CV3012 20-21 Sem 2 (Renardi M. @pardirenardi) glhf :)

1. Figure Q1(a) in the next page shows the footprint of a heavy equipment, supported by steel frame structure. Maximum design load from the equipment is given as 1200 kN, the load from equipment is assumed to distribute evenly over the footprint.

   (a) Ignoring self-weight of the steel framing, calculate point loads acting on points B, C and D, as shown in Figure Q1(b). Draw the shear force and bending moment diagrams acting on beam ABCDE. State your assumptions, if any.

   (6 Marks)

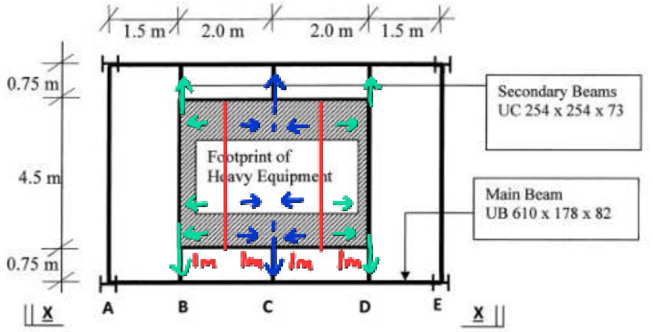
   (b) Using steel grade of S355, check the adequacy of Beam ABCDE with regard to its web bearing and buckling resistance in accordance with EN1993-1-5 guidelines.

   (16 Marks)

   (c) Regardless of the results check in Q1(b), suggest a practical solution to reduce the risk of web bearing and buckling failure in the Beam ABCDE. Provide a sketch of your suggested solution. No calculation is required.

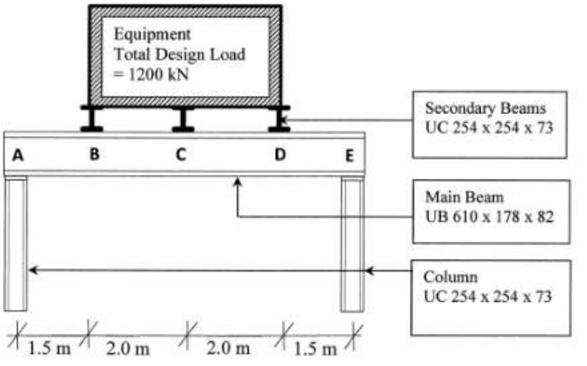
   (3 Marks)

Note:
design → factored
characteristic → unfactored



Plan View of Equipment Support Frame

Figure Q1(a)



Section X-X

Figure Q1(b)

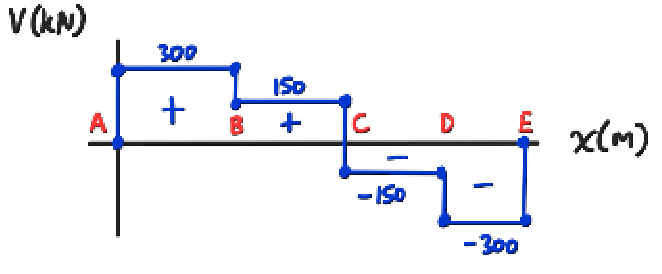
(Note: drawings are not drawn to scale)

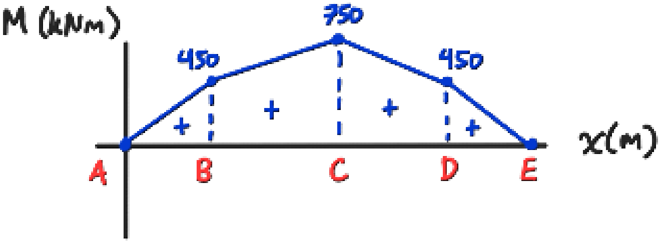
1. (a) • Assume all beams are simply supported.
   • Assume load by the heavy equipment is a UDL with one-way load distribution.

Area load by equipment, $n = \frac{1200 \text{ kN}}{4 \times 4.5 \text{ m}^2} = 66.67 \text{ kN/m}^2$

Point load at B = Point load at D = $\frac{1 \times 4.5 \times 66.67}{2} = 150$ kN

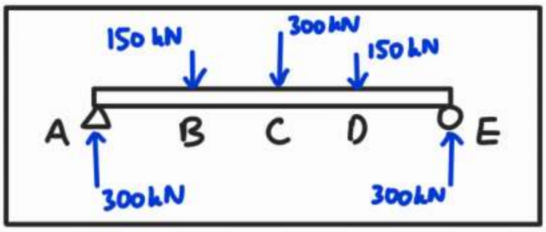
Point load at C = $\frac{2 \times 4.5 \times 66.67}{2} = 300$ kN





(b) $f_y = 355 \ N/mm^2$

Most critical point load might be at C or A/E with $F_{Ed} = 300 \ kN$.



Point C check

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

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

- $F_{cr} = 0.9(6)(210\,000)\dfrac{(10)^3}{598.6 - 2(12.8)} \times 10^{-3} \ kN = 1979.1 \ kN$

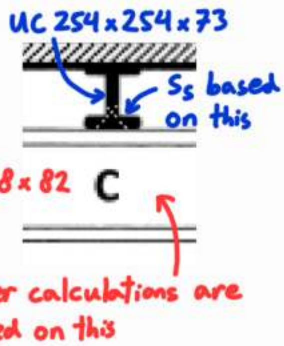
Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area	
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne
		h	b	t_w	t_f	r	d	c_f / t_f	c_w / t_w	C	N	n		
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2
610x178x100 +	100.3	607.4	179.2	11.3	17.2	12.7	547.6	4.14	48.5	8	94	30	1.89	18.8
610x178x92 +	92.2	603.0	178.8	10.9	15.0	12.7	547.6	4.75	50.2	7	94	28	1.88	20.4
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

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
610x178x100 +	72500	1660	23.8	3.60	2390	185	2790	296	0.854	38.7	1.44	95.0	128
610x178x92 +	64600	1440	23.4	3.50	2140	161	2510	258	0.850	42.7	1.24	71.0	117
610x178x82 +	55900	1210	23.2	3.40	1870	136	2190	218	0.843	48.5	1.04	48.8	104

- $m_1 = \dfrac{\cancel{355} \times 177.9}{\cancel{355} \times 10} = 17.79 \qquad m_2 = 0.02\left(\dfrac{598.6 - 2(12.8)}{12.8}\right) = 0.895$

- S_s is based on the secondary beam (UC $254 \times 254 \times 73$)

$$S_s = 8.6 + 1.6(12.7) + 2(14.2) = 57.32 \ mm$$



$$l_y = 57.32 + 2(12.8)\left(1 + \sqrt{17.79 + 0.895}\right) = 193.58 \ mm$$

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
254x254x107	107.1	266.7	258.8	12.8	20.5	12.7	200.3	5.38	15.6	8	134	34	1.52	14.2
254x254x89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	6.38	19.4	7	134	30	1.50	16.9
254x254x73	73.1	254.1	254.6	8.6	14.2	12.7	200.3	7.77	23.3	6	134	28	1.49	20.4

- $\bar{\lambda}_F = \sqrt{\dfrac{(193.58)(10.0)(355)}{1979.1 \times 10^3}} = 0.589 > 0.5$ (m_2 ok!)

- $\chi_F = \dfrac{0.5}{0.589} = 0.849 \leq 1.0$

- $L_{eff} = 0.849 \times 193.58 = 164.35$ mm

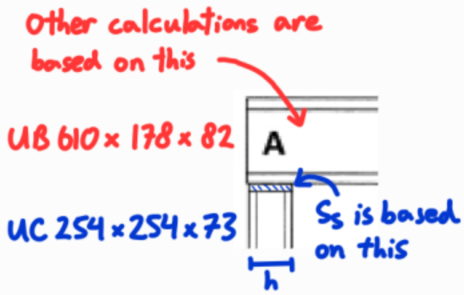
- $F_{Rd} = \dfrac{355 \times 164.35 \times 10}{1.0} \times 10^{-3}$ kN $= 583.4$ kN

- $\eta_2 = \dfrac{300}{583.4} = 0.514 < 1.0$ ok!

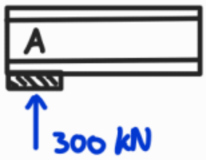
  $\eta_1 = \dfrac{750 \times 10^3}{\dfrac{355 \times 2190}{1.0}} = 0.965 < 1.0$

  $\eta_2 + 0.8\eta_1 = 0.514 + 0.8(0.965) = 1.29 \leq 1.4$ ok!

Point A/E Check



- S_s = h of column = 254.1 mm



- Type (c), assume point of contact is at the edge so $c = 0$

  $k_F = 2 + 6\left(\dfrac{254.1 + 0}{598.6 - 2(12.8)}\right) = 4.66 \leq 6.0$

- $F_{cr} = 0.9\,(4.66)(210\,000)\dfrac{(10)^3}{598.6 - 2(12.8)} \times 10^{-3}$ kN $= 1537$ kN

- $m_1 = \dfrac{\cancel{355} \times 177.9}{\cancel{355} \times 10} = 17.79$ $\quad m_2 = 0.02\left(\dfrac{598.6 - 2(12.8)}{12.8}\right) = 0.895$

- $l_y = 254.1 + 2(12.8)\left(1 + \sqrt{17.79 + 0.895}\right) = 390.4$ mm

- $\bar{\lambda}_F = \sqrt{\dfrac{(390.4)(10.0)(355)}{1537 \times 10^3}} = 0.950 > 0.5$ (m_2 ok!)

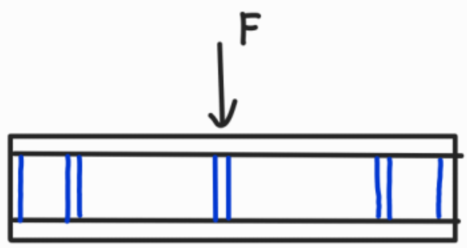
- $\chi_F = \frac{0.5}{0.950} = 0.526 \leq 1.0$
- $L_{eff} = 0.526 \times 390.4 = 205.4$ mm
- $F_{Rd} = \frac{355 \times 205.4 \times 10}{1.0} \times 10^{-3}$ kN $= 729$ kN

- $\eta_2 = \frac{300}{729} = 0.412 < 1.0$ Ok!

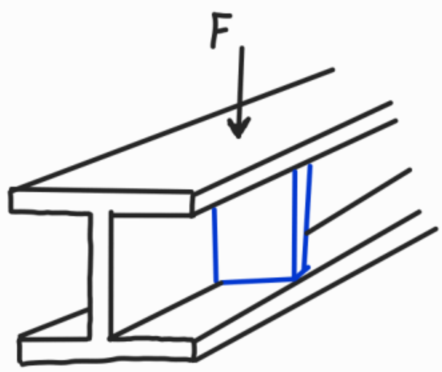
  $\eta_1 = \frac{750 \times 10^3}{\frac{355 \times 2190}{1.0}} = 0.965 < 1.0$

  $\eta_2 + 0.8\eta_1 = 0.412 + 0.8(0.965) = 1.18 \leq 1.4$ Ok!

(c)



(add stiffeners)



2. Figure Q2 shows a steel frame supporting a storage tank, the tank is sitting on 4 number of stumps. When fully loaded, total unfactored load of the tank is 600 kN. Self-weight of tank can be taken as 60 kN.

(a) Ignoring self-weight of the structural framing, calculate and draw the design shear force and bending moment diagrams of beam ABCD under this storage load.

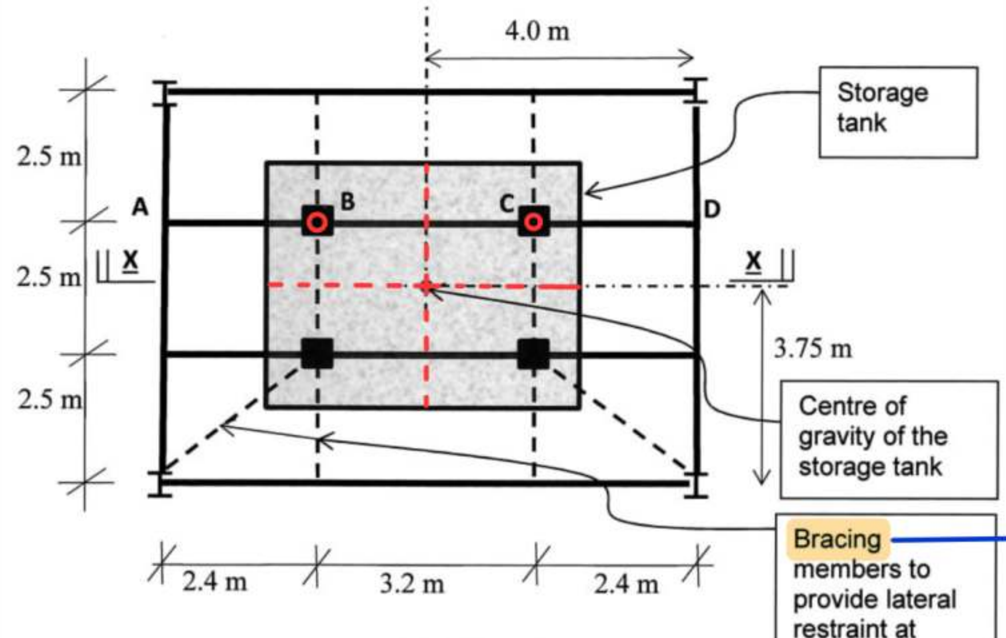
(6 Marks)

(b) If rolled section Grade S355, UB610x178x82 kg/m is used as beam ABCD, with lateral restraints provided at supports A, D and points B & C, check if this beam is adequate with regard to Lateral Torsional Buckling (LTB) resistance in accordance with EC3-1-1 to take the storage loads. Other design checks are not required.

(15 Marks)

(c) Regardless of the design check results in Q2(b) above, assumed that design check on LTB failed by a margin of 10%, propose two possible solutions to improve the LTB resistance of beam ABCD, provide sketches for your proposed solutions. No calculation is required for the proposed solutions. Assumed that the support structures have been built, but the storage tank is not yet installed.

(4 Marks)

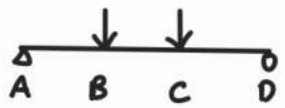


PLAN VIEW

Figure Q2(a)

No slabs i.e., void

There are point loads at B & C and no UDL elsewhere, so we are expecting:



2.) (a)

- Characteristic loads:

$G_k = 60 \text{ kN}$

$Q_k = 600 - 60 = 540 \text{ kN}$

- Design load:

$n = 1.35(60) + 1.5(540)$

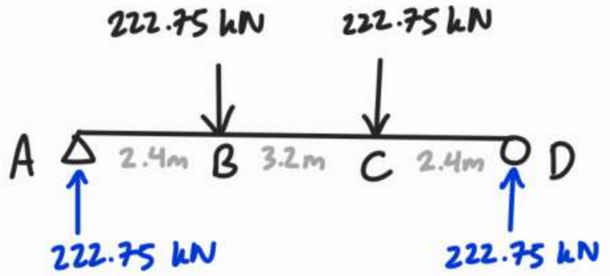
$= 891 \text{ kN}$

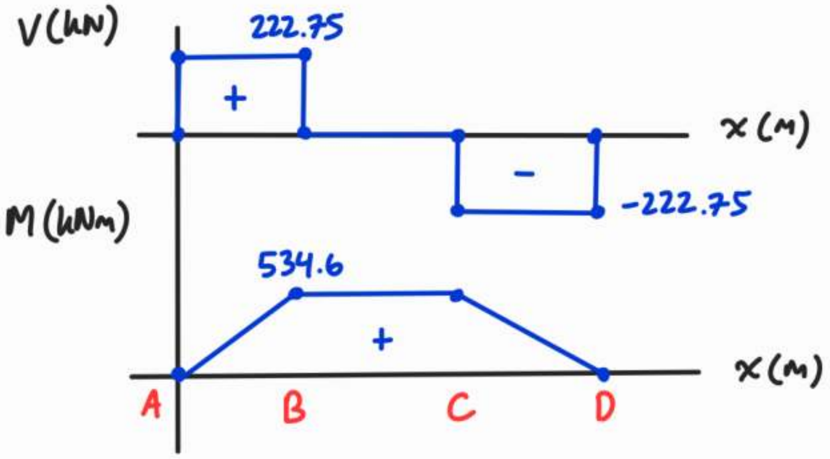
- Point load at B = Point load at C $= \frac{891}{4} = 222.75 \text{ kN}$

- Since the loading on beam ABCD is symmetrical,

Reaction at A = Reaction at D $= \frac{222.75 \times 2}{2} = 222.75 \text{ kN}$

•





(b) For LTB, just need to check BC as it is:

1.) The longest unrestrained length

2.) The section with the biggest M

$f_y = 355 \ N/mm^2$

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
610x178x100 +	100.3	607.4	179.2	11.3	17.2	12.7	547.6	4.14	48.5	8	94	30	1.89	18.8
610x178x92 +	92.2	603.0	178.8	10.9	15.0	12.7	547.6	4.75	50.2	7	94	28	1.88	20.4
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

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
610x178x100 +	72500	1660	23.8	3.60	2390	185	2790	296	0.854	38.7	1.44	95.0	128
610x178x92 +	64600	1440	23.4	3.50	2140	161	2510	258	0.850	42.7	1.24	71.0	117
610x178x82 +	55900	1210	23.2	3.40	1870	136	2190	218	0.843	48.5	1.04	48.8	104

• $L_{BC} = 3.2m$

• $\psi = \frac{534.6}{534.6} = +1.0 \Rightarrow C_1 = 1.0$

- $M_{cr} = 1.0 \times \dfrac{\pi^2 (210 \times \cancel{10^9})(1210 \times \cancel{10^{-8}})}{(3.2)^2} \times \sqrt{\dfrac{1.04 \times 10^{\cancel{-6}\,2}}{1210 \times \cancel{10^{-8}}} + \dfrac{(3.2)^2 (81 \times \cancel{10^9})(48.8 \times \cancel{10^{-8}})}{\pi^2 (210 \times \cancel{10^9})(1210 \times \cancel{10^{-8}})}} \times 10^{-3}$ kNm

  $= 782.522$ kNm

- $\bar{\lambda}_{LT} = \sqrt{\dfrac{2190 \times 355}{782.522 \times 10^3}} = 0.997 > 0.2$, LTB check needed

- $\dfrac{h}{b} = \dfrac{598.6}{177.9} = 3.36 > 2$, rolled $\Rightarrow$ (b) $\Rightarrow \alpha_{LT} = 0.34$

- $\phi_{LT} = 0.5 \left[1 + 0.34 (0.997 - 0.2) + 0.997^2 \right] = 1.132$

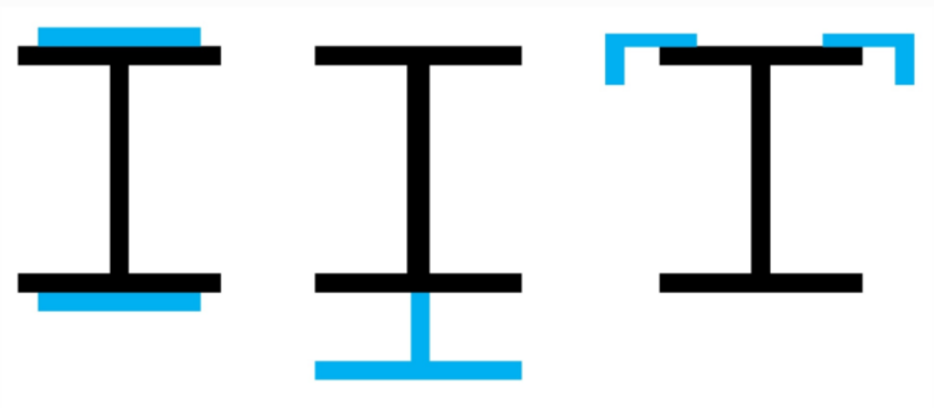
- $\chi_{LT} = \dfrac{1}{1.132 + \sqrt{1.132^2 - 0.997^2}} = 0.599 < 1.0$

- $M_{b,Rd} = 0.599 \times \dfrac{2190 \times 355}{1.0} \times 10^{-3}$ kNm

  $= 465.7$ kNm $< M_{Ed} = 534.6$ kNm // NOT OK !

$\therefore$ Beam is not adequate to resist LTB.

(c)



(increase the second moment of area, I_y)

3. A column FG, shown in Figure Q3(b), supports a simply supported beam EF where a flexible end plate joint is used at joint F. Figure Q3(a) shows the sectional view of the column and the two secondary beams. The base at G is a fixed joint. All actions have been factored; the self-weight of the column and the two secondary beams may be neglected in the calculations.

(a) Calculate the design compression action at F, and the nominal moments about y-y and z-z axes which are to be carried by the 254 x 254 x 89 UC column.

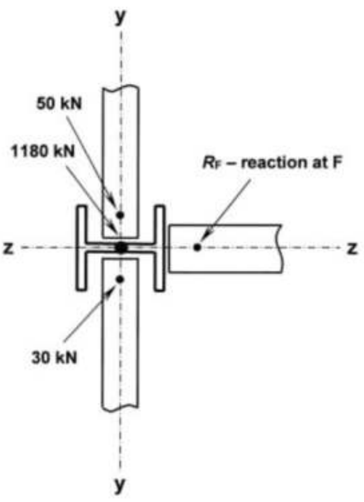
(6 Marks)

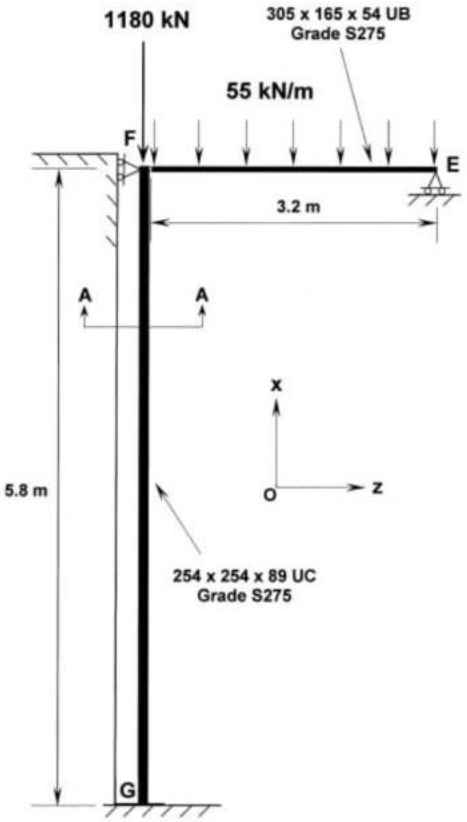
(b) Check the buckling and lateral torsional buckling resistance of this column, and use the appropriate column effective length in your calculations.

(12 Marks)

(c) If the base of the column at G is changed to a pinned joint, will the column still be strong to resist the design actions?

(7 Marks)





$f_y = 275\ N/mm^2$

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
254x254x89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	6.38	19.4	7	134	30	1.50	16.9
254x254x73	73.1	254.1	254.6	8.6	14.2	12.7	200.3	7.77	23.3	6	134	28	1.49	20.4

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
254x254x89	14300	4860	11.2	6.55	1100	379	1220	575	0.850	14.46	0.717	102	113
254x254x73	11400	3910	11.1	6.48	898	307	992	465	0.849	17.24	0.562	57.6	93.1

③ (a) Reaction at F, $R_F = \frac{55 \times 3.2}{2} = 88$ kN

- $N_{Ed} = 1180 + 50 + 30 + 88 = 1348$ kN
- $M_{Ed,y} = R_F \times e_y = 88 \times \left(100 + \frac{260.3}{2}\right) \times 10^{-3}$ kNm $= 20.25$ kNm
- $M_{Ed,z} = F_{R,z} \times e_z = (50-30) \times \left(100 + \frac{10.3}{2}\right) \times 10^{-3}$ kNm $= 2.10$ kNm

(b) Buckling Check

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

- $L_{cr} = \underset{\text{(pin-fixed)}}{\underline{0.85}} \times 5.8 = 4.93$ m

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

- $\bar{\lambda} = \frac{4.93}{6.55 \times 10^{-2}} \times \frac{1}{86.8} = 0.867$

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

  Curve ⓒ, $\alpha = 0.49$

- $\phi = 0.5\left[1 + 0.49(0.867 - 0.2) + 0.867^2\right] = 1.039$

- $\chi = \frac{1}{1.039 + \sqrt{1.039^2 - 0.867^2}} = 0.621$

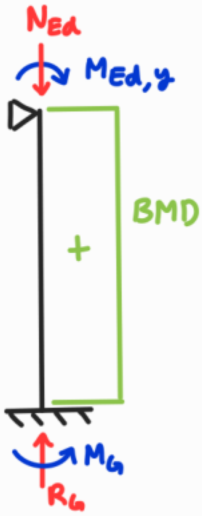
- $N_{b,Rd} = 0.621 \times \frac{(113 \times 10^{-4})(275 \times 10^6)}{1.0} \times 10^{-3}$ kN
  $= 1930$ kN $> N_{Ed} = 1348$ kN OK!

LTB Check

- $L_{FG} = 5.8m$ (Lateral restraints at F & G)
- $\psi = +1.0 \Rightarrow C_1 = 1.0 \longrightarrow$

  (fixed - pin)

  (if it's pin-pin, $\psi = 0$, $C_1 = 1.77$)



- $$M_{cr} = 1.0 \times \frac{\pi^2 (210\times10^9)(4860\times\cancel{10^{-8}})}{(5.8)^2} \times \sqrt{\frac{0.717\times10^{\cancel{-6}\,2}}{4860\times\cancel{10^{-8}}} + \frac{(5.8)^2(81\times10^9)(102\times\cancel{10^{-8}})}{\pi^2(210\times10^9)(4860\times\cancel{10^{-8}})}} \times 10^{-3}\ kNm$$

  $= 616.2\ kNm$

- $\bar{\lambda}_{LT} = \sqrt{\dfrac{1220\times275}{616.2\times10^3}} = 0.738 > 0.2$, LTB check needed

- $\dfrac{h}{b} = \dfrac{260.3}{256.3} = 1.02 < 2$, rolled $\Rightarrow$ a $\Rightarrow \alpha_{LT} = 0.21$

- $\phi_{LT} = 0.5\left[1 + 0.21(0.738 - 0.2) + 0.738^2\right] = 0.829$

- $\chi_{LT} = \dfrac{1}{0.829 + \sqrt{0.829^2 - 0.738^2}} = 0.829 < 1.0$

- $M_{b,Rd} = 0.829 \times \dfrac{1220\times275}{1.0} \times 10^{-3}\ kNm$

  $= 278.1\ kNm > M_{Ed,y} = 20.25\ kNm$ ok!

(c) From (b) we know that flexural buckling is much more critical than LTB, so we just use flexural buckling check as reference.

Buckling Check

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

- $L_{cr} = \underbrace{1.0}_{\text{(pin-pin)}} \times 5.8 = 5.8\ m$

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

- $\bar{\lambda} = \frac{5.8}{6.55 \times 10^{-2}} \times \frac{1}{86.8} = 1.02$

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

  Curve ©, $\alpha = 0.49$

- $\phi = 0.5\left[1 + 0.49(1.02 - 0.2) + 1.02^2\right] = 1.221$

- $\chi = \frac{1}{1.221 + \sqrt{1.221^2 - 1.02^2}} = 0.528$

- $N_{b,Rd} = 0.528 \times \frac{(113 \times 10^{-4})(275 \times 10^6)}{1.0} \times 10^{-3}\ kN$

  $= 1640\ kN > N_{Ed} = 1348\ kN$ OK!

∴ Column will still be strong to resist design actions.

4. Figure Q4 shows the joint details of a latticed truss where the diagonal and vertical web members are bolted along the longer legs to the 12 mm thick gusset plate. All the discontinuous web members are fabricated using a Grade S275 steel, and they are subjected to factored design axial actions as indicated in Figure Q4.

(a) Determine the adequacy of the discontinuous diagonal web member which carry a factored design tensile action of 55 kN, taking end distance e_1 = 60 mm and edge distance e_2 = 40 mm.

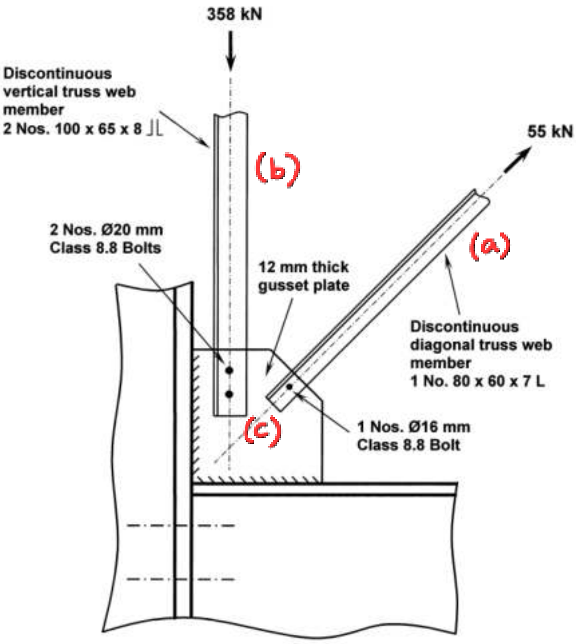
(7 Marks)

(b) Determine the adequacy of the discontinuous vertical web member which is subjected to a factored design compressive action of 358 kN. Assume an effective length L_E of 1.8 m when calculating the slenderness ratios about all the axes for this back-to-back angle sections.

(12 Marks)

(c) Check the shear resistances of the 1-bolt group of ϕ16 mm and the 2-bolt group of ϕ20 mm.

(6 Marks)



S275, t < 40 mm

f_y = 275 N/mm^2

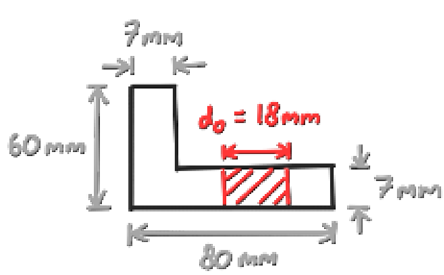
f_u = 430 N/mm^2

4. (a) • Angle is bolted along the longer leg.

• Diameter of hole, $d_0 = d + 2$ mm (for $d \leq 24$ mm)

$= 16 + 2 = 18$ mm

•



$\boxed{80 \times 60 \times 7 \text{ L}}$

$A_{net} = (60 \times 7) + (73 \times 7) - (18 \times 7)$

$= 805$ mm^2

or, using sections property table,

$A_{net} = A - d_0 \times t = 938 - (18 \times 7)$

$= 812$ mm^2

Either one is OK.

- $N_{pl,Rd} = \dfrac{938 \times 275}{1.0} \times 10^{-3}\ kN = 258.0\ kN$

- For 1 bolt, $N_{u,Rd} = \dfrac{2.0(40 - (0.5 \times 18)) \times 7 \times 430}{1.25} \times 10^{-3}\ kN = 149.296\ kN$

Using the default EC3 value of $\gamma_{M2} = 1.25$;
for Singapore Annex use $\gamma_{M2} = 1.1$.

- $N_{t,Rd} = \min\{258.0,\ 149.296\} = 149.3\ kN > N_{Ed} = 55\ kN$ OK!

*Note: A_{net} only used for $N_{u,Rd}$ of > 1 bolt.
See relevant formulas for $N_{u,Rd}$.

(b)
- $L_E = 1.8\ m$
- $\lambda_1 = 93.9\sqrt{\dfrac{235}{275}} = 86.8$

$\boxed{100 \times 65 \times 8\ \text{JL}}$

12 mm gusset plate in-between the back-to-back angles.

Composed of Two Angles		Total Mass per Metre	Distance	Total Area	Properties about Axis y-y			Radius of Gyration i_z about Axis z-z (cm)				
			n_y		I_y	i_y	$W_{el,y}$	Space between angles, s, (mm)				
h x b mm	t mm	kg/m	cm	cm²	cm⁴	cm	cm³	0	8	10	12	15
100x65	10 +	24.6	6.64	31.2	308	3.14	46.4	2.43	2.72	2.79	2.87	2.99
	8 +	19.9	6.73	25.4	254	3.16	37.8	2.39	2.67	2.74	2.82	2.93
	7 +	17.5	6.77	22.4	226	3.17	33.2	2.37	2.65	2.72	2.79	2.91

- $\bar{\lambda}_z = \dfrac{(1.8 \times 10^2) \div 2.82}{86.8} = 0.735$

- For discontinuous internal member,

$\bar{\lambda}_{eff,z} = 0.35 + 0.7(0.735) = 0.865$

- Angles → curve ⓑ → $\alpha = 0.34$

- $\phi = 0.5\left[1 + 0.34(0.865 - 0.2) + 0.865^2\right] = 0.987$

- $\chi = \dfrac{1}{0.987 + \sqrt{0.987^2 - 0.865^2}} = 0.684 < 1.0$

- $N_{b,Rd} = 0.684 \times \dfrac{(25.4 \times 10^{-4}) \times (275 \times 10^6)}{1.0} \times 10^{-3}\ kN = 478\ kN$

$> N_{Ed} = 358\ kN$ OK!

(c) <u>1 No. Ø16mm Class 8.8 (S275)</u>

From section properties table, $F_{v,Rd} = 60.3\ kN$

$\Sigma F_{v,Rd} = 60.3\ kN > F_{Ed} = 55\ kN$ // ok!

<u>2 No. Ø20mm Class 8.8 (S275)</u>

From section properties table, $F_{v,Rd} = 94.1\ kN$

$$\Sigma F_{v,Rd} = \underbrace{2}_{\text{2 bolts}} \times \underbrace{2}_{\text{double shear}} \times 94.1 = 376.4\ kN > F_{Ed} = 358\ kN$$ // ok!