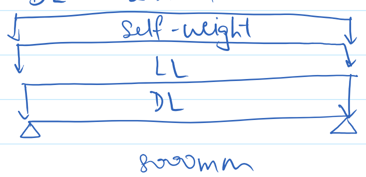
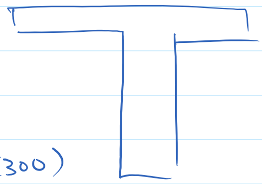
CV3011 - RC design 2017-2018 Sem 1

Sunday, 18 February 2018 9:53 PM

1. a) DL = 25 kN/m LL = 30 kN/m

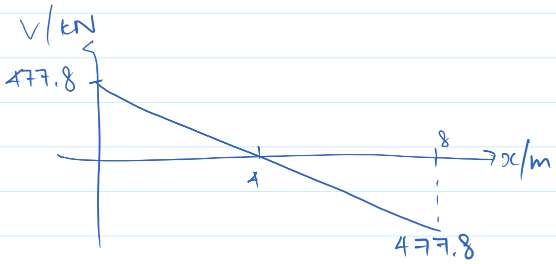


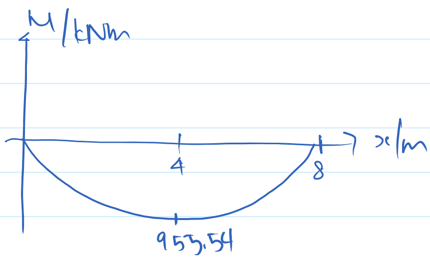


Cross-sectional area = 100(1200) + 700(300)
= 330 000 mm^2

Self-weight = $\frac{330\ 000}{1000^2} \times 1 \times 24$ = 7.92 kN/m

Total factored load = (25+7.92)(1.35) + 30(1.5)
= 119.442 kN/m × 8m
= 955.54 kN





Use ϕ32 reinforcement.

$d = 100 + 700 - 20 - \frac{32}{2} - 10 = 754$ mm

$k = \frac{M}{f_{ck} b d^2} = \frac{955.54 \times 10^6}{40(1200)(754^2)}$

$= 0.035016 < 0.167$

$z = \left(0.5 + \sqrt{0.25 - \frac{k}{1.134}}\right) d$

$= 0.9681 d \leq 0.95 d$

$z = 0.95 d = 716.3$ mm

$s = 2(754 - 716.3)$

$= 75.4$ mm < 100 mm

$\therefore s < h_f \Rightarrow$ neutral axis lies in flange

$A_s = \frac{955.54 \times 10^6}{0.87(500)(716.3)}$

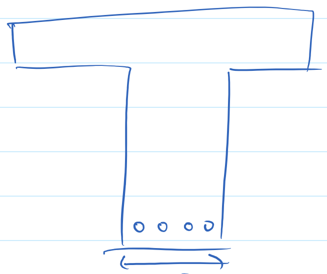
$= 3066.653$ mm^2

$A_{s,min} = 0.18\% = \frac{0.18}{100} \times 330\ 000 = 594$ mm^2

$A_s > A_{s,min} \rightarrow$ OK

Cross-sectional area of ϕ rebar = 804.248 mm^2

Use 4T32 rebar



b) Using T10 @ 200mm → $\frac{A_w}{s} = 0.785$

Shear @ 1d from support face $= 477.8 - \frac{477.8(0.754)}{4}$

$= 387.735$ kN

$$v_{Ed} = \frac{387.735}{300(754)(0.9)} = 1.9046 \text{ N/mm}^2$$

$$v_{Rd,max} \cot\theta_{(\theta=22^\circ)} = 4.63$$

$$v_{Ed} < v_{Rd,max} \cot\theta_{(\theta=22^\circ)}$$

$\therefore \theta = 22^\circ$

$$\frac{A_{sw}}{s} = \frac{v_{Ed}\, b_w}{f_{ywd} \cot\theta}$$

$$= \frac{1.9046(300)}{0.78 \times 250\,(2.5)}$$

$$= 1.0407 > 0.785$$

$\therefore$ Insufficient shear link reinforcement

$$S = \frac{A_{sw}}{1.0407} = \frac{2\pi\left(\frac{10}{2}\right)^2}{1.0407} = 150.94 \text{ mm}$$

Required link reinforcement = T10 @ 125 mm

Note: I honestly don't know why the answer is 100 mm.

→ Will lead to diagonal tension shear failure. EC2 uses a truss model where concrete stress is in compression and stirrups in tension. When there is insufficient stirrups, inclined cracks form in concrete, usually along 45° angle. Stirrups will yield and there will be further rotation of concrete cracks. Finally web crushing of beam occurs and element fails.

2. $F_{st} = 0.87(10^2\pi \times 3)(500) = 409977.8$ N

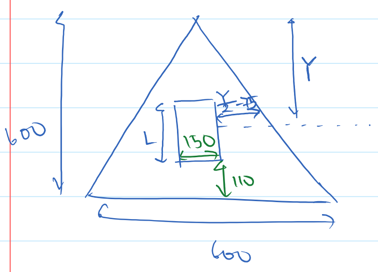
$d = 60 + 490 = 550$ mm

Without affecting beam flexural capacity: $\frac{x}{d} < 0.617$ → from a checker position

Assume concept of simplified stress block : $s = 0.8x$

$s < 0.8(0.617)(550)$

$s < 271.48\text{mm}$



Assume 2 cases :

① neutral axis lies above opening

② neutral axis lies along opening

Case ① :

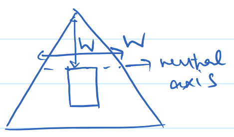


Fig A

Area above neutral axis = $\frac{W \times W}{2} = \frac{W^2}{2}$ (see Fig A)

$F_{st} = F_{cc}$

$409977.8 = 0.567(40) \times A = 0.567(40)\left(\frac{W^2}{2}\right)$

$= 11.34W^2$

$W^2 = 36153.25 \rightarrow A = 18076.63\ \text{mm}^2$

$W = 190.14\ \text{mm} < 271.48 \rightarrow$ beam flextural capacity not affected

Max $L = 490 - 190.14$

$= 299.86$

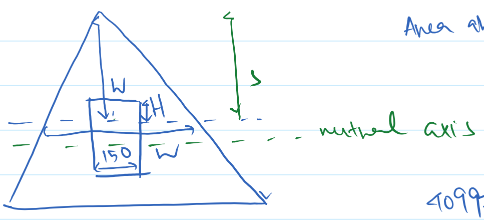
$\approx 300\ \text{mm}$

Case ② :

Assume $W = 271.48$ (max s possible to maintain beam flextural capacity)

Area above neutral axis $= \frac{W \times W}{2} - H \times 150$

$= \frac{W^2}{2} - 150H$

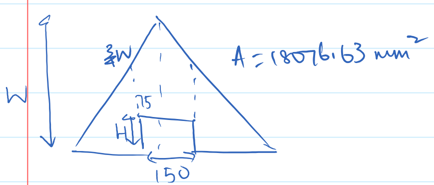


$F_{st} = F_{cc}$

$409977.8 = 0.567(40) \times A$

$A = 18076.63\ \text{mm}^2$

$= \frac{W^2}{2} - 150H$



$H < \frac{300-75}{300} W$

$H < 0.75W$

sub in $W = 271.48$:

$H = \frac{\frac{271.48^2}{2} - 18076.63}{150}$

$= 125.16 = 0.461W < 0.75W$

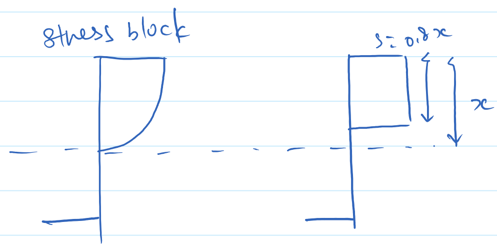
$\therefore L = 490 - (271.48 - 125.16)$

$= 343.68$

$\approx 343\ \text{mm}$

Largest $L = 343\ \text{mm}$ //

b) In the actual stress block, concrete stress is not constant above the neutral axis. For simple calculation of concrete strength mobilised, the simplified stress block is used. Concrete lying in ~~the~~ 80% of neutral axis depth is assumed to mobilise the maximum concrete stress of $0.567 f_{ck}$. As such, $s = 0.8x$ is used to calculate area of concrete mobilised.



mobilise the maximum concrete stress of 0.567 fck. As such, s = 0.8x is used to calculate area of concrete mobilised.

Maximum concrete strength used is $\frac{0.85 f_{ck}}{1.5} = 0.567 f_{ck}$

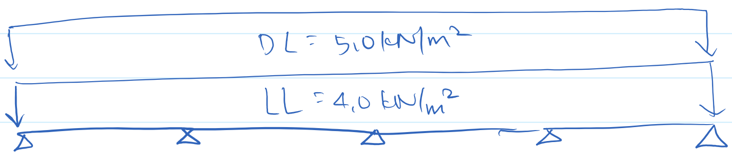
where $1.5 = \gamma_m$, the factor of safety assigned to concrete strength and 0.85 is the difference between concrete cylinder strength and concrete strength in bending.

Together, $s = 0.8x$, $0.567 f_{ck}$ and the simplified stress block are used to estimate compressive concrete force with reasonable accuracy.

c) Over-reinforced beams are beams with too much tensile steel rebar provided. The concrete in the beam will reach its strain limit before the steel rebar, resulting in failing of the concrete first. As concrete gives way and there is nothing to take compression, the beam will have brittle failure and fail suddenly. As such, beams should not be over-reinforced.

In under-reinforced beams, steel is provided such that the steel yields prior to concrete failing. This results in steel yielding till ultimate strain before failure. As steel fails in ductile failure, the beam will not fail suddenly.

3.



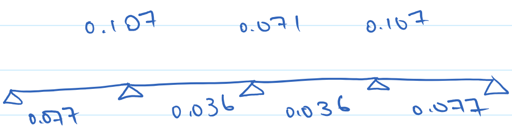
Factored load, $w = (1.35(5.0) + 1.5(4.0))$
$= 12.75$ kN/m

Bay area $= 4.5(7) = 31.5\ m^2 > 30$

$\frac{Qk}{Gk} = \frac{4}{5} = 0.8 < 1.25$

$Qk = 4 kN/m^2 < 5 kN/m^2$

$\therefore$ Use single load case method ⇒ assume non-restrained



$F = wl = 12.75 \times 4.5 = 57.375$ kN

Elastic moment at : AB midspan & DE midspan $= 0.077(57.375)(4.5) = 19.88$ kNm/m

B & D $= -0.107(57.375)(4.5) = 27.626$ kNm/m

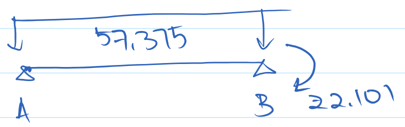
BC & CD midspan $= 0.036(57.375)(4.5) = 9.295$ kNm/m

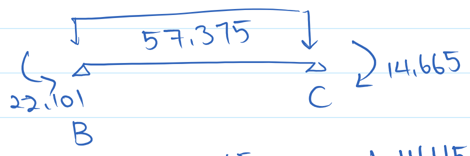
C $= -0.071(57.375)(4.5) = 18.331$ kNm/m

Allow 20% redistribution at B, C & D

Moment at B & D $= -27.626 \times 0.8 = -22.101$ kNm

C $= -18.331 \times 0.8 = -14.665$ kNm





$F_{AB} = \dfrac{57.375\left(\frac{4.5}{2}\right) - 22.101}{4.5}$

$= 23.7762$ kN/m

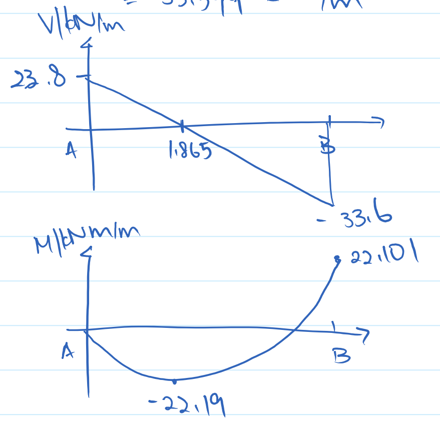
$F_{BA} = 57.375 - 23.776$

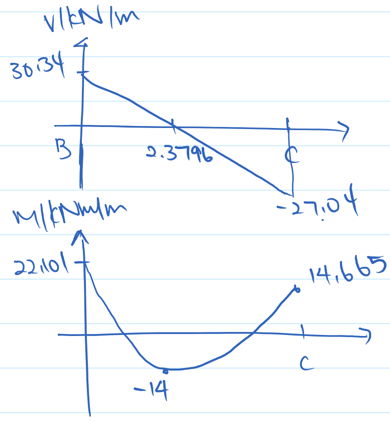
$= 33.599$ kN/m

$F_{BC} = \dfrac{57.375\left(\frac{4.5}{2}\right) + 22.101 - 14.665}{4.5}$

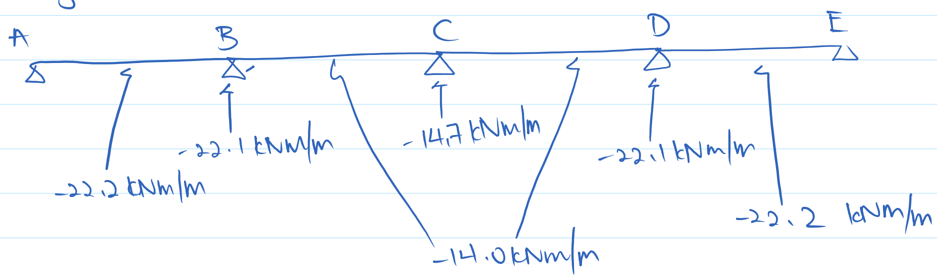
$= 30.34$ kN/m

$F_{CB} = 27.035$ kN/m





slab design moments



b) Using moment coeff:

Moments at A & E : $-0.04(57.375)(4.5)$

$= -10.328$ kNm

AB & DE midspan $= 0.075(57.375)(4.5)$

$= 19.364$ kNm

B & D $= -0.086(57.375)(4.5)$

$= -22.204$ kNm

BC & CD midspan $= 0.063(57.375)(4.5)$

$= 16.266$ kNm

C $= -0.063(57.375)(4.5)$

$= -16.266$ kNm

Most critical section = at support B & D

$= 22.204$ kNm

$d = 200 - 25 - \frac{8}{2} = 171$ mm

$k = \dfrac{22.204 \times 10^6}{30(1000)(171^2)} = 0.02531 < 0.116 \Rightarrow k_{bal}$ for 20% redistribution

$z = \left(0.5 + \sqrt{0.25 - \frac{k}{1.134}}\right) d = 0.97716\, d > 0.95d$

$\therefore\ z = 0.95\, d = 0.95(171)$

$= 162.45\ \text{mm}$

$A_s = \dfrac{22.204 \times 10^6}{0.87(500)(162.45)}$

$= 314.21\ \text{mm}^2/\text{m}$

For $f_{ck} = 30\ \text{MPa}$, $A_{s,min} = 0.15\%$

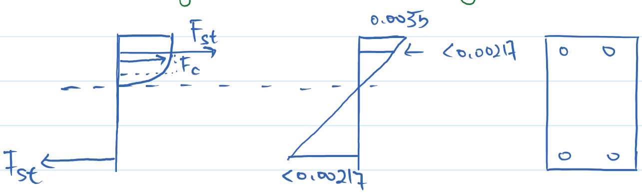
$= 300\ \text{mm}^2 < A_s = 314.21\ \text{mm}^2$

$\therefore$ Use $315\ \text{mm}^2/\text{m}$ main flexural reinforcement

4. a) $A_{sc} = 350 \times 500 \times 0.02 = 3500\ \text{mm}^2$

$d = 0.9(500) = 450\ \text{mm}$

Stress diagram Strain diagram



Pure bending $(N = 0)$.

Assume compression steel has not yielded:

$\varepsilon_{s1} = \dfrac{x - 50}{x} \times 0.0035$

$f_{s1} = \varepsilon_{s1} E_s = \dfrac{x-50}{x} \times 0.0035 \times 200 \times 10^3$

$= 700\left(\dfrac{x-50}{x}\right)$

$0.567(30) \times 350 \times 0.8x + 700\left(\dfrac{x-50}{x}\right)\left(\dfrac{3500}{2}\right) = 0.87(500)\left(\dfrac{3500}{2}\right)$

$4762.8x + \dfrac{1225000x - 61250000}{x} = 761250$

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

$x = 74.726\ \text{mm}$ or $x = -172.096$ (rej.)

$f_{s1} = 231.622\ \text{N/mm}^2$

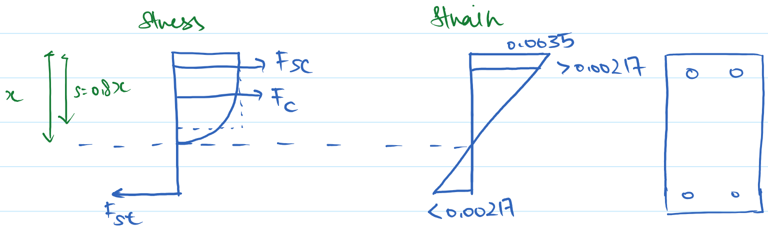
About tension steel:

$M = 0.567(30)(350)(0.8 \times 74.726)(450 - 0.4 \times 74.726) + \dfrac{3500}{2}(231.622)(450-50)$

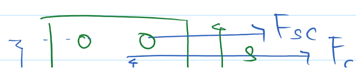
$= 311.656\ \text{kNm}$

b) Compression steel has yielded,

Stress Strain



$M = 311.656\ \text{kNm}$

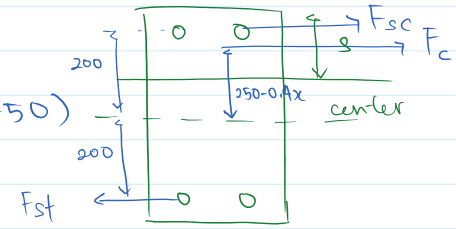


f_{st} < 0.00217

$M = 311.656$ kNm

About center of column:

$$311.656 = F_c(250 - 0.4x) + (F_{sc} + F_{st})(250 - 50)$$



$$\varepsilon_{st} = 0.0035 \times \frac{450 - x}{x}$$

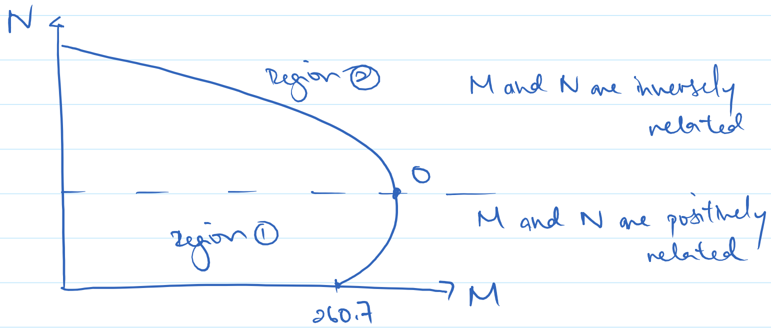
$$f_{st} = 0.0035 \times \frac{450 - x}{x} \times 200 \times 10^3$$

$$= \frac{315000 - 700x}{x}$$

$$311.656 = 0.567(30)(350)(0.8x)(250 - 0.4x) + \left(\frac{3500}{2} \times 0.87 \times 500 + \frac{3500}{2} \times \frac{315000 - 700x}{x}\right)(200)$$

$$= 4762.8x(250 - 0.4x) + \left(761250 + \frac{55125 \times 10^4 - 1225 \times 10^3 x}{x}\right)(200)$$

c)



In region ① : As axial load increases, the column can withstand more bending moment. This is because axial load acts against the bending moment. It stabilises the concrete and reduces tensile cracking.

In region ② : As axial load increases past point of balanced failure (point O) the bending moment capacity of column starts to decrease. This is caused by axial load starting to cause concrete crushing. Concrete fails first in compression and cannot withstand as much bending moments

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Signature :

Note : This paper was tough.
Good luck for your papers!