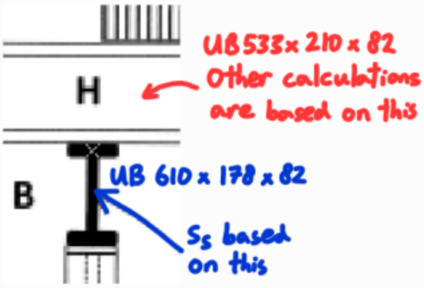
- $\eta_2 = \dfrac{F_{Ed}}{F_{Rd}} = \dfrac{250}{840.4} = 0.297 < 1.0$ ok!

Point H

Assuming that the beam is simply supported (discontinuous),

- $F_{Ed,H} = \dfrac{500}{2} + \dfrac{60 \times 8}{2} = 490$ kN ; $f_y = 355$ N/mm²

S_s is based on the secondary beam (UB 610 × 178 × 82), which is the point of contact of the force.



- $S_s = 10.0 + 1.6(12.7) + 2(12.8) = 55.92$ mm

- Type (a), no web stiffeners → $a = \infty$, $\dfrac{h_w}{a} = 0$

  $k_F = 6 + 2(0)^2 = 6$

- $F_{cr} = 0.9(6)(210\,000)\dfrac{(9.6)^3}{528.3 - 2(13.2)} \times 10^{-3}$ kN $= 1999$ kN

- $m_1 = \dfrac{\cancel{355} \times 208.8}{\cancel{355} \times 9.6} = 21.75$

  $m_2 = 0.02\left(\dfrac{528.3 - 2(13.2)}{13.2}\right)^2 = 28.91$

- $\ell_y = 55.92 + 2(13.2)\left(1 + \sqrt{21.75 + 28.91}\right) = 270.2 \text{ mm}$

- $\bar{\lambda}_F = \sqrt{\dfrac{(270.2)(9.6)(355)}{1999 \times 10^3}} = 0.679 > 0.5$ $\quad$ (M_2 ok!)

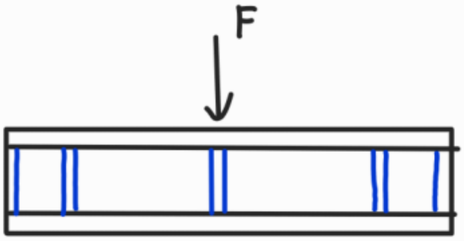
- $\chi_F = \dfrac{0.5}{0.679} = 0.736 \leq 1.0$

- $L_{eff} = 0.736 \times 270.2 = 198.9 \text{ mm}$

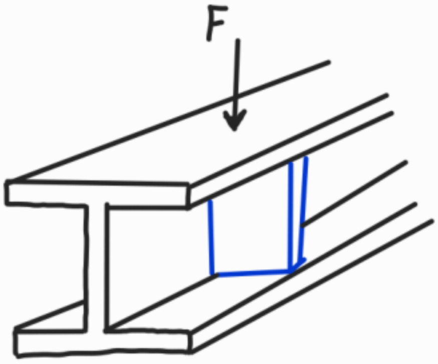
- $F_{Rd} = \dfrac{355 \times 198.9 \times 9.6}{1.0} \times 10^{-3} \text{ kN} = 677.9 \text{ kN}$ //

- $\eta_2 = \dfrac{F_{Ed}}{F_{Rd}} = \dfrac{490}{677.9} = 0.723 < 1.0$ $\quad$ ok!

(c)



(add stiffeners)



3. Figure Q3(a) shows a typical section of a 3-storey office building; it is to be designed as a simple, non-sway braced frame in Grade S275 steel. A factored compression action of 1,900 kN is acting at the centroid of a 254 x 254 x 89 kg/m UC column marked 'X'. The primary and secondary beams factored design reactions and their positions with respect to the column is given in Figure Q3(b).

(a) List three advantages of using "simple frame construction" in designing steel structures, and calculate the total compression action and nominal moments carried by this column 'X'.

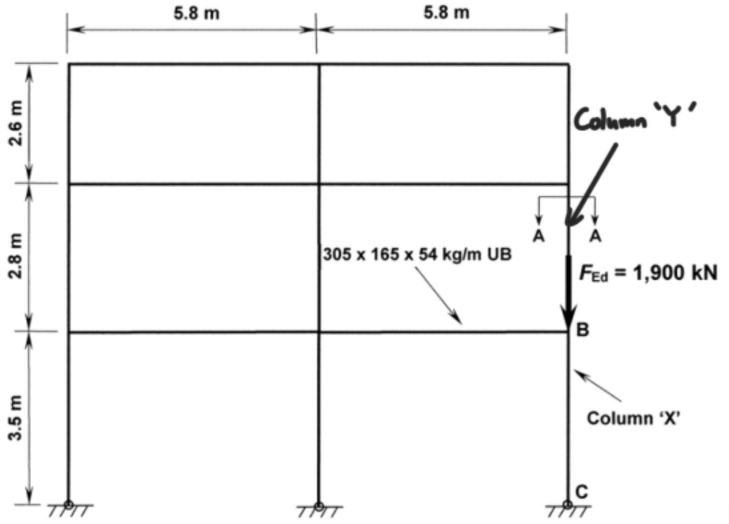
(9 Marks)

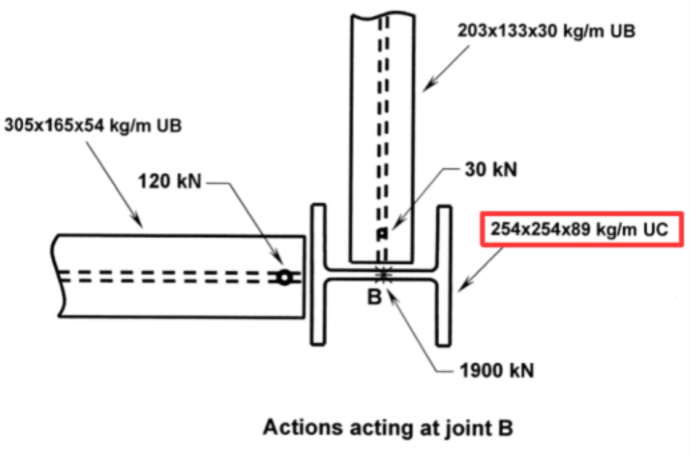
(b) Show that the proposed 254 x 254 x 89 kg/m UC section is adequate. The classification of the section is Class 1. When considering column buckling, the effective length factor may be taken as 1.0.

(12 Marks)

(c) If the joint at C is replaced with a fixed joint, without any further calculations, explain why the column is still adequate to resist the total factored design actions.

(4 Marks)





Actions acting at joint B

Section A-A

Figure Q3(a)

③ (a) Pros of simple frame constructions

1. Pin joints are simpler to construct than rigid/fixed joints
2. Easier to analyze (pin joints only have 2 unknowns; fixed joints have 3, increasing the deg. of indeterminacy)
3. Allows gaps in welding/connection
4. Simplifies the structure into simply supported

- $N_{Ed} = 1900 + 120 + 30 = 2050$ kN ✓

Stiffness ratio between column 'X' and column 'Y':

$$\text{ratio} = \frac{\frac{\cancel{EI}}{2.8}}{\frac{\cancel{EI}}{3.5}} = 1.25 < 1.5$$

∴ Moment is divided equally between column 'x' and 'Y'.

- $M_{Ed,y} = \frac{1}{2} \times 120 \times e_y = \frac{1}{2} \times 120 \times \left(100 + \frac{260.3}{2}\right) \times 10^{-3}\ kNm = 13.81\ kNm$ //

- $M_{Ed,z} = \frac{1}{2} \times 30 \times e_z = \frac{1}{2} \times 30 \times \left(100 + \frac{10.3}{2}\right) \times 10^{-3}\ kNm = 1.58\ kNm$ //

(b) Buckling Check

Check z-z axis as it is more critical than y-y.
(Same $L_z = L_y$ but $I_z < I_y$)

- $L_{cr} = \underset{\text{(pin - pin)}}{\underline{1.0}} \times 3.5 = 3.5\ m$

- $\lambda_1 = 93.9 \times \sqrt{\frac{235}{275}} = 86.8$

- $\bar{\lambda} = \frac{3.5}{6.55 \times 10^{-2}} \times \frac{1}{86.8} = 0.616 > 0.2$; need check buckling

- $\Rightarrow \frac{h}{b} = \frac{260.3}{256.3} = 1.02 \leq 1.2$
  $\Rightarrow t_f = 17.3\ mm \leq 100\ mm$
  $\Rightarrow$ Buckling about z-z , S275
  } Curve ⓒ, $\alpha = 0.49$

- $\phi = 0.5\left[1 + 0.49\,(0.616 - 0.2) + 0.616^2\right] = 0.792$

- $\chi = \frac{1}{0.792 + \sqrt{0.792^2 - 0.616^2}} = 0.775$

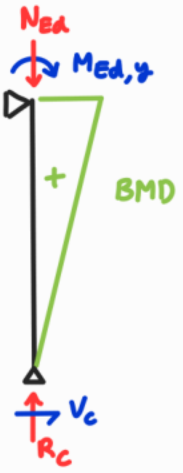
- $N_{b,Rd} = 0.775 \times \frac{(113 \times 10^{-4})(275 \times 10^6)}{1.0} \times 10^{-3}\ kN$

  $= 2408\ kN > N_{Ed} = 2050\ kN$ // OK!

LTB Check

- $L_{FG} = 3.5\,m$ (Lateral restraints at B & C)
- $\psi = 0 \Rightarrow C_1 = 1.77 \longrightarrow$

(pin-pin)



- $$M_{cr} = 1.77 \times \frac{\pi^2 (210 \times \cancel{10^9})(4860 \times \cancel{10^{-8}})}{(3.5)^2} \times \sqrt{\frac{0.717 \times 10^{\cancel{-6}\,2}}{4860 \times \cancel{10^{-8}}} + \frac{(3.5)^2 (81 \times \cancel{10^9})(102 \times \cancel{10^{-8}})}{\pi^2 (210 \times \cancel{10^9})(4860 \times \cancel{10^{-8}})}} \times 10^{-3}\ kNm$$

$$= 2292\ kNm$$

- $\bar{\lambda}_{LT} = \sqrt{\dfrac{1220 \times 275}{2292 \times 10^3}} = 0.383 > 0.2$, LTB check needed
- $\dfrac{h}{b} = \dfrac{260.3}{256.3} = 1.02 < 2$, rolled $\Rightarrow$ ⓐ $\Rightarrow \alpha_{LT} = 0.21$
- $\phi_{LT} = 0.5 \left[1 + 0.21 (0.383 - 0.2) + 0.383^2 \right] = 0.593$
- $\chi_{LT} = \dfrac{1}{0.593 + \sqrt{0.593^2 - 0.383^2}} = 0.956 < 1.0$
- $M_{b,Rd} = 0.956 \times \dfrac{1220 \times 275}{1.0} \times 10^{-3}\ kNm$

$$= 320.7\ kNm > M_{Ed,y} = 13.81\ kNm \quad // \quad \text{OK!}$$

Moment Resistance Check (z-z)

$M_{pl,Rd,z} = \dfrac{575 \times 275}{1.0} \times 10^{-3}\ kNm = 158.1\ kNm > M_{Ed,z} = 1.58\ kNm$ // OK!

Combined Check

$$\frac{2050}{2408} + 1.0\,\frac{13.81}{320.7} + 1.5\,\frac{1.58}{158.1} = 0.91 < 1.0 \quad // \quad \text{OK!}$$

(c) Reduces buckling length (length factor = 0.85 instead of 1.0), hence column 'X' is less susceptible to buckling.

CV3012 21-22 Sem 2 #4 (Bolted & Welded connection, Tension members)

4. Figure Q4 shows a 114.3 x 114.3 x 6.3 circular hollow section (CHS) welded to a 12 mm gusset plate using a 6 mm fillet welds, and in turn it is bolted to another 12 mm thick plate using 2 numbers of M24 Class 8.8 non-preloaded bolts in Grade S275 steel. The plates are also made from Grade S275 steel with the connection subjected to a factored design tension action of 260 kN.

(a) Show that the 2-bolt group is sufficient in terms of shear and bearing resistances.

(10 Marks)

(b) Determine the tension resistance of the circular hollow section (CHS) and gusset plate members.

(7 Marks)

(c) Check the adequacy of the 6 mm fillet welds. Where is the weakest part of the bolted connection?

(8 Marks)

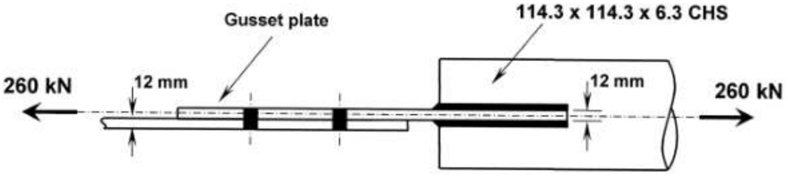
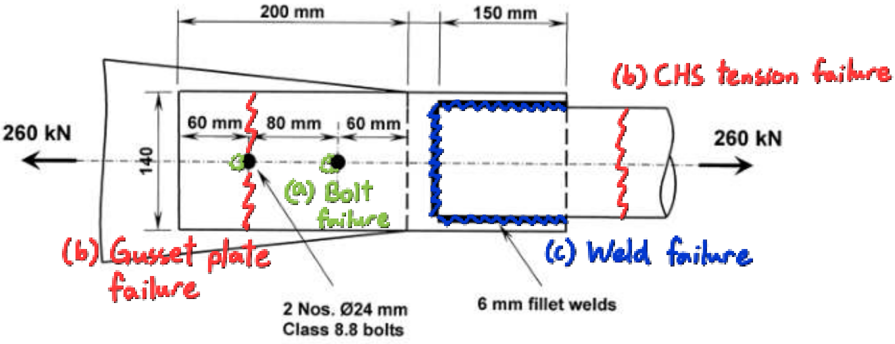


Figure Q4

(Note: drawings are not drawn to scale)

(All dimensions are in mm unless otherwise stated)



④ (a) Bolt:

Shear Resistance

2 No. Ø24 mm Class 8.8 (S275)

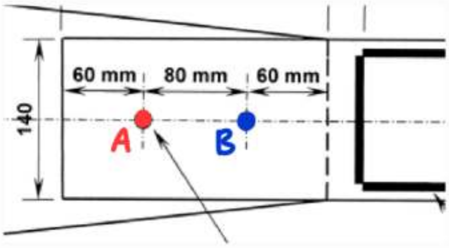
From section properties table, $F_{v,Rd} = 136$ kN (single shear)

$$\Sigma F_{v,Rd} = \underbrace{2}_{\text{2 bolts}} \times \underbrace{1}_{\text{single shear}} \times 136 = 272 \text{ kN} > F_{Ed} = 260 \text{ kN}$$

ok!

Bearing Resistance

- $\frac{\overset{\text{bolt}}{f_{ub}}}{\underset{\text{plate}}{f_u}} = \frac{800}{430} = 1.86$

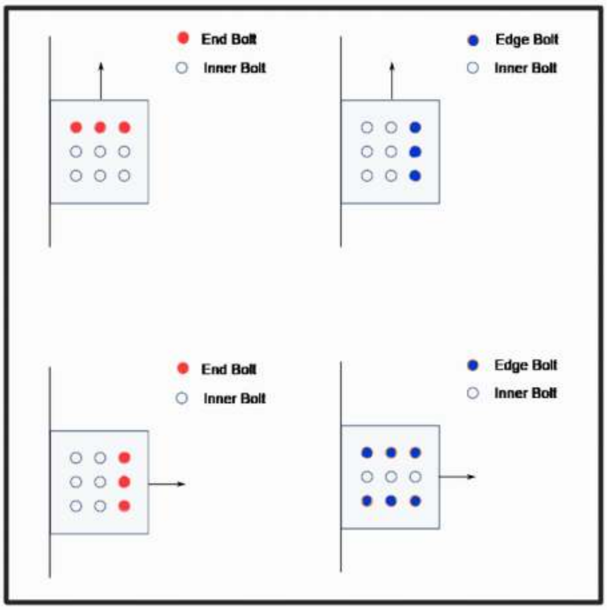
- 

$\parallel \quad \perp$

A is (end, edge) bolt

B is (inner, edge) bolt

For reference :



Note : d_o → hole diameter

d → bolt diameter

- Bolt A

$$\alpha_d = \frac{60}{3(24+2)} = 0.769$$

$$\alpha_b = \min\{0.769, 1.86, 1.0\}$$
$$= 0.769$$

$$k_1 = 2.8\left(\frac{70}{24+2}\right) - 1.7 = 5.84 > 2.5$$
$$\Rightarrow \text{use } k_1 = 2.5$$

- Bolt B

$$\alpha_d = \frac{80}{3(24+2)} - \frac{1}{4} = 0.776$$

$$\alpha_b = \min\{0.776, 1.86, 1.0\}$$
$$= 0.776$$

$$k_1 = 2.8\left(\frac{70}{24+2}\right) - 1.7 = 5.84 > 2.5$$
$$\Rightarrow \text{use } k_1 = 2.5$$

- $F_{b,Rd,A} = \dfrac{2.5(0.769)(430)(24)(12)}{1.25} \times 10^{-3}\ kN = 190.5\ kN$

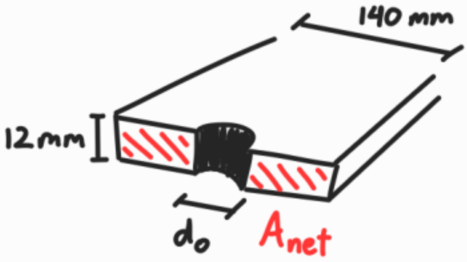
- $F_{b,Rd,B} = \dfrac{2.5(0.776)(430)(24)(12)}{1.25} \times 10^{-3}\ kN = 192.2\ kN$

Bolt Group Resistance

$$\Sigma F_{b,Rd} = 190.5 + 192.2 = 382.7\ kN > F_{Ed} = 260\ kN \quad //$$

OK!

(b) Tension Resistance : Gusset Plate



- $f_y = 275 \text{ N/mm}^2$; $f_u = 430 \text{ N/mm}^2$
- $A_{net} = (140 \times 12) - (24+2)(12) = 1368 \text{ mm}^2$
- $N_{pl,Rd} = \dfrac{(140 \times 12)(275)}{1.0} \times 10^{-3} \text{ kN} = 462 \text{ kN}$
- $N_{u,Rd} = \dfrac{0.9(1368)(430)}{1.25} \times 10^{-3} \text{ kN} = 424 \text{ kN}$
- $N_{t,Rd} = \min\{462, 424\} = 424 \text{ kN} > N_{Ed} = 260 \text{ kN}$ // OK!

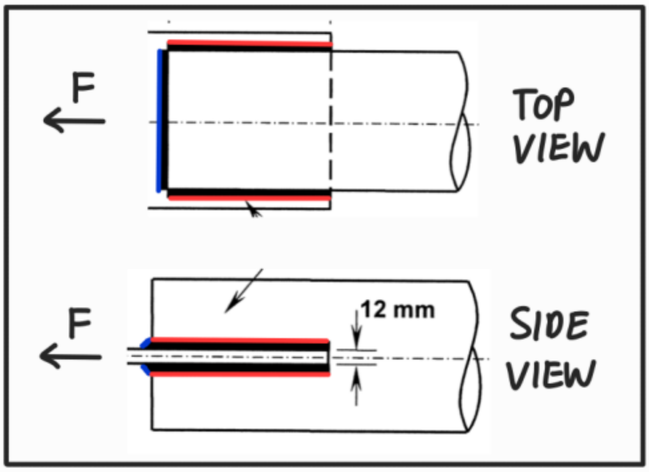
Tension Resistance : CHS

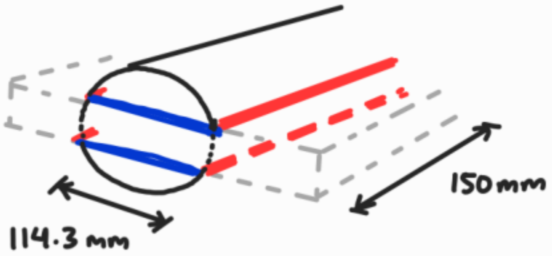
- $f_y = 275 \text{ N/mm}^2$; $f_u = 430 \text{ N/mm}^2$
- $A = 21.4 \text{ cm}^2$
- $A_{net} = A = 21.4 \text{ cm}^2$
- $N_{pl,Rd} = \dfrac{(21.4 \times 10^2)(275)}{1.0} \times 10^{-3} \text{ kN} = 589 \text{ kN}$
- $N_{u,Rd} = \dfrac{0.9(21.4 \times 10^2)(430)}{1.25} \times 10^{-3} \text{ kN} = 662.5 \text{ kN}$
- $N_{t,Rd} = \min\{589, 663\} = 589 \text{ kN} > N_{Ed} = 260 \text{ kN}$ // OK!

(c) Weld : Directional Method

From Section Properties Table, S275 weld of leg length 6 mm has :

$F_{w,L,Rd} = 0.94 \text{ kN/mm}$ $\quad F_{w,T,Rd} = 1.15 \text{ kN/mm}$



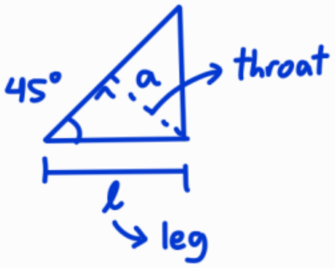


$$F_{w,Rd} = (2 \times 114.3 \times 1.15) + (4 \times 150 \times 0.94)$$
$$= 827 \text{ kN} > F_{Ed} = 260 \text{ kN} \text{ // OK!}$$

Weld : Simplified Method

- $f_u = 430\ N/mm^2$ (based on welded member i.e., CHS & plate)
- $\beta_w = 0.85$ for S275 fillet weld
- $$f_{vw,d} = \frac{430/\sqrt{3}}{0.85 \times 1.25} = 233.7\ N/mm^2$$
- $$f_{w,Rd} = f_{vw,d} \times a = 233.7 \times 6 \sin 45^\circ \times 10^{-3}\ kN/mm$$
  $$= 0.992\ kN/mm$$

  throat thickness



- $$F_{w,Rd} = 0.992 \times (150 + 150 + 114.3) \times 2$$
  $$= 822\ kN > F_{Ed} = 260\ kN$$ OK!

∴ Bolt is the weakest link in the overall connection (lowest F_{Rd})