CV3011 2019/20 PYP.

1). For section 1-1 $\overset{C}{\underset{T}{\smile}}$ sag

Assume x above flange,

$F_{st} = F_{cc} + F_{sc}$

$0.87 f_{yk} A_{st} = 0.567 f_{ck} b (0.8x) + 0.87 f_{yk} A_{sc}$

$0.87 f_{yk} (2414) = 0.567(40)(500)(0.8x) + 0.87(500)(1006)$

$x = 67.513 \text{ mm} < h_f$ ∴ assumption correct

$\frac{d'}{d} = \frac{30}{510} = 0.0588 < 0.171$ ∴ comp steel yield!

$\frac{x}{d} = \frac{67.513}{510} = 0.132 < 0.617$ ∴ tension steel yield

$M_{Rd} = F_{cc}(d - 0.4x) + F_{sc}(d - d')$

$= 0.567(40)(500)(0.8 \times 67.513)(510 - 0.4 \times 67.513)$

$+ 0.87(500)(1006)(510 - 30)$

$= 505.88 \text{ kNm}$

For section 2-2 $\overset{T}{\underset{C}{\frown}}$

Assume x above flange,

$F_{cc} + F_{sc} = F_{st}$

$0.567(40)(250)(0.8x) + 0.87(500)(402)$

$= 0.87(500)(1571)$

$x = 112.1 \text{ mm}$

$\frac{d'}{d} = \frac{30}{520} = 0.0577 < 0.171$ ∴ ~~tension~~ comp. steel yielded

$\frac{x}{d} = \frac{112.1}{520} = 0.216 < 0.617$ ∴ tension steel yielded

$M_{Rd} = F_{cc}(d - 0.4x) + F_{sc}(d - d')$

$= 0.567(40)(250)(0.8 \times 112.1)(520 - 0.4 \times 112.1) + 0.87(500)(402)(520-30)$

$= 327.3 \text{ kNm}$

Permanent load

= self weight

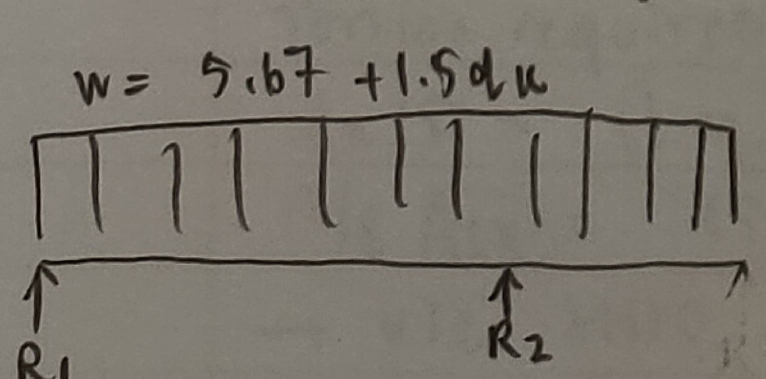
$= [(500 \times 150) + (400 \times 250)] \times 24$

$= 4.2 \text{ kN/m}$

Load $= 1.35 G_k + 1.5 Q_k$

$= 5.67 + 1.5 q_k$

$w = 5.67 + 1.5 q_k$



Take moment @ R_2 $\sum M_{R2} = 0$

$6R_1 + w \times 2 \times 1 = w \times 6 \times 3$

$R_1 = \frac{8}{3} w$

$\sum F_y = 0$ $R_2 = 8w - \frac{8}{3} w$

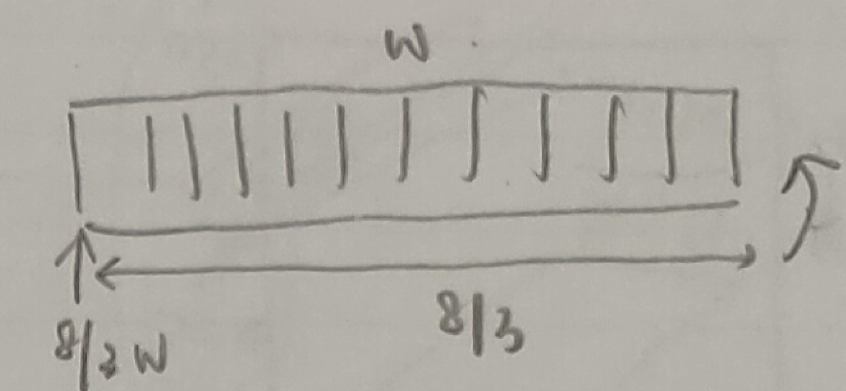
$= \frac{16}{3} w$

Max sag moment location

⇒ zero shear location

$= \frac{8}{3} w \div w$

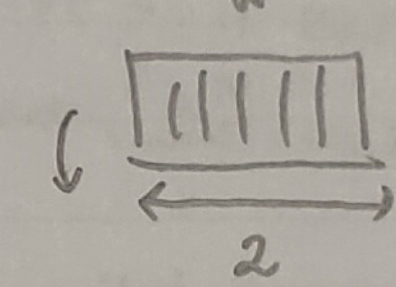
$= \frac{8}{3} = 2.67$



$M_{max} = (\frac{8}{3} w)(\frac{8}{3}) - (\frac{8}{3})^2 w / 2$

$M_{max} = \frac{32}{9} w$

at R_2,



$M = w(2)^2/2$

$= 2w$

$M_{max} < 505.88$

$\frac{32}{9} w < 505.88$

$w < 142.28$

$5.67 + 1.5 q_k < 142.28$

$q_k < 91.07 \text{ kN/m}$

$M_{max R2} < 327.3$

$2w < 327.3$

$w < 163.65$

$5.67 + 1.5 q_k < 163.65$

$q_k < 105.32 \text{ kN/m}$

∴ $q_{k,max} = 91.07 \text{ kN/m}$ //

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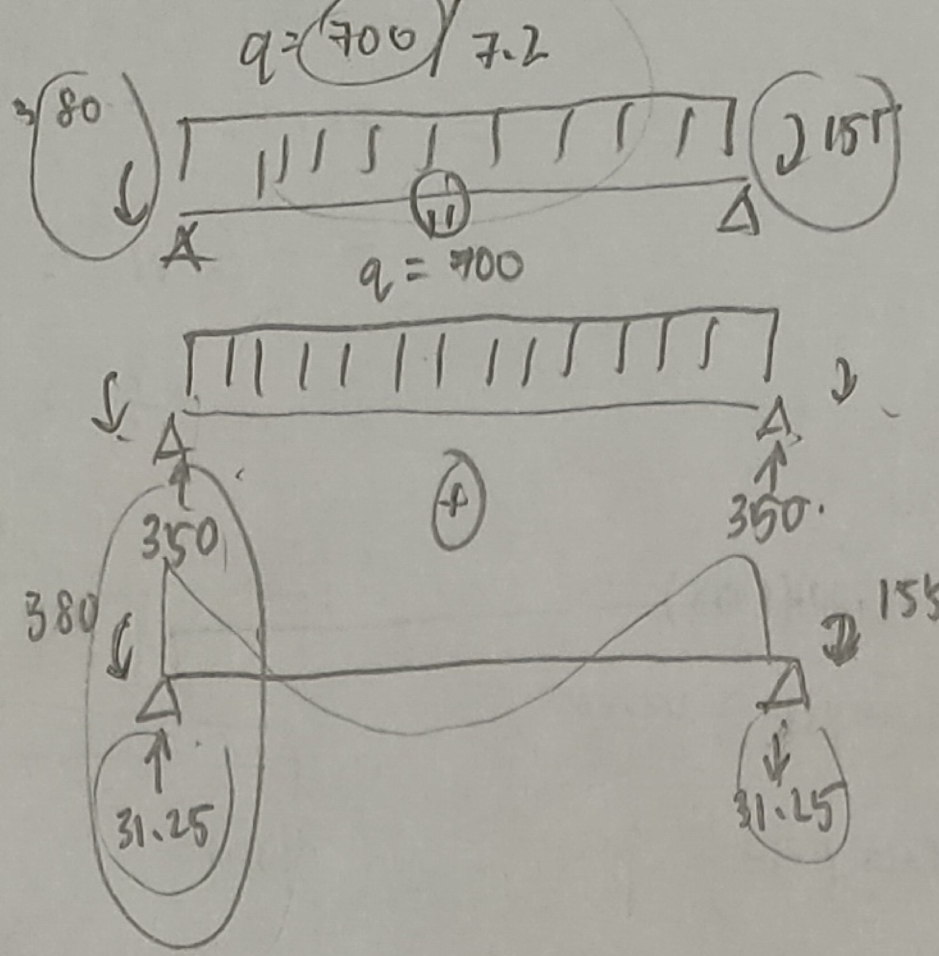
2) $F = 700\ kN$

$M_L = 380\ kNm$

$M_R = 155\ kNm$

$L = 7.2\ m$



$R_1 = 350 + 31.25 = 381.25$

$R_2 = 350 - 31.25 = 318.75$

LHS

Zone 1, at support face, $V_{Ef} = 381.25 - (\frac{700}{7200}) \times \frac{350}{2} = 364.24\ kN$

$V_{Rd,max\,(22)} = 0.124\, b_w d\, (1 - f_{ck}/250)\, f_{ck}$

$= 0.124\,(300)(600)\,(1 - 25/250)(25)$

$= 502200\ N = 502.2\ kN > 364.24\ kN$

Since $V_{Ef} < V_{Rd,max(22)}$, then $\theta = 22°$ & $\cot\theta = 2.5$

Zone 2, at 1d from support face,

$V_{Ed} = 381.25 - \frac{700}{7200} \times (\frac{350}{2} + 600) = 305.9\ kN$

$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78 d f_{yk} \cot\theta} = \frac{305.9 \times 10^3}{0.78(600)(500)(2.5)} = 0.523$

→ choose $\phi 8$ @ 175 spacing ⇒ $A_{sw}/s = 0.575 > 0.523$

Zone 3

$\frac{A_{sw,min}}{s} = \frac{0.08 f_{ck}^{0.5} b_w}{f_{yk}} = \frac{0.08(25)^{0.5}(300)}{500}$

$= 0.24$

⇒ choose $\phi 8$ @ 400 spacing ⇝ $\frac{A_{sw}}{s} = 0.251 > 0.24$

Actual shear resistance of stirrup is

$V_{min} = \frac{A_{sw}}{s} 0.78 d f_{yk} \cot\theta$

$= 0.251 \times 0.78 \times 600 \times 25 \times 2.5 \times 500$

$= 146.8\ kN$

For zone 1 & 2, no. of stirrup

⇒ $X_L = \frac{V_{Ef} - V_{min}}{w} = \frac{364.24 - 146.8}{700/7.2} = 2.24\ m$

no. of links $= 1 + \frac{X_L}{S_L} = 1 + \frac{2.24}{0.175} = 13.8$

$= 14$

$L_{left} = 13 \times 175 = 2275\ mm$.

RHS

If zone 1 at LHS (with higher support reaction) showed that $V_{Ef} < V_{Rd,max(22)}$, then RHS also similar ⇒ $\theta = 22°$, $\cot\theta = 2.5$

Zone 2

$V_{Ed} = 318.75 - (\frac{700}{7200}) \times (\frac{350}{2} + 600) = 243.4\ kN$

$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78 d f_{yk} \cot\theta} = \frac{243.4 \times 10^3}{0.78 \times 600 \times 500 \times 2.5} = 0.42$

⇒ choose $\phi 8$ @ 225 spacing $\frac{A_{sw}}{s} = 0.447$ (> 0.42).

no. of stirrup

⇒ $X_R = \frac{V_{Ef} - V_{min}}{w}$

$= \frac{243.4 - 146.8}{700/7.2}$

$= 0.9936$.

no. of links $= 1 + \frac{X_R}{S_R} = 1 + \frac{0.9936}{0.225}$

$= 5.416$

≈ 6.

$L_{right} = 5 \times 225 = 1125\ mm$.

no. of stirrup in zone 3

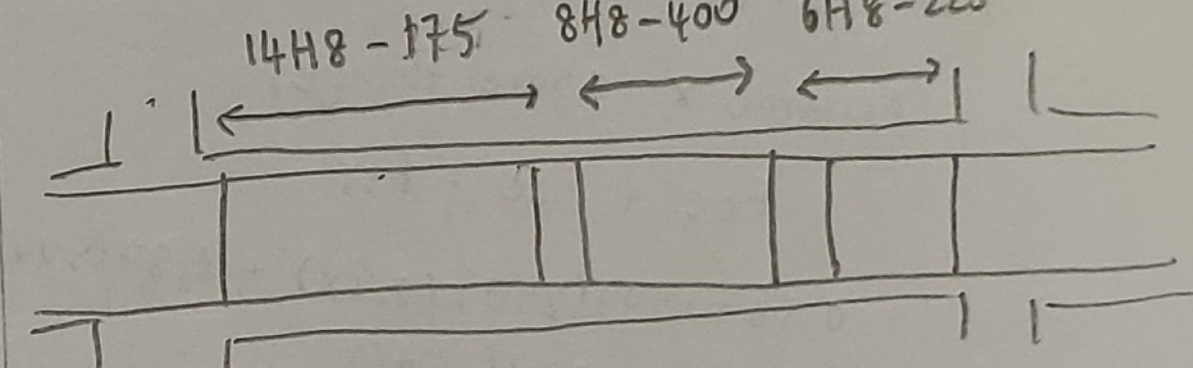
Length $= 7200 - 2(\frac{350}{2}) - 2275 - 1125$

$= 3450\ m$

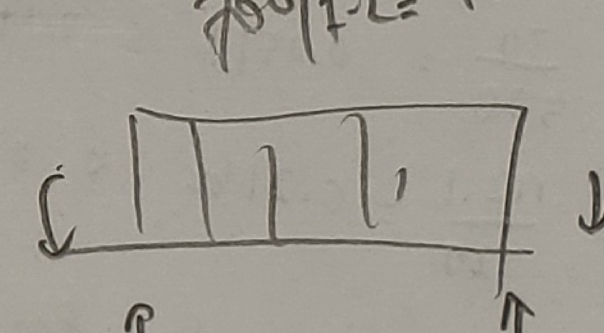
no. of links $= \frac{X}{s} - 1 = \frac{3450}{400} - 1$

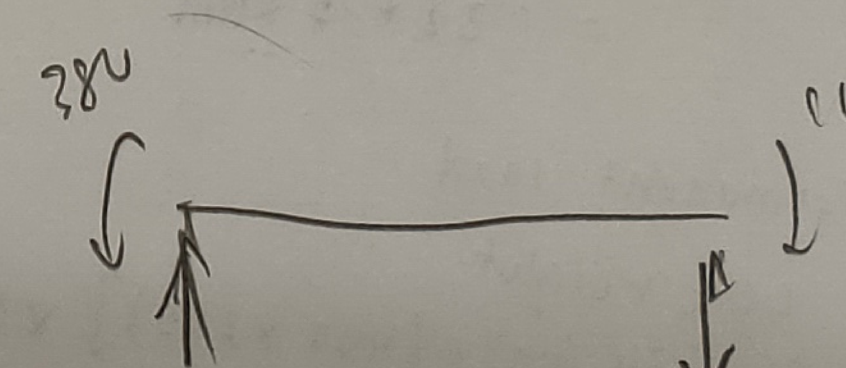
$= 7.625$

≈ 8.



$700/7.2 = 97.2$





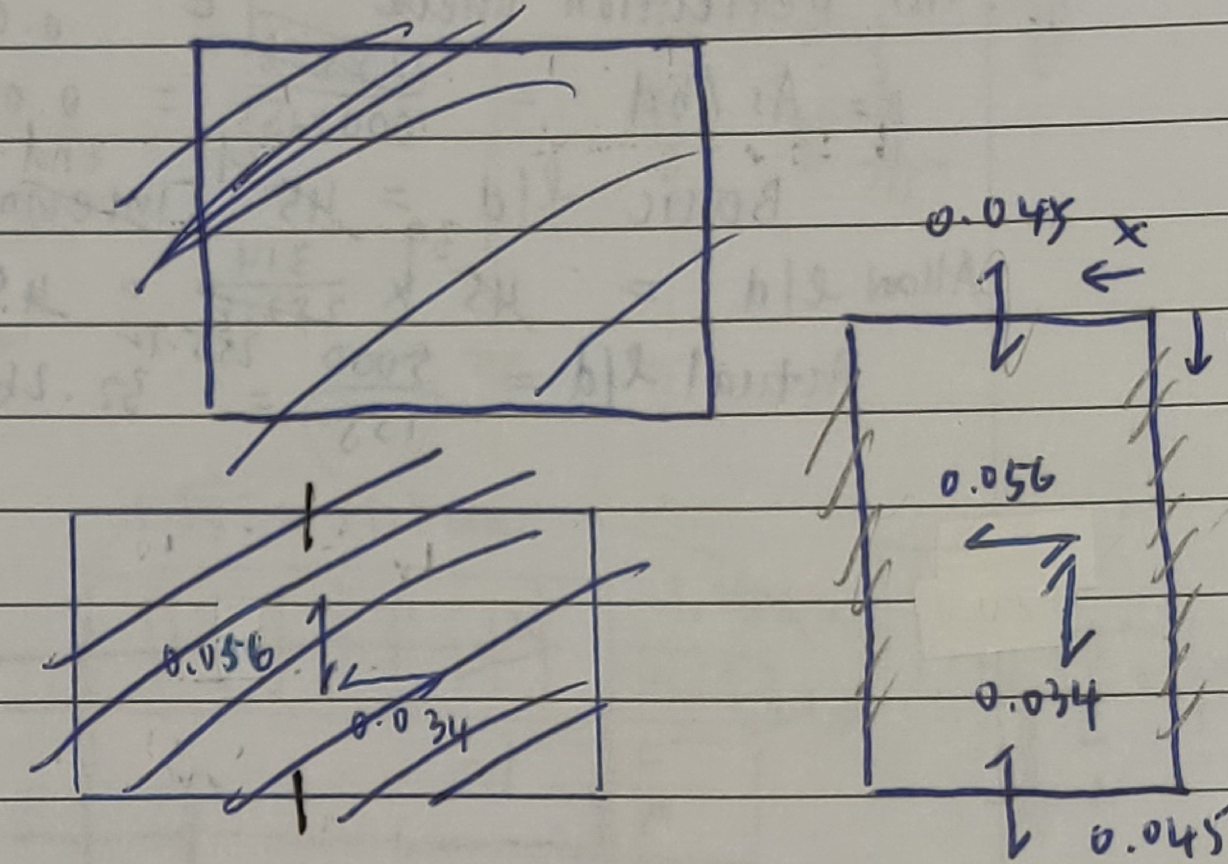
Date No.

3) a) Slab thickness = 190mm

$n = 1.35 \times 6 + 1.5 \times 2.4 = 11.7\ kN/m^2$

$d_x = 190 - 30 - 5 = 155mm$

$d_y = 155 - 10 = 145mm$



From Table 3.14, Case 6, $l_y/l_x = 6/5 = 1.2$

$\beta_{sx} = 0.056$

$\beta_{sy}' = 0.045$

$\beta_{sy} = 0.034$

$A_{s,min} = 0.026 \frac{f_{ctm}}{f_{yk}} bd$

$= 0.026 \left(\frac{0.3(40)^{2/3}}{500}\right) 1000 (155)$

$= 282.8\ mm^2$

$M_{sx} = \beta_{sx} \cdot n \cdot l_x^2 = 16.38\ kNm$

$M_{sy} = \beta_{sy} \cdot n \cdot l_x^2 = 9.945\ kNm$

$M_{sy}' = \beta_{sy}' \cdot n \cdot l_x^2 = 13.1625\ kNm$

$A_{sy\,min} = 0.26 \frac{f_{ctm}}{f_{yk}} bd$

$= 264.57\ mm^2$

x-dir

$K = 0.017 < 0.167$

$d = 0.985d < 0.95d$

$A_{s,req} = 255.72\ mm^2 < A_{s,min}$ ← use $A_{s,min}$?

⇒ Provide H10-250 ($A_{s,prov} = 314\ mm^2$)

0.25 A_{sx}

For discontinuous edges,

⇒ $A_{s,min}$

⇒ Provide H10-275 ($A_{s,prov} = 286\ mm^2$)

y-dir (midspan)

$K = 0.012 < 0.167$

$d = 0.989d < 0.95d$

$A_{s,req} = 165.97\ mm^2 < A_{s,min}$ ← use $A_{s,min}$?

⇒ Provide H10-275 ($A_{s,prov} = 286\ mm^2$) ←

y-dir (continuous edge)

$K = \frac{M}{f_{ck}bd^2} = 0.0157 < 0.167$

$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right) = 0.986d \times 0.95d$

$A_{s,req} = \frac{M}{0.87 f_{yk} z} = 219.66\ mm^2$

⇒ provide H10-250 ($A_{s,prov} = 314\ mm^2$)

☆ max bar spacing

$S_{max} = 2h \leq 250$

$= 2(190) \leq 250$

$= 380 \leq 250$

∴ $S_{max} = 250$

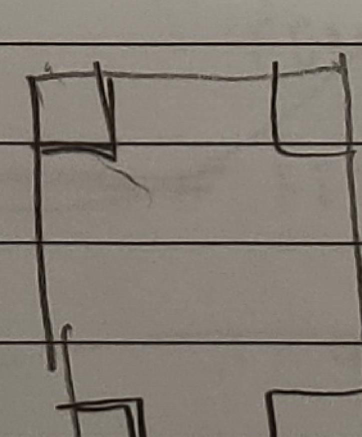
$8/5 = 1.0$

Torsion requirement $l_x/5$

$l_x/5 = 1m.$

$3/8\ A_{sx} = 95.9\ mm^2$

→ use 8H10 with 1000m width (=157mm²)



b) Deflection check

$\rho = A_s / bd = \dfrac{\cancel{282.8}\ 255.72}{1000 \times 155} = \cancel{0.00182}\ 0.00165 < 0.0035$ — $A_{s,req}$

Basic $l/d = \cancel{45}\ 39$ (~~interior~~ end span)

Allow $l/d = \cancel{45}\ 39 \times \dfrac{314}{\cancel{282.8}\ 255.72} = \cancel{49.96}\ 47.89$

Actual $l/d = \dfrac{5000}{155} = 32.26 < \cancel{49.96}\ 47.89$ ok!

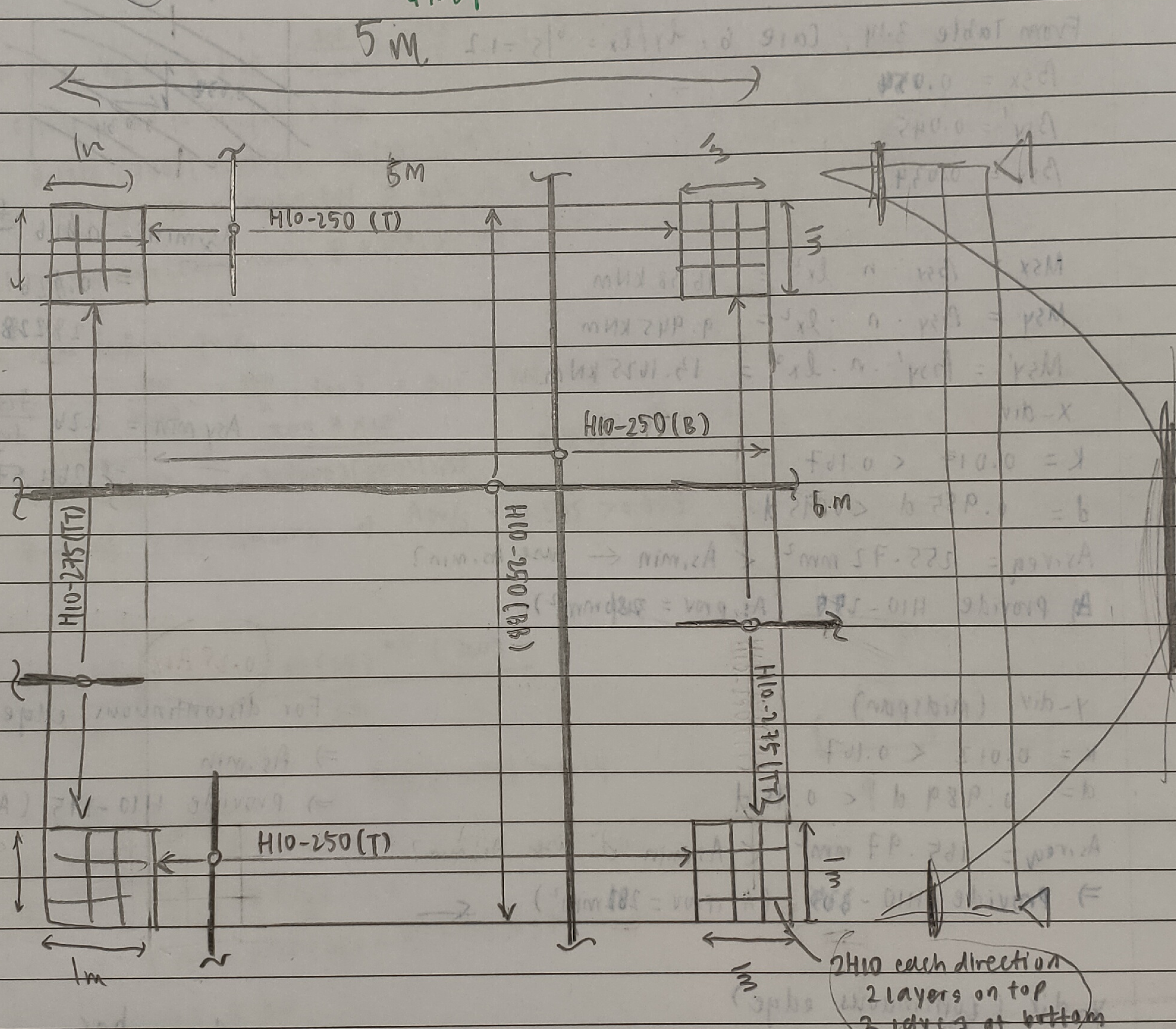
255.72

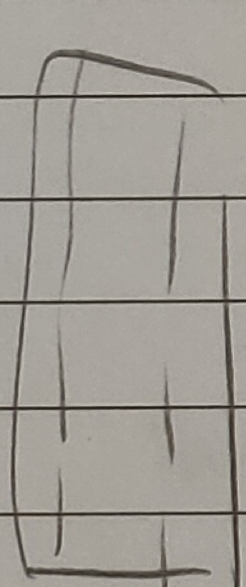
0.00165

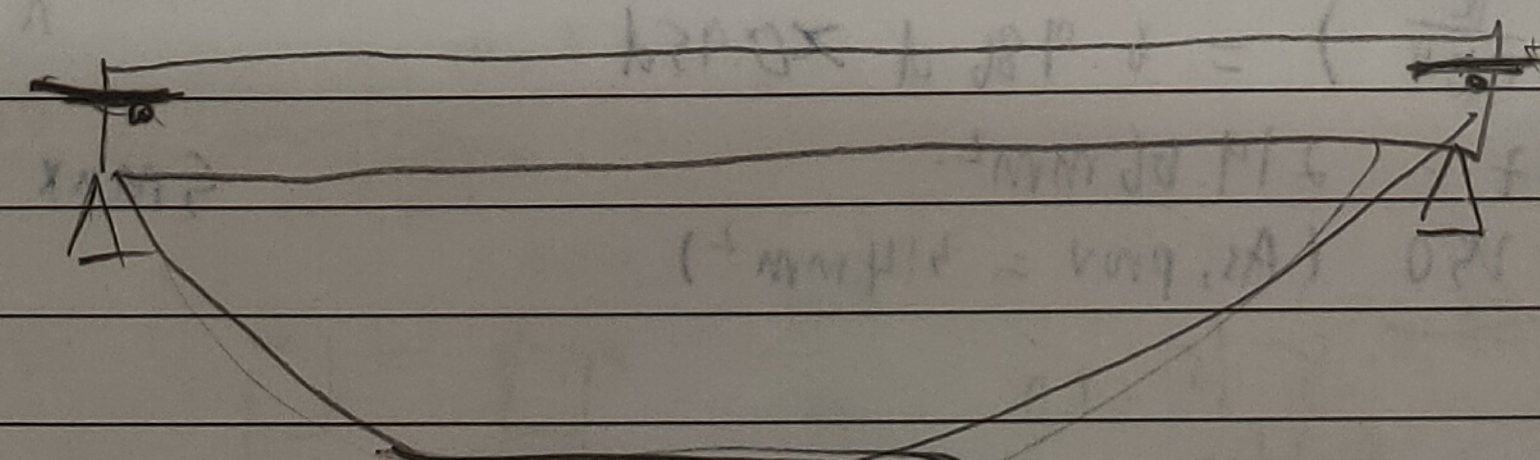
↓

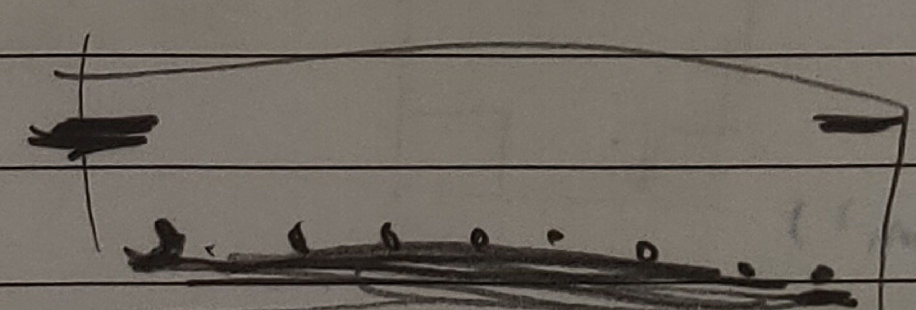
45

↓ $45 \times \dfrac{314}{255.72} = 55.26 > 32.26$









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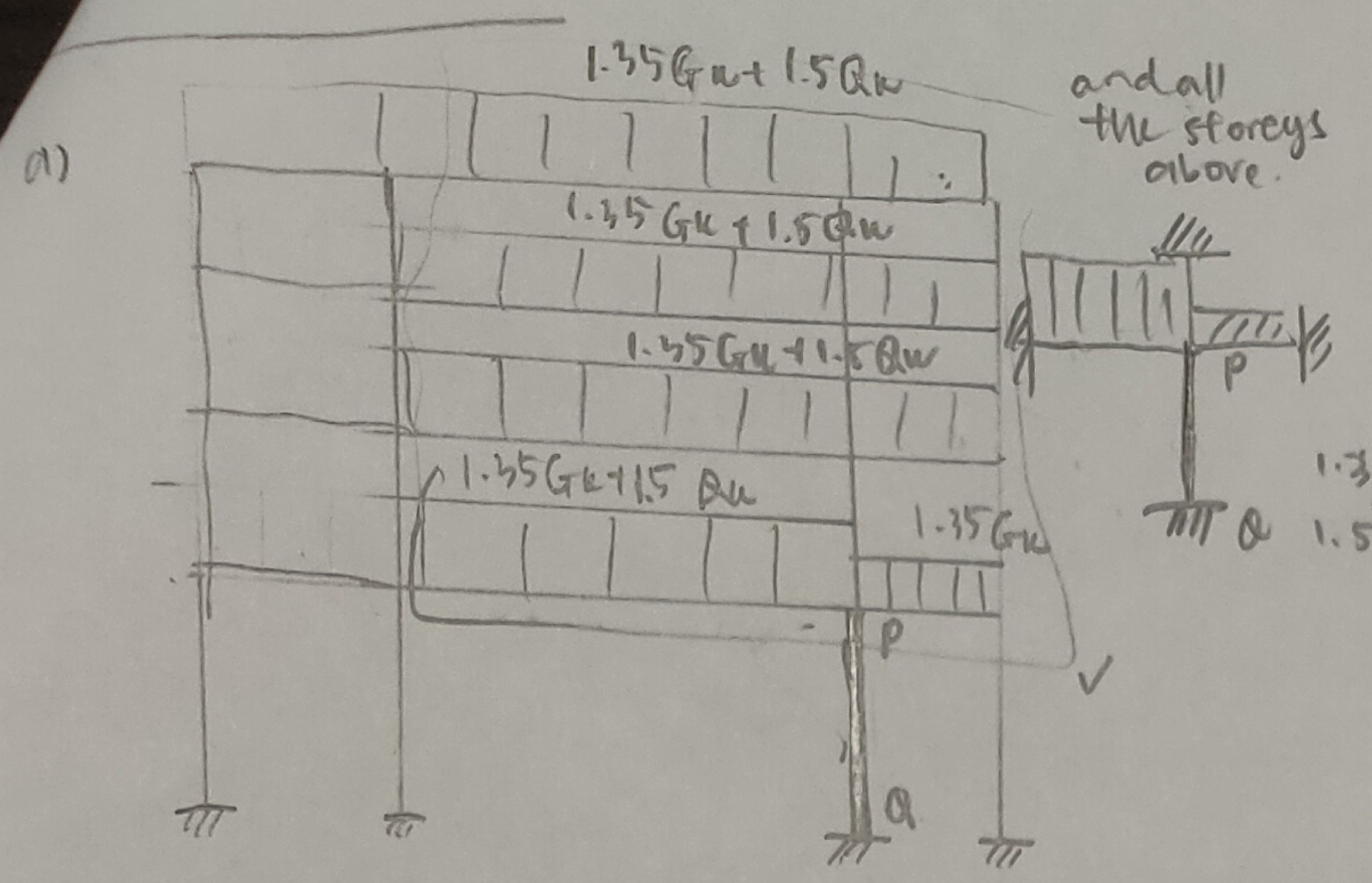
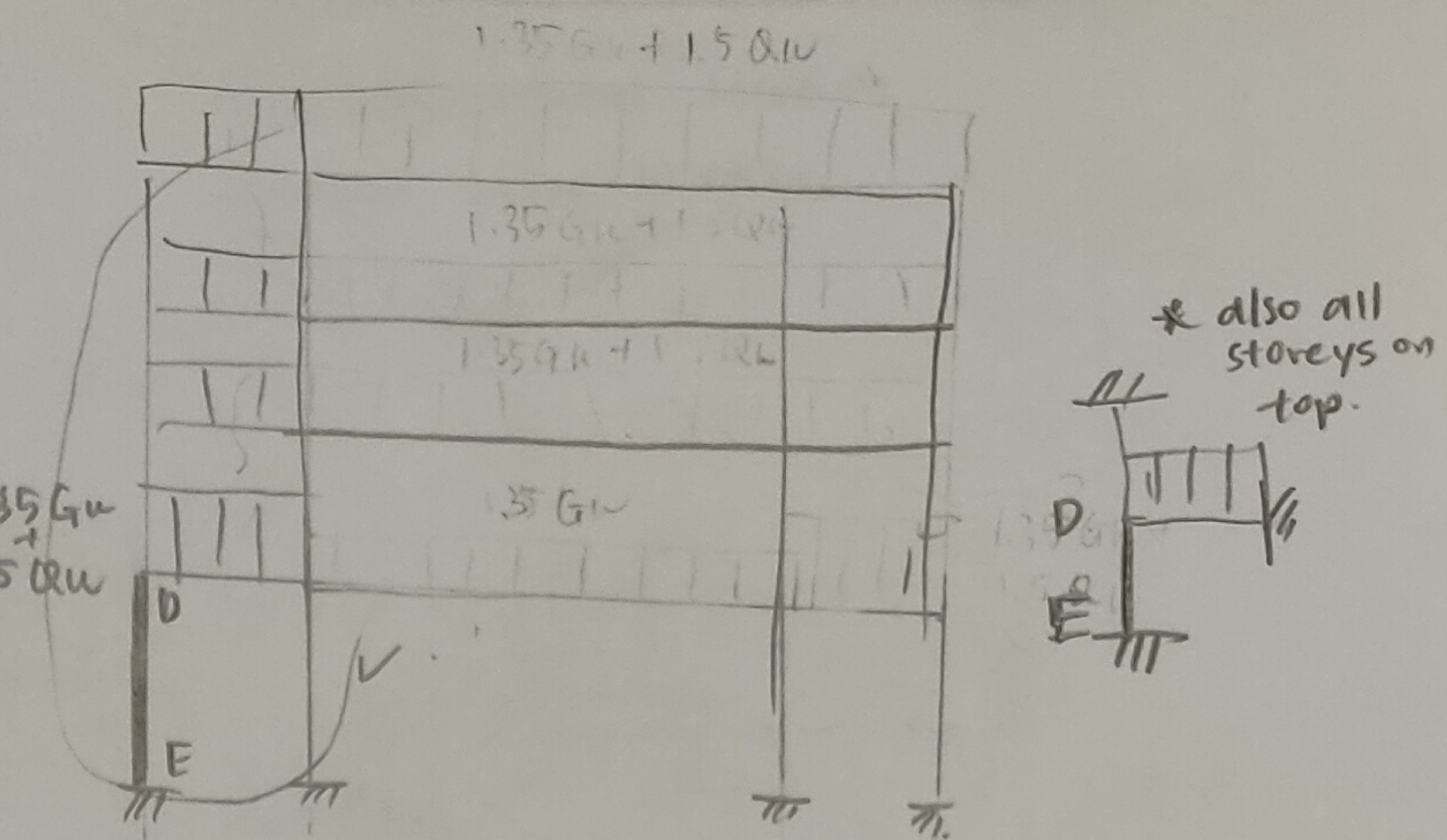
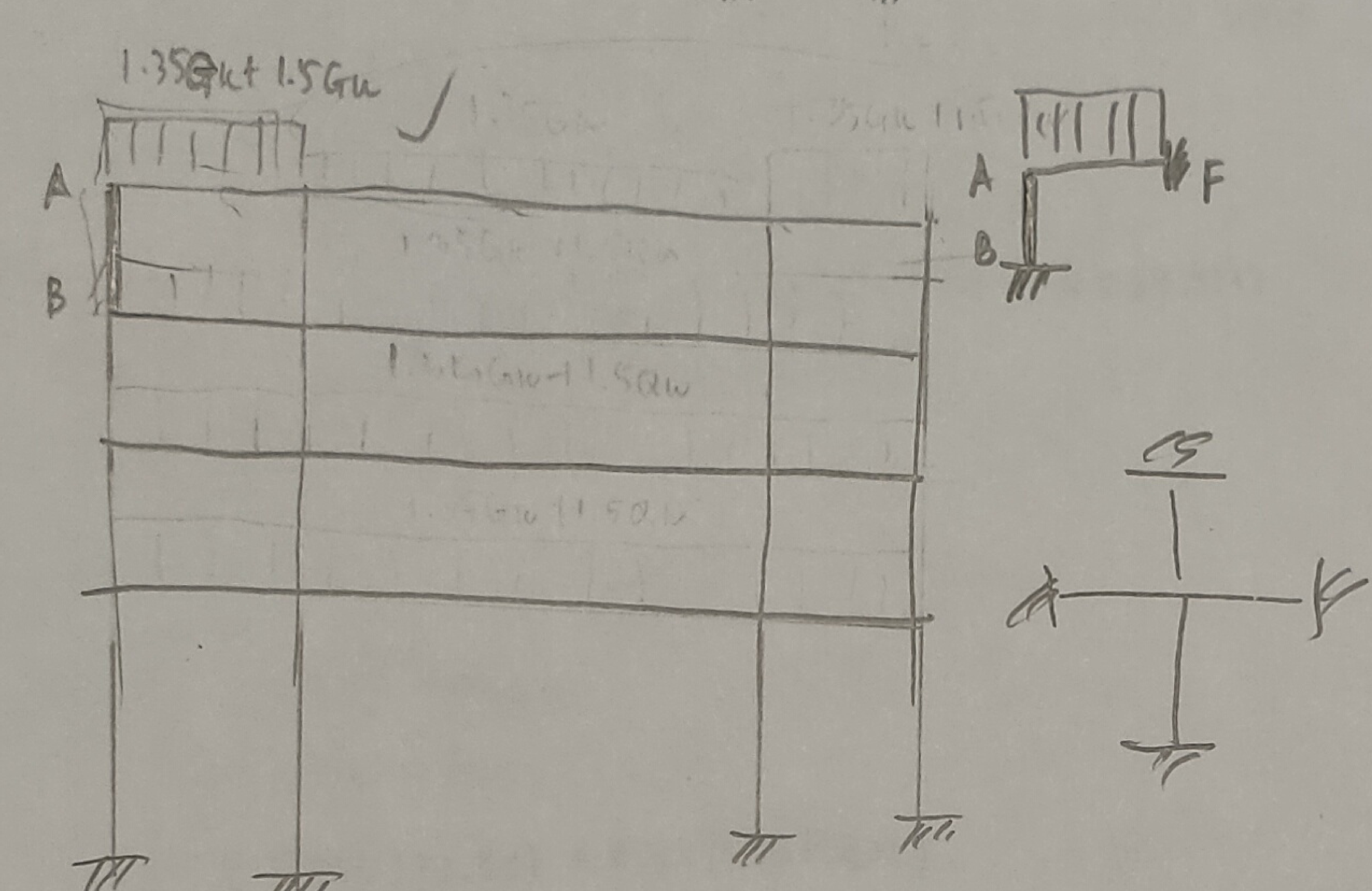
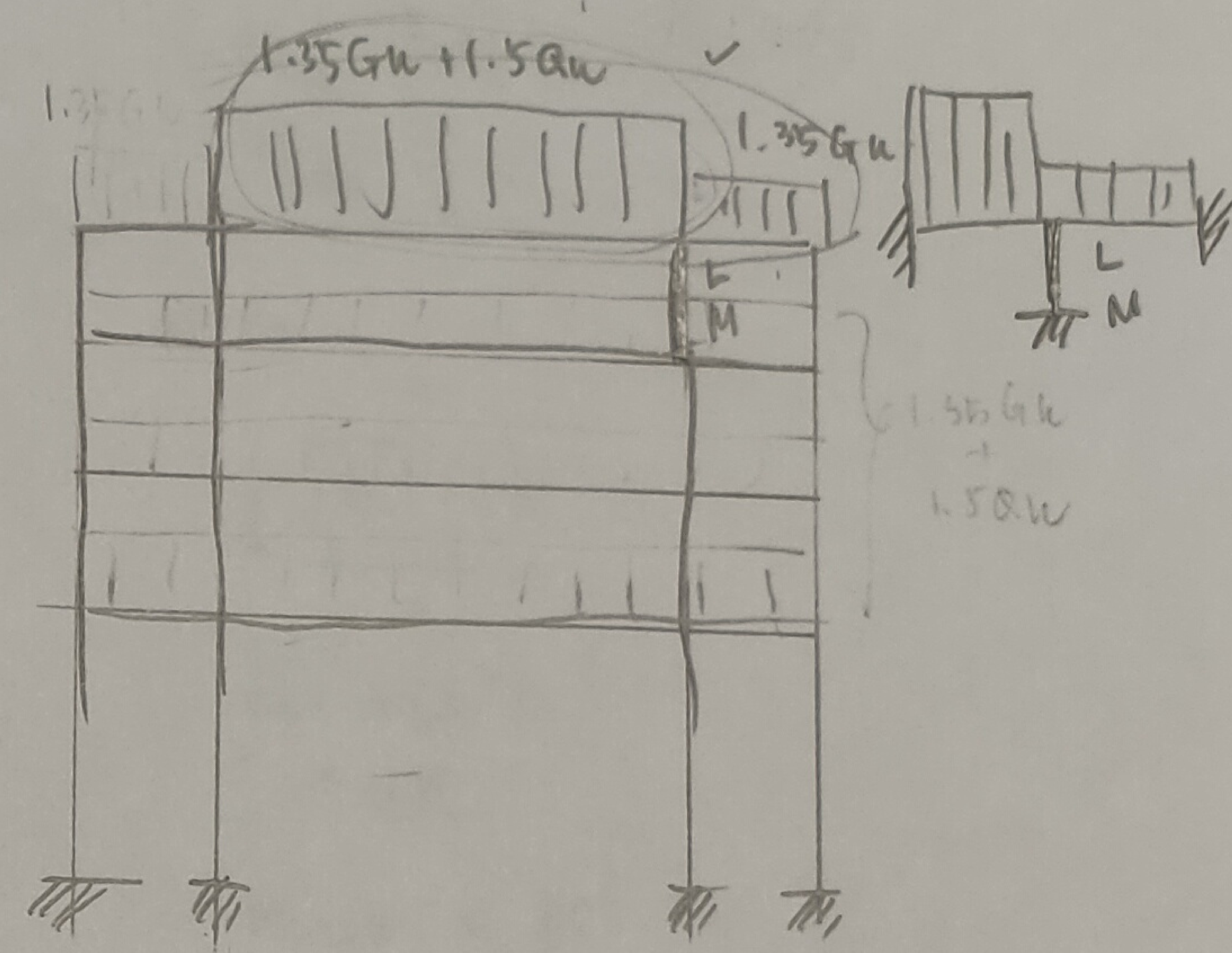
a)



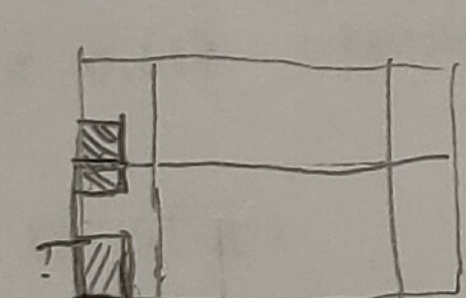


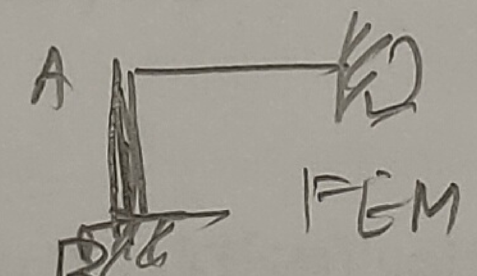


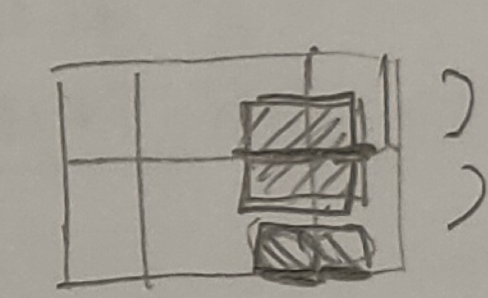


☆ (more axial load).

⇒ PQ will need more reinforcement as it is at the bottom most storey and needs to bear the weight of the storey above it while LM is at roof level hence does not need to bear as much load.







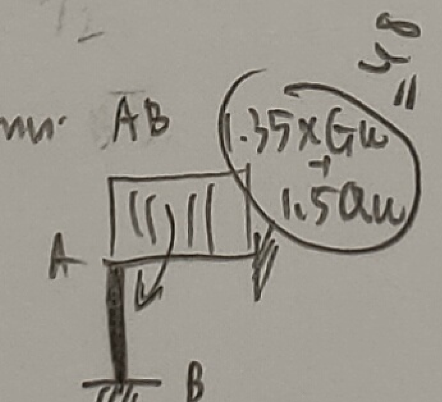
Column AB

b) Loadings :

$= (1.35 \times 16 + 1.5 \times 18) \times 4/2$

$= 97.2$ kN = total axial force in column AB

column self weight = 0 (at roof level)



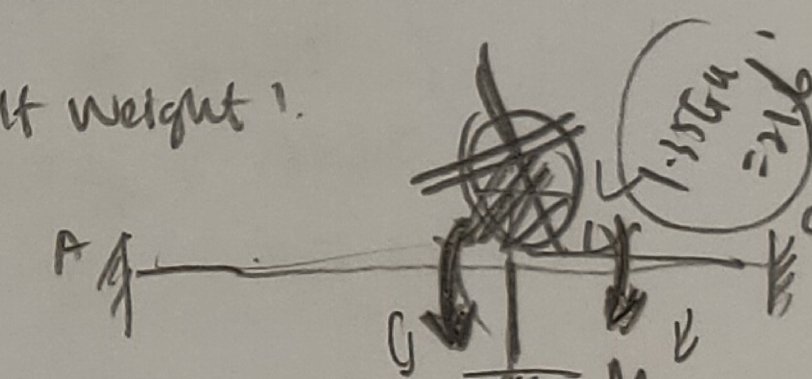
Column LM

Loadings

$= (1.35 \times 16) \times \frac{6+4}{2} + (1.5 \times 18) \times \frac{6}{2}$

$=$ 189 kN = total axial force on LM

(no column self weight!



Max moment

Max design load $= 1.35 \times 16 + 1.5 \times 18 = 48.6$ kN/m

Min design load $= 1.35 \times 16 = 21.6$ kN/m

Member stiffness

Columns : $k = \frac{EI}{L} = \frac{0.4}{3} = 0.133$

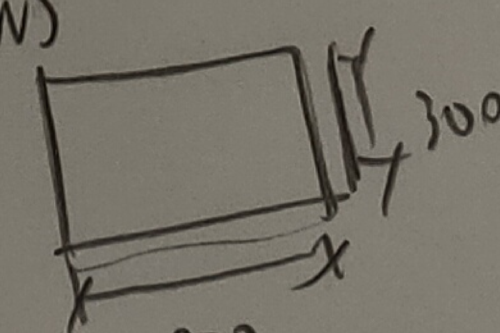
Beam $k = \frac{EI}{L} = \frac{1}{4}$

$\Sigma k = \boxed{0.133 + \frac{1}{4}(0.5)} = 0.258$

☆ no other FEM ??

$FEM_{AB} = \frac{1}{12}(48.6)(4)^2 = 64.8$ kNm (CW)

Moment $= 64.8 \times \frac{0.133}{0.258} = 33.4$ kNm.



$M_{min} = N \cdot e_{min} = 97.2 \times (h/30, 20)$

$= 1.94$ kNm < 33.4 $h = 300$mm

OK!

$FEM_{BA} = \frac{1}{12} \times 48.6 \times 6^2 = 145.8$ (CCW)

$FEM_{BC} = \frac{1}{12} \times 21.6 \times 4^2 = 28.8$ (CW)

column $\Rightarrow EI = 0.4 \Rightarrow k = \frac{EI}{L} = \frac{0.4}{3} = 0.133$

beam $\Rightarrow EI = 1 \Rightarrow k = \frac{EI}{L} \rightarrow \frac{1}{6}, \frac{1}{4}$

$\Sigma k = 0.133 + 0.5(\frac{1}{6}) + 0.5(\frac{1}{4})$

$= 0.341$

Moment $= (145.8 - 28.8) \times \frac{0.133}{0.341}$

$= 45.6$ kNm. (CCW)

$M_{min} = N \cdot e_{min}$

$= 189 \times \max(h/30, 20)$

$= 3.78$ kNm. < 45.6