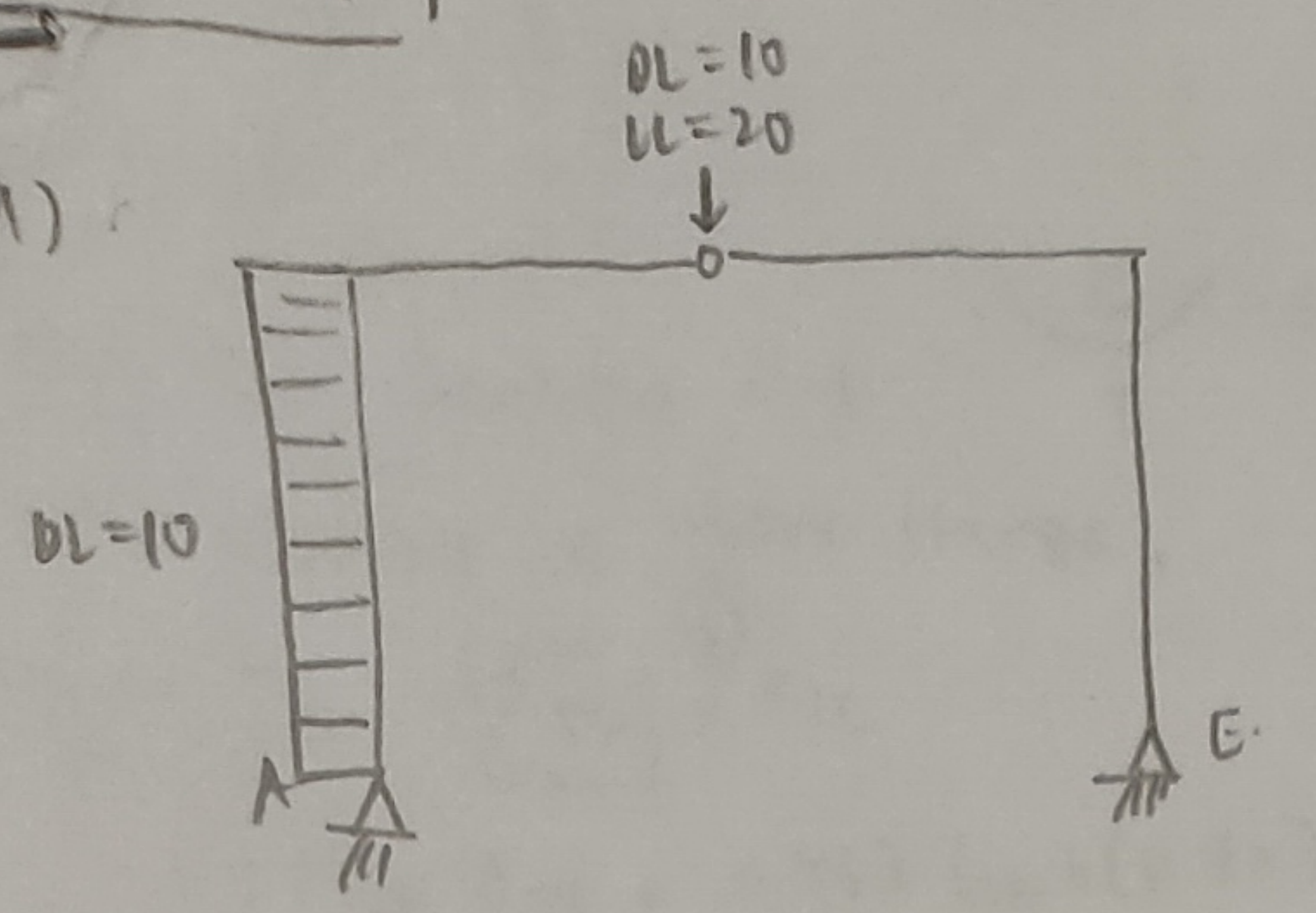
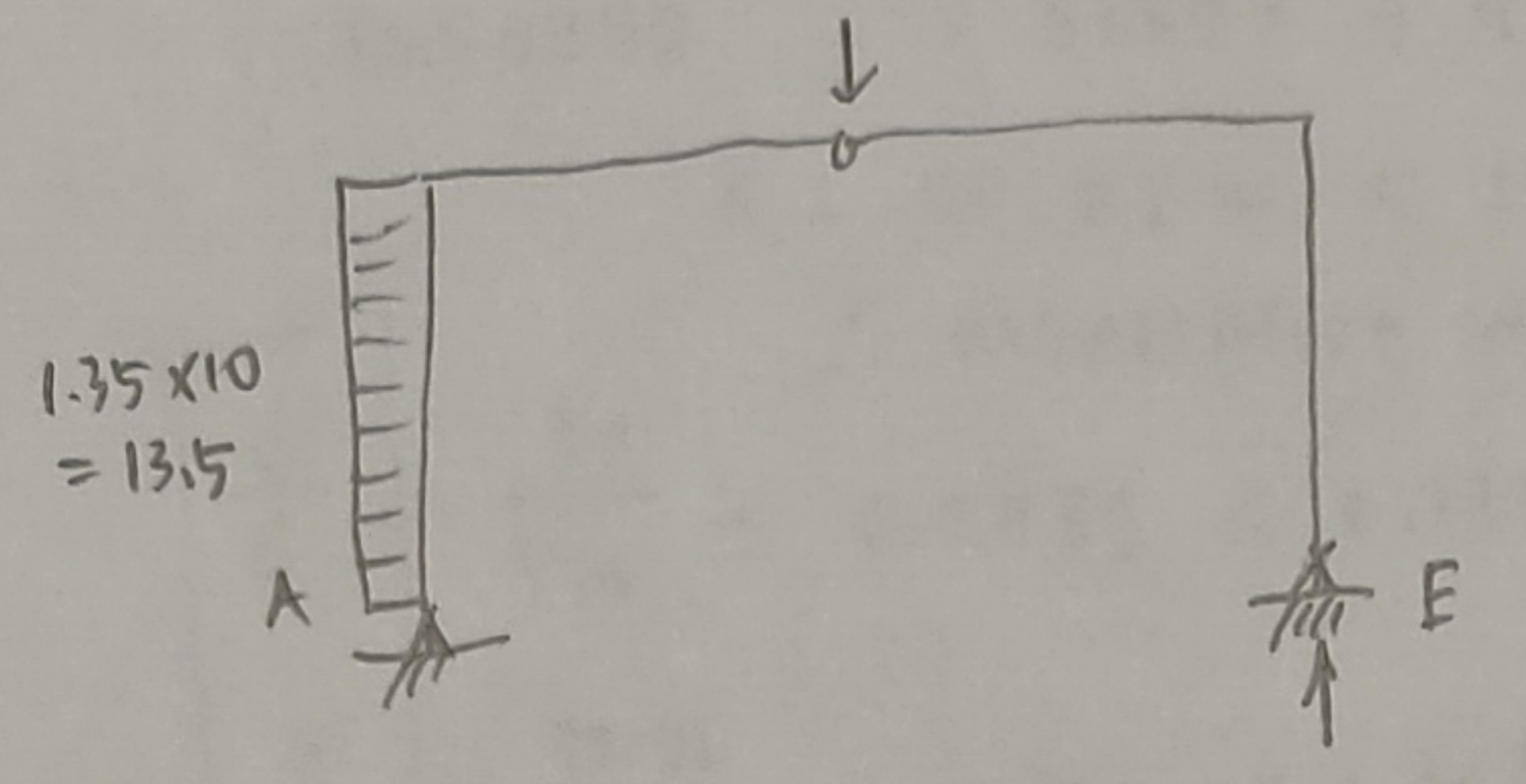
CV3011 2016/17

1) a)



i) For maximum upward reaction at E,

$1.35 \times 10 + 1.5 \times 20 = 43.5$



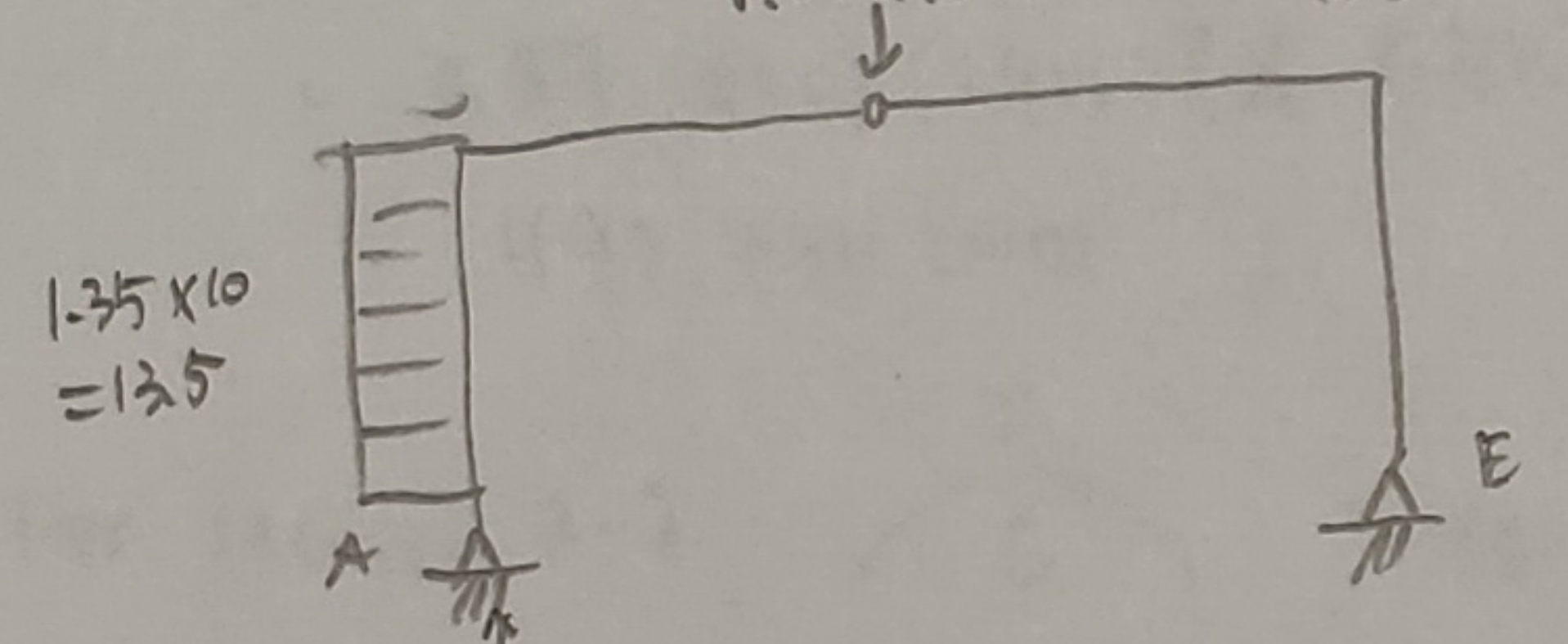
$\Sigma M_A = 0$

$F_E(6) = 43.5(3) + 13.5(3)(\frac{3}{2})$

$F_E = \underline{31.875}$ KN (↑)

ii) For upward reaction at A

$1.35 \times 10 + 1.5 \times 20 = 43.5$



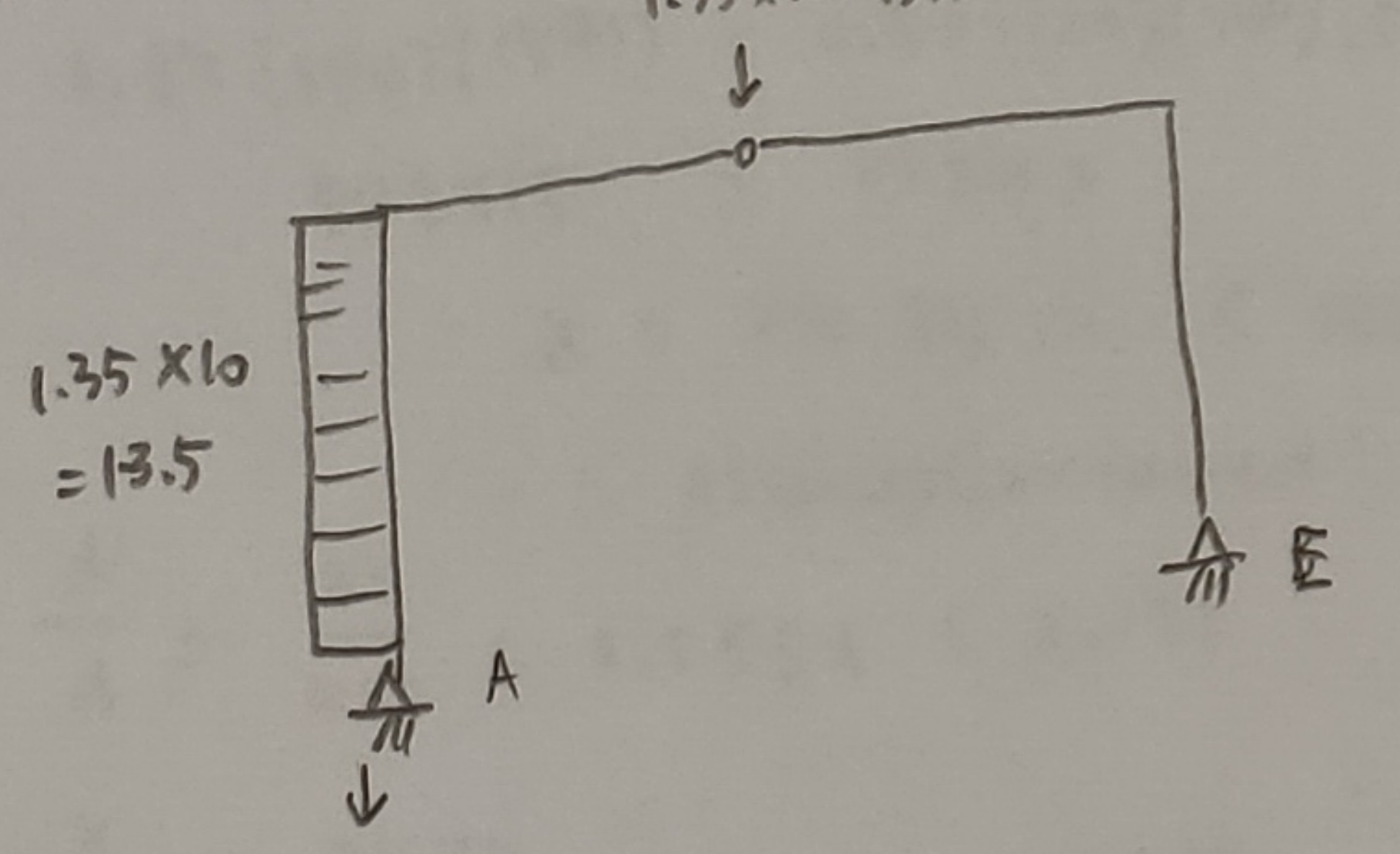
$\Sigma M_E = 0$

$F_A(6) + 13.5(3)(3/2) = 43.5(3)$

$F_A = \underline{11.625}$ kN (↑)

iii) For maximum downward reaction at A

$1.35 \times 10 = 13.5$

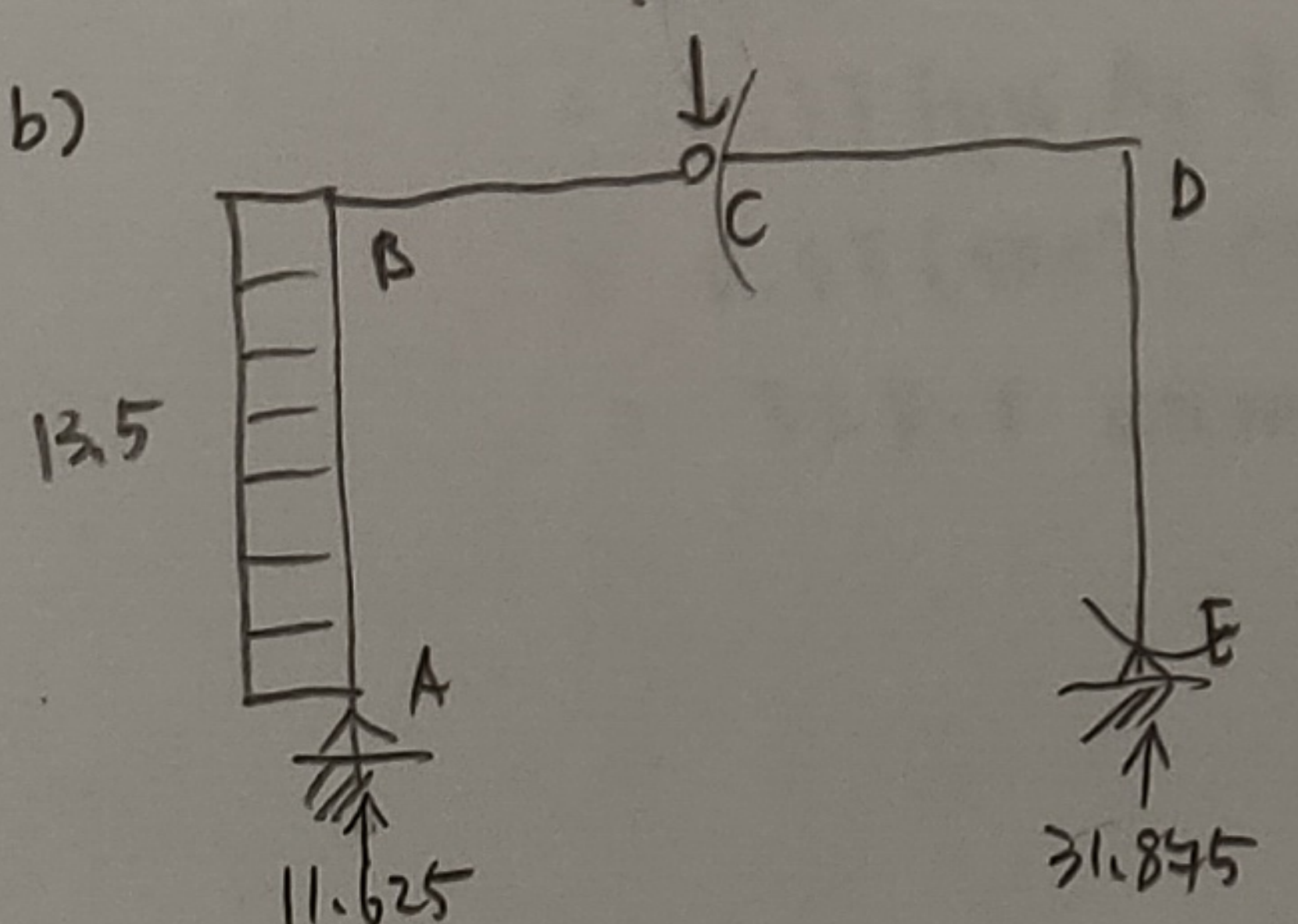


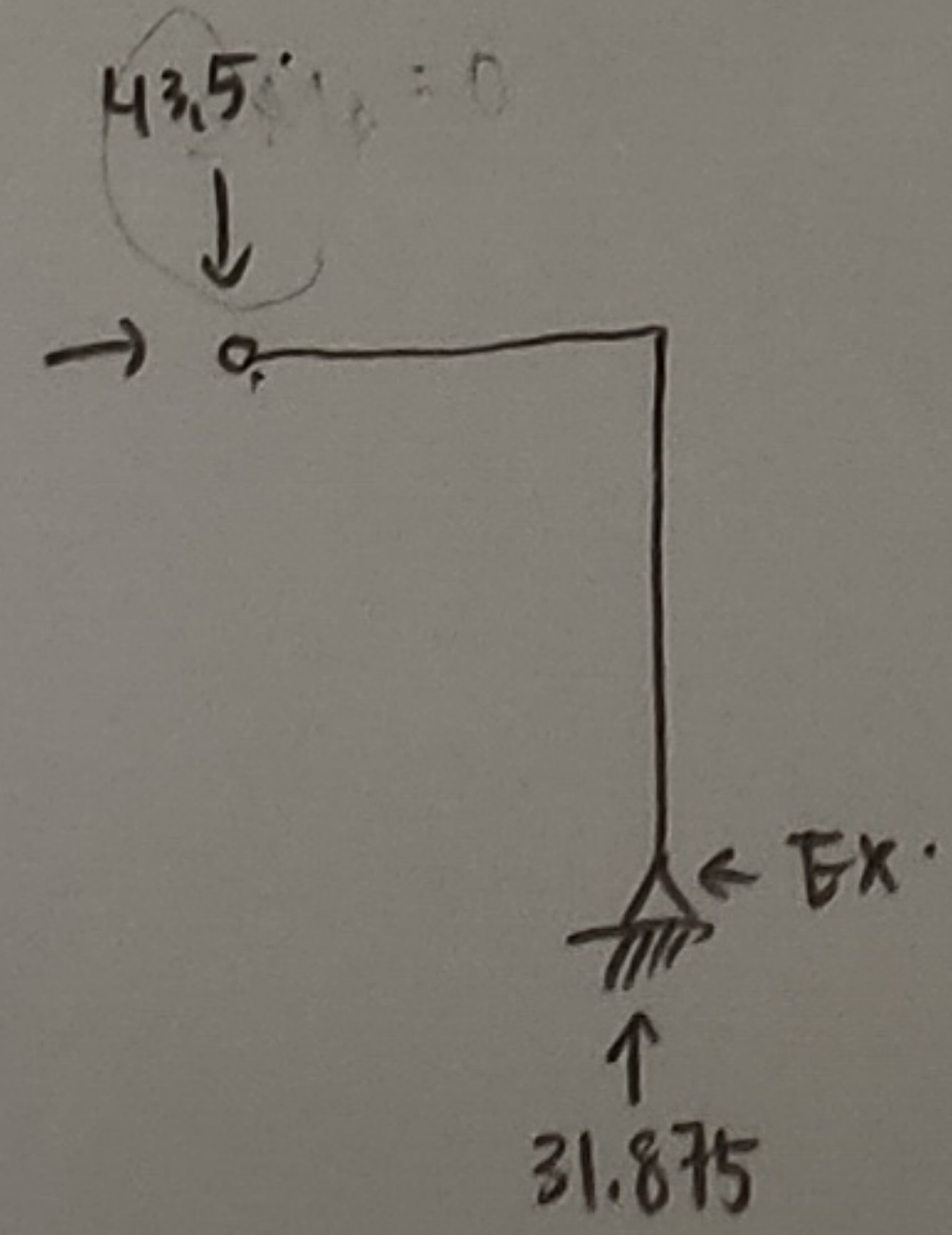
$\Sigma M_E = 0$

$F_A(6) + 13.5(3)(3/2) = 13.5(3)$

$FA = \underline{3.375}$ kN (↓).

b)





$\Sigma M_C = 0$

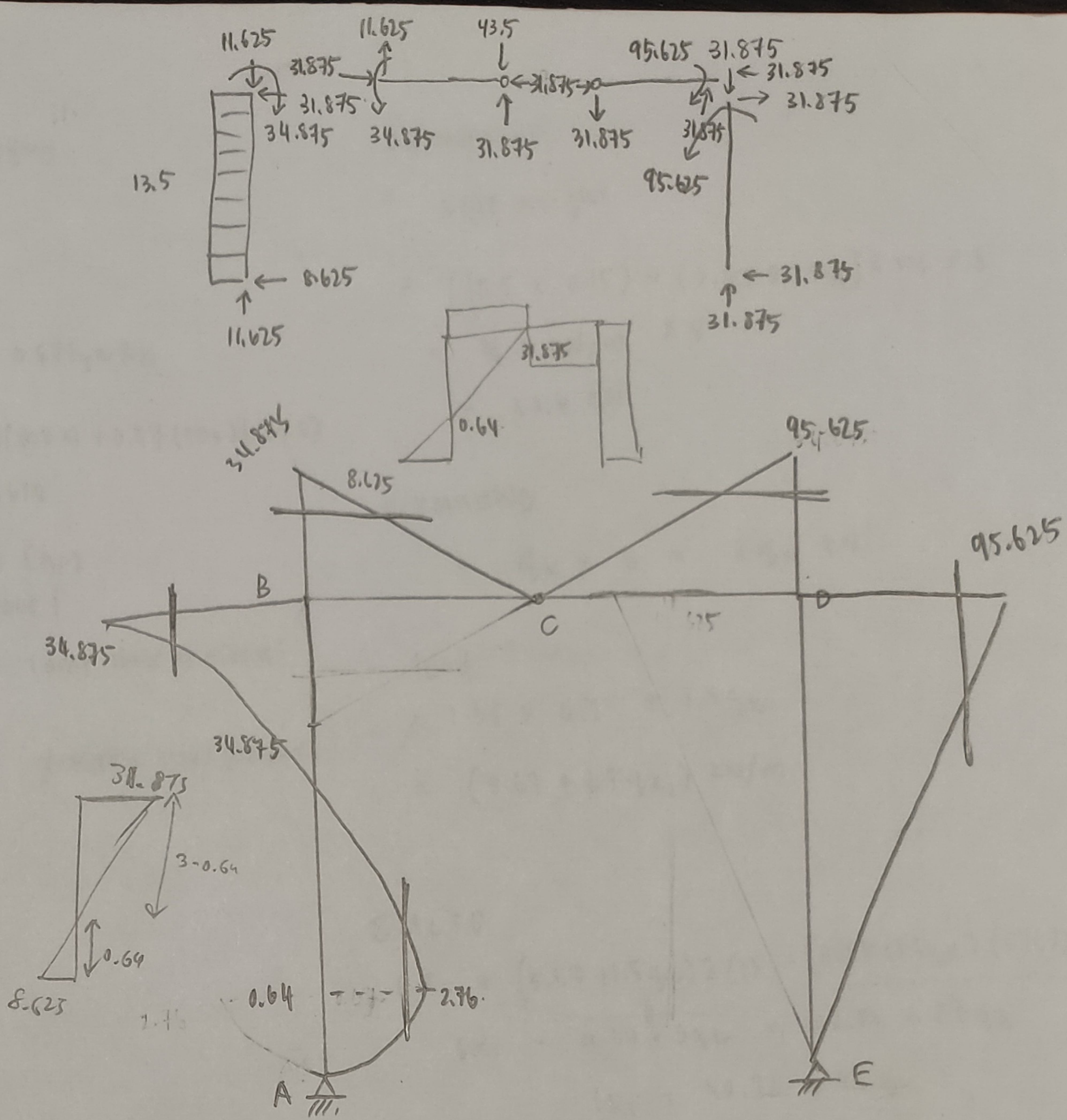
$31.875(3) = Ex(3)$

$Ex = 31.875$ kN

$\Sigma Fx = 0$ · (For whole)

$31.875 + Ax = 13.5(3)$.

$Ax = 8.625$ kN



c) For member BD,

highest moment = 95.625 KNm

$F_c = F_{st}$ (Assume steel yield)

$0.567 f_{ck} bs = 0.87 f_{yk} A_s$ $\overset{2H25}{\downarrow}$

$0.567(30)(250)(0.8x) = 0.87(500)(982)$

$x = 125.56m$

$d = 400$ mm

$z = d - s/2 = d - 0.4x = 400 - 0.4(125.56)$

$= 350$

$\frac{x}{d} = 0.314 < 0.617$ ok! steel yield

$M = 0.87 f_{yk} z A_s$

$= 0.87(500)(350)(982)$

$= 149.41$ KNm > 95.625 KNm ok!

11.625 43.5 95.625 31.875 21.825

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1 a) For section 1-1 C T sagging

Assume x above flange,

$F_{st} = \boxed{F_{cc}}$ ① $+ F_{sc}$

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

$0.87 f_{yw}(2414) = 0.567(30)(500)(0.8x) + 0.87(500)(1006)$

$1050090 = 6804x + 437610$

$x = 90.02 m < 150 (h_f)$

$\therefore$ assumption correct!

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

$\boxed{\frac{x}{d}} = \frac{90.02}{510} = 0.176 < 0.617$ tension steel yielded!

$M = F_{st} z$

$= 0.87 f_{yw} A_s z$

$= 0.87(500)(2414)(d - 0.4x)$

$= 497.734$ kNm

For section 2-2 T C Hogging

Assume x above flange

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

$0.87 f_{yw} A_{st} = 0.567 f_{cu} b(0.8x) + 0.87 f_{yw} A_{st}$

$0.87(500)(1571) = 0.567(30)(500)(0.8x) + 0.87(500)(402)$

$508515 = 6804x$

$x = 74.74 m < 150 (h_f)$

$\therefore$ assumption correct!

$\frac{d'}{d} = \frac{30}{510} = 0.0588 < 0.171$

$\frac{x}{d} = \frac{74.74}{510} = 0.147 < 0.617$

$M = F_{st} z$

$= 0.87 f_{yw} A_s z$

$= 0.87(500)(1571)(d - 0.4x)$

$= 328.1$ kNm

W = —

Permanent

= self weight

$= [(0.5 \times 0.15) + (0.4 \times 0.25)] \times 24 \times 8$

$= 4.2$ kN/m $\times 8$

$= 33.6$ kN

variable

$= q_k \times 8 = 8q_k$ kN/

load

$= 1.35 \times 4.2 + 1.5 q_w$

$= (5.67 + 1.5 q_k)$ kN/m

$\Sigma M_2 = 0.$

$6R_1 + \underline{(5.67 + 1.5q_w)2(1)} = \underline{(5.67 + 1.5q_k)(6)(3)}$

$6R_1 + 11.34 + 3q_w = 102.06 + 27q_w$

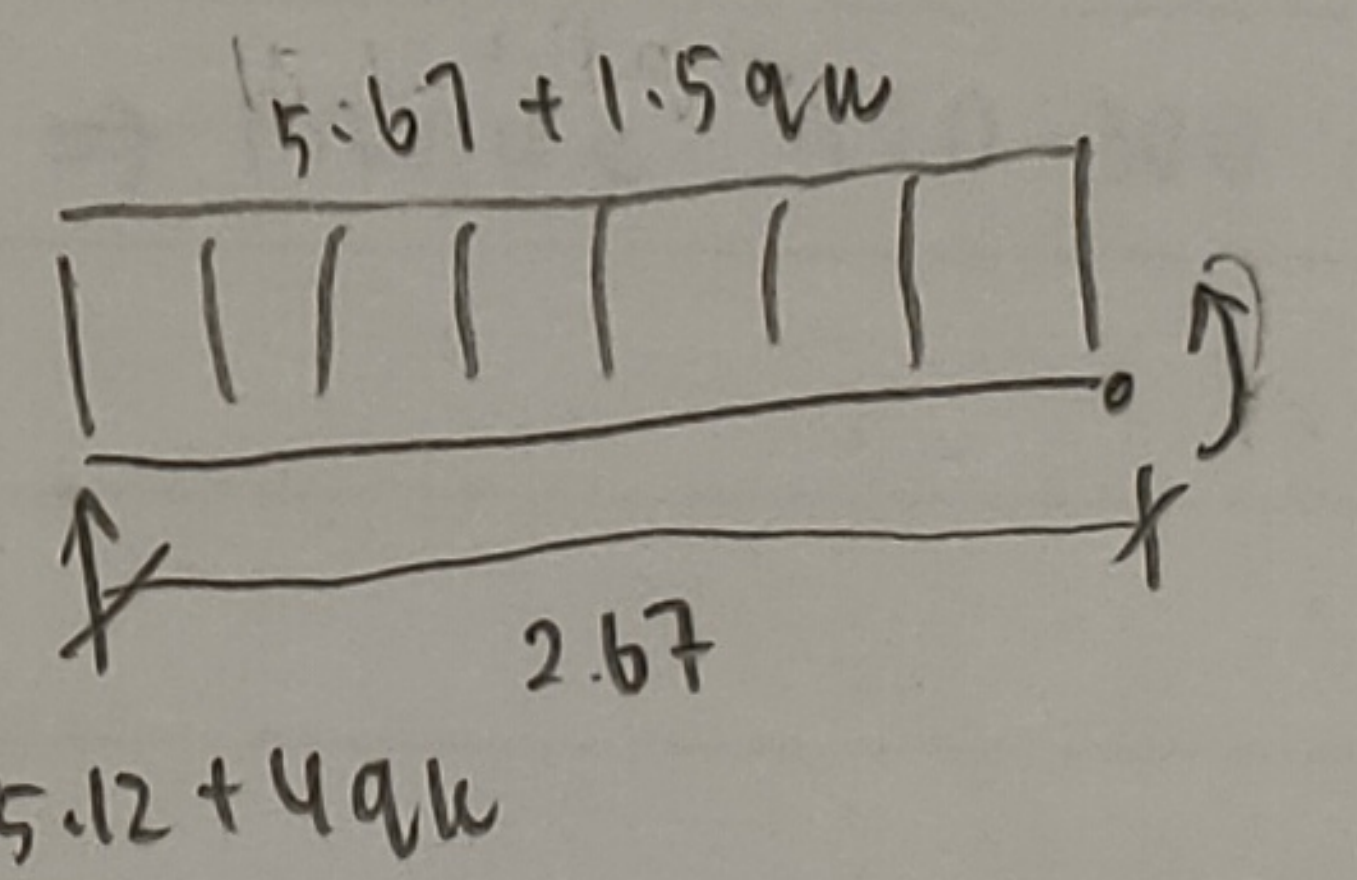
$6R_1 = 90.72 + 24q_k$

$R_1 = 15.12 + 4q_w.$

pt. of max moment

$\Rightarrow \frac{15.12 + 4q_w}{5.67 + 1.5q_w} = \frac{\frac{8}{3}(5.67 + 1.5q_w)}{5.67 + 1.5q_w}$

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



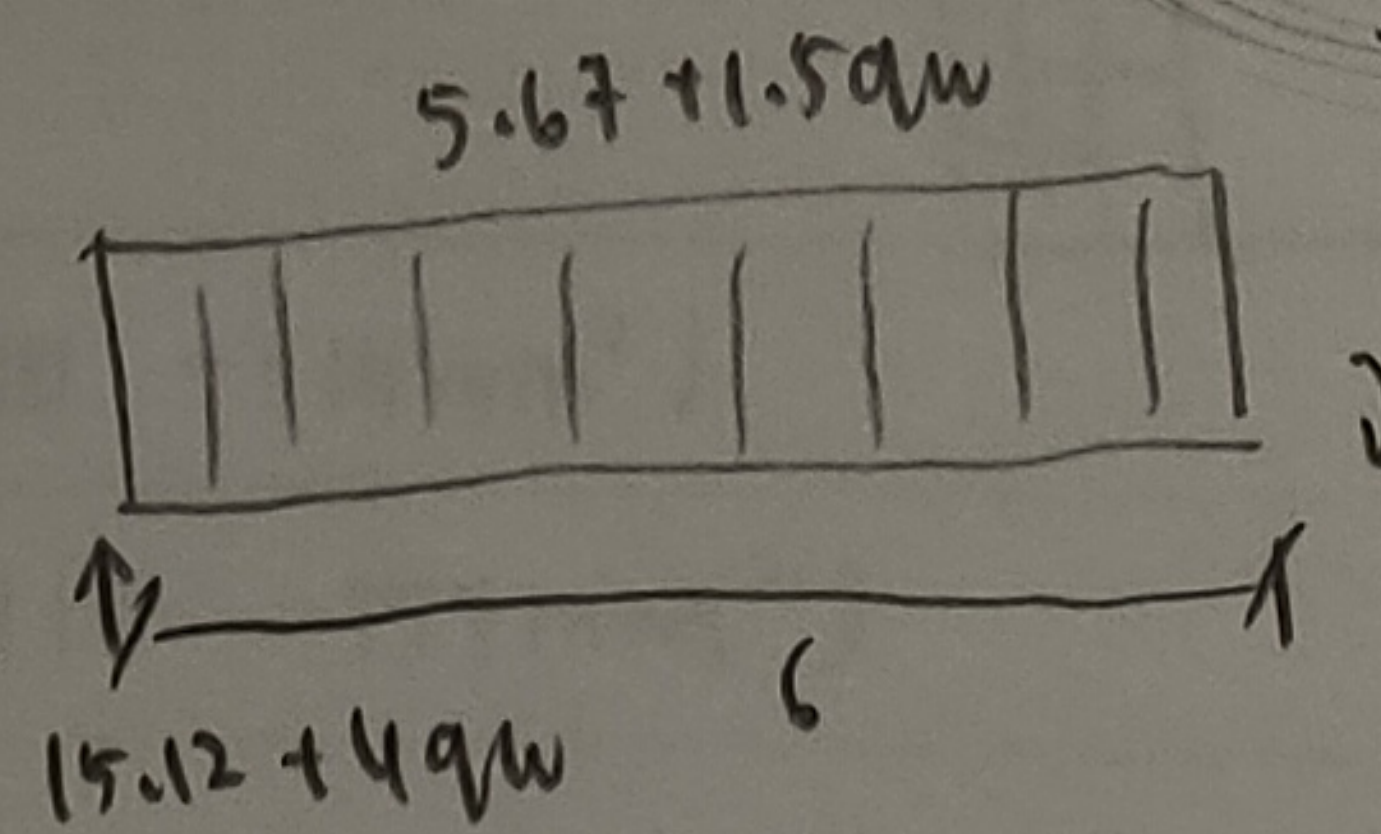
$\Sigma M = 0$

$M + (5.67 + 1.5q_w)\frac{(2.67)^2}{2} = (15.12 + 4q_k)(2.67) + M$

$M + 20.21 + 5.35q_w = 40.37 + 10.68q_w$

$497.734 = 20.16 + 5.33q_w$

$\boxed{\underline{q_w = 89.6 \text{ kN/m}}}$



$M + (15.12 + 4q_w)6 = (5.67 + 1.5q_w)\frac{(6)^2}{2}$

$M + 90.72 + 24q_k = 102.06 + 27q_w$

$328.1 - 11.34 = 3q_w$

$q_k = 105.59$ kN/m

2) b) At section 1-1,

At support face, $V_{Ef} = A_y = 15.12 + 4 \times 89.6$

$\doteq 373.52$ KN

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

$= 0.124(250)(1 - 30/250)(30)$

$= 818.4 > 373.52$

$\Rightarrow \theta = 22°\ ,\ \cot\theta = 2.5$

stirrup spacing = 250 , H8

H8@250 $\rightarrow$ $A_w/s = 0.402$

$V_{Rd} = \frac{A_{sw}}{s} 0.78\, d\, f_{yw} \cot\theta$

$= 0.402 \times 0.78 \times 510 \times 500 \times 2.5$

$= 199.89$ KN

$w = 5.67 + 15 \times 89.6 = 140.07$

$V_{1Ed} = 373.52 - 0.51(140.07)$

$= 302.08$ KN.

Midspan
Continuous edge
Discontinuous edge
As,min
~~Edge strip~~

:Torsion requirement

No.: ....

Date: ....

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3) For corner slab, slab thickness = 190 mm

$n = 1.35 \times 6.5 + 1.5 \times 2.23 = 12.12\ kN/m^2$

$l_y / l_x = 6/6 = 1$

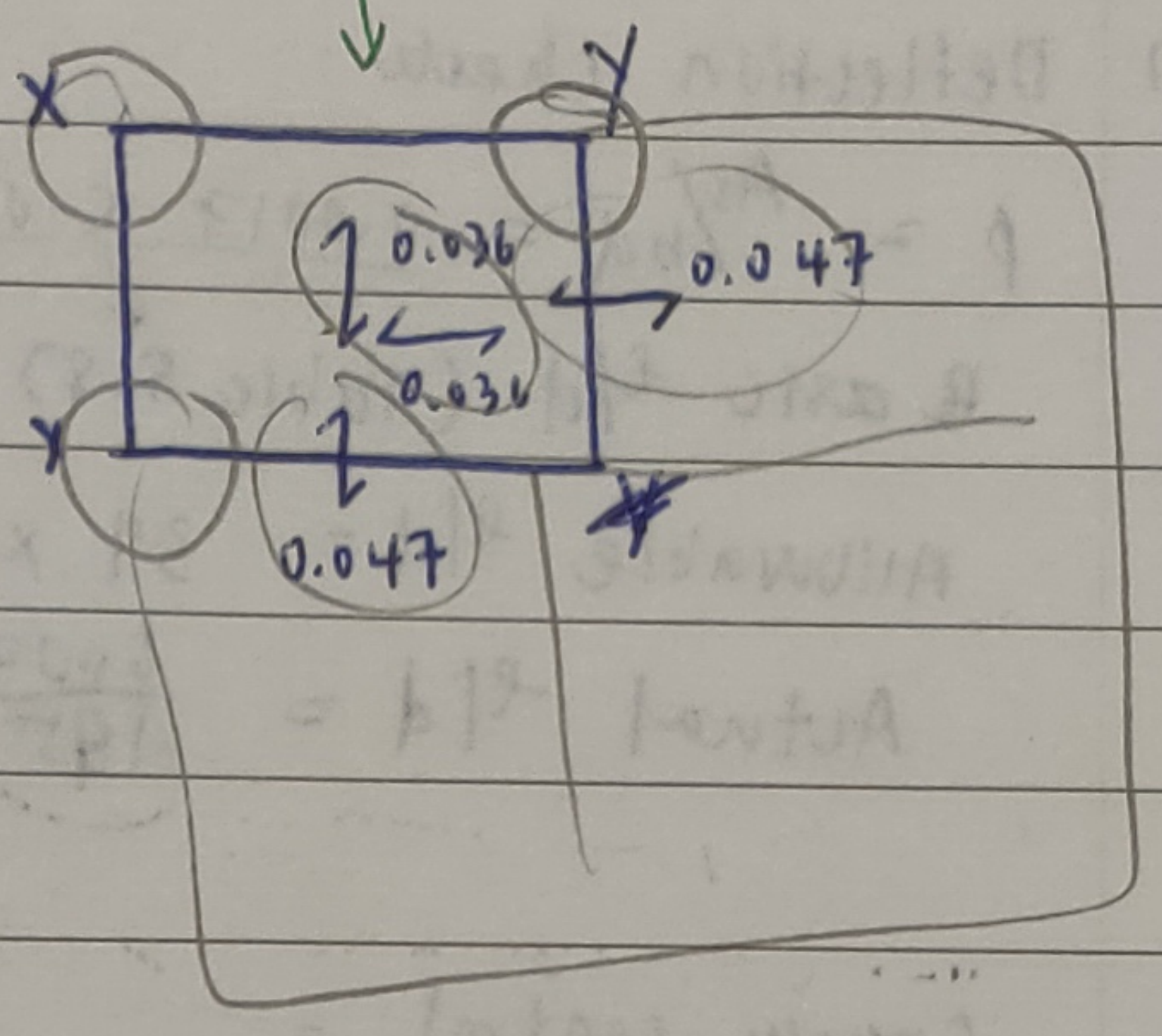
From Table 3.14, Case 4,

$\beta_{sx}' = 0.047$, $\beta_{sx} = 0.036$

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

$d_y = 155 - 10 = 145mm$

~~no need~~ count for y-dir ?
since d diff



$M_{sx} = \beta_{sx} \cdot n \cdot l_x^2 = 15.7\ kN\ m/m.$

$M_{sx}' = \beta_{sx}' \cdot n \cdot l_x^2 = 20.5\ kN\ m/m$

Midspan

$k = \frac{M}{f_{cu} b d^2} = 0.019$ 0.021

$z = d(0.5 + \sqrt{0.25 - \frac{k}{1.134}}) = 0.983d > 0.95d$

$A_{s,req} = \frac{M}{0.87 f_{yk} z} = 245\ mm^2 < A_{s,min}$

⇒ Provide H10 - 250 ($A_{s,prov} = 314 mm^2$)
↳ max bar spacing requirement

$A_{sx,min} = \max(0.26 \frac{f_{ctm}}{f_{yk}} bd, 0.0013bd)$

$= 0.26 \frac{0.3(35)^{2/3}}{500}(1000)(155)$

$= 258\ mm^2$

Continuous edge

$k = \frac{M}{f_{cu} b d^2} = 0.024$

$z = 0.978d \ngtr 0.95d$

$A_{s,req} = \frac{M}{0.87 f_{yk} z} = 320\ mm^2$

⇒ Provide H10 - 200 ($A_{s,prov}$ 393 mm^2)

Discontinuous edge

↳ $0.25 \times A_{sx} = 61.25 mm^2 < A_{s,min}$

⇒ Provide H10-300 ($A_{s,prov}$ 262mm^2)

Edge strip

⇒ Provide minimum reinforcement (258mm^2)

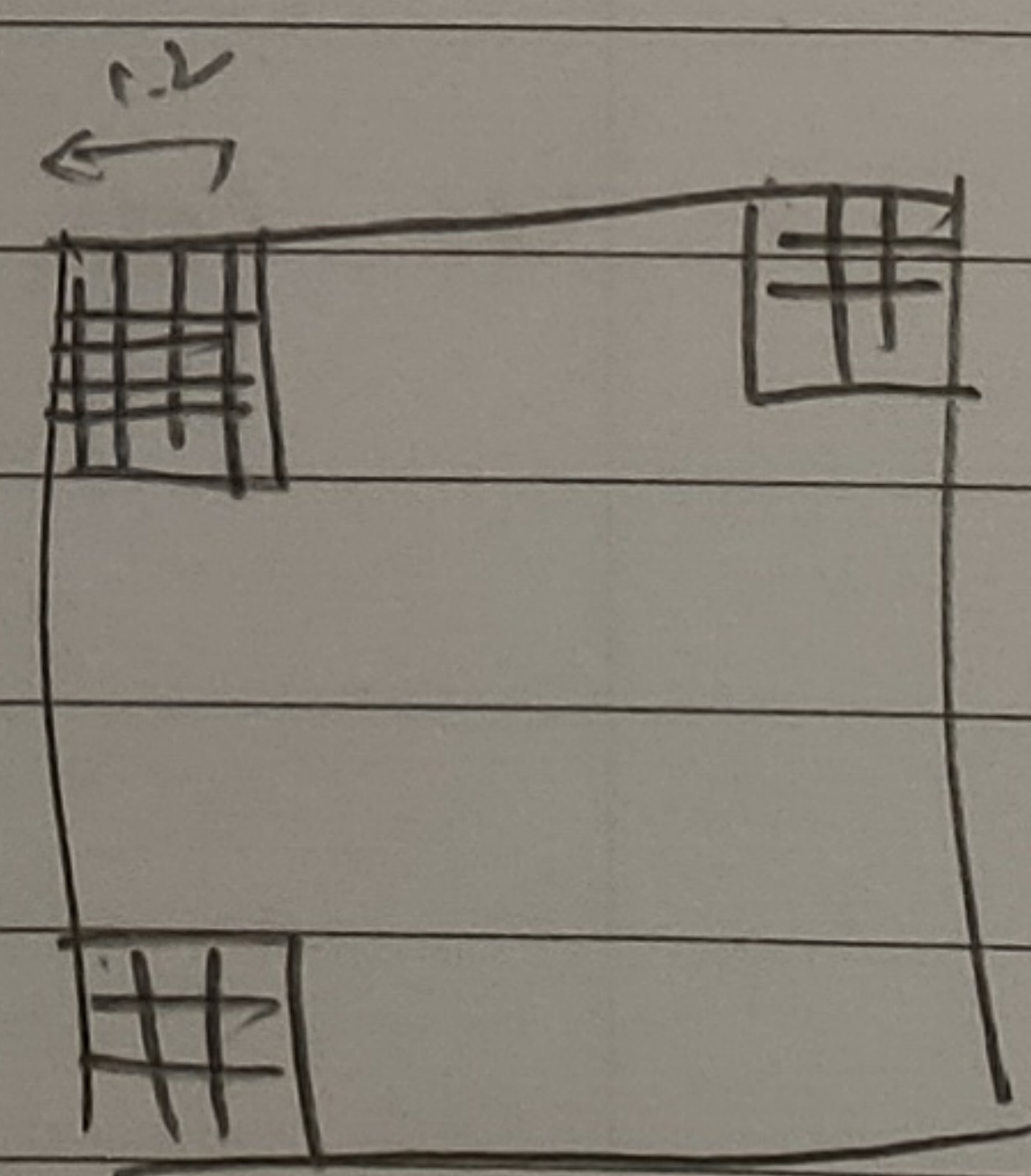
→ provide H10-300 (262mm^2) ← not bound by max bar spacing

Torsion requirement

⇒ at length $l_x/5 = 1.2m$ are needed at 2 corners.

Corner X : $3/4 \times A_{sx}$ = ~~[illegible]~~ 183.75 mm^2/m. (provide 4H10 -314mm^2)

Corner Y: $3/8 \cdot A_{sx}$ = 92 mm^2/m (provide 2H10 -157mm^2)



take 145 to be conservative? → If its same length

Reinforcement details ⇒ lbd ?

No.: Date:

b) Deflection check

$\rho = A_s/bd = \underline{0.0017 < 0.0035}$

Basic l/d (Table 5.8) for End span $= 39$

Allowable $l/d = 39 \times \frac{314}{245} = 49.98$ mm.

Actual $l/d = \frac{6000}{\boxed{145}} = 41.38 < 49.98$ ← ok!

Crack control

$$\sigma_s = \left(\frac{f_{yk}}{\gamma_{ms}}\right)\left\{\frac{\psi_2 Q_k + G_k}{1.5Q_k + 1.35G_k}\right\}\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$$

$$= \left(\frac{500}{1.15}\right)\left(\frac{0.3\times2.5+6.2}{1.5\times2.5+1.35\times6.2}\right)\left(\frac{245}{314}\right)\left(\frac{1}{1}\right) =$$

$= 195$ MPa

For $w = 0.4$ mm, $\sigma_s \le \boxed{195\text{ MPa}}$, $\boxed{\approx 200}$

max bar size $\le \underline{32\text{mm}}$ or

max bar spacing $\le \underline{300}$ mm.

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) check for adequacy of base area (SLS)

$$P_1 = \frac{N+W}{A} + \frac{6M}{BL^2} \leq 230$$

$P_1 < 230$

$P_1 > 0$

$$P_1 = \frac{1600}{3600^2} + 0.8(25) + \frac{6(500)}{3600(3600)^2} = 207.8 < 230$$

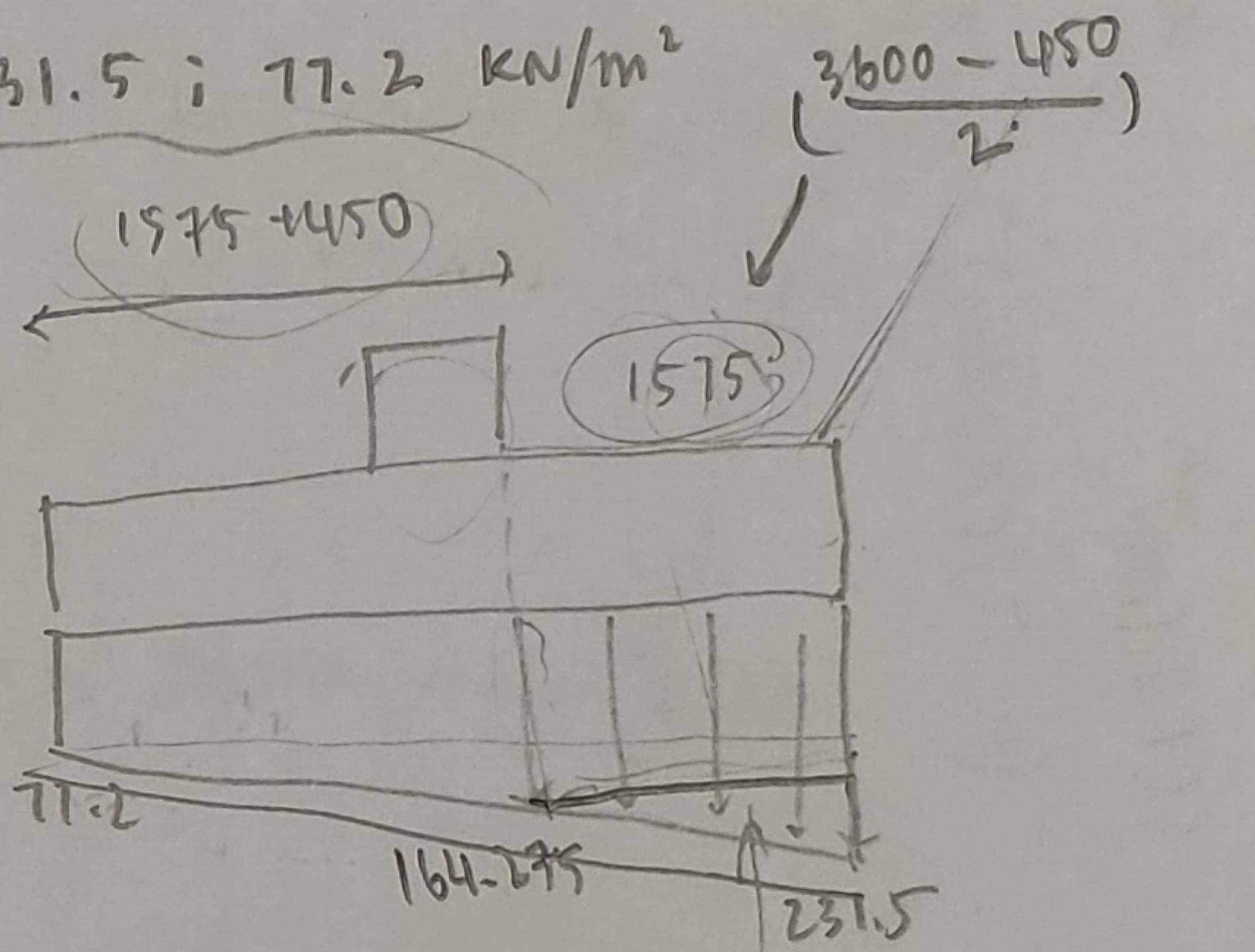
∴ Base area is adequate.

Check that $\frac{M}{N} < \frac{L}{6}$, $L > \frac{6M}{N} = \frac{6(500)}{1600} = 1.875$

∴ ok!

Design for bending (ULS).

Design pressure ⇒ $\frac{N_u}{A} \pm \frac{6M_u}{BL^2} = \frac{2000}{3.6^2} \pm \frac{6(600)}{3.6^3} = 231.5 \,;\, 77.2 \text{ kN/m}^2$



on what basis we assume?

Assume use H16 bars, $d_x = 800 - 70 - 8 = 722 \text{ mm}$

length.

$$M = \frac{1}{2}(164.275)(1.575)(3.6)\left(\frac{1.575}{2}\right) + \frac{1}{2}(231.5 - 164.275)(1.575)(3.6)\left(\frac{2(1.575)}{3}\right)$$

$$= 933.6 \text{ kNm}$$

similar Δ :

$$\frac{231.5 - 77.2}{y - 77.2} = \frac{3600}{x}$$

$$y - 77.2 = \frac{154.3x}{3600}$$

$$y = 0.043x + 77.2$$

At $x = 1575 + 450 = 2025$

$\therefore y = 0.043(2025) + 77.2$

$y = 164.275$ KPa

$$K = \frac{M}{f_{ck}bd^2} = \frac{933.6 \times 10^6}{30 \times 3600 \times 722^2} = 0.0166 < 0.167$$

$$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right) = 0.985d > 0.95d$$

$$A_{s,req} = \frac{M}{0.87 f_{yk} z} = \frac{933.6 \times 10^6}{0.87(500)(0.95 \times 722)} \approx 3129 \text{ mm}^2$$

$$A_{s,min} = 0.26\,(f_{ctm}/f_{yk})\,bd \geq 0.0013\,bd$$

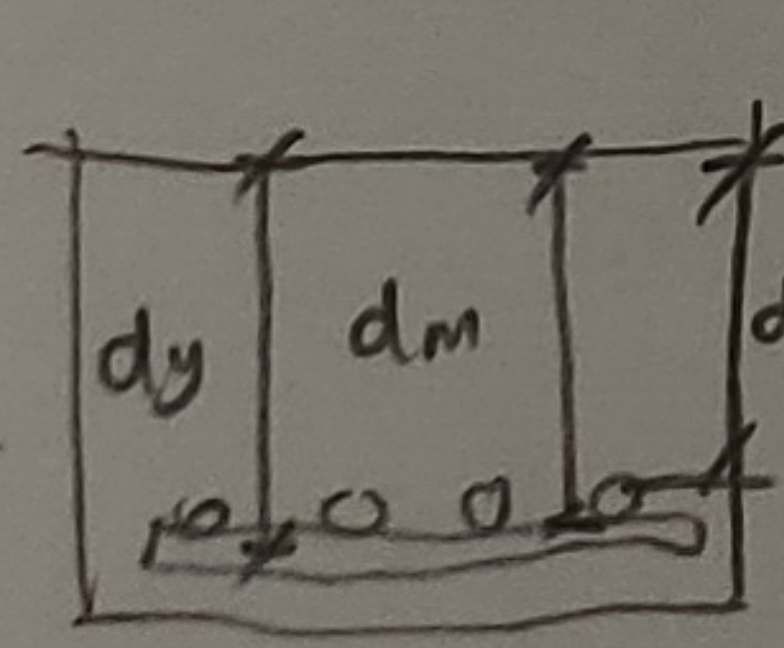
$$= 0.26\left(\frac{0.3(30)^{2/3}}{500}\right)(3600)(722)$$

$$= 3914 \text{ mm}^2$$

↳ provide 20H16 ($A_{s,prov} = 4021 \text{ mm}^2$).

Check bar spacing $= \frac{3600 - 70 \times 2 - 16}{19} = 181.26 \text{ mm} < \min(2h, 250)$ ok!

b) Shear resistance



$d_x = 800 - 70 - 8 = 722 \text{ mm}$

$d_m = 722 - 8 = 714 \text{ mm}$

$d_y = 722 - 16 = 706 \text{ mm}$

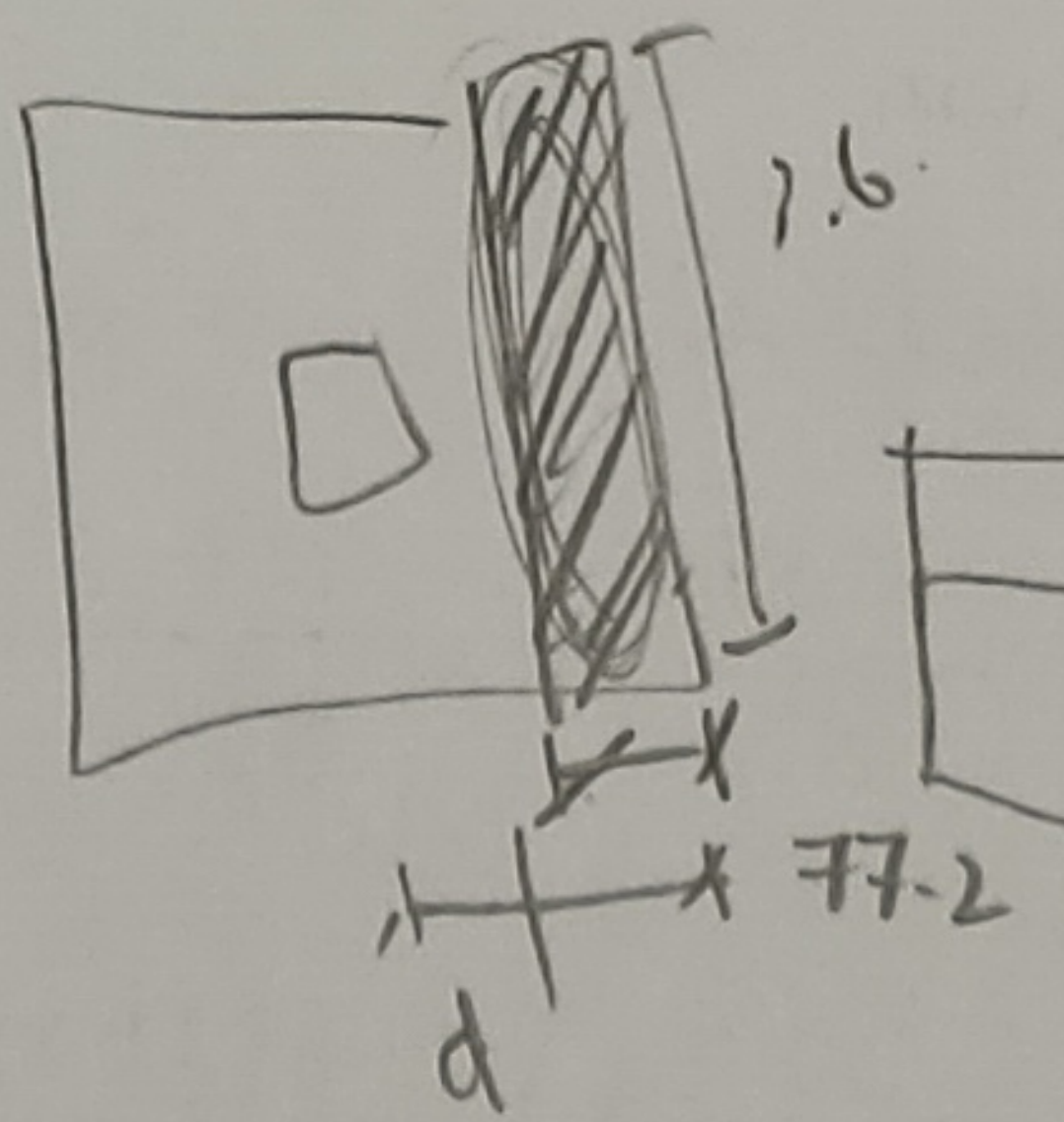
Max shear at column perimeter ($V_{Ed} = N_u = 2000 \text{ kN}$)

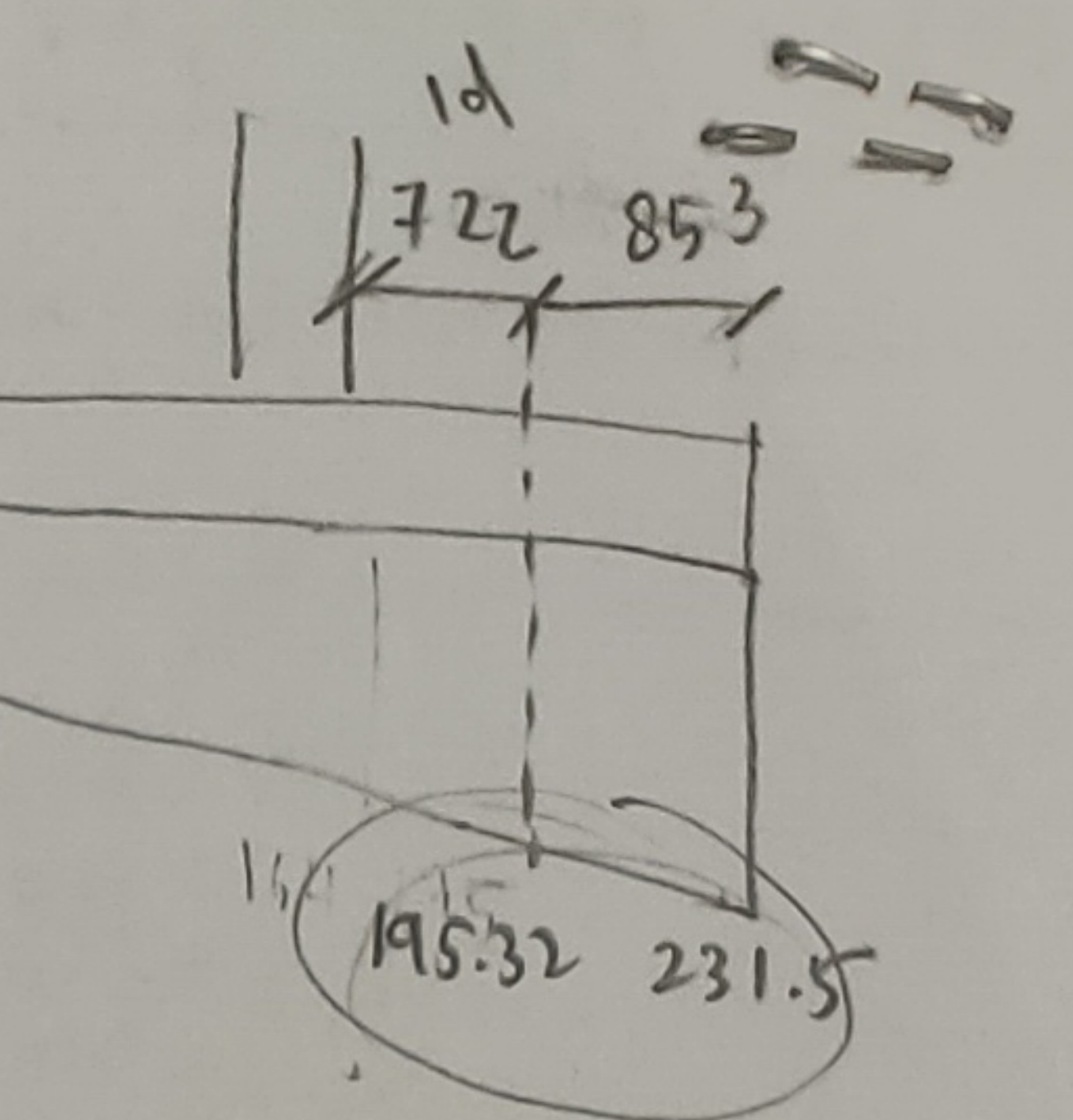
$$V_{Rd,max} = 0.5u_0 d_m \left[0.6\left(1 - \frac{f_{ck}}{250}\right)\right]\frac{f_{ck}}{1.5}$$

$$= 0.5(4 \times 450)(714)\left[0.6\left(1 - \frac{30}{250}\right)\right]\frac{30}{1.5}$$

$$= 6786 \text{ KN} > V_{Ed} = N_u = 2000$$

vertical shear at 1.0 d from column face (in x-dir) ($V_{Ed} = 655.34$)





Design shear, $V_{Ed} = \frac{195.32+231.5}{2}(3.6 \times 0.853) = 655.34$ kN

(x-dir only)
reinforcement ratio, $\underline{\underline{P_x}} = \frac{4021}{3600 \times 722} \times 100 = 0.155\% < 2\%$.

Shear resistance of concrete w/o shear reinforcement

$V_{Rd,c} = \frac{0.18}{\gamma_c} k (\underline{100} p_1 f_{cu})^{1/3} = 0.12 \left(\left(1+\sqrt{\frac{200}{722}}\right)\left(\boxed{0.155}(30)\right)\right)^{1/3} = 0.306$ C2

$V_{Rd,c} > V_{min} = 0.035 k^{3/2} f_{cu}^{1/2} = 0.035\left(1+\sqrt{\frac{200}{722}}\right)^{3/2}(30)^{1/2} = 0.361$

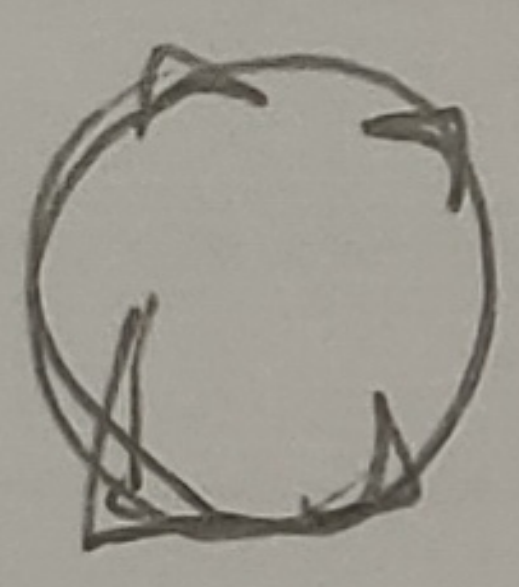
$V_{Rd,c} = \boxed{V_{Rd,c} \times \boxed{bd_x}} = 0.361 \times 3600 \times 722 = 939.58$ KN $> V_{Ed} = 655$ kN $= 195.32$ KPa

OK!

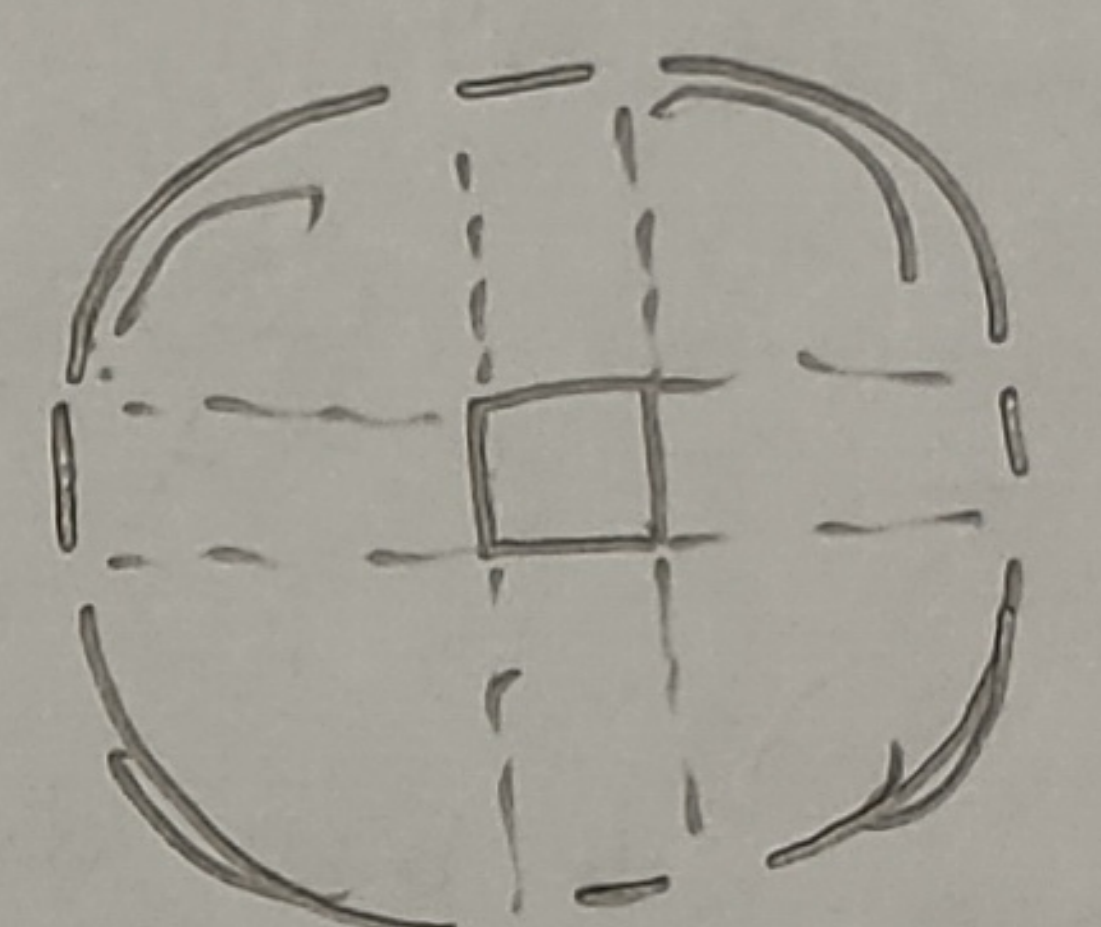
P(1575 + 450 + 722)
= 0.043 (1575 + 450 + 722)
+ 77.2

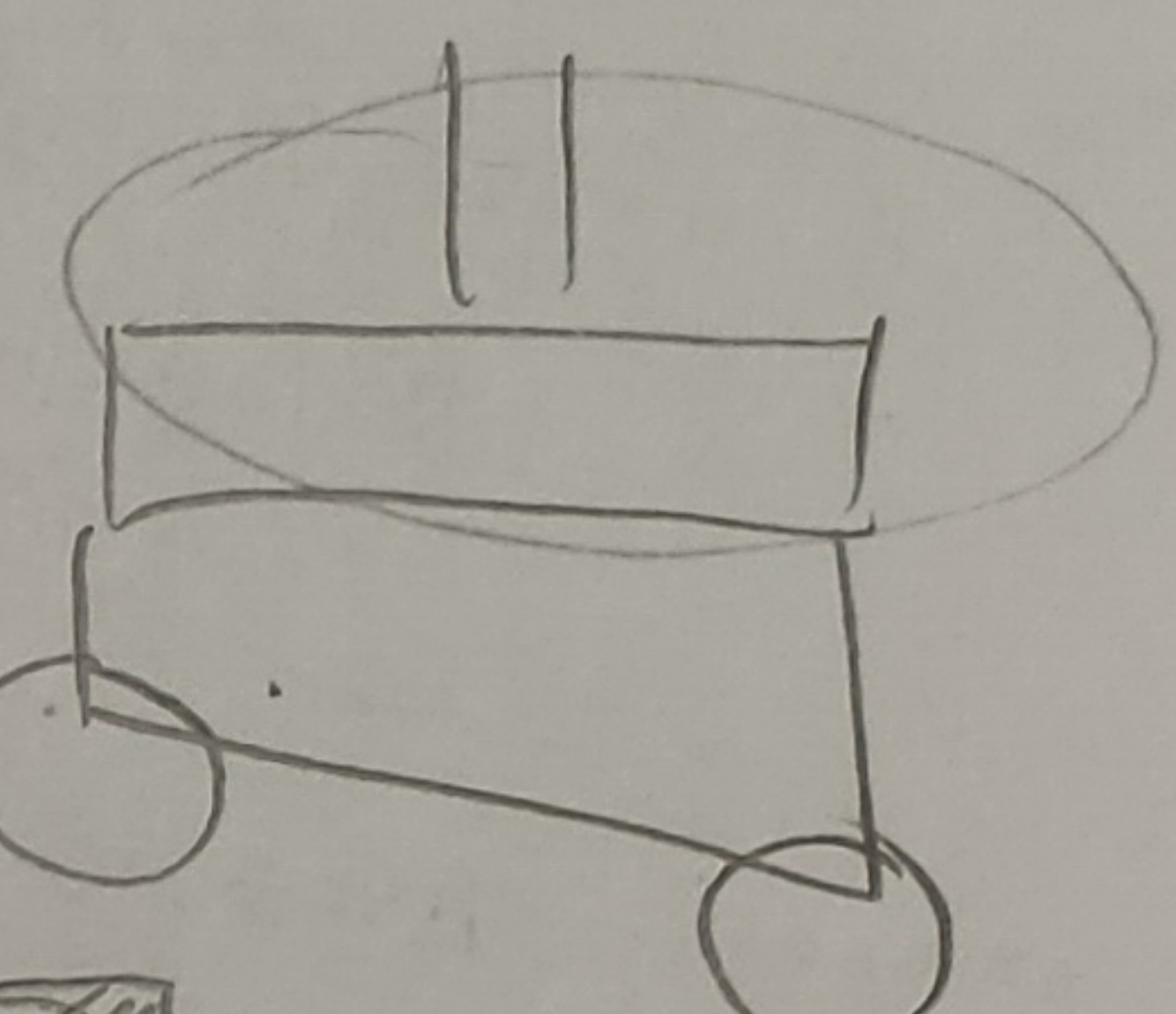
$u_0 = 4 \times 450$
$u_1 = 4 \times 450 + \underline{4\pi d_m}$

$((C + 4dm)^2$
$- (4-\pi)(2.0dm)^2$

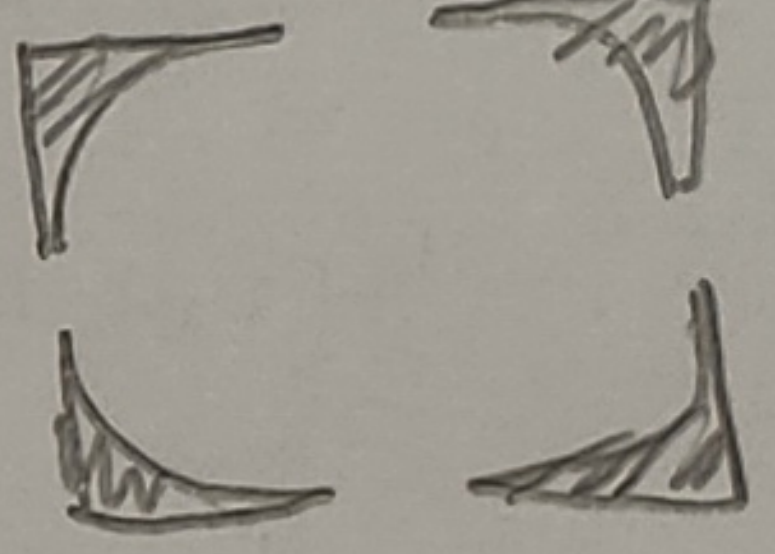


Punching shear at 2.0 d from column face ($V_{Ed} = 583.443$ kN)





Critical perimeter $\underset{\rightarrow}{u_1}$ = column perimeter + $4\pi d_m$
= $4 \times 450 + 4\pi(714)$
= 10772 mm



Area within critical perimeter = $(c + 4dm)^2 - (4-\pi)(2.0\,dm)^2$
$= [450 + 4(714)]^2 - (4-\pi)(2 \times 714)^2$
$= 9.18 \times 10^6$ mm^2

Punching force, $V_{Ed} = \left(\frac{231.5 + 77.2}{2}\right) \times (3.6^2 - 9.18) = 583.443$ KN

(two-dir)
average reinforcement ratio, $P_1 = \sqrt{\underset{d_x}{\frac{4021}{3600 \times 722}} * \underset{d_y}{\frac{4021}{3600 \times 706}}} = 0.156\% < 2\%$

shear resistance of concrete

$V_{Rd,c} = \frac{0.18}{\gamma_c} k (100 p_1 f_{cu})^{1/3} = 0.12\left(1+\sqrt{\frac{200}{714}}\right)(0.156(30))^{1/3} = 0.307$.

$V_{Rd,c} > V_{min} = 0.035 K^{3/2} f_{cu}^{1/2} = 0.035\left(1+\sqrt{\frac{200}{714}}\right)^{3/2}(30)^{1/2} = 0.363$

$V_{Rd,c} = \boxed{V_{Rd,c} \times \boxed{u_1 d_m}} = 0.363 \times (10772) \times 714 = \cancel{116.5}$ 2791.9 kN > 583 kN

OK!

∴ shear resistance is adequate