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1) b) T10 @ 200 → shear links

V_{Ef} = ~~480~~ 477.768 kN

V_{Eld} = ~~480~~ 477.768 − ~~120~~ 119.443 (~~0.744~~ 0.754)
= ~~390.72~~ 387.72 kN

Assume
cc = 30   dia = 32
LD = 10
d = 800 − 30 − 10 − 16
= 744

$\frac{8000}{744}$ = 10.

$V_{Rd,max(22)}$ = $0.124 b_w d (1 - f_{ck}/250) f_{ck}$
= $0.124 (300)(\cancel{744}\ 754)(1 - \frac{40}{250})(40)$
= ~~929.94~~ 942.439 kN > ~~480kN~~ 477.768

⇒ θ = 22°, cot θ = 2.5

T10 @ 200 → $\frac{A_{sw}}{s}$ = 0.785

V_{Rd} = $\frac{A_{sw}}{s} 0.78 d f_{yk} \cot\theta$ = 0.785 × 0.78 × 744 × 250 × 2.5
= 284.72 kN < ~~390.72~~ 387.72 kN

☆ ⇒ collapse behaviour ??

$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78 d f_{yw} \cot\theta} = \frac{390.72}{0.78(744)(250)(2.5)}$ = 1.078

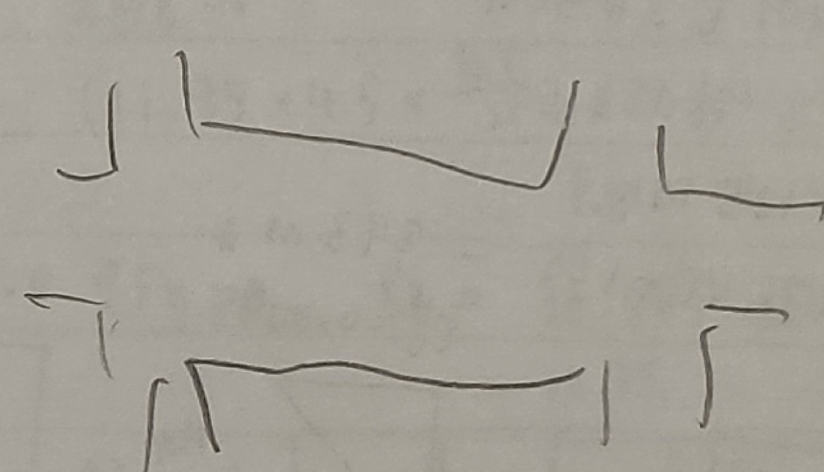
$\frac{L}{d} = \frac{8000}{744}$ = 10.75 ≥ 4
Diagonal tension failure mode

choose ϕ10 @ 125 mm ⇒ A_{sw}/s = 1.256

→ need design for min links ?

⇒ $\frac{A_{s,min}}{s} = \frac{0.08 f_{ck}^{0.5} b_w}{f_{yw}} = \frac{0.08(25)^{0.5}(300)}{500}$
= 0.24

choose ϕ10 @ 400, $\frac{A_{sw}}{s}$ = 0.393 > 0.24



1) a) DL = 25 + self weight
LL = 50
Self weight = [(1.2 × 0.1) + (0.7 × 0.3)] × 24
= 7.92 kN/m
Design load = (25 + 7.92)1.35 + 1.5(50)
= 119.442 kN/m

Simply supported
M_{Ed} = $wL^2/8$ = 955.536 kN
V_{Ed} = $wL/2$ = 477.768 kN
Assume use ϕ32 bar, assume 20
d = 800 − 32/2 − 20 − 10
= 754mm

~~Assume x above flange~~,
$K = \frac{M}{f_{ck} b d^2} = \frac{955.536 \times 10^6}{40 \times 1200 \times 754^2}$ = 0.035 < 0.167
comp steel not needed!

$z = d(0.5 + \sqrt{0.25 - \frac{K}{1.134}})$ < 0.95d
= 716.3 mm
s = 2(d − z) = 75.4 mm < 100m ∴ assumption correct

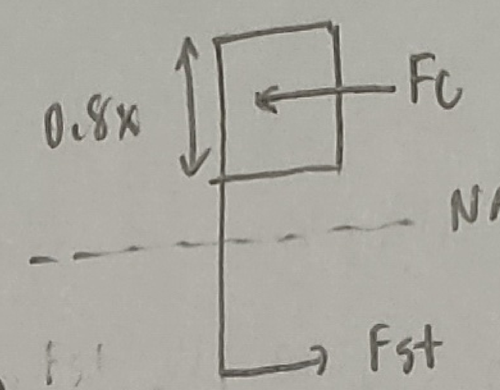
$F_{st} = M/z$
$A_s = \frac{M}{0.87 f_{yk} z}$
= $\frac{955.536 \times 10^6}{0.87(500)(716.3)}$
= 3066.64 mm²
⇒ Provide 4H32 $A_{s,prov}$ = 3216 mm²

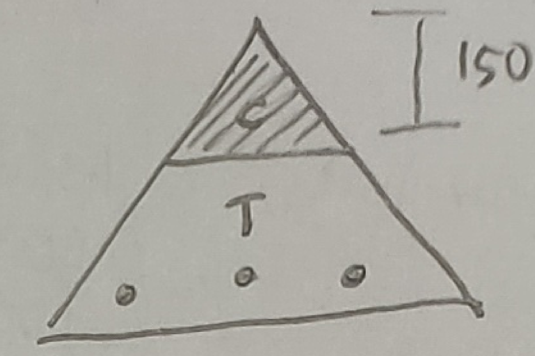
2) a) $d = 600 - 50 = 550$

$F_{cc} = F_{st}$ (Assume yield)

$0.567 f_{cu} b s = 0.87 f_{yk} A_s$ (3T20)

$0.567(40)(\frac{x}{600} \times 600)(0.8x) = 0.87(500)(943)$





$x = \frac{bx \times \frac{x}{2} + \frac{E_s}{E_c} A_s d}{bx + \frac{E_s}{E_c} A_s}$

$x^2 = 22608.3$

$x = 150.36 \text{ mm}$

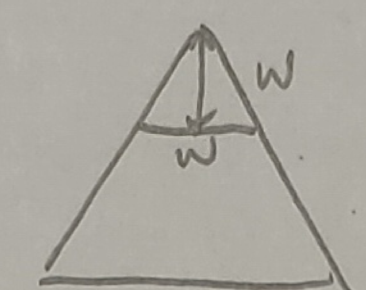
$\frac{x}{d} = \frac{150.36}{550} = 0.27 < 0.617$ ∴ steel yielded

$z = d - s/2 = 550 - 0.4x = 489.856$

$M = 0.87 f_{yk} z A_s$

$= 0.87(500)(489.856)(943)$

$= 200.94 \text{ kNm}$



$L = 490 - 150 = 340 \text{ m}$.

$d = 490 + 60 = 550$

$F_{st} = 0.87(500)(943)$

$= 410205$

$\frac{x}{d} < 0.617$

$x < 339.35$

$s = 0.8x < 271.48 \text{ mm}$

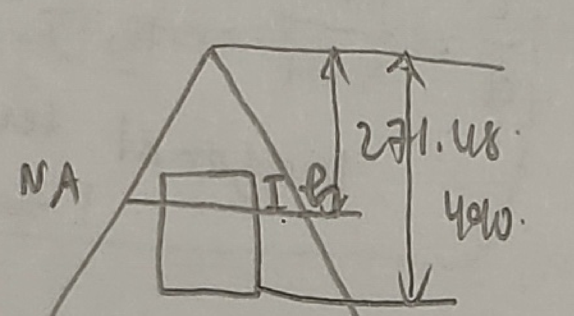
$F_{st} = F_{cc}$

$0.87(500)(943) = 0.567(40)(\frac{w^2}{2})$

$w = 190.19 \text{ mm} < 271.48 \text{ mm}$

$\max L = 490 - 190.19$

$= 300 \text{ mm}$.



$w = 271.48$

area above NA $= \frac{w^2}{2} - 150h$

$F_{st} = F_{cc}$

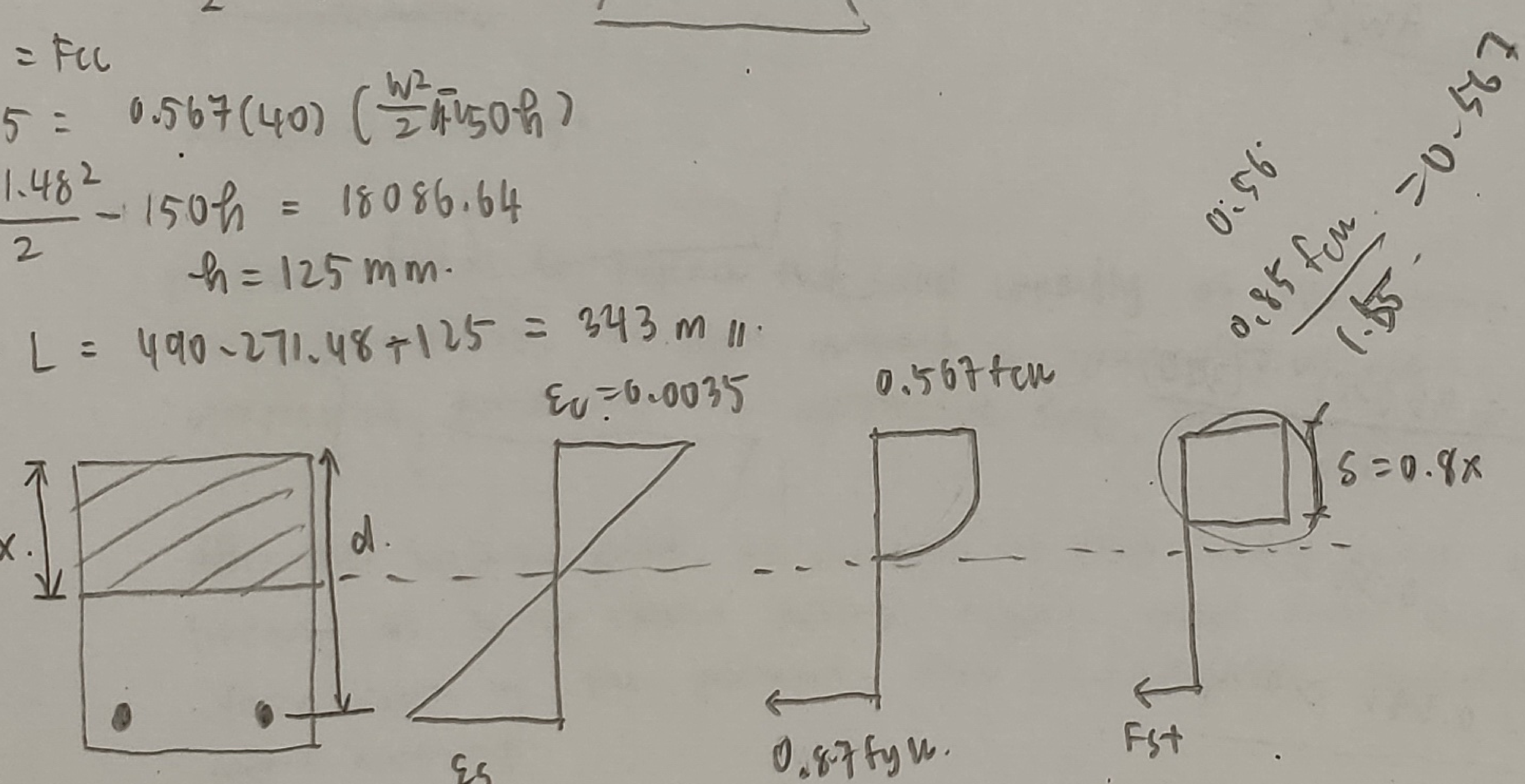
$410205 = 0.567(40)(\frac{w^2}{2} - 150h)$

$\frac{271.48^2}{2} - 150h = 18086.64$

$h = 125 \text{ mm}$.

$\max L = 490 - 271.48 + 125 = 343 \text{ m}$ //

b)



Actual stress block, concrete stress is not constant above neutral axis. For simple calc. of concrete strength mobilized, a simplified stress block is used. ~~s=0.8~~ A rectangular stress block with length $s = 0.8x$ is used to calculate the concrete strength mobilised.
fyt with $0.567 f_{cu}$ ⇒ use to estimate compressive concrete force with reasonable accuracy.

concrete crush at 0.0035 strain, which is at a stress of $0.85 f_{cu}$, where f_{cu} refers to the cylinder crushing strength of the concrete. A factor 0.85 is included to account for the difference between the bending strength & cylinder strength.
Then, a factor, γ_{m2} is applied ~~to account for uncertainty~~ where $\gamma_{m2} = 1.5$

⇒ $\frac{0.85 f_{cu}}{1.5} = 0.567 f_{cu}$. $0.567 f_{cu}$ is used in calculating concrete compression force, $F_{cc} = 0.567 f_{cu} b s$.

the depth s is the depth for the equivalent stress block when $s = 0.8x$ and x is the depth of neutral axis.

concrete reach strain before steel. ↑

c) Over-reinforced beams refer to beams where $\varepsilon_y < 0.00217$ and steel reinforcement do not yield at ULS and beams fail when concrete reach its crushing strain. $x/d > 0.45$ and $K > 0.167$. (Brittle failure). (too much reinforcement steel provided)

Under-reinforced beams refer to beams where $\varepsilon_y > 0.00217$ and steel reinforcement yield before concrete reach crushing strain. $x/d < 0.45$ & $K < 0.167$.
Steel provided such that it yields prior to concrete crushing strain.
ductile failure.

① need to check for single load analysis ?

* School building ≈ office area

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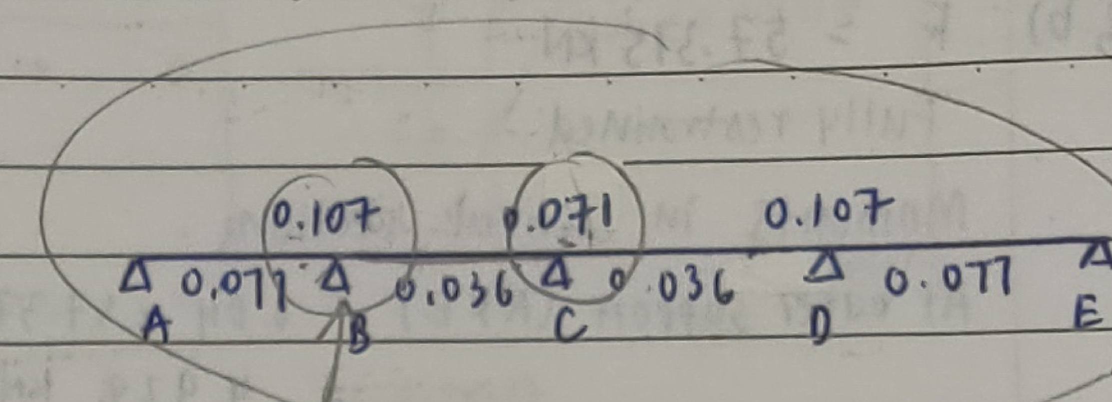


3)a) Bay area = 7 × 4.5 = 31.5 m² > 30 m² (span length)

Imposed / Dead load = 4/5 = 0.8 < 1.25

Q_K = 4 kN/m² < 5 kN/m²

→ use single load case analysis

(Required)

Consider a strip of 1-m wide

$V = F \times coe$ (wl)

$M = F \times coe \times l$

$w = (1.35 \times 5) + (1.5 \times 4) = 12.75$ kN/m

$F = w \cdot l = 12.75 \times 4.5 = 57.375$ kN

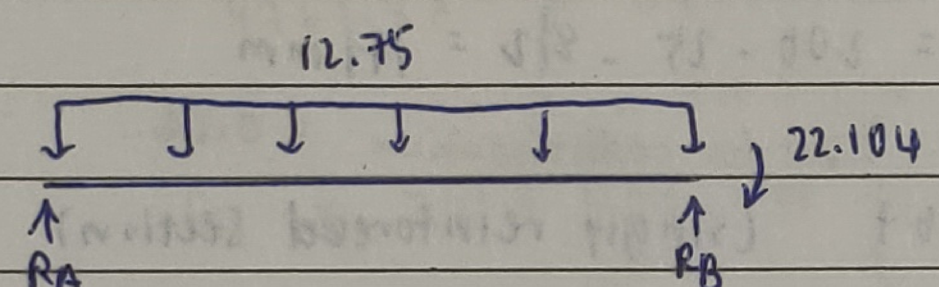
$F \cdot l \cdot coe$

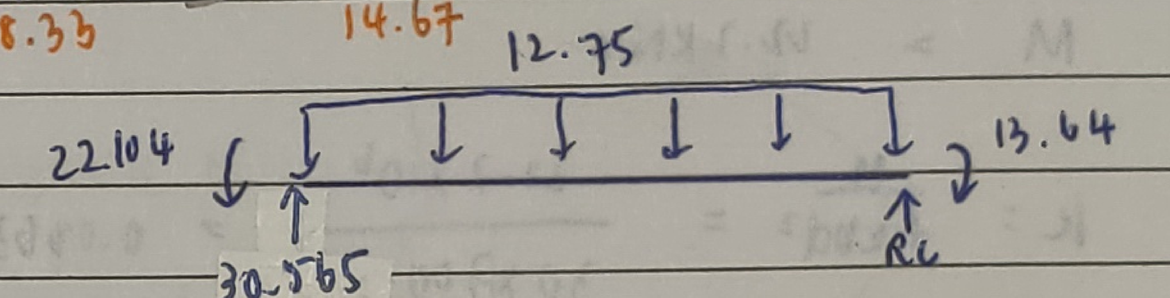
Elastic moment at support B & D = ~~53.375~~ 57.375 × 4.5 × 0.107 = 27.63 kNm

" " " " C = ~~53.375~~ 57.375 × 4.5 × 0.071 = ~~17.05~~ 18.33 kNm

New support moment @ support B & D = 27.63 × 0.8 = 22.104 kN m

" " " @ " C = ~~17.05~~ 18.33 × 0.8 = ~~13.64~~ 14.67 kNm





$\frac{23.78}{x} = \frac{33.595}{4.5 - x}$

$\Sigma M_B = 0$

$(12.75 \times 4.5 \times \frac{4.5}{2}) - 22.104 = R_A(4.5)$

R_A = 23.78 kN (= RE)

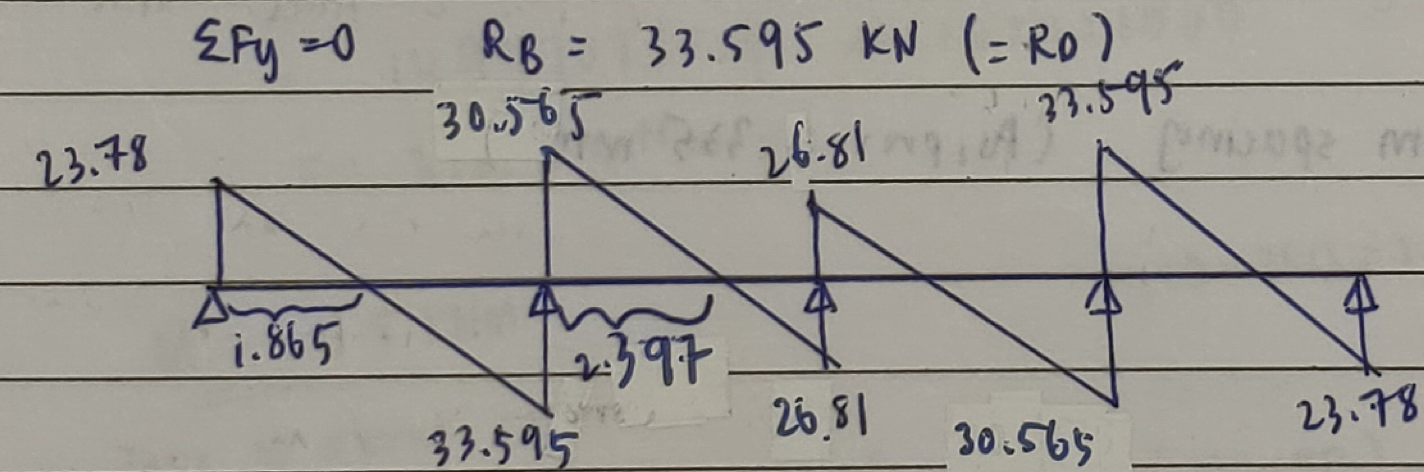
$\Sigma F_y = 0$ R_B = 33.595 kN (= RD)

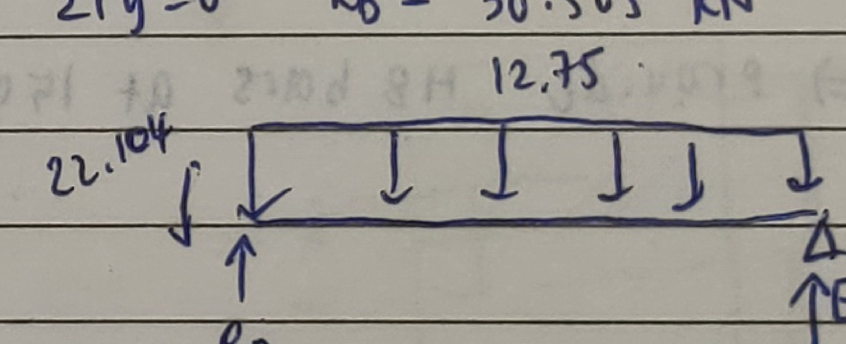
$\Sigma M_B = 0$

$(12.75 \times 4.5 \times \frac{4.5}{2}) + 13.64 - 22.104 = R_C(4.5)$

R_C = 26.81 kN

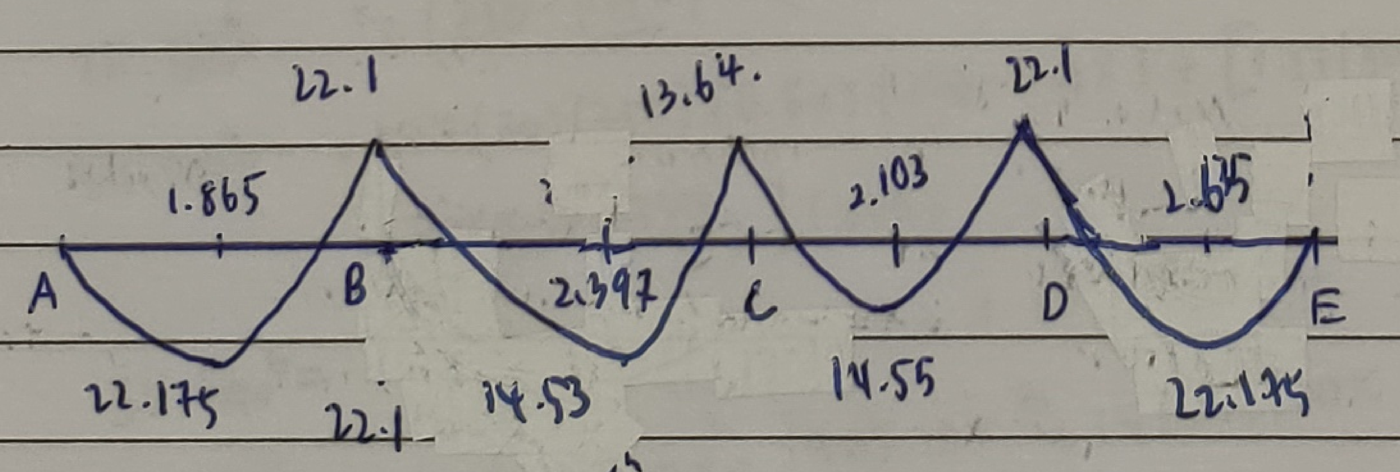
$\Sigma F_y = 0$ R_B = 30.565 kN

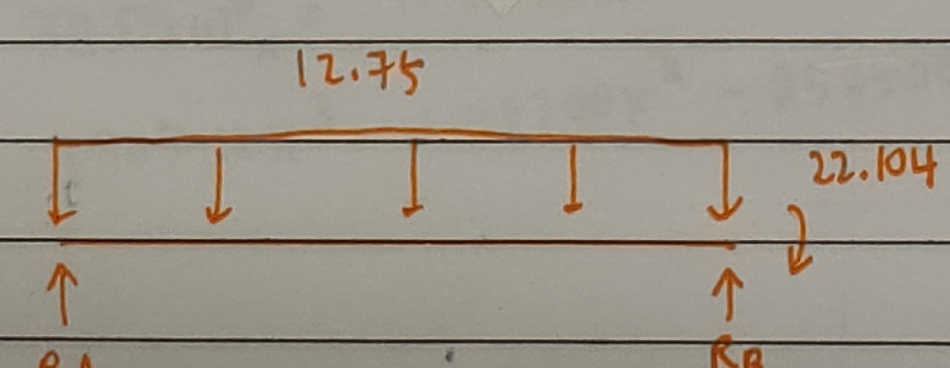


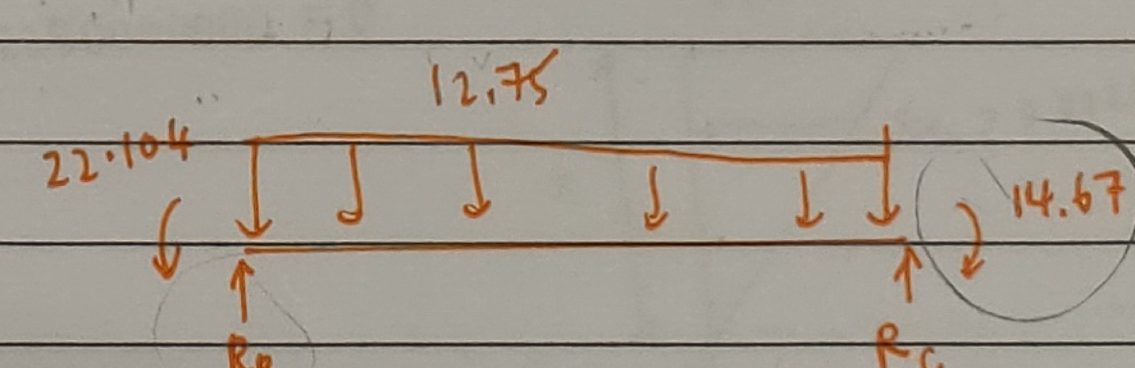


$\Sigma M_D = 0$ $12.75(4.5)(\frac{4.5}{2}) - 22.104 = RE(4.5)$

RE = 23.78







$\Sigma M_B = 0$ $(12.75 \times 4.5^2/2) - 22.104 = R_A(4.5)$

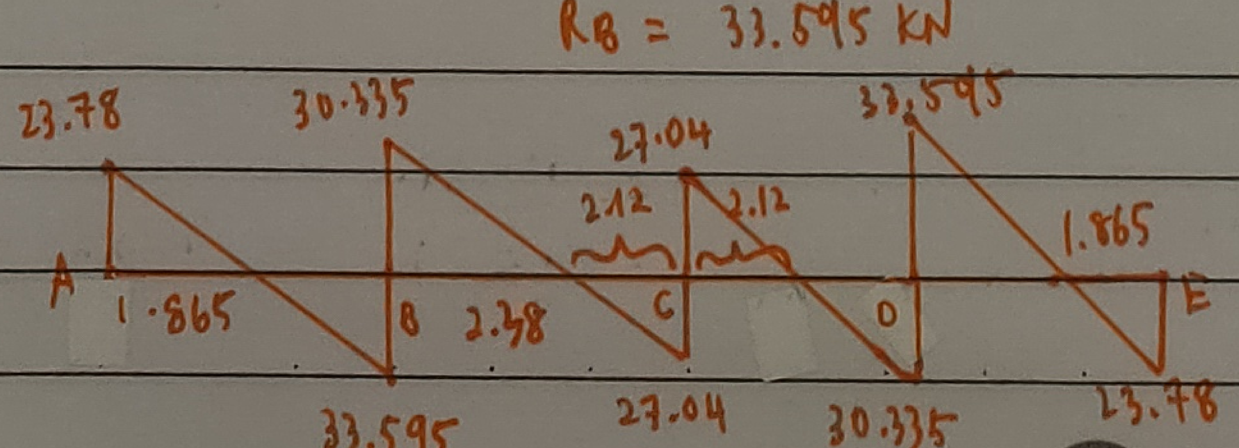
R_A = 23.78 kN

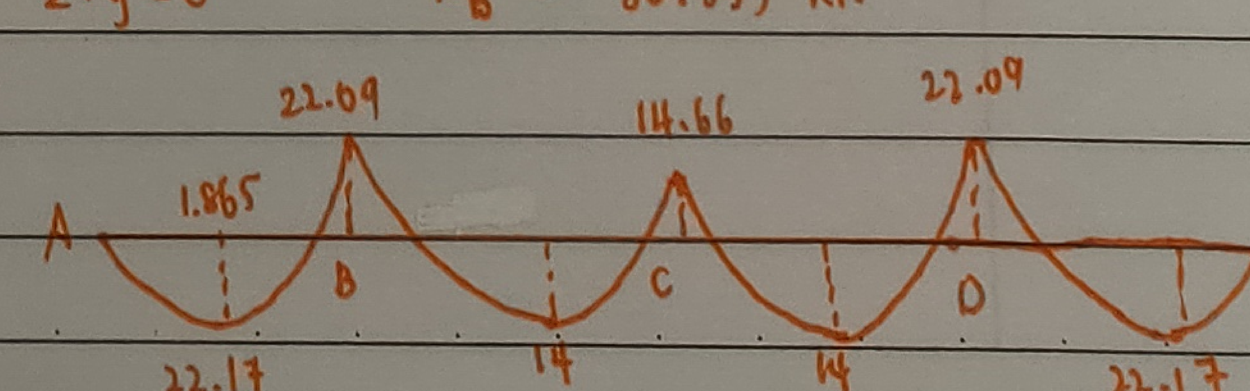
R_B = 33.595 kN

$\Sigma M_B = 0$ $12.75(\frac{4.5^2}{2}) + 14.67 - 22.104 = R_C(4.5)$

R_C = 27.04 kN

$\Sigma F_y = 0$ R_B = 30.335 kN





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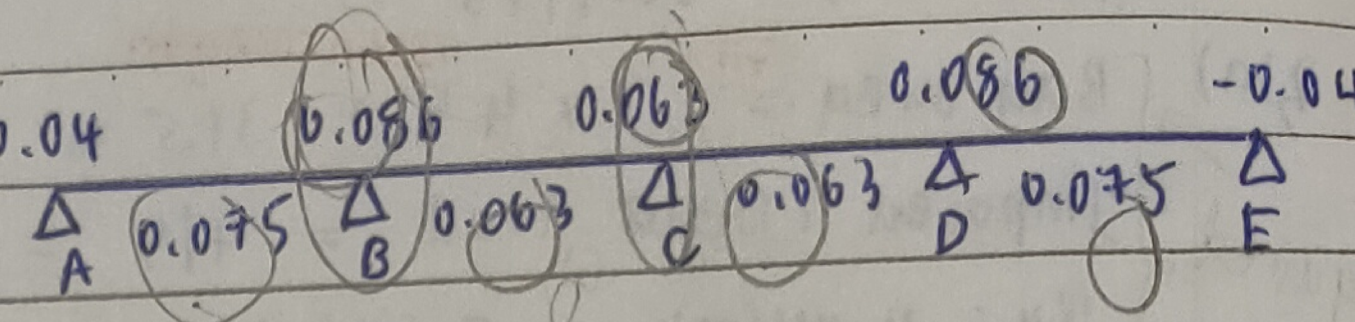
3 b) $F = 57.375$ kN

Fully restrained



Moments in critical sections :

At outer support (A & E) $= 0.04 \times 57.375 \times 4.5$

$= 4.928$ kN m

At near middle of end span $= 0.075 \times 57.375 \times 4.5$

$= 19.36$ kNm

at support B & D $= 0.086 \times 57.375 \times 4.5$

$= 22.2$ kNm

at support C $= 0.063 \times 57.375 \times 4.5$

$= 16.27$ kNm

middle of span BC & CD $= 0.063 \times 57.375 \times 4.5$

$= 16.27$ kNm

Most critical ⇒ at support B & D

$M = 22.2$ kNm

$d = 200 - 25 - 8/2 = 171$ mm

k_{bal} for redistribution $= 0.116$.

$k = \frac{M}{f_{ck} b d^2} = \frac{22.2 \times 10^6}{30 \times 1000 \times 171^2} = 0.0253 < 0.167$ (singly reinforced section)

$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right) = 0.977d < 0.95d$

$A_s = \frac{M}{0.87 f_{yk} z} = \frac{22.2 \times 10^6}{0.87 \times 500 \times 0.95 \times 171} = 314.15$ mm²/m

⇒ provide H8 bars at 150 mm spacing ($A_{s,prov} = 335$ mm²)

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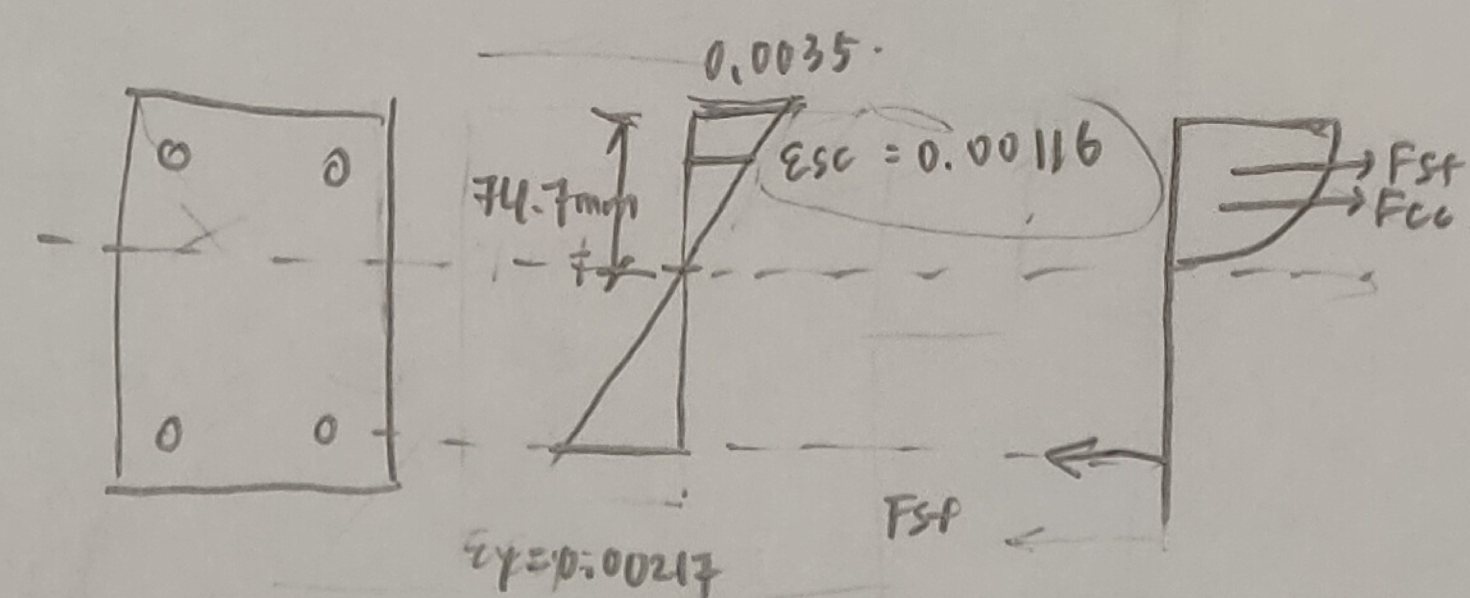
Pure Bending failure ($N=0$)

Stress strain diagram?

Assume compression steel not yielded.

$d' = 0.1h = 0.1(500) = 50$

$d = h - d' = 450.$



$\varepsilon_{s1} = \frac{x-50}{x}(0.0035)$

$f_{s1} = E_{s1}\varepsilon_{s1} = 200\times10^3 \times \frac{x-50}{x}(0.0035)$

$= \frac{700(x-50)}{x}$

$\Sigma F = 0$

$F_c + F_{s1} = F_{s2}$

$0.567 f_{cu} b s + f_{s1} A_{sc1} = 0.87 f_{y} A_{sc2}$

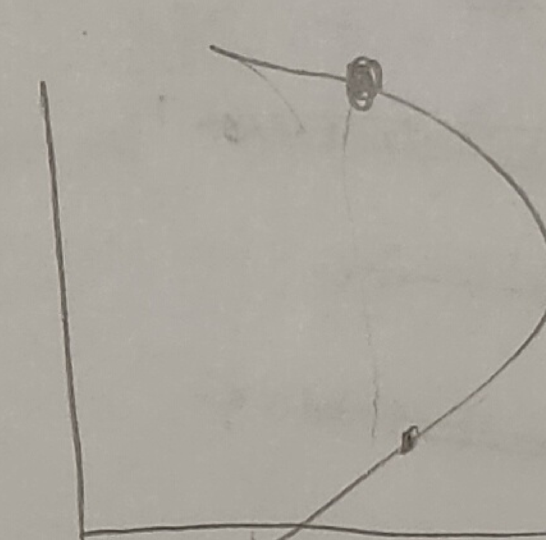
$0.567(30)(350)(0.8x) + \frac{700(x-50)}{x}(\frac{1}{2}\times0.02\times350\times500) = 0.87(500)(\frac{1}{2}\times0.02\times350\times500)$

$4762.8x^2 + 1225000(x-50) = 761250x$

$4762.8x^2 + 463750x - 61250000 = 0$

$x = 74.7\ mm$ //

$f_{s1} = 231.5\ MPa \Rightarrow F_{s1} = 405054\ N$



Taking moment abt ~~tension~~ centroid of tension steel:

$M = F_c(d - 0.4x) + F_s(d - d')$

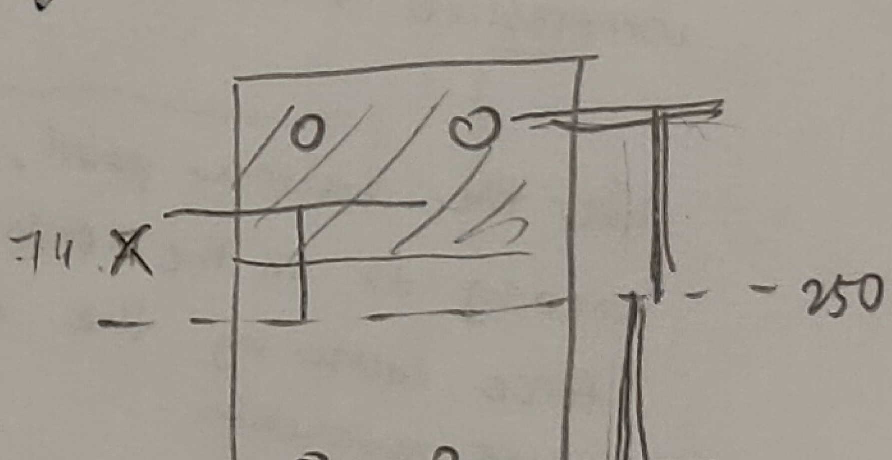
$= 4762.8x(d - 0.4x) + 405054(450 - 50)$

$= 149470780.9 + 162021600$

$= 311.5\ kNm$ //

b). $M = 311.5\ kNm$

$450 - 250 = 200$



Take M about centroid:

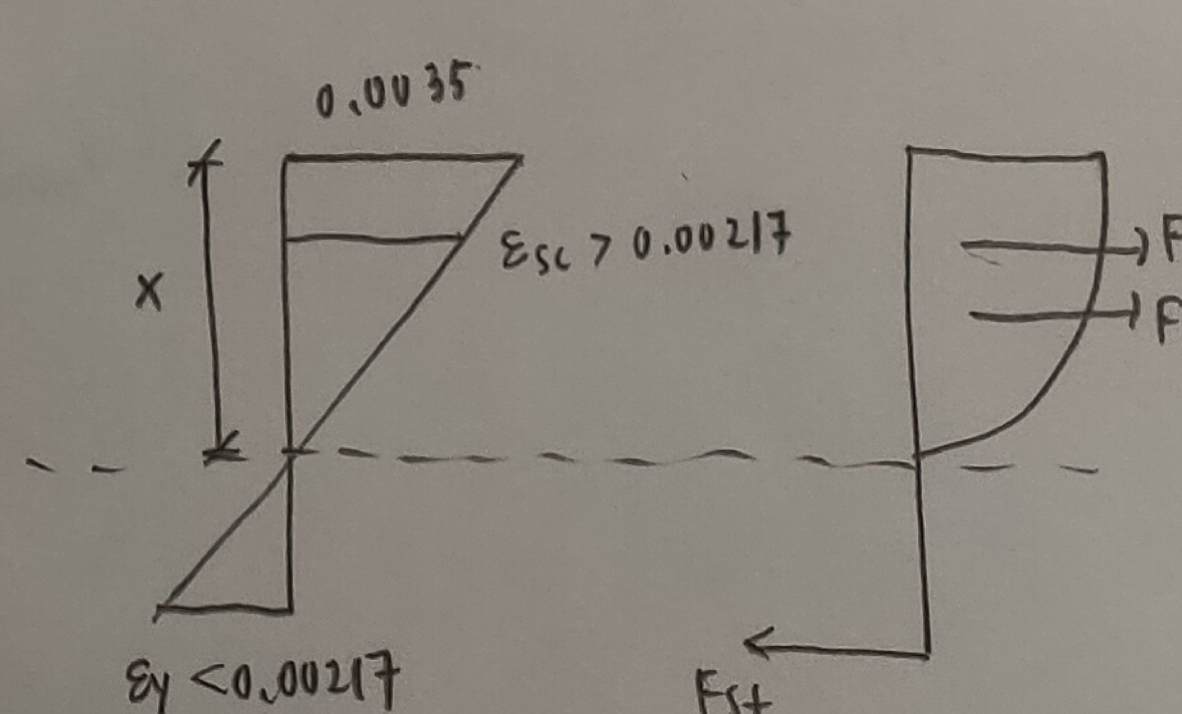
$311.5\times10^6 = F_c(250 - 0.4x) + (F_{sc} + F_{st})(250 - 50)$

$311.5\times10^6 = 0.567(30)(350)(0.8x)(250 - 0.4x) + [0.87(500)(\frac{3500}{2})$
$+ \frac{700(x-50)}{x}(\frac{3500}{2})](200)$

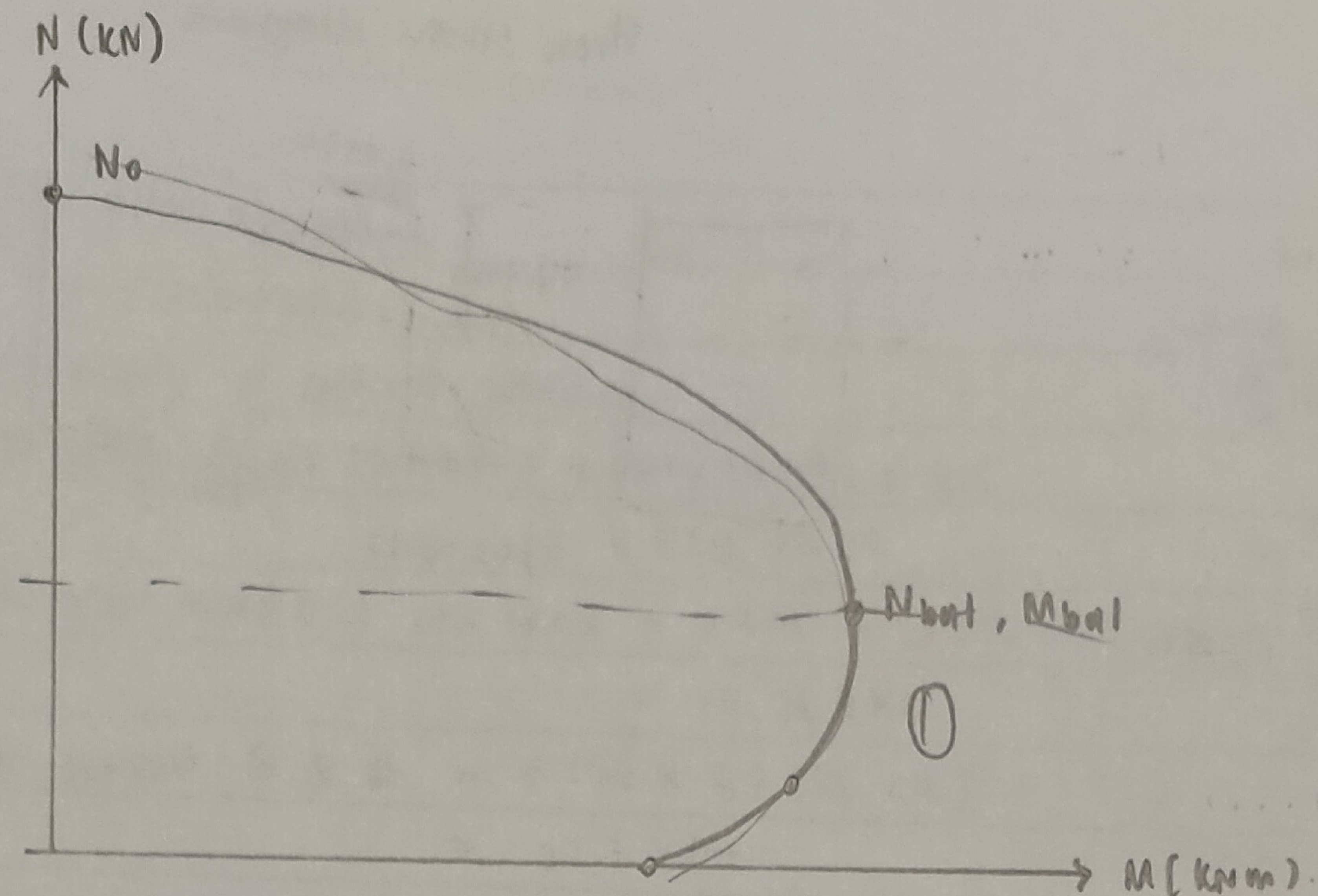
$311.5\times10^6 = 4762.8x(250 - 0.4x) + 152250000 + 245000000\frac{(x-50)}{x}$

$311.5\times10^6 x = 1190700x^2 - 1905.12x^3 + 152250000x + 245000000x - 12250000000$

$1905.12x^3 - 1190700x^2 - 85750000x + 12250000000 = 0$



c)



The region before the balance point, M & N are ~~positively~~ inversely related where the decrease in N is follow by an increase in M. The region after the balance point is where M & N are positively related, where a decrease in N is follow by a decrease in M as well.

Generally, moment tends to reduce the load capacity of a column. Hence, the larger the moment the column carries, the smaller is the axial load, hence depicts the first region.

~~Tension? ... ~~?

~~In After the~~

~~As axial loa~~dings

moment tends to reduce the load capacity of the column. Initially the whole column is under compression, then when moment is gradually increase, the moment introduce more compressive force onto the compressive side, thus decrease the capacity to take axial load

After the balance point, an increase in axial loady cause the moment taken to increase linearly as in the tension failure region, axial force helps to reduce the ~~...~~ tension force cause by the moment, thus increasing the capacity of the column to take more moment.

act against bending moment, stabilises concrete & reduce tension cracking