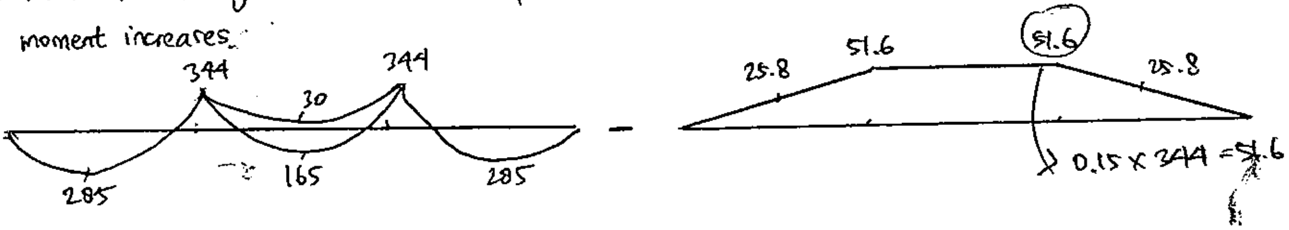
CV3011

RC Design

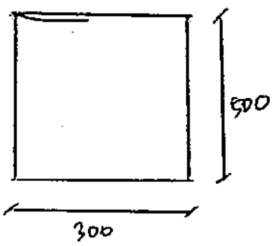
PYP Sem 1 2014/2015

) (a) → Design for sagging moment at endspan. Due to moment redistribution factor of $\delta = 0.85$, sagging moment increases.



$M_{Ed} = 285 + 25.8 = 310.8$ kNm

→



$f_{ck} = 35 N/mm^2$ cover = 35mm $f_{yk} = 500 N/mm^2$

ϕ stirrup = 13mm $E_s = 200000 N/mm^2$

assume we are using ϕ32 bar, $d = 500 - 35 - 13 - \frac{32}{2} = 436$mm

→ Since the sagging moment increase, take $\delta = 1.0$

$$K = \frac{M_{Ed}}{f_{ck} b d^2} = \frac{310.8 \times 10^6}{35 \times 300 \times 426^2} = 0.156 < K_{bal} = 0.167$$

∴ No compression steel needed

$$z = \left\{0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right\} d = 0.836d = 364.36 \text{mm}$$

$$A_{s,req} = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{310.8 \times 10^6}{0.87 \times 500 \times 364.36} = 1960.9 \text{ mm}^2$$

Use 3H32 steel bar, $A_{s,prov} = 2412.74 \text{mm}^2$

→ Check : $\rho = \frac{2412.74}{300 \times 500} = 1.6\% <$ $< 4\%$, $> 0.13\%$ ⇒ OK!!

→ Due to $\delta = 0.85$, hogging moment decreases.

$M_{Ed} = 344 - 51.6 = 292.4\ kNm$

→ take $\delta = 0.85$ ($K_{bal} = 0.129$)

$$K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{292.4\times10^6}{35\times300\times436^2} = 0.146 > K_{bal} = 0.129$$

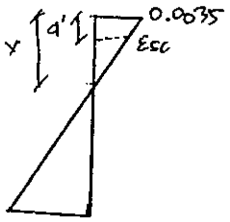
∴ Compression reinforcement is needed

→ check if compression steel has yielded or not. (For $\delta = 0.85$, $\frac{d'}{d}_{lim} = 0.125$)

$$\frac{d'}{d} = \frac{64}{436} = 0.147 > 0.125$$

∴ Compression steel has not yielded → find f_{sc} from strain diagram

For $\delta = 0.85$, $x = 0.328d$ (refer to redistribution table)



$$\frac{(x - d')}{x} = \frac{\varepsilon_{sc}}{0.0035}$$

$$\varepsilon_{sc} = 0.0035\left(1 - \frac{d'}{x}\right) = 0.0035\left(1 - \frac{d'}{0.328d}\right)$$

$$\varepsilon_{sc} = 0.00193$$

$$f_{sc} = E_s\varepsilon_{sc} = 200000\times0.00193 = 386\ N/mm^2$$

→ Compression steel required calculation

$$A_{sc,req} = \frac{M - K_{bal}f_{ck}bd^2}{f_{sc}(d-d')} = \frac{292.4\times10^6 - 0.129\times35\times300\times436^2}{386(436-64)} = 242.7\ mm^2$$

use 3H16 steel bar, $A_{sc,prov} = 603.2\ mm^2$

→ Check: $\rho = 0.4\% \; < 4\%,\; > 0.13\%$ ⇒ ok!

→ Tension steel required

$$z = d - 0.4x = 436 - 0.4(0.328\times436) = 378.8\ mm$$

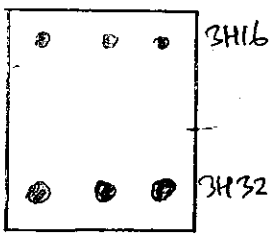
$$A_{st} = \frac{K_{bal}f_{ck}bd^2}{0.87f_{yk}z} + A_s'\left(\frac{f_{sc}}{0.87f_{yk}}\right) = \frac{0.129\times35\times300\times436^2}{0.87\times500\times378.8} + 242.7\left(\frac{386}{0.87\times500}\right)$$

$$= 1778.4\ mm^2$$

use 3H32 steel bar, $A_{st,prov} = 2412.74\ mm^2$

→ check $\rho = 1.6\% \; < 4\%,\; > 0.13\%$ ⇒ ok!

→ Cross section arrangement



(Drawing is not to scale)

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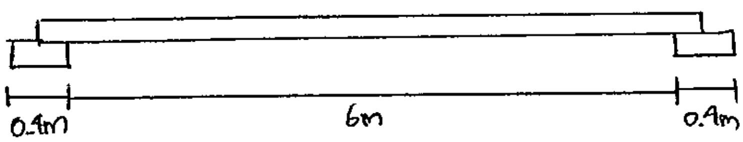


Drawing is not to scale

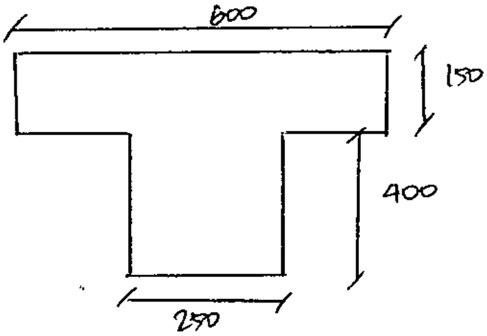
→ Find the actions

$g_k = 40\,kN/m \quad q_k = 35\,kN/m$

$w = 1.35 g_k + 1.5 q_k = 1.35(40) + 1.5(35) = 106.5\,kN/m$

Maximum moment occur at midspan, $M_{Ed} = \frac{wL^2}{8} = \frac{106.5 \times 6.4^2}{8} = 545.28\,kNm$

a) → Find tension reinforcement



$f_{ck} = 30\,N/mm^2$

$f_{yk} = 500\,N/mm^2$

Cover = 30mm

Stirrups ϕ = 10mm

Assume use H32 bar,

$d = 550 - 30 - 10 - \frac{32}{2}$

$d = 494\,mm$

$K = \frac{M_{Ed}}{f_{ck} b d^2} = \frac{545.28 \times 10^6}{30 \times 600 \times 494^2} = 0.124 < 0.167$

∴ no compression reinforcement needed

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

$= 0.875 d = 432.19\,mm$

$\frac{s}{2} = d - z \rightarrow s = 123.62\,mm < h_f = 150\,mm$

$A_s = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{545.28 \times 10^6}{0.87 \times 500 \times 432.19} = 2900.38\,mm^2$

use 4T32 bars, $A_{s,prov} = 3217\,mm^2$

→ check $\rho = 2.34\%$ $< 4\%$, $> 0.13\%$ ⇒ ok!

→ Deflection Check

See Fig 7 from IStructE, basic l/d for $\rho \geq 1.5\%$ and simply supported beam is 14

∴ basic $l/d = 14$

See Fig 6, $F_1 = 1 - 0.1\left\{\left(\frac{b_f}{b_w}\right) - 1\right\} = 0.86$

$F_2 = 1 \quad (l_{eff} < 7)$

$F_3 = \frac{310}{\sigma_s}$

→ Find σ_s

$\sigma_s = \sigma_{su}\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right) = \boxed{\frac{50.5}{106.5}} \times 0.87 \times 500 \times \left(\frac{2900.38}{3217}\right) = 186$

↓

at SLS : $\psi_2 = 0.3$ (residential)

$E_{d,SLS} = G_k + \psi_2 Q_k = 50.5$

at ULS : $E_{d,ULS} = 1.35 G_k + 1.5 Q_k = 106.5$

∴ $F_3 = \frac{310}{186} = 1.67$

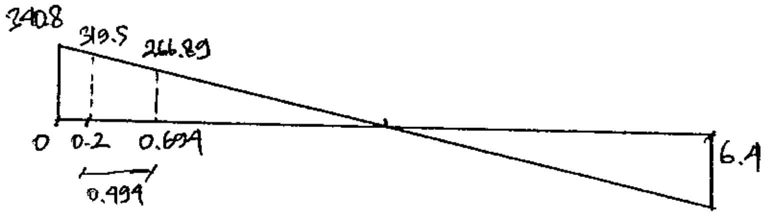
→ Limiting Deflection (δ_{lim}) = $basic\ l/d \times F_1 \times F_2 \times F_3$

$= 14 \times 0.86 \times 1 \times 1.67 = 20.1$

actual deflection $= \frac{6.4m}{550mm} = 11.64$

since $\delta_{actual} < \delta_{lim}$, deflection is satisfying.

(b) Shear Force Diagram



→ find θ

V_{Ef} (Shear at support face) = 319.5 kN

$V_{Rd,max(45)} = 0.18\ b_w d\ (1 - \frac{f_{ck}}{250}) f_{ck}$

$= 0.18\ (250)(494)(1 - \frac{30}{250})\ 30 = 586.872\ kN > V_{Ef} = 319.5\ kN$

∴ Shear resistance at support face is adequate.

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

$= 404.3\ kN$

Since $V_{Rd,max(22)}$ is still greater than V_{Ef}, we take $\theta = 22°$

→ At distance 1d from support face

$V_{E1d} = 266.89\ kN$

$\frac{A_{sw}}{s} = \frac{V_{E1d}}{0.78\ d\ f_{yk} \cot\theta} = \frac{266.89 \times 10^3}{0.78 \times 494 \times 500 \times \cot 22} = 0.56\ mm^2/mm$

use ϕ8–175 stirrups

→ Minimum stirrups

$\frac{A_{sw,min}}{s} = \frac{0.08\ f_{ck}^{0.5} b_w}{f_{yk}} = \frac{0.08 \times 30^{1/2} \times 250}{500} = 0.219\ mm^2/mm$

use ϕ8 – 350 stirrups

$V_2 = 0.0624\ (f_{ck}^{0.5} b_w d) \cot\theta$

$= 0.0624\ (30^{0.5} \times 250 \times 494) \times \cot 22 = 104.47\ kN$ → Shear at distance $X_2 = 2.219\ m$,

measured from face of support,

$X = 2.219 - 0.412 = 2.019\ m$

→ For zone 1 and 2

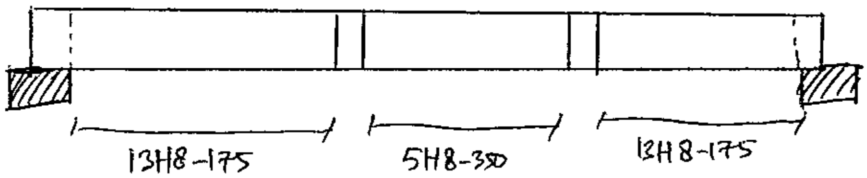
∴ Provide φ8-175

Number of links : $1 + \frac{2.019}{0.175} = 13$ stirrups spaced over a distance of (13-1) 175 = 2.1m

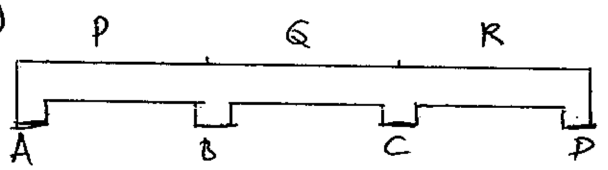
→ For zone 3

Number of links = $\frac{6000 - 2 \times 2100}{350} - 1 = 5$ stirrups

→ longitudinal sketch



3) (a)



→ Find the actions

$G_k = 5\ kN/m^2$

$Q_k = 3.2\ kN/m^2$

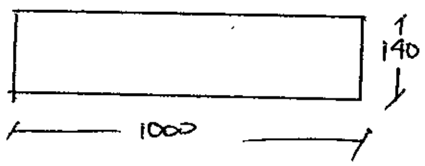
$w = 1.35G_k + 1.5Q_k = 11.55\ kN/m^2$

$f_{ck} = 30\ N/mm^2$ $f_{yk} = 500\ N/mm^2$ $f_{ctm} = 2.9\ N/mm^2$ $\gamma_c = 25\ kN/m^3$

Slab Thickness = 140mm

Cover = 25mm

ϕ bar = 8mm



$d = 140 - 25 - \frac{8}{2}$
$= 111mm$

→ Find Moments

$F = wl = 11.55 \times 3.5 = 40.425\ kN$

Point	Design Moment
D	$-0.04Fl = 5.66\ kNm$
R	$0.075Fl = 10.61\ kNm$
C	$-0.086Fl = 12.17\ kNm$
Q	$0.063Fl = 8.91\ kNm$

→ Find Steel Reinforcement

Point	Moment	$K = \frac{M_{Ed}}{f_{ck}bd^2}$	z (for $z > 0.95d$, take $z = 0.95d$)	$A_{s,req} = \frac{M}{0.87f_{yk}z}$	$A_{s,min} = 0.0015bd$	$A_{s,prov}$	Steel provided
D	5.66	0.0153	105.45	123.39	166.5	201	H8-250
R	10.61	0.029	105.45	231.3	166.5	251	H8-200
C	12.17	0.033	105.45	265.31	166.5	279	H8-180
Q	8.91	0.0241	105.45	194.24	166.5	251	H8-200

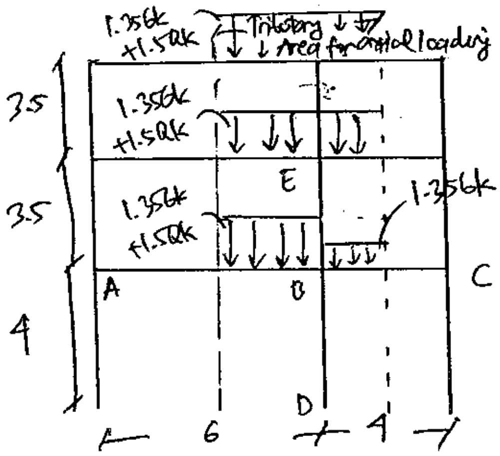
Also provide transverse reinforcement : $0.2A_{s,min} = 33.3mm^2$ but not less than $0.0015bd = 166.5mm^2$

→ provide H8-300 ($168mm^2$)

b) $EI_{col} : EI_{beam} = 0.35 : 1$

$G_k = 16\ kN/m$ $\quad Q_k = 17.6\ kN/m \quad \gamma = 25\ kN/m^3$

$w = 1.35G_k + 1.5Q_k = 48\ kN/m$



→ For design axial loading

∘ loading on floor / beam

$= 48 \times (3+2) \times 2 + 48 \times 2 + 1.35(16) \times 2$

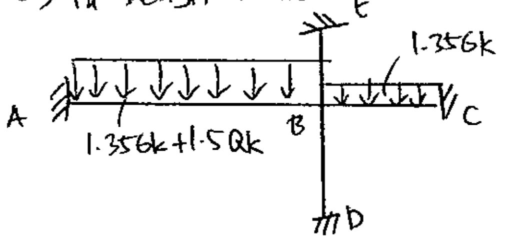
$= 667.2\ kN$

∘ loading of column self weight

$= 1.35 \times [0.3 \times 0.3 \times 2.5 \times 25] \times 2 = 21.2625\ kN$

∘ Total Axial Force $(N) = 667.2 + 21.2625 = 688.4625\ kN$ //

→ For Design Moment



∘ Stiffness of each member

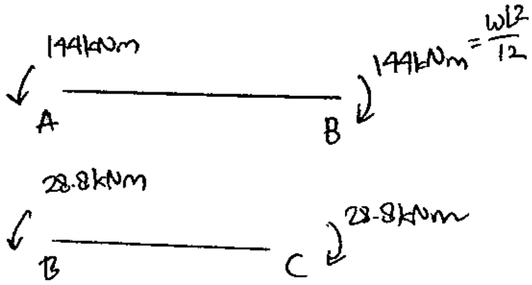
$k_{BE} = \dfrac{EI_{col}}{3.5}$ $\quad$ Total $k = \dfrac{0.35EI_{beam}}{3.5} + \dfrac{0.35EI_{beam}}{4} + \dfrac{EI_{beam}}{6} + \dfrac{EI_{beam}}{4}$

$k_{BD} = \dfrac{EI_{col}}{4}$ $\quad = 0.604\ EI_{beam}$

$k_{AB} = \dfrac{EI_{beam}}{6}$

$k_{BC} = \dfrac{EI_{beam}}{4}$

∘ FEM in each beam



Moment in col BD $= (144 - 28.8) \times \dfrac{\frac{0.35EI_{beam}}{4}}{0.604\ EI_{beam}}$

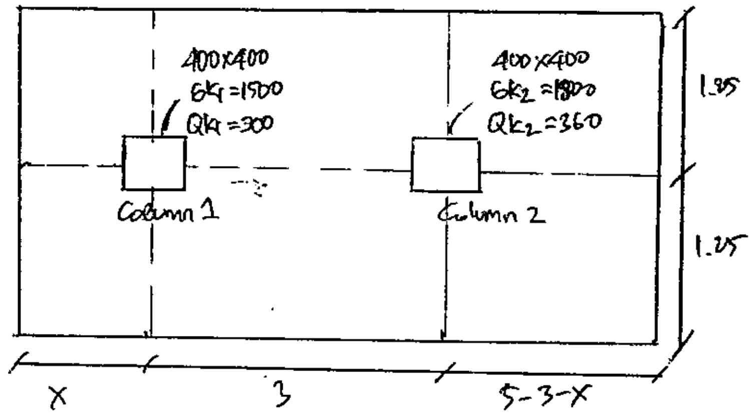
$= 16.68\ kNm\ (CCW)$

$M_{BD,min,Ed} = N e_{min} = 688.4625 \times \max\left(\frac{h}{30}, 20\right)$

$= 688.4625 \times \max\left(\frac{300}{30}, 20\right)$

$= 13.8\ kNm < M_{BD} = 16.68\ kNm$

∴ Design Moment $= 16.68\ kNm$ //



1m thick
$f_{ck} = 30\ N/mm^2$
$f_{yk} = 500\ N/mm^2$
Allowable bearing pressure on soil = 350 kN/m^2

a) Resultant Force must be in the centroid of pad footing, force are calculated in SLS

$$1800x + 2160(3+x) = (1800+2160) \times 2.5$$
$$x = 0.864m$$

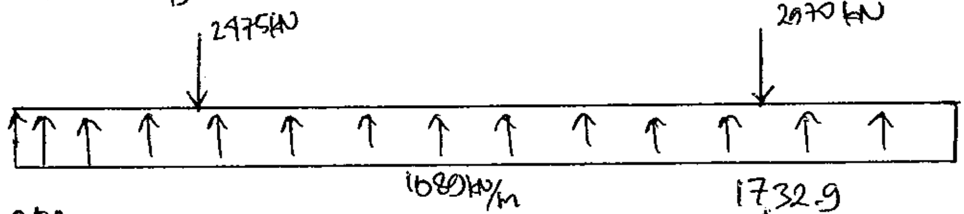
$$\text{Earth pressure} = \frac{1800+2160}{5 \times 2.5} = 316.8\ kPa$$

b) Based on ULS

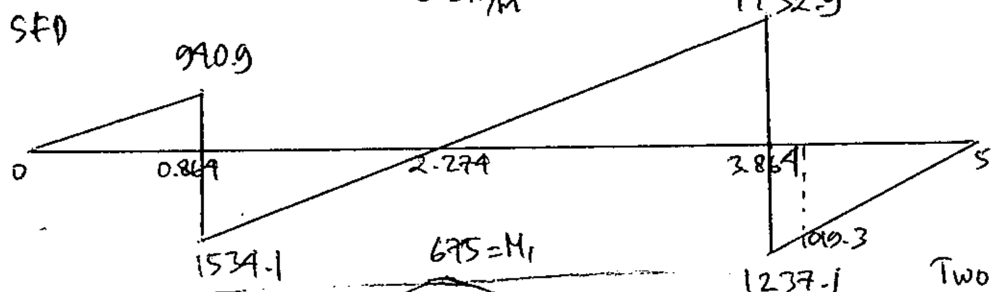
Loading on col 1 = 1.35 (1500) + 1.5 (300) = 2475 kN
Loading on col 2 = 1.35 (1800) + 1.5 (360) = 2970 kN

Uniform Earth Pressure $= \frac{2475+2970}{5} = 1089\ kN/m$

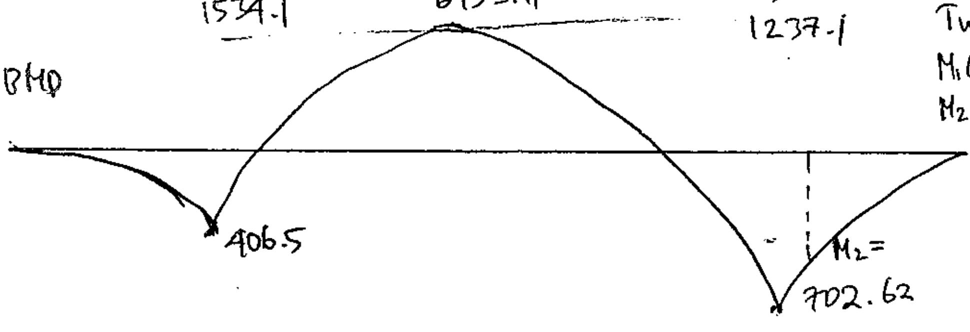
Take 1m strip,



SFD



BMD



Two critical moments,
M_1 (max sagging) = 675 kNm
M_2 (max hogging, at column face) = 476.98 kNm

c) Cover = 60mm , use $\phi 20$ steel bar

→ Design in the longitudinal direction

- Two critical moments, M_1 and M_2, use M_1 (maximum one) = 675 kNm
- $d_x = 1000 - 60 - \frac{20}{2} = 930mm$
- Design as a beam section of 2.5m width

$K = \frac{M_1}{f_{ck} b d^2} = \frac{675 \times 10^6}{30 \times 2500 \times 930^2} = 0.0104$

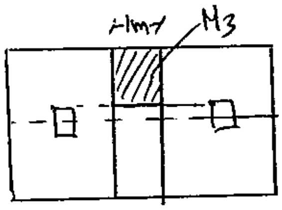
$z = \{0.5 + \sqrt{0.25 - \frac{k}{1.134}}\} d = 0.99d$, use $z = 0.95d = 883.5$ mm

$A_s = \frac{M_1}{0.87 f_{yk} z} = \frac{675 \times 10^6}{0.87 \times 500 \times 883.5} = 1756.34\ mm^2$

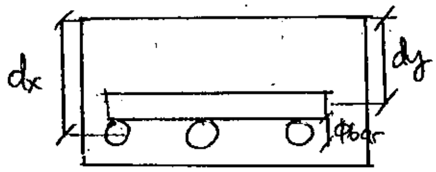
$A_{s,min} = 0.0015\ bd = 3487.5\ mm^2$ → governs the steel required

Provide 12H20 ($A_s = 3769.91 mm^2$)

→ Design in transverse direction



- Take a 1 m strip in transverse direction
- $M_3 = \frac{1}{2}\left(\frac{1089}{2.5}\right) \times \left(\frac{2.5}{2} - \frac{0.4}{2}\right)^2 = 240.1245\ kNm/m$
- $d_y = d_x - \phi_{bar} = 930 - 20 = 910mm$



- $K = \frac{M_3}{f_{ck} b d^2} = 0.0097$
- $z = \{0.5 + \sqrt{0.25 - \frac{k}{1.134}}\} d = 0.99d$,

take $z = 0.95d = 864.5mm$

$A_{sy} = \frac{M_3}{0.87 f_{yk} z} = \frac{240.1245 kN}{0.87 \times 500 \times 864.5} = 638.53\ mm^2$

$A_{sy,min} = 0.0015bd = 0.0015 \times 1000 \times 910 = 1365mm^2$ → governs the steel required

Provide H20 – 225 ($A_s = 1396mm^2/m$)