CV3011 – REINFORCED CONCRETE DESIGN

AY 20/21 S1

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1 (a) Redistributed sagging moment $= 285 + 350\left(\frac{0.15}{2}\right) = 311.25\,kNm$.

Assume $\phi 20$ used, $d = 550 - 35 - 10 - \frac{20}{2} = 495mm$

$K = \frac{M}{f_{ck}bd^2} = \frac{311.25 \times 10^6}{35 \times 300 \times 495^2} = 0.121 < 0.167 \rightarrow$ no comp. steel required.

$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right) = 495\left(0.5 + \sqrt{0.25 - \frac{0.121}{1.134}}\right) = 434.9 < 0.95d = 470.25$.

$A_s = \frac{M}{0.87 f_{yk} z} = \frac{311.25 \times 10^6}{0.87(500)(434.9)} = 1645.2mm^2 \rightarrow$ Provide 6H20 (1886mm²)

NOTE: My solution is different from the numerical solution of 1900mm². I deduced that in the calculation of the numerical solution, the height was taken to be 500mm like in various examples. The resulting d value would then be 445mm and the calculated values of K, z and A_s would be 0.150, 375 and 1908mm² respectively.

(b) Redistributed hogging moment $= 350 \times 0.85 = 297.5\,kNm$

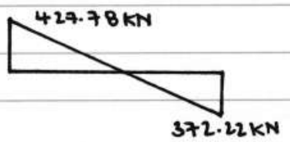
$K = \frac{M}{f_{ck}bd^2} = \frac{297.5 \times 10^6}{35 \times 300 \times 495^2} = 0.1156 < 0.129 \rightarrow$ no comp. steel required

$z = d\left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right) = 495\left(0.5 + \sqrt{0.25 - \frac{0.1156}{1.134}}\right) = 438mm < 0.95d = 470.25mm$

$A_s = \frac{M}{0.87 f_{yk} z} = \frac{297.5 \times 10^6}{0.87(500)(438)} = 1561mm^2 \rightarrow$ provide 5H20 (1571mm²).

2 $w = 800/7.2 = 111.11\,kN/m$, $d = 600$

SFD:



$\rightarrow$ LHS: shear force $= (800 \times 3.6 + 400 - 200)/7.2 = 427.78\,kN$

RHS: shear force $= (800 \times 3.6 + 200 - 400)/7.2 = 372.22\,kN$

zone ①:

$V_{Ef} = V_1 - w \times \frac{width}{2} = 427.78 - 111.11 \times \frac{0.4}{2} = 405.56\,kN$

$V_{Rd,max(22)} = 0.124 b_w d\left(1 - \frac{f_{ck}}{250}\right) f_{ck} = 0.124(300)(600)\left(1 - \frac{25}{250}\right)(25)(10^{-3}) = 502.2\,kN$

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

zone ②:

At 1.0d from support face, $V_{E1d} = 427.78 - 111.11\left(\frac{0.4}{2} + 0.6\right) = 338.89\,kN$.

$\frac{A_{sw}}{s_2} = \frac{V_{E1d}}{0.78 d f_{yk} \cot\theta} = \frac{338.89 \times 10^3}{0.78(600)(500)(2.5)} = 0.5793 \rightarrow$ provide $\phi 8$ @ 150 ($\frac{A_{sw}}{s} = 0.671$)

spacing $= 150 < 0.75d = 450mm \rightarrow$ ok!

zone ③:

$\frac{A_{sw,min}}{s} = \frac{0.08 f_{ck}^{0.5} b_w}{f_{yk}} = \frac{0.08 \times 25^{0.5} \times 300}{500} = 0.24 \rightarrow$ provide $\phi 8$ @ 400 ($\frac{A_{sw}}{s} = 0.251$)

spacing $= 400 < 0.75d = 450mm \rightarrow$ ok!

$V_{min} = \frac{A_{sw}}{s} 0.78 d f_{yk} \cot\theta = 0.251(0.78)(600)(2.5)(10^{-3}) = 146\,kN$

Calculate no. of stirrups in ① & ②:

$x_2 = \frac{405.56 - 146}{111.11} = 2.336m$

no. of links $= 1 + \frac{x_2}{s_2} = 1 + \frac{2.336}{0.150} = 16.57 \approx 17$.

length $= (\text{no. of links} - 1) \times s = (17 - 1) \times 0.15 = 2.4m$

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3 (a) $\phi10@250 = 314\ mm^2/m$, $d = 190 - 30 - \frac{10}{2} = 155mm$, $z = 0.95d = 147.25mm$

For three edges discontinuous (one long edge continuous), mid-span coefficients are as follows, $\beta_{sx} = 0.060$, $\beta_{sy} = 0.044$.

$M_{sx} = \beta_{sx} \cdot n \cdot l_x^2 = 0.060 \cdot n \cdot 5^2 = 1.5n$

$n = 1.35(25 \times 0.19 + 1.25) + 1.5(q_k) = 8.1 + 1.5q_k$

$A_s = \frac{M \times 10^6}{0.87 f_{yk} z} \rightarrow 314 = \frac{M \times 10^6}{0.87(500)(147.25)} \rightarrow M = 20.113 kNm/m$

$= 1.5n$

$= 1.5(8.1 + 1.5q_k)$

$\therefore q_k = 3.5391 kN/m^2$

(b) $n = 8.1 + 1.5q_k = 8.1 + 1.5(4.5) = 14.85 kN/m^2$

$M = 1.5n = 22.275 kNm/m$.

$K = \frac{M}{f_{ck}bd^2} = \frac{22.275 \times 10^6}{40(1000)(155)^2} = 0.0232 < 0.167$ (no comp. steel required).

$z = d(0.5 + \sqrt{0.25 - \frac{K}{1.134}}) = 155(0.5 + \sqrt{0.25 - \frac{0.0232}{1.134}}) = 152mm > 0.95d = 147.25mm$.

$A_s = \frac{M}{0.87 f_{yk} z} = \frac{22.275 \times 10^6}{0.87(500)(147.25)} = 347.75 mm^2/m \rightarrow$ provide H10@ 200mm (393mm²/m)

(c) actual L/D $= \frac{5000}{155} = 32.3$

end span basic L/D = 39

allowable L/D $= 39 \times \frac{393}{347.75} = 44.1$

Since actual L/D < allowable L/D, deflection check pass.

4 (a) $N + W = 600 + 800 + (3 \times 3 \times 0.6) \times 25 = 1535 kN$

$P = \frac{N+W}{A} = \frac{1535}{3 \times 3} = 170.56 kN/m^2 \leq q_a = 250 kN/m^2 \rightarrow$ ok!

ult. load $= 1.35 \times 600 + 1.5 \times 800 = 2010 kN$

ult. pressure $= \frac{2010}{3 \times 3} = 223.33 kN/m^2$

moment at face of column $= 223.33 \times (3 \times 1.3) \times \frac{1.3}{2} = 566.15 kN/m$

$C = T \rightarrow 0.567 f_{ck} bd = 0.87 f_{yk} A_s \rightarrow 0.567(40)(3000)(0.8x) = 0.87(500)(3770) \rightarrow x = 24.103mm$

$z = d - 0.4x = (600 - 60 - 10) - 0.4(24.103) = 520.36mm$

$z/d = 0.982 > 0.95$, use $z = 0.95d = 503.5mm$

moment resistance $= T \cdot z = (0.87 f_{yk} A_s)(z)$

$= 0.87(500)(3770)(503.5)$

$= 825.7 kN/m > 566.15 kN/m \rightarrow$ ok!

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4 (b) $d_x = 600-60-10 = 530mm$, $d_m = 600-60-20 = 520mm$, $d_x = 600-60-10-20 = 510mm$

① Max shear stress v_{Ed} at column circumference:

$v_{Ed} = \frac{N_u}{\pi D \cdot d_m} = \frac{2010}{\pi(400)(520)} = 3.076 N/mm^2$

max shear resistance, $v_{Rd,max} = 0.5\left[0.6\left(1-\frac{f_{ck}}{250}\right)\right]\frac{f_{ck}}{1.5} = 6.72 N/mm^2 > v_{Ed} = 3.076 N/mm^2$

② Vertical shear at 1.0d from edge (x-dir):

design shear $V_{Ed} = P_{ult} \times area = 223.33 \times (1.3-0.53)(3) = 515.89kN$

reinforcement ratio $\rho_x = \frac{3770}{3000(530)} = 0.237\% < 2\% \rightarrow$ OK!

shear resistance of concrete w/o shear reinforcement, $v_{Rd,c} = \frac{0.18}{\gamma_c}k(100\rho_x f_{ck})^{1/3}$

$= 0.12\left(1+\sqrt{\frac{200}{520}}\right)(0.237 \times 40)^{1/3}$

$= 0.410$

$v_{Rd,c} \leq v_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.035\left(1+\sqrt{\frac{200}{520}}\right)^{3/2}(40)^{1/2} = 0.4540$

$V_{Rd,c} = v_{Rd,c}(bd) = 0.454(3000)(530)(10^{-3}) = 721.86kN > V_{Ed} = 515.89kN \rightarrow$ OK!

③ Punching shear at 2.0d from edge of column:

critical perimeter $u_1 = 2\pi(200+2\times520) = 7791.1mm$

punching force $V_{Ed} = 223.33(3\times3 - \pi(0.2+2\times0.52)^2) = 931.17kN$

average reinforcement ratio, $\rho_l = \sqrt{\left(\frac{3142}{3000\times530}\right)\left(\frac{3142}{300\times510}\right)} \times 100\% = 0.20145\% < 2\% \rightarrow$ OK!

shear resistance w/o shear reinforcement, $v_{Rd,c} = \frac{0.18}{\gamma_c}k(100\rho_l f_{ck})^{1/3}$

$= 0.12\left(1+\sqrt{\frac{200}{520}}\right)(0.20145 \times 40)^{1/3}$

$= 0.38978$

$v_{Rd,c} \leq v_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.035\left(1+\sqrt{\frac{200}{520}}\right)^{3/2}(40)^{1/2} = 0.4565$

$V_{Rd,c} = v_{Rd,c}(u_1 d_m) = 0.4565(7791.1)(520)(10^{-3})$

$= 1849.4kN > V_{Ed} = 931.17kN \rightarrow$ OK!