NANYANG TECHNOLOGICAL UNIVERSITY

SEMESTER 1 EXAMINATION 2019-2020

CV4102 – ADVANCED STEEL DESIGN

November / December 2019 **Time Allowed: 2½ hours**

INSTRUCTIONS

1. This paper contains **FOUR (4)** questions and comprises **SIX (6)** pages.
2. Answer **ALL FOUR (4)** questions.
3. All questions carry equal marks.
4. This paper is an Open Book Examination.

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1. Figure Q1 shows a non-overlapped N-joint where the compressive stress in the chord is $\sigma_{p,Ed}$=0.35f_{y0}. The gap between the two braces is 75 mm and all members are made of circular hollow sections (CHS) in accordance to EN 10210, and the steel Grade is S355 as given in Table 3.1 of EN 1993-1-1: 2014. Use design recommendations given in Eurocode 3, EN 1993-1-8: 2010,

(a) Calculate the joint parameters γ and β, and the ranges of validity of the non-overlapped N-joint.

(6 marks)

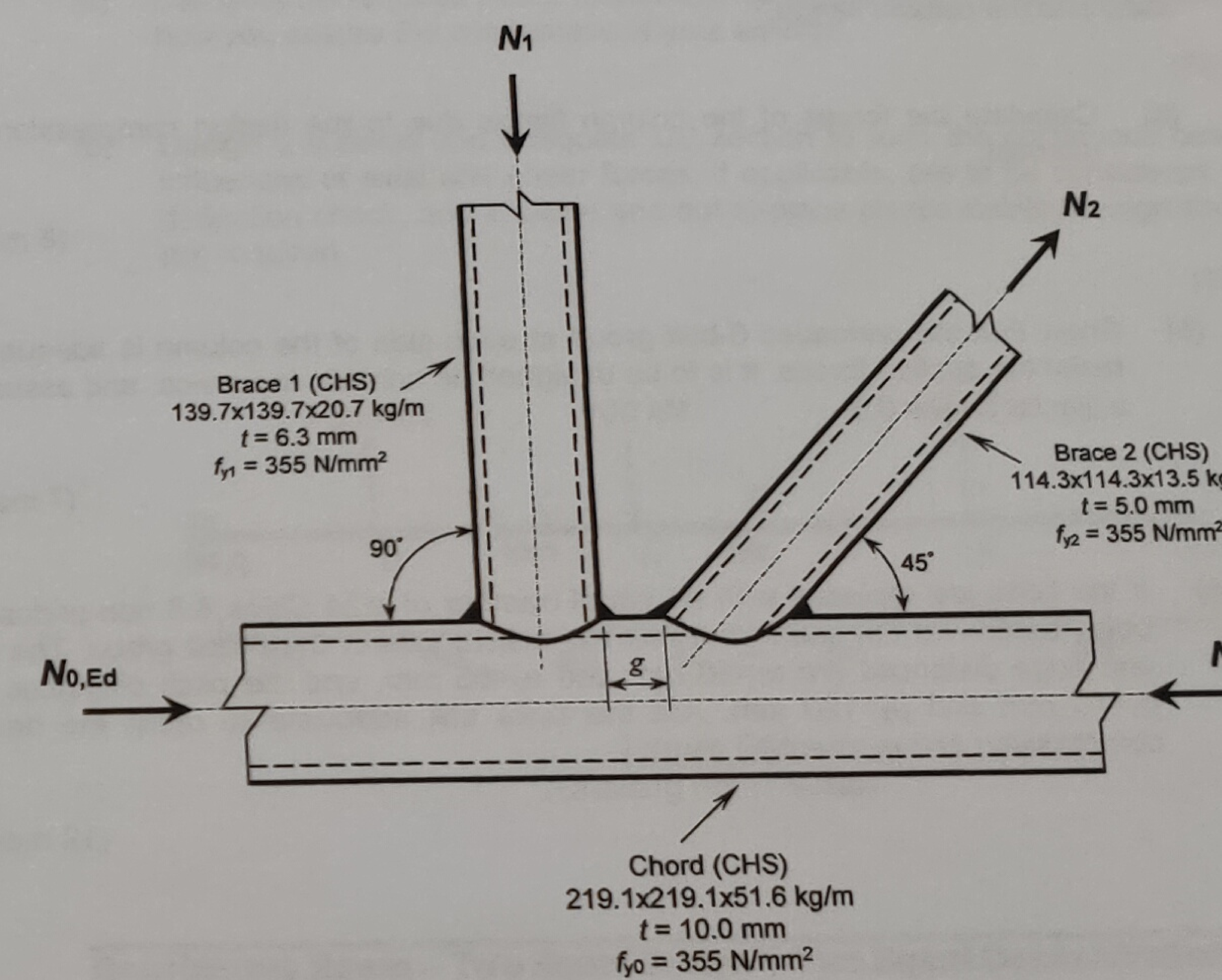
(b) Determine the design resistances N_1 and N_2 of the joint using the appropriate design formulae given in Table 7.2, and check the punching shear resistances.

(15 marks)

(c) What is the failure criterion of the N-joint? What is the effect of decreasing the gap g between the two braces?

(4 marks)

Note: Question No. 1 continues on Page 2.



① non-overlapped (N gap joint) (HS

② column bolted

③

④

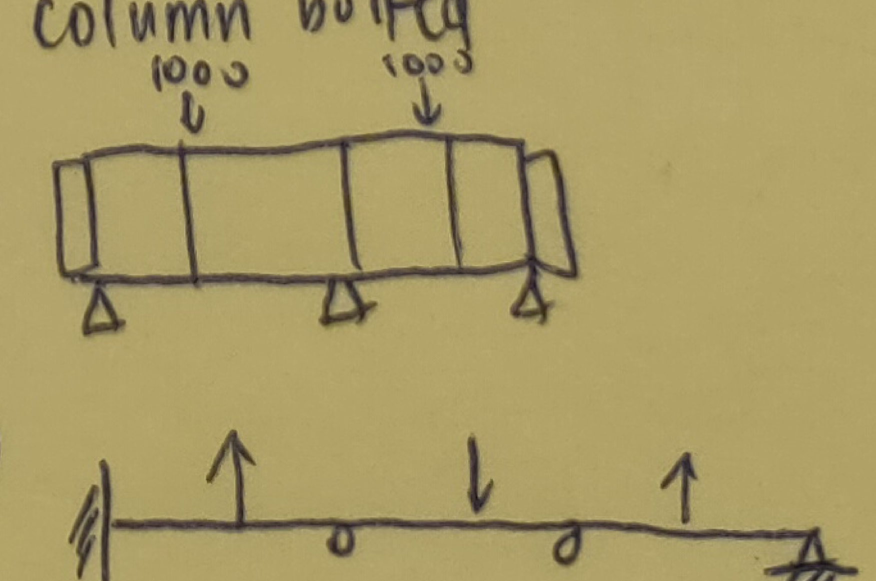


Figure Q1

(Note: drawings are not drawn to scale)

2. The bolted column splice connection shown in Figure Q2 is required to transmit a design compression of 230 kN and a design moment of 75 kNm, respectively. The flange splice plate dimensions are 250 mm x 450 mm x 10 mm thick, and it is in Grade S275 steel. The column is made from 254 x 254 x 89 UC of Grade S275, and M24 Class 8.8 preloaded bolts in Grade S275 with standard clearance holes used to connect the splice plate and the column flange.

(a) Calculate the forces of the column flange due to the design compression and moment.

(6 marks)

(b) Show that the preloaded 6-bolt group at each side of the column is adequate to resist the applied forces. It is to be designed as non-slip in service, and assuming a slip factor $\mu = 0.5$.

(7 marks)

(c) If the bolts are replaced with an equal number of M24 Class 8.8 non-preloaded bolts, determine the shear and bearing resistances of the 6-bolt group. The end and edge distances are e_1=50 mm and e_2=65 mm, and the pitch distances are p_1=70 mm and p_2=120 mm. Are the bolts still adequate to resist the design compression and moment?

(12 marks)

Note: Question No. 2 continues on Page 4.

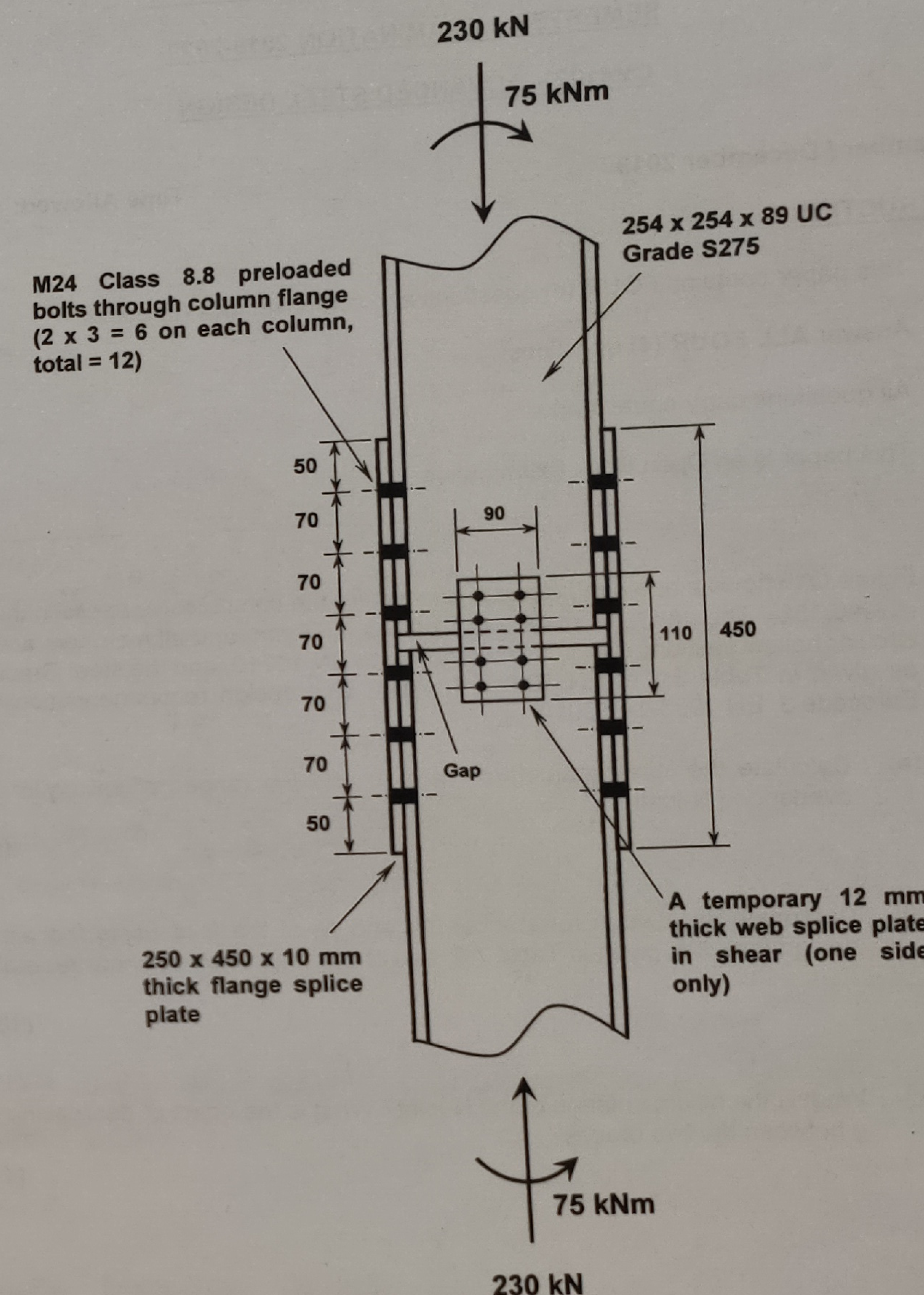


Figure Q2

(All dimensions are in mm unless otherwise stated)

(Note: drawings are not drawn to scale)

3. The 12 m 2-span continuous plate girder ABC shown in Figure Q3(a) is fabricated from Grade S275 steel plates throughout. It is simply supported at end posts A and C, and effectively restrained in the lateral direction along the entire plate girder. It is supporting a factored design load of 1000 kN at mid-span D and E. Details of the stiffeners are given in Figure Q3(b). Assume all the end posts and stiffeners are rigid.

(a) Check the adequacy of the girder in terms of its bending resistance.
(b) Check the adequacy of the girder in terms of its shear resistance.
(c) Check the adequacy of the rigid end posts.
(d) Explain the considerations needed to ensure minimum maintenance cost of the plate girder throughout its service life.

(25 marks)

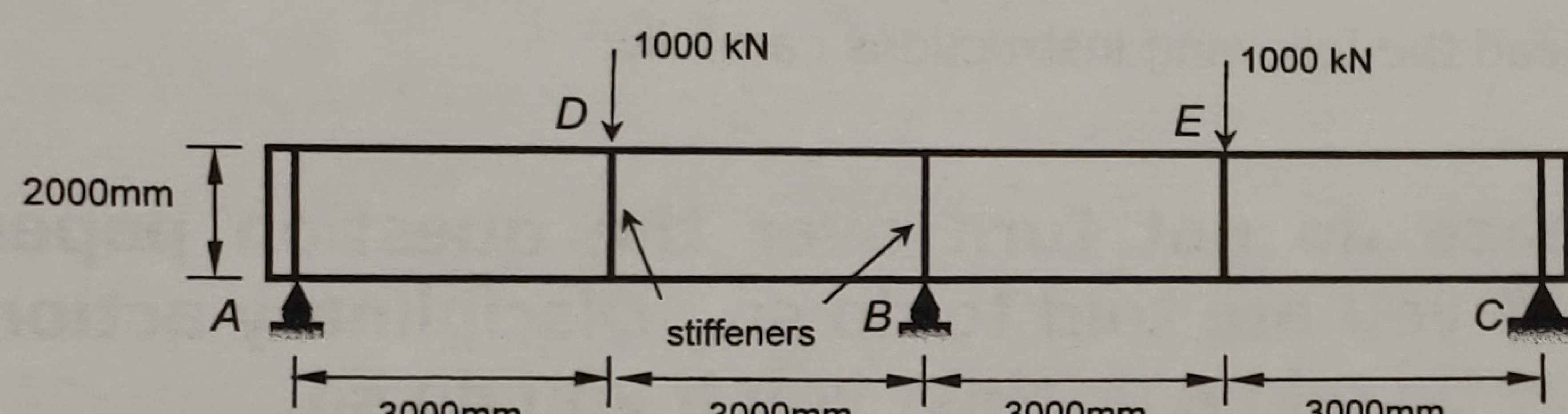
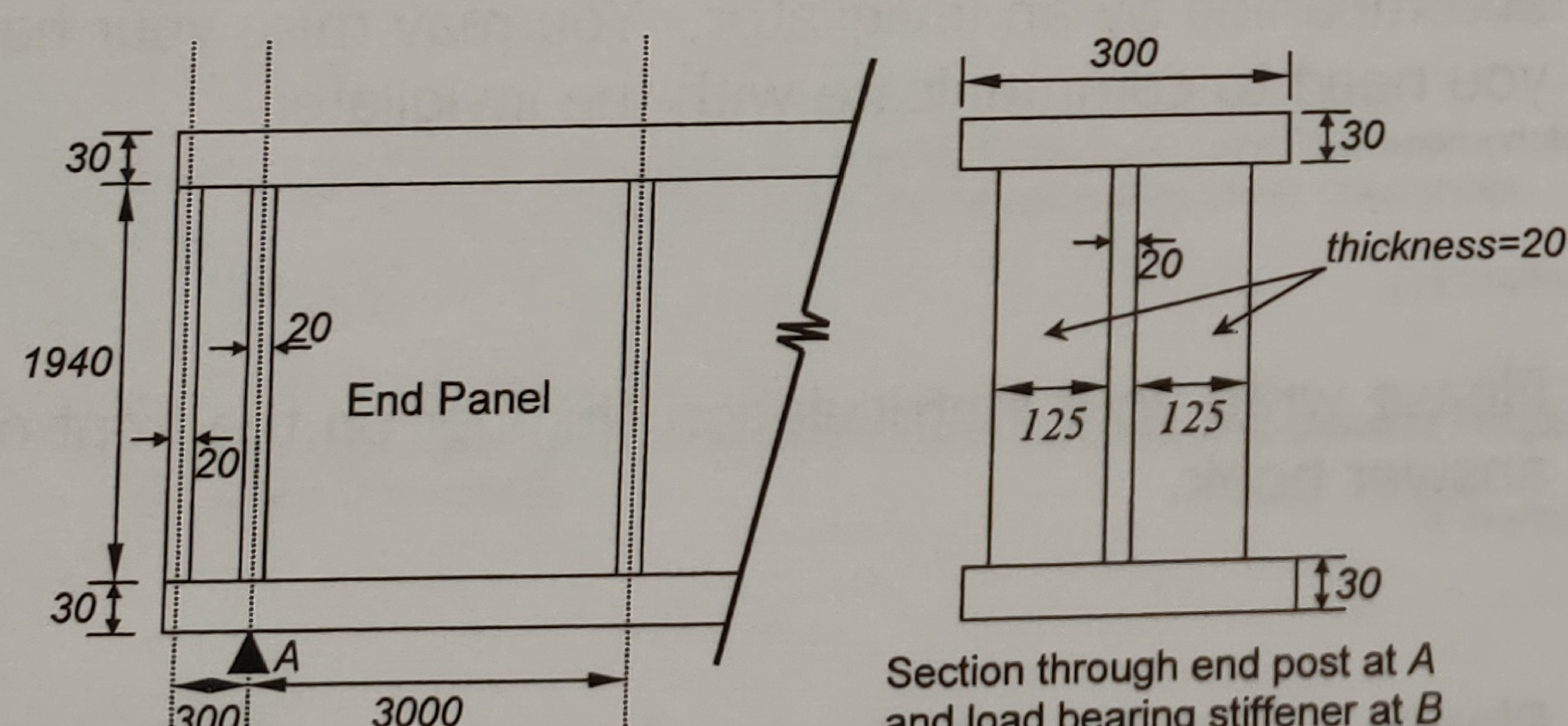


Figure Q3(a)
(Drawing not to scale)



Elevation view of end post at A

Section through end post at A and load bearing stiffener at B

Note:
(i) Elevation details for end post at C is identical to end post at A.
(ii) All dimensions in mm.

Figure Q3(b)
(Drawing not to scale)

4. The continuous beam ABCDEFG shown in Figure Q4 is fixed at A, roller-supported at C and E, and pinned at G. It is subjected to factored design concentrated loads at B, D and F. It is fabricated using uniform S355 steel section throughout. The plastic moment resistances for Member ABC is $2M_p$, Member CDE is $3M_p$ and Member EFG is $4M_p$. Adequate restraints against stability are provided.

(a) Calculate the required plastic moment of resistance for the continuous beam. Show how you ensure the correctness of your solution.

(13 marks)

(b) Design a suitable and adequate UB section to form the continuous beam. The influences of axial and shear forces, if applicable, are to be considered. Further deflection check, and in-plane and out-of-plane plastic stability design checks are not required.

(12 marks)

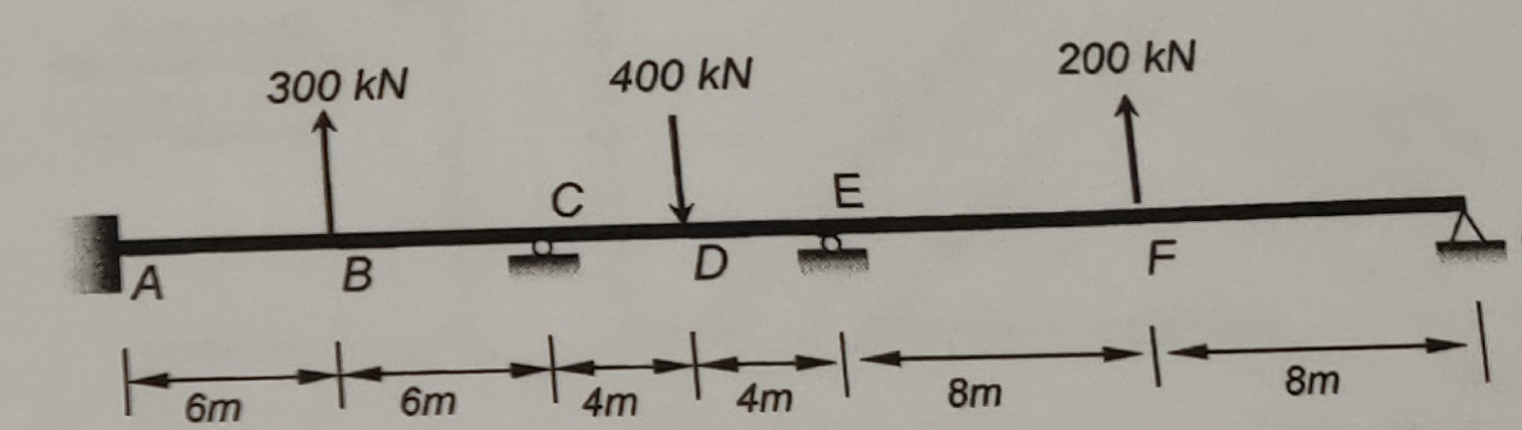
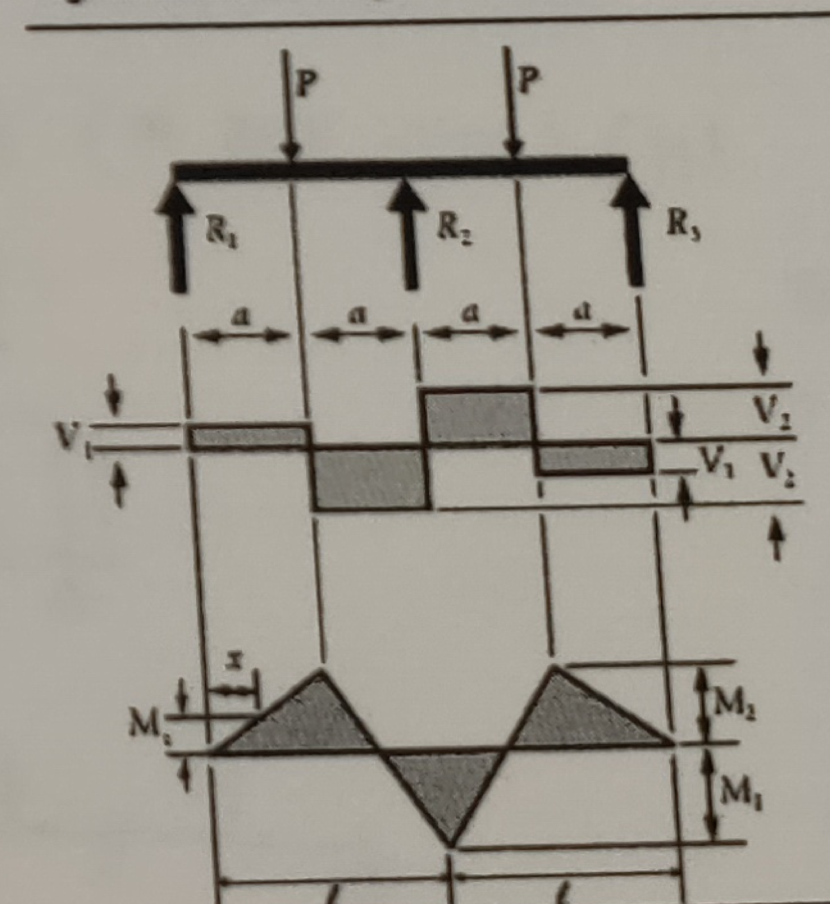


Figure Q4
(Drawing not to scale)

Continuous Beam – Two Equal Spans – Two Equal Concentrated Loads Symmetrically Placed



$R_1 = V_1 = R_3 = V_3 \ldots\ldots = \frac{5P}{16}$

$R_2 = 2V_2 \ldots\ldots = \frac{11P}{8}$

$V_2 = P - R_1 \ldots\ldots = \frac{11P}{16}$

$V_{max} \ldots\ldots = V_2$

$M_1 \ldots\ldots = -\frac{3P\ell}{16}$

$M_2 \ldots\ldots = \frac{5P\ell}{32}$

M_x (when $x < a$) $\ldots\ldots = R_1 x$

END OF PAPER

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CV4102 PYP 2019/2020

1) a) $\gamma = \dfrac{d_0}{2t_0} = \dfrac{219.1}{2(10)} = 10.955$ ✓

$\beta = \dfrac{d_1 + d_2}{2d_0} = \dfrac{139.7 + 114.3}{2(219.1)} = 0.58$ ✓

Range of validity

$0.2 < \dfrac{d_1}{d_0} = \dfrac{139.7}{219.1} = 0.638 < 1.0$

$0.2 < \dfrac{d_2}{d_0} = \dfrac{114.3}{219.1} = 0.522 < 1.0$

chords in compression

* For $f_y = 355$
$\varepsilon = \sqrt{235/355} = 0.814$
$70\varepsilon^2 = 70(0.814)^2 = 46.382$

$10 \leq \dfrac{d_0}{t_0} = \dfrac{219.1}{10} = 21.91 \leq 50 \leq 46.382$ (class 2; ok)

Braces

Tension

$\dfrac{d_2}{t_2} = \dfrac{114.3}{5} = 22.86 \leq 50$

compression

$10 \leq \dfrac{d_1}{t_1} = \dfrac{139.7}{6.3} = 22.17 \leq 50 \leq 46.382$ (class 2; ok!)

min gap, $g = t_1 + t_2 = 5 + 6.3 = 11.3$

$g = 75 \text{ mm} \geq 11.3\text{mm}$ OK!

b) Efficiency formula, $\dfrac{N_{1,Rd}}{A_1 f_{y1}}$

Table 7.2, For N-joint.

$N_{1,Rd} = \dfrac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1.8 + 10.2\dfrac{d_1}{d_0}\right) / \gamma_{M5}$

$N_{2,Rd} = \dfrac{\sin\theta_1}{\sin\theta_2} N_{1,Rd}$

$k_g = \gamma^{0.2}\left(1 + \dfrac{0.024\gamma^{1.2}}{1 + e^{0.5\frac{g}{t_0} - 1.33}}\right)$

$= 10.955^{0.2}\left(1 + \dfrac{0.024(10.955)^{1.2}}{1 + e^{0.5\left(\frac{75}{10}\right) - 1.33}}\right)$

$= 1.67$ ✓

$\sigma_{p,Ed} = 0.35 f_{y0}$

$n_p = \dfrac{\sigma_{p,Ed}}{f_{y0}} = 0.35 > 0$

$k_p = 1 - 0.3 n_p (1 + n_p) \leq 1.0$

$= 1 - 0.3(0.35)(1 + 0.35) \leq 1.0$

$= 0.858 \leq 1.0$ ✓

$N_{1,Rd} = \dfrac{1.67 \times 0.858 \times 355 \times 10^2}{\sin 90^\circ}\left(1.8 + 10.2 \times \dfrac{139.7}{219.1}\right) / 1.0$

$= 422.375$ KN ✓

$N_{2,Rd} = \dfrac{\sin 90^\circ}{\sin 45^\circ}(422.375)$

$= 597.33$ kN ✓

$d_1 = 139.7 \leq 219.1 - 2(10) = 199.1$

Need to check for punching shear for both braces

$N_{1,Rd} = \dfrac{f_{y0}}{\sqrt{3}} t_0 \pi d_1 \dfrac{1 + \sin\theta_1}{2\sin^2\theta_1} / \gamma_{M5}$

$= \dfrac{355}{\sqrt{3}}(10)(\pi)(139.7)\left(\dfrac{1 + \sin 90}{2(\sin 90)^2}\right) / 1.0$

$= 899.526$ kN > 422.375

$N_{2,Rd} = \dfrac{f_{y0}}{\sqrt{3}} t_0 \pi d_2 \dfrac{1 + \sin\theta_2}{2\sin^2\theta_2} / \gamma_{M5}$

$= 1256.39$ kN > 597.33.

$\therefore$ Brace 1 resistance = 422.375 KN

Brace 2 resistance = 597.33 kN.

c) Brace capacity for Brace 1 = $A_1 f_{y1} = \frac{\pi}{4}\{139.7^2 - 127.1^2\}(355) = 937.3$ kN

Brace capacity for Brace 2 = $A_2 f_{y2} = \frac{\pi}{4}\{114.3^2 - 104.3^2\}(355) = 609.5$ kN

$\therefore$ Failure criterion for both brace is chord face failure.

By decreasing the gap, k_g will be increased, increasing $N_{1,Rd}$ & $N_{2,Rd}$. If $N_{1,Rd}$ & $N_{2,Rd}$ are increased and surpass the resistance for punching shear, the brace might fail in punching shear

or surpass brace capacity
=> fail by brace failure

CV4102 2018/19 PYP

1) a) $\gamma = \frac{b_0}{2t_0} = \frac{150}{2(12.5)} = 6$ ✓

$\beta = \frac{b_1+b_2}{2b_0} = \frac{90+90}{2(150)} = 0.6$ ✓

$\varepsilon = \sqrt{235/355} = 0.814$

Ranges of validity :

$\frac{b_1}{b_0} = \frac{b_2}{b_0} = \frac{90}{150} = 0.6 \geq 0.25$

compression brace $\frac{b_2}{t_2} = \frac{90}{10} = 9 \leq 33\varepsilon = 33(0.814) = 26.85$

Tension brace $\frac{b_1}{t_1} = \frac{90}{10} = 9 \leq 35$

$\frac{h_1}{t_1} = \frac{90}{10} = 9 \leq 35$

~~overlap not need check~~ ~~Additional for SHS~~

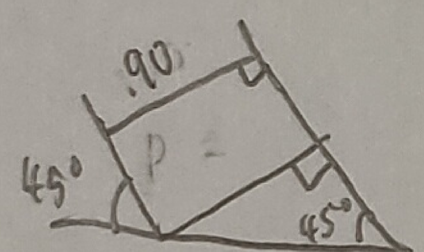
$0.5 \leq \frac{h_0}{b_0} = \frac{h_1}{b_1} = \frac{h_2}{b_2} = 1.0 \leq 2.0$

$\cancel{0.6 \leq \frac{b_1+b_2}{2b_1} = \frac{90+90}{2(90)} = 1 \leq 1.3}$

$\frac{h_0}{t_0} = \frac{b_0}{t_0} = \frac{150}{12.5} = 12 \leq 33\varepsilon = 26.85$

$\cancel{\frac{b_0}{t_0} = \frac{150}{12.5} = 12 \leq 15}$

~~∴ Table 7.10 cannot be used~~



$\sin 45° = \frac{90}{p}$ $q = 40$

$p = 127.3$

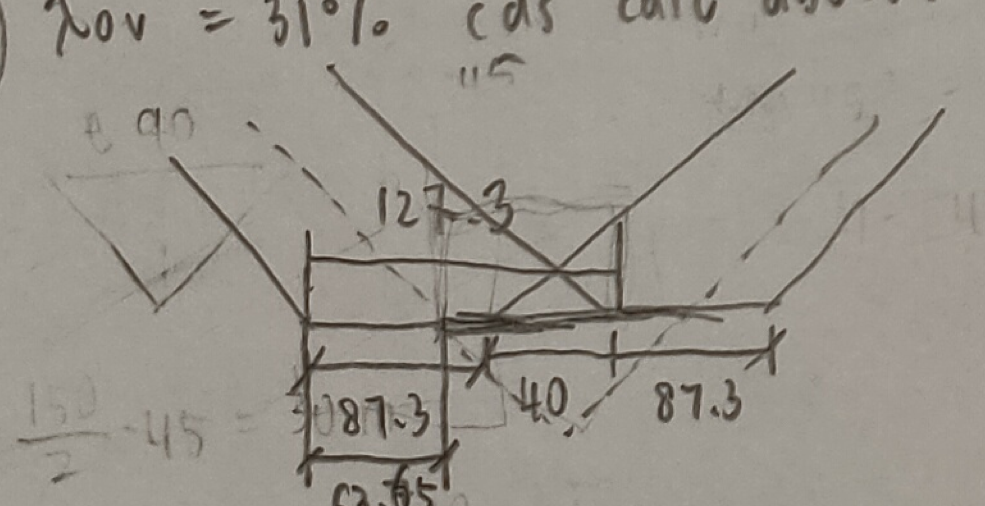
$\lambda_{ov} = \left(\frac{q}{p}\right) \times 100\% = \frac{40}{127.3} \times 100\% = 31\%$

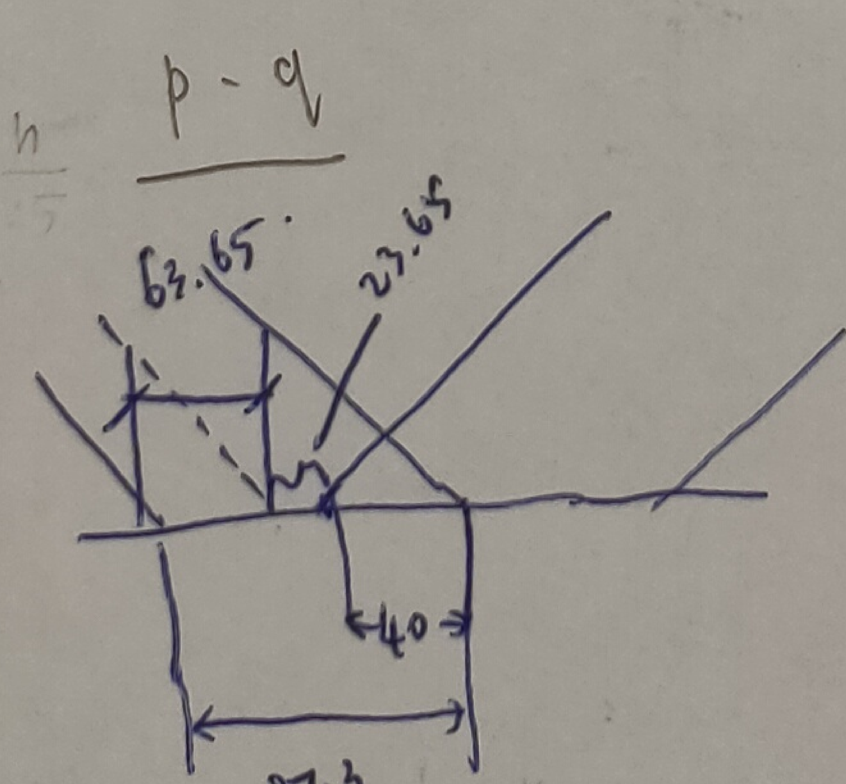
since ~~overlapping brace is partially welded~~
~~⇒ hidden seam not welded~~
⇒ ~~$\lambda_{ov,lim} = 60\%$~~

$25\% \leq \lambda_{ov} = 31\% \leq \cancel{60}\%$ 50% ← lecture notes

$\boxed{or}$ ~~$\frac{b_1}{b_j} = \frac{b_1}{b_2} = \frac{90}{90} = 1$~~ ≤ 0.75 ✗

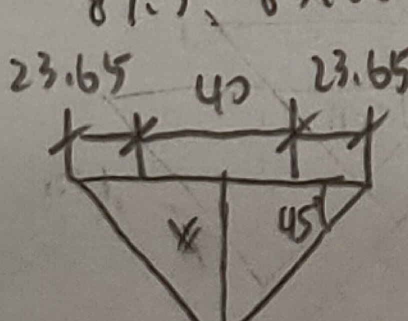
b) $\lambda_{ov} = 31\%$ (as calc above)





$127.3 - 40 = 87.3$

$87.3 - 63.65 = 23.65$



length $= 40 + 23.65 \times 2 = 87.3$

$x = \frac{87.3\ mm}{2} = 43.65\ mm$

$e = \frac{150}{2} - 43.65 = 31.35\ mm$

above ⇒ $e = -31.35\ mm$

check

$-0.55 d_0 \leq e \leq 0.25 d_0$

$-75 \leq -31.35 \leq 37.5$

ok! //

c) $\beta = 0.6$

~~$(1-\frac{1}{\gamma}) = (1-\frac{1}{6}) = 0.833$~~

~~since $\beta = 0.6 \leq 0.85$~~

~~∴ punching shear is not necessary~~

$25\% < \lambda_{ov} < 50\%$

$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + 2h_i \frac{\lambda_{ov}}{50} - 4t_i \right) / \gamma_{M5}$

$b_{eff} = \frac{10}{b_0/t_0} \frac{f_{y0} t_0}{f_{yi} t_i} b_i \leq b_i$

$= 93.75 \leq b_i = 90$

$= 90$

$b_{e,ov} = \frac{10}{b_j/t_j} \frac{f_{yj} t_j}{f_{yi} t_i} b_i \leq b_i$

$= \frac{10}{90/10} \times \frac{355 \times 10}{355 \times 10} \times 90 \leq 90$

$= 100 \leq 90$

$= 90$

$N_{i,Rd} = 355(10)(90+90+2(90)(\frac{31}{50}) - 4(10))/10$

$= 893.18\ kN$ //

2019/20

a) $F_{t,Ed} = \dfrac{M}{h-t_f} - \dfrac{N}{2}$

$= \dfrac{75\times10^6}{(260.3-17.3)\,mm} - \dfrac{230\times10^3}{2}$

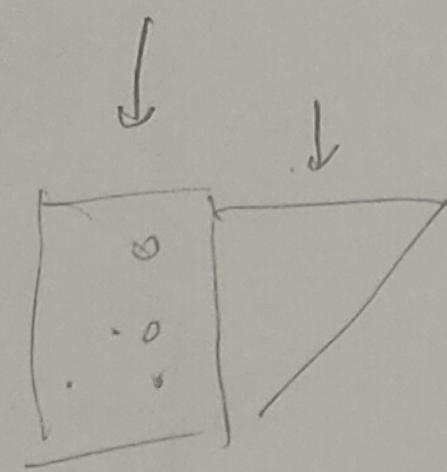
$= 193641.97 = 193.642\,kN$ ✓

$F_{c,Ed} = \dfrac{M}{(h-t_f)} + \dfrac{N}{2}$

$= \dfrac{75\times10^6}{(260.3-17.3)} - \dfrac{230\times10^3}{2}$

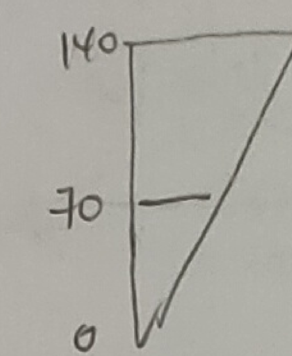
$= 423641.97$

$= 423.642\,kN$ ✓



Bolts

$F_{max} = F_{t,Ed} = \dfrac{Re\,y_{max}}{n\sum y^2}$



$= \cancel{\dfrac{75(140)}{2(70^2+140^2)}}$

$= \cancel{214.286}$

$\dfrac{260.3}{2}$

$= \dfrac{230(130.15)(140)}{2(70^2+140^2)}$

=

b) $\mu = 0.5$

M24 class 8.8 preloaded.

24mm diameter $\Rightarrow A_s = 353\,mm^2$

Class 8.8 $\Rightarrow f_{ub} = 800\,N/mm^2$

Preload $F_{p,C} = 0.7 f_{ub} A_s$

$= 0.7(800)(353)$

$= 197680\,N$

Check for tension capacity

$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}} = \dfrac{0.9(800)(353)}{1.25} = 203\,kN > 193.642\,kN$

$F_{s,Rd} = \dfrac{K_s n \mu}{\gamma_{M3}} F_{p,C} = \dfrac{(1)(1)(0.5)}{1.25}(197680)$

$= 79072 = 79.1\,kN$

~~$F_{v,Ed} = \dfrac{193.642}{6} = 32.27\,kN$~~ ~~$< 79.1\,kN$ adequate!~~

~~$= \dfrac{423.642}{6} = 70.607\,kN$~~

$F_v = \dfrac{230}{\boxed{12}} = 19.17\,kN$

$F_{p,C} = 0.7(800)A_s$

$= 0.7(800)(353)$

$= 197.68\,kN$

$F_{s,Rd} = \dfrac{k_s n \mu (F_{p,C} - 0.8F_{t,Ed})}{\gamma_{M3}}$

$= \dfrac{(1)(1)(0.5)(197.68 - 0.8\times214.286)}{1.25}$

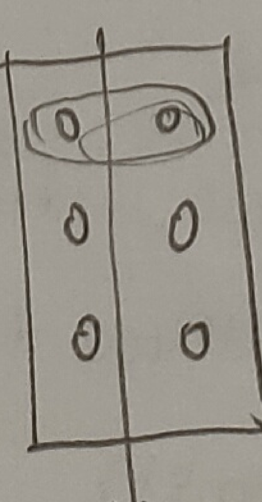
$= 10.5\,kN$

c) $F_{v,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$

class 8.8 $\Rightarrow f_{ub} = 800$, $\alpha_v = 0.6$

24 diameter $\Rightarrow A = 353\,mm^2$

$F_{v,Rd} = \dfrac{0.6(800)(353)}{1.25} = 135.55\,kN$



$F_{b,Rd} = \dfrac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$

$\alpha_b = \min\left(\alpha_d, \dfrac{f_{ub}}{f_u}, 1.0\right)$ $\quad \dfrac{f_{ub}}{f_u} = \dfrac{800}{430} = 1.86$

In direction of load transfer,

end bolts $\Rightarrow \alpha_d = \dfrac{e_1}{3d_0} = \dfrac{50}{3(26)} = 0.641$ $\quad \alpha_b = 0.641$

inner bolts $\Rightarrow \alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4} = \dfrac{70}{3(26)} - \dfrac{1}{4} = 0.647$ $\quad \alpha_b = 0.647$

perpendicular to direction of load

end $k_1 = \min\left(2.8\dfrac{e_2}{d_0} - 1.7, 2.5\right) = \min\left(2.8\dfrac{65}{26} - 1.7, 2.5\right)$

$= \min(5.3, 2.5)$

$= 2.5$

inner $k_1 = \min\left(1.4\dfrac{p_2}{d_0} - 1.7, 2.5\right)$

$= \min\left(1.4\dfrac{120}{26} - 1.7, 2.5\right)$

$= \min(4.76, 2.5)$

$= 2.5$

plate thickness

✓ end bolts, $F_{b,Rd} = \dfrac{2.5(0.641)(430)(24)(10)}{1.25}$

$= 132.3\,kN$

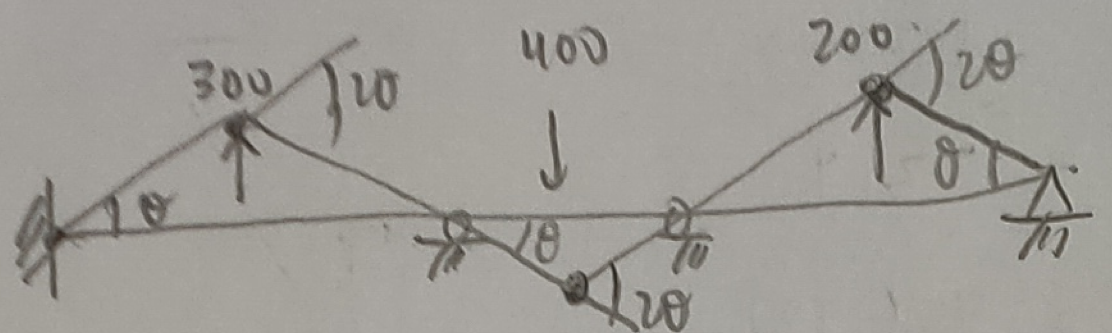
✓ inner bolts, $F_{b,Rd} = \dfrac{2.5(0.647)(430)(24)(10)}{1.25}$

$= 133.54\,kN$

Total bearing resistance

$= 2(132.3) + 4(133.54)$

$= 798.76\,kN$

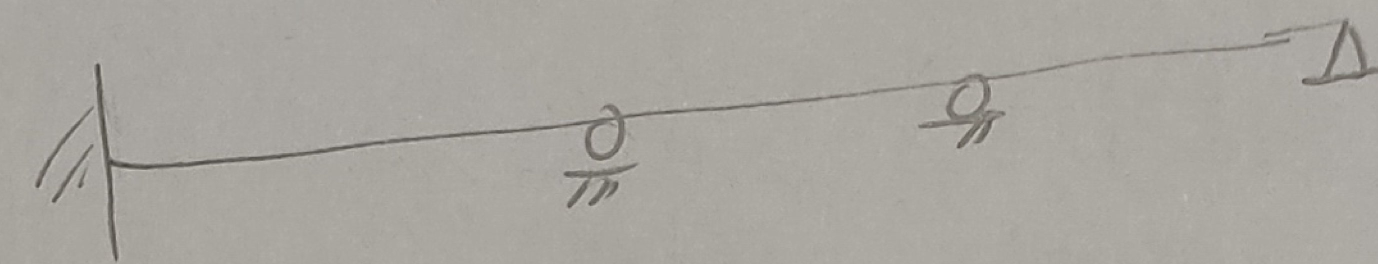


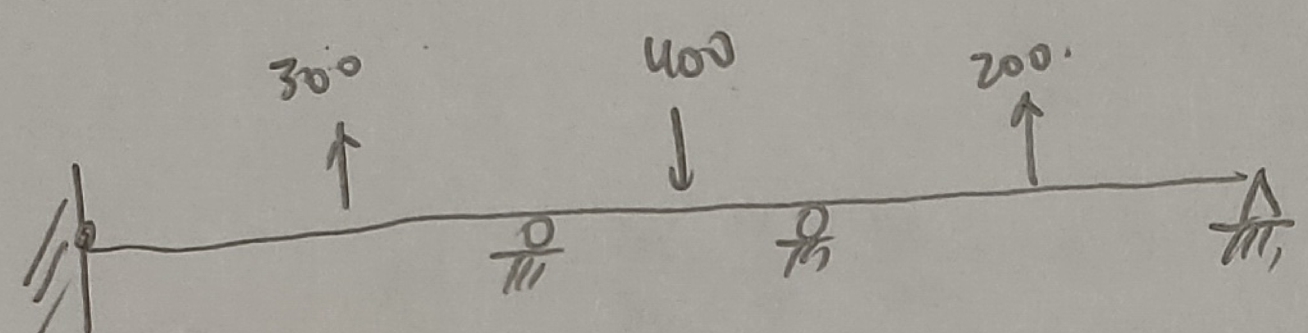
$$300(6\theta) + 400(4\theta) + 200(8\theta)$$

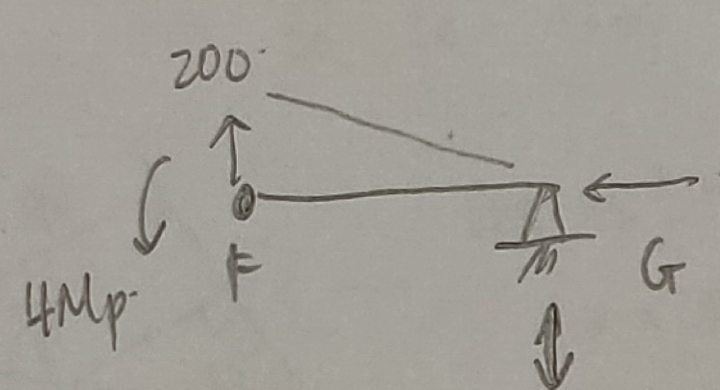
$$= 2M_p(\theta) + 2M_p(2\theta) + 3M_p(2\theta) + 4M_p(2\theta)$$

$$5000 = 20M_p$$

$$M_p = 250 \text{ kNm}$$

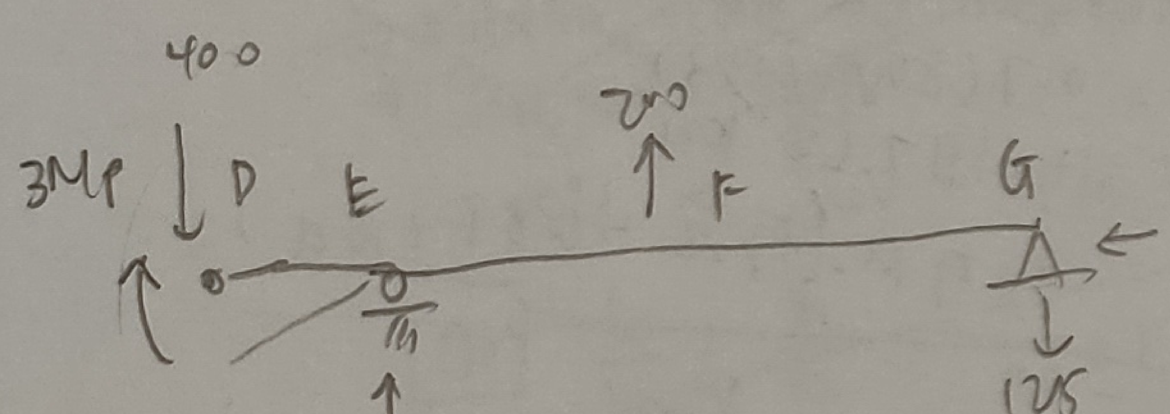






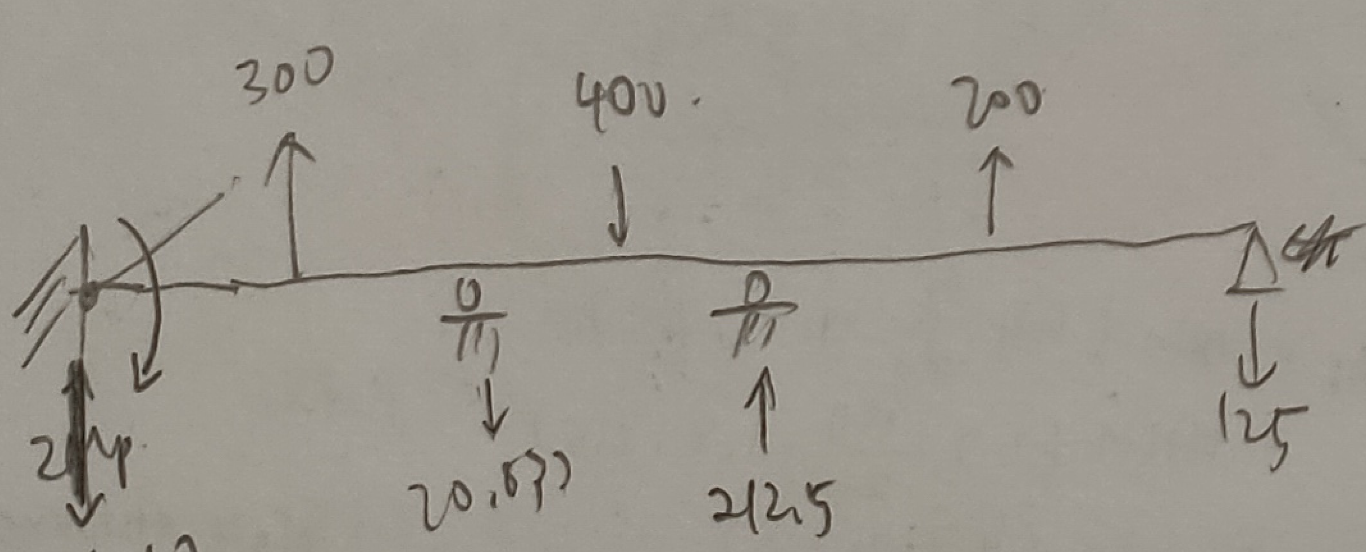
$\Sigma M_F = 0$ $\quad G_y(8) = 4M_p$

$$G_y = 125 \text{ kN}$$



$\Sigma M_D = 0$ $\quad 125(20) + 3M_p = 200(12) + E_y(4)$

$$E_y = 212.5 \text{ kN}$$



$\Sigma M_A = 0$ $\quad 300(6) + 200(28) + 212.5(20) = 400(16) + 125(36) + 2M_p + C_y(12)$

$$11650 = 11400 + C_y(12)$$

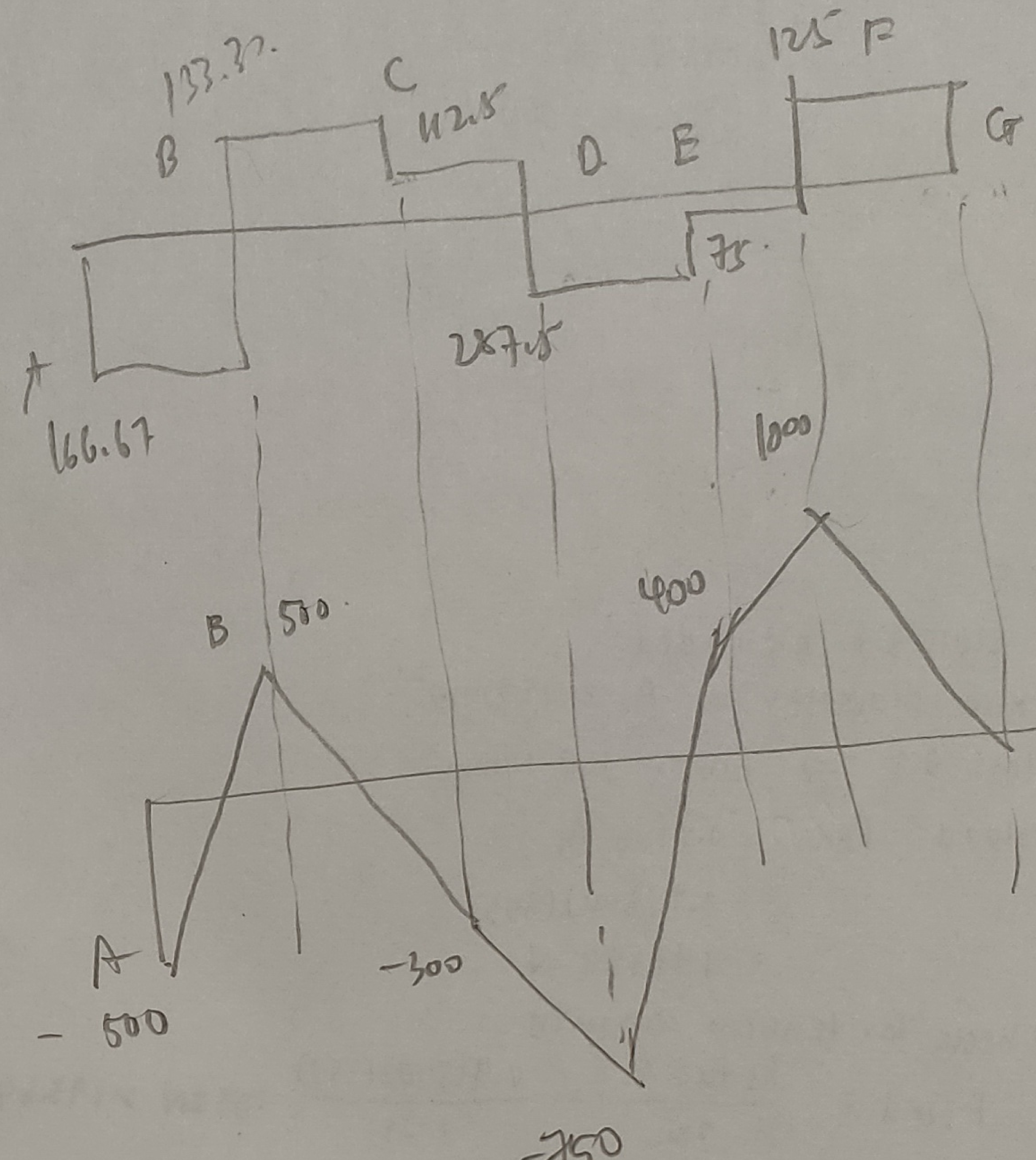
$$C_y = 20.833$$

$\Sigma M_G = 0$

$$200(8) + 300(30) + 212.5(16) + 2M_p = 20.833(24) + 400(20) + A_y(36)$$

$$14500 = 8500 + A_y(36)$$

$$A_y = 166.67$$



For S275,

$\varepsilon = \sqrt{235/275} = 0.924$

Flange $(300-20)/2$

$\frac{c_f}{t_f} = \frac{\cancel{150-20}}{30} = \cancel{4.33}\ 4.67 < 9\varepsilon = 8.316$

∴ class 1

Web

$\frac{c_w}{t_w} = \frac{1940}{20} = 97 < 124\varepsilon = 114.576$

∴ class 3

For class 1 Flange & Class 3 web

⇒ use Method 1

$M_{y,Rd} = W_{pl,f} f_{yf} + W_{el,w} f_{yw}$

$W_{pl,f} f_{yf} = A_f(h_w + t_f) f_{yf}$

$= (300\times30)(1940+30)(275)/10^6$

$= 4875.75$ kNm

$I_w = \frac{1940^3 \times 20}{12} = 1.217\times10^{10}\ mm^4$

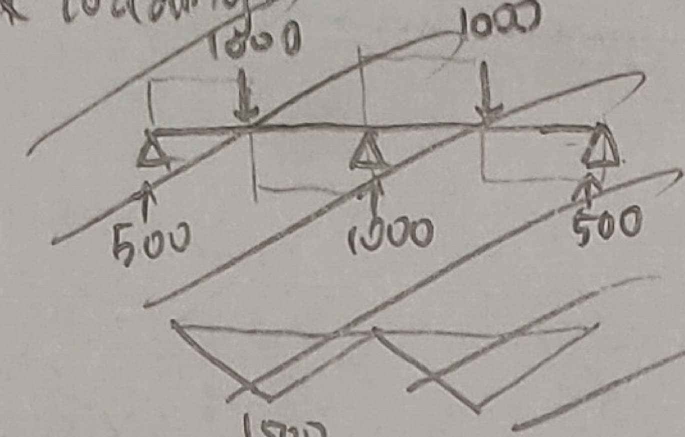
$W_{el,w} = \frac{I_w}{z} = \frac{1.217\times10^{10}}{1940/2} = 12545333.33\ mm^3$

$W_{el,w} f_{yw} = 3449.97$ kNm.

$M_{y,Rd} = 4875.75 + 3449.97 = 8325.72$ kNm $> \cancel{1500}\ 1125$ kNm.

ok!

* loadings



b) $a/h_w = 3000/1940 = 1.55 \geq 1$

$\Rightarrow k_\tau = 5.34 + 4\left(\frac{1940}{3000}\right)^2 = 7.013$

$\frac{h_w}{t_w} \leq 31\frac{\varepsilon}{\eta}\sqrt{k_\tau}$

$= \frac{1940}{20} = 97 \leq 31\frac{(0.924)}{1}\sqrt{7.013} = 75.86$

web is NOT stocky.

$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon\sqrt{k_\tau}} = \frac{1940}{37.4(20)(0.924)\sqrt{7.013}}$

$= 1.06$

$0.83/\eta \leq \bar{\lambda}_w < 1.08$

Rigid End Post,

$\chi_w = 0.83/\bar{\lambda}_w = 0.783$

$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}}$

$= \frac{0.783(275)(1940)(20)}{\sqrt{3}(1.0)}$

$= 4823.54$ kN $> \cancel{500}$ kN $\cancel{1125}$ 687.5 ok!

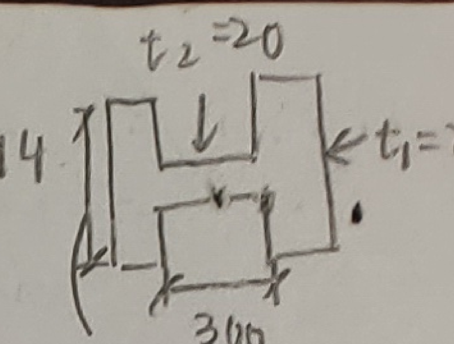
$c = \cancel{3000}\ [\cancel{0.25 + 1.6(300)(30)^2(275)}$

* Assume no contribution from flange at support, $V_{Ed} = 500$ kN, $M_{Ed} = 0$

$\eta_3 = \frac{687.5}{4823.54} = 0.1425 < 0.5$

No need interaction check!

c) Rigid End Post



$e = 300 + \cancel{10+10} = 320 > 0.1 h_w = 194$ 300

$A_e = A_u = 270\times20 = 5400$

$\frac{4 h_w t_w^2}{e} = \frac{4\times1940\times20^2}{300}$

☆ → $= \cancel{9500}\ 10346.67$

Minimum requirement not satisfied!

Axial Force due to TFA.

$\bar{\lambda}_w = 1.06$.

$N_{st,ten} = V_{Ed} - \frac{1}{(\bar{\lambda}_w)^2}\frac{h_w f_{yw} t_w}{\sqrt{3}\gamma_{M1}}$

$= 500 - \frac{1}{1.06^2}\frac{1940\times275\times20}{\sqrt{3}(1.0)}$

$= -4982.7$ kN

∴ TFA not mobilized.

$N_{Ed} = \cancel{500+500}$ ~~kN~~ 312.5 kN.

$I_{st} = 2(270^3\times20/12) + 300\times20^3/12$

$= 65810000\ mm^4$!!

$A_{st} = 2(270\times20) + (\cancel{300}\ 270\times20)$

$= 16800\ mm^2$

$i_{st} = \sqrt{\frac{I_{st}}{A_{st}}} = 62.59$ mm

$\lambda_1 = 93.9\varepsilon = 86.764$.

$\bar{\lambda} = \frac{1940\ \ 0.75\ \cancel{h_w}}{62.59}\times\frac{1}{86.764} = 0.357 > 0.2$

$\phi = 0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]$ (α = 0.49)

$= 0.602$

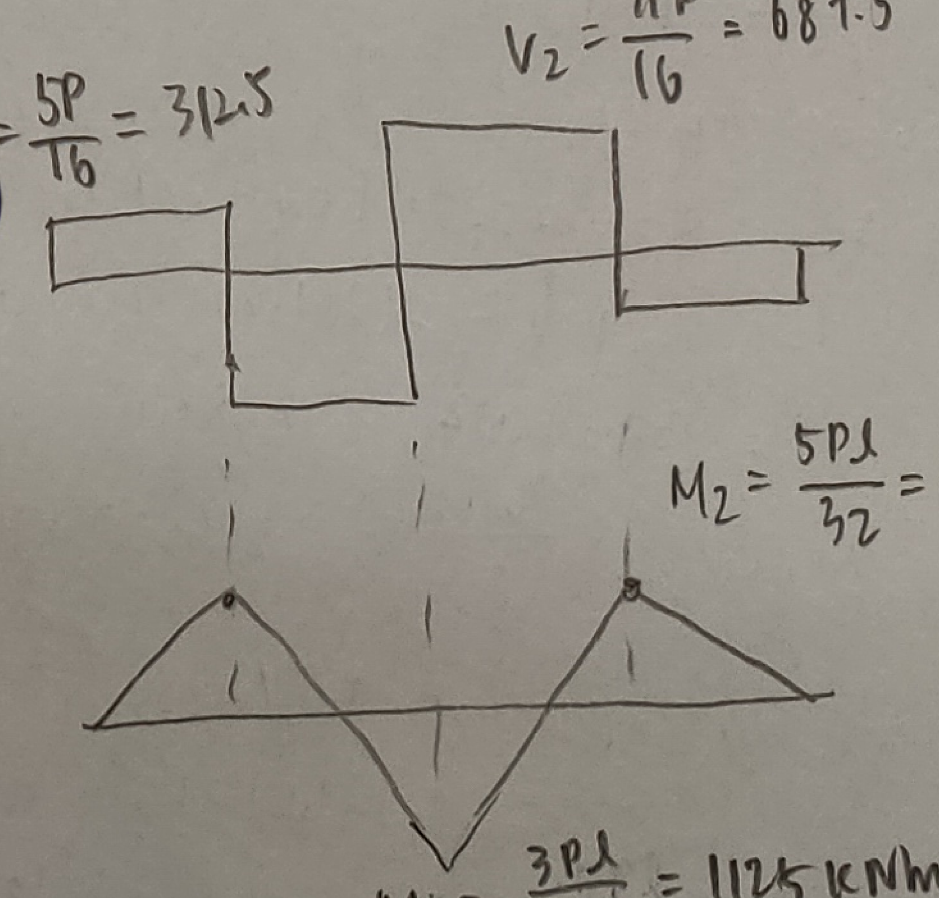
$\chi = \frac{1}{\phi+\sqrt{\phi^2-\bar{\lambda}^2}} = 0.92$

$N_{b,Rd} = 0.92\times A_{st}\times f_{yw}$

$= 4249.66$ kN $> \cancel{1000}\ 312.5$ kN ok!

end post is adequate.

* d).



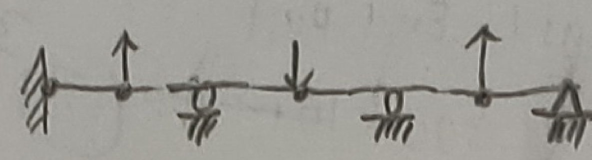
$V_1 = \frac{5P}{16} = 312.5$

$V_2 = \frac{11P}{16} = 687.5$

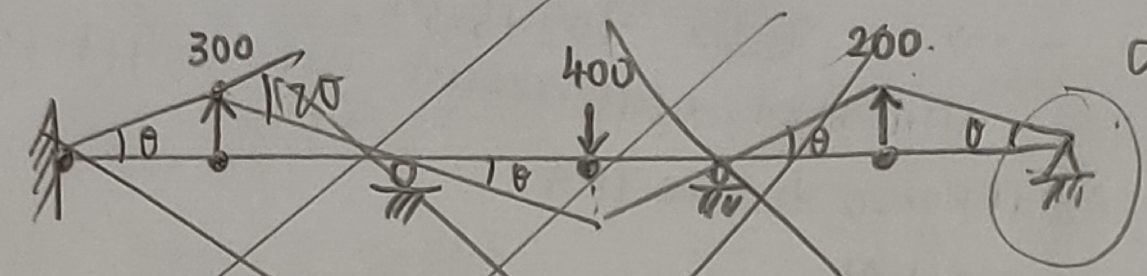
$M_2 = \frac{5PL}{32} = 937.5$

$M_1 = \frac{3PL}{16} = 1125$ kNm

4) a) $n_s = (3+1+1+2)-3$
$= 4$
$n_{ph} = n_s + 1 = 4+1 = 5$

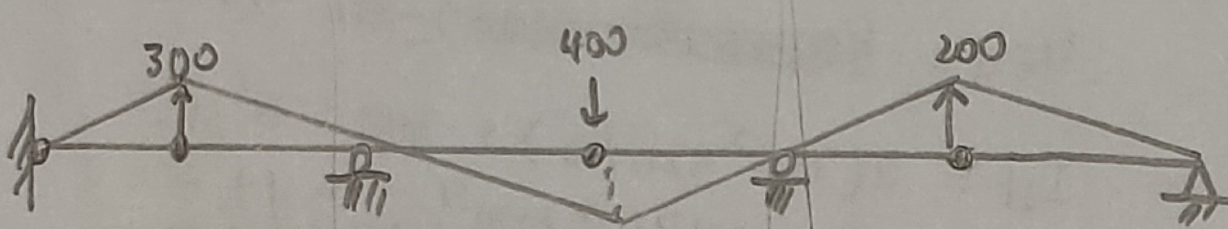


Mechanism 1 (A - - - - G)



$300(6\theta) + 400(4\theta) + 200(8\theta)$
$= \cancel{2Mp(\theta) + 2Mp(2\theta) + 3Mp}$

Mechanism 2 (A - B - D - F)



$300(6\theta) + 400(4\theta) + 200(8\theta)$
$= 2Mp(\theta) + 2Mp(2\theta) + 3Mp(2\theta) + 4Mp(2\theta)$

$5000 = 20Mp$
$Mp = 250$ kNm. (Hypothesis : correct mechanism)

b). $W_{pl} \geq \frac{4Mp}{f_y} = \frac{4(250)}{355} = 2816.9 \text{ cm}^3$

Use $533 \times 210 \times 109$, $W_{pl,y} = 2830 \text{ cm}^3$

$h_w = 539.5$ mm
$b_f = 210.8$ mm
$t_w = 11.6$ mm
$t_f = 18.8$ mm
$r = 12.7$ mm
$C_f/t_f = 4.62$
$C_w/t_w = 41.1$
$A = 139 \text{ cm}^2$

$\varepsilon = \sqrt{235/355} = 0.814$
$C_f/t_f = 4.62 < 9\varepsilon = 7.32$
$C_w/t_w = 41.1 < 72\varepsilon = 58.61$
class 1 web & flange

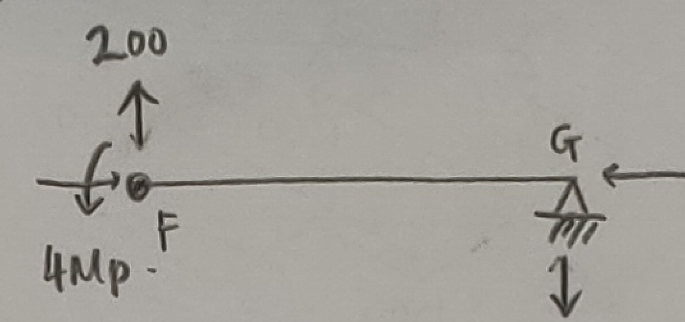
shear capacity
$A_v = A - 2b_f t_f + (t_w + 2r)t_f > h_w t_w$
$= 13900 - 2(210.8)(18.8) + (11.6 + 2(12.7))(18.8) > 539.5(11.6)$
$= 6669.52 > 6258.2$

$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}} = 1366.98 \text{ kN} > V_{max}$

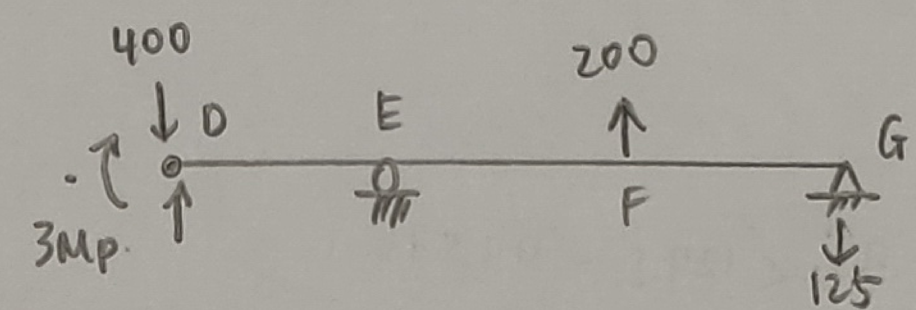
low shear → $0.5 V_{pl,Rd} > V_{max}$
no reduction in moment capacity!

UB $533 \times 210 \times 109$ is adequate!

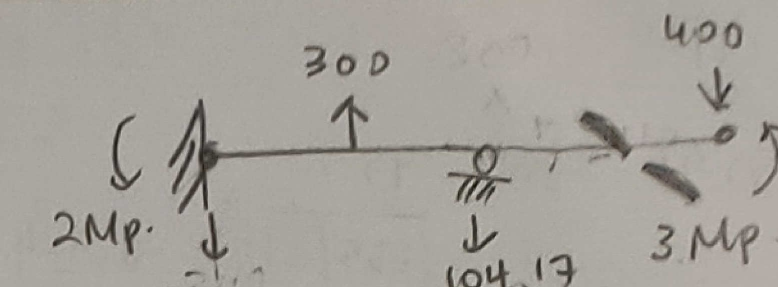
Equilibrium Check



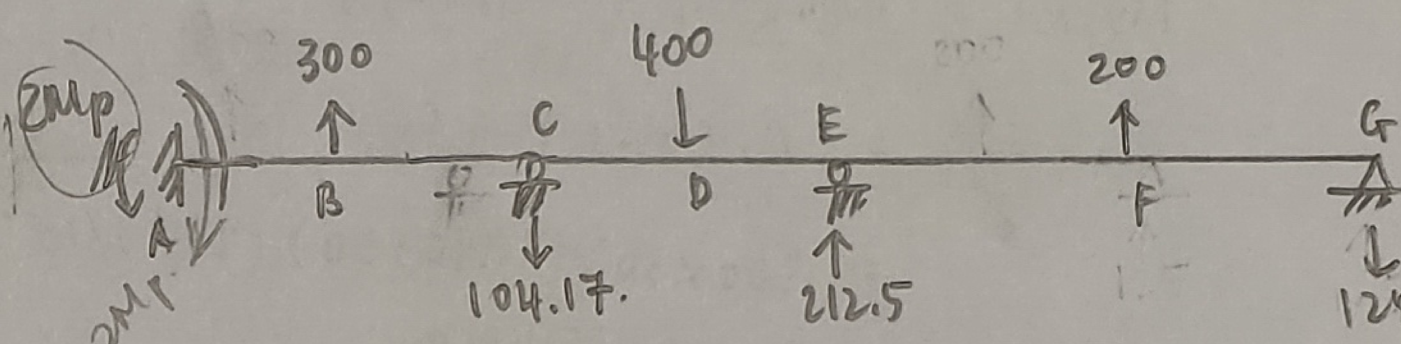
$\Sigma M_F = 0$ $G_y(8) = 4Mp$
$G_y = 125$ kN



$\Sigma M_D = 0$
$125(8+8+4) + 3Mp = 200(12) + R_E(4)$
$3250 = 2400 + 4R_E$
$R_E = 212.5$ kN.



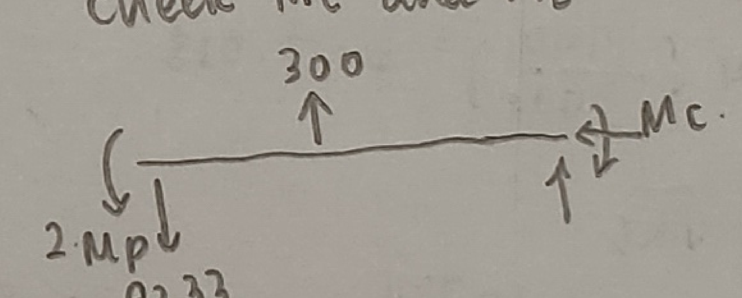
$\Sigma M_D = 0$
$5Mp + 104.17(4) = 300(10) + R_A(16)$



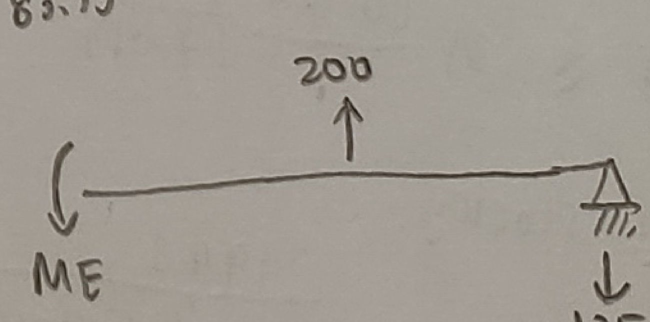
$\Sigma M_A = 0$
$125(36) + 400(16) + R_C(12) = 300(6) + 200(28) + 212.5(20) + 2Mp$
$R_C(12) = 1250$
$R_C = 104.17$ kN

$\Sigma F_y = 0$. $R_A = 300 - 104.17 - 400 + 212.5 + 200 - 125$
$= \cancel{83.33}$ kN //

Check Mc and ME



$\Sigma M_C = 0$
$2Mp + 83.33(12) + Mc = 300(6)$
$M_C = 300 \text{ kNm} < 500 \text{ kNm}$ (2Mp)



$\Sigma M_E = 0$
$125(16) = 200(8) + ME$
$ME = 400 \text{ kNm} < 750 \text{ kNm} = 3Mp$

plasticity satisfied. //