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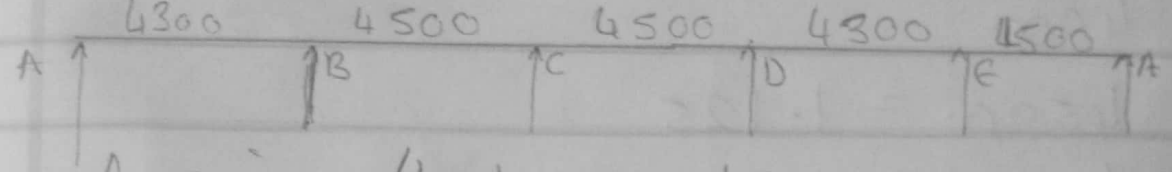
Civil Engineering

18/Eng03/059 /Structural design

(CVE 308)

Assignment 2

2Q1



Assuming thickness; 150 mm

$f_{cu} = 25 N/mm^2$

$f_y = 410 N/mm^2$

Slab loading

Slab weight = $0.15 \times 24 = 3.8 kN/m^2$

Partition = $1.0 kN/m^2$

Finishes = $1.2 kN/m^2$

Total G.k. = $5.8 kN/m^2$

assuming for Chorw

D.L = $1.4(5.8) + 1.6(3.0) = 13 kN/m^2$

Beam loading

Self wt of beam = $0.225 \times 0.6 \times 24 = 5.24 kN/m^2$

finishes = $1.2 kN/m^2$

wall load = $3 \times 3.47 = 10.41 kN/m^2$

Total G.k = 14.85

D.L = 14 (14.85) = 20.79 kN/m²

Slab load on beam in longer direction = ½ w lx (1 - 1/3x)

k = Ly / Lx

4300 / 4000 = 1.075

4500 / 4000 = 1.125

½ x 13 x 4.3 (1 - 1/3 x (1.075)²) = 19.89 kN/m²

½ x 13 x 4.5 (1 - 1/3 x (1.125)²) = 21.55 kN/m²

Slab load on beam in Shorter direction = 1/3 w lx

= 1/3 x 13 x 1.5 =

6.5 kN/m²

Total load = 19.89 + 20.79 = 40.68 kN/m

= 21.55 + 20.79 = 42.34 kN/m

= 6.5 + 20.79 = 27.29 kN/m

Distribution factor

KAB = 1

KBA = 1/LBA / (1/LBA + 1/LBC) = 1/4.3 / (1/4.2 + 1/4.3) = 0.51

$K_{BC} = 1 - 0.51 = 0.49$

$K_{CB} = \frac{1/4 \cdot 5}{1/4 \cdot 5 + 1/4 \cdot 5} = 0.5$

$K_{CB} = 1 - 0.5 = 0.5$

$K_{DC} = 0.49$

$K_{DE} = 0.51$

$K_{ED} = \frac{1/4 \cdot 3}{1/4 \cdot 3 + 1/1 \cdot 5} = 0.26$

$K_{EA} = 1 - 0.26 = 0.79$

$K_{AE} = 1$

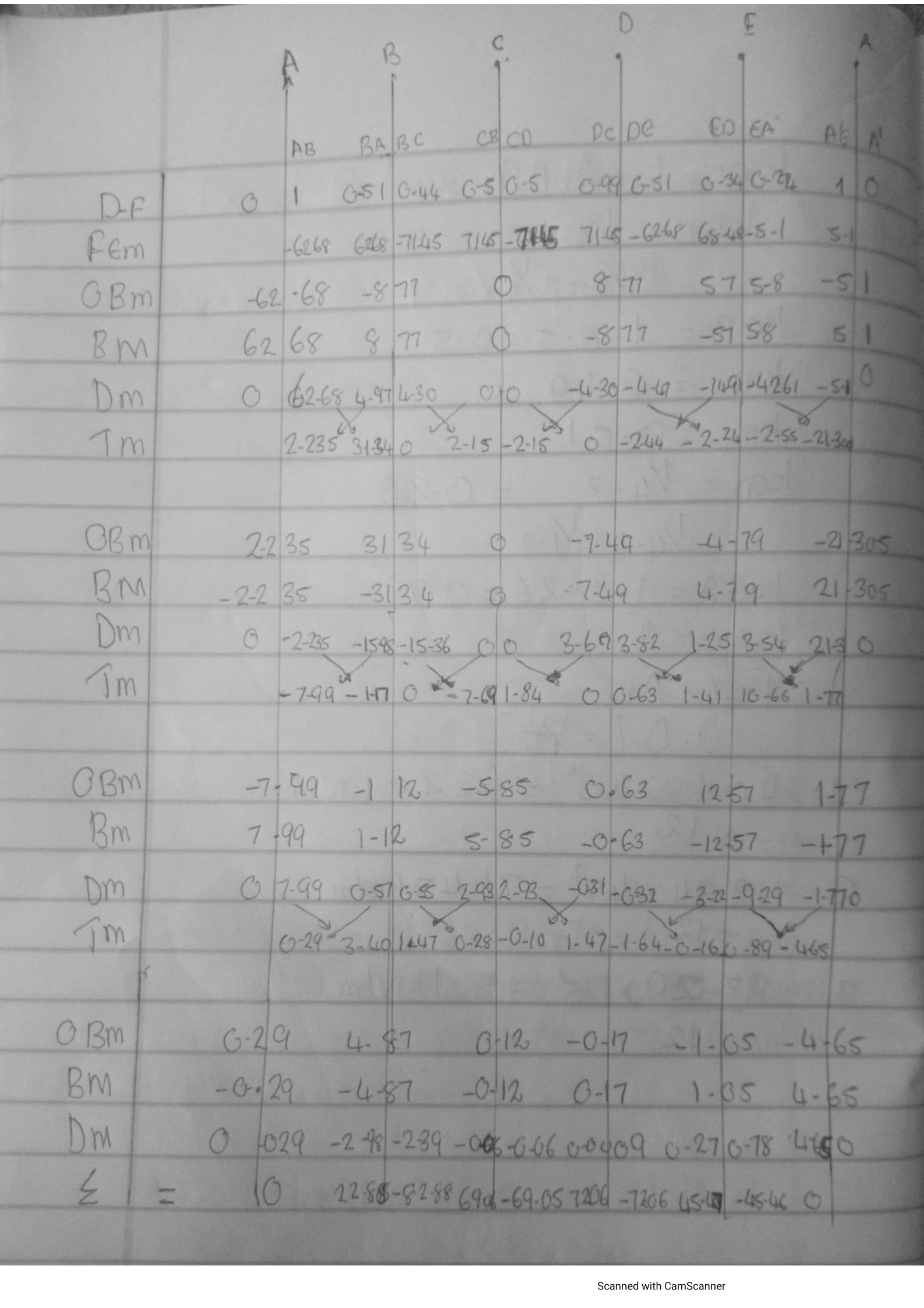
F.E.M

U.D.L $= \frac{wL^2}{12}$

1 $\frac{40.68 \times 4.3^2}{12} = 62.68$ kN/m

2 $\frac{42.34 \times 4.5^2}{12} = 71.45$ kN/m

3 $\frac{27.29 \times 1.5^2}{12} = 5.1$ kN/m



Moments

$M_A = 0 kNm$

$M_B = 82.88 kNm$

$M_C = 69.06 kNm$

$M_D = 72.06 kNm$

$M_G = 45.47 kNm$

$M_{A_1} = 0 kNm$

Free moment

For U.D.L $= \frac{WL^2}{8}$

1. $\frac{40.68 \times 4.3^2}{8} = 94.02 kN/m^2$

2. $\frac{42.38 \times 4.5^2}{8} = 107.17 kN/m^2$

3. $\frac{27.29 \times 1.5^2}{8} = 8.24 kN/m^2$

Span moment

$M_{AB} = M^F = \left(\frac{M_A + M_B}{2}\right) = 94.02 - \left(\frac{0 - 82.88}{2}\right) = 52.58 kNm$

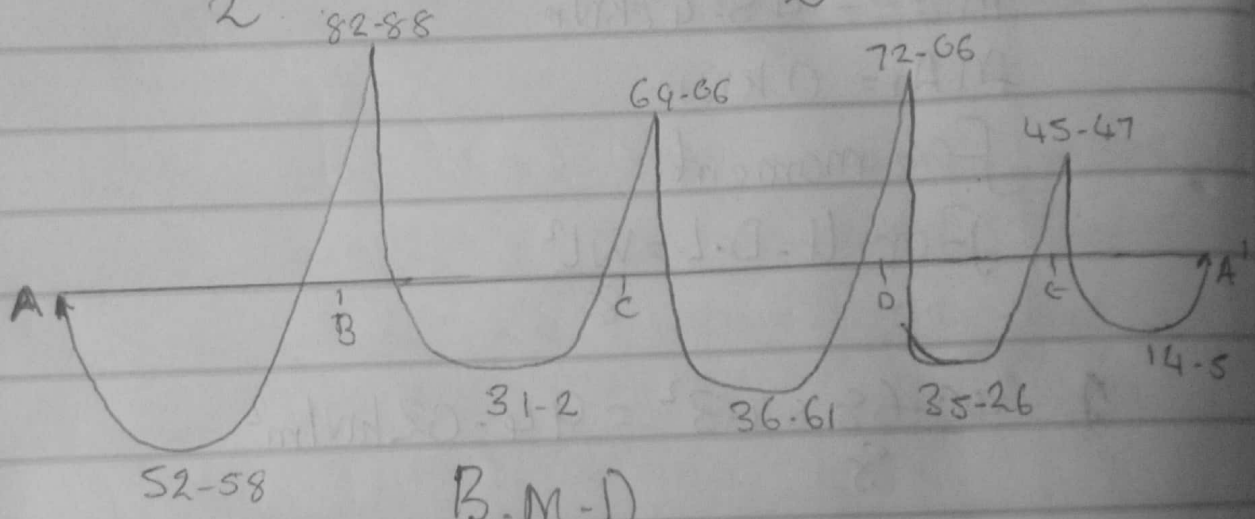
$M_{BC} = M^F - \left(\frac{M_B + M_C}{2}\right) = 107.17 - \left(\frac{82.88 + 69.06}{2}\right)$

$= 31.2 kNm$

$$M_{CD} = M^F = \left(\frac{m_C + M_D}{2}\right) = 107.17 - \left(\frac{69.06 + 72.06}{2}\right) = 36.6\,kN_m$$

$$M_{DE} = M^F = \left(\frac{m_D + m_E}{2}\right) = 94.02 - \left(\frac{72.06 + 45.47}{2}\right) = 35.26\,kN_m$$

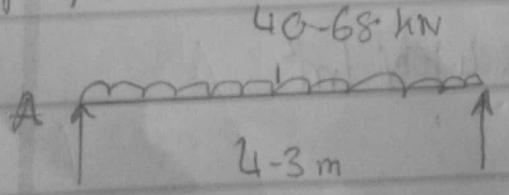
$$M_{EA'} = M^F - \left(\frac{m_E + M_{A'}}{2}\right) = 8.24 - \left(\frac{45.47 + 0}{2}\right) = -14.5\,kN_m$$



B.M.D

Shear Force

For A



$$V_A = W/2 = V_B$$

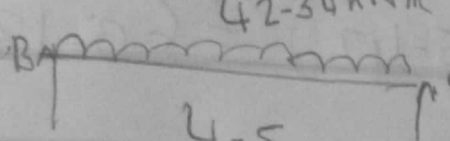
$$= \frac{40.68 \times 4.3}{2} = 87.462\,kN$$

$$V_{AB} = V_a + \left(\frac{m_A - m_B}{2}\right) = 87.462 + \left(\frac{0 - 82.88}{4.3}\right) = 68.19$$

$$V_{BA} = Wl - V_{AB}$$

$$= (40.68 \times 4.3) - 68.19 = 106.78\,kN$$

For B



$V_B' = \frac{wL}{2} = V_C$

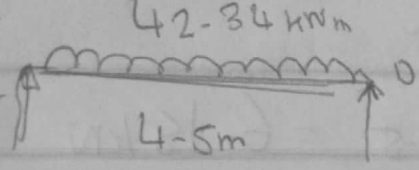
$= \frac{42.34 \times 4.5}{2} = 95.27 kN$

$V_{BC} = V_B + \left(\frac{m_a + m_c}{L}\right) = 95.27 + \left(\frac{82.68 + 69.06}{4.5}\right)$

$V_{CB} = (42.34 \times 4.5) = 129.03$ ≈ 129.03kN

$= 61.5 kN$

For C



$V_C = \frac{wL}{2} = V_D$

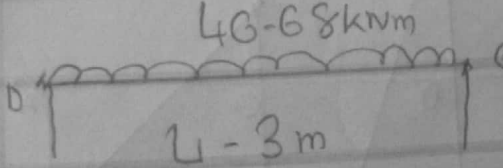
$\frac{42.34 \times 4.5}{2} = 95.27 kN$

$V_{CD} = 95.27 + \left(\frac{69.06 + 72.06}{4.5}\right) = 126.63 kN$

$V_{CD} = (42.34 \times 4.5) - 126.63$

$= 63.19 kN$

For D



$V_D' \frac{wL}{2} = V_C$

$= \frac{40.68 \times 4.3}{2} = 87.46 kN$

$V_{DC} = V_D - \left(\frac{M_D + M_C}{L}\right)$

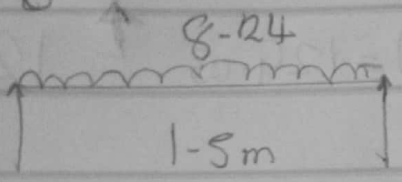
$= 87.46 - \left(\frac{72.66 + 45.47}{4.3}\right) = 60.13 kN$

$V_{ED} = WL - V_{DE}$

$= (40.68 \times 4.3) - 60.13$

$= 114.79 kN$

For E



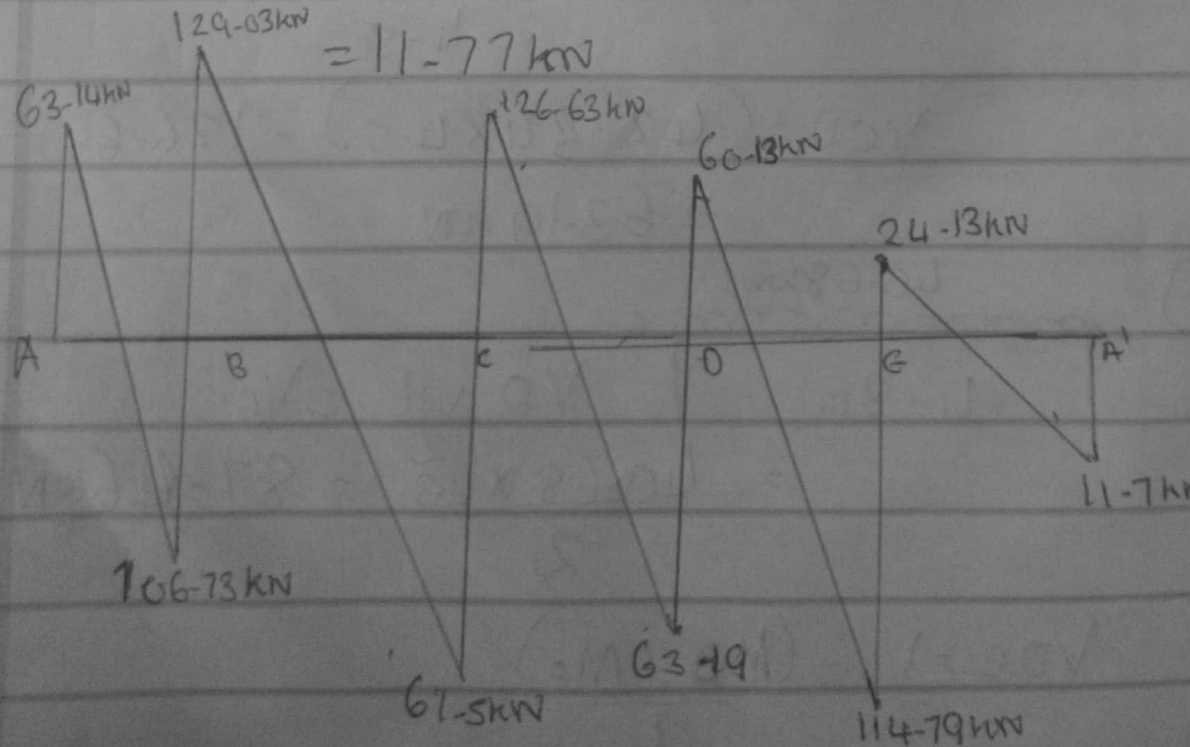
$V_E' = \frac{WL}{2} = V_A'$

$= \frac{8.24 \times 1.5}{2} = 6.18 kN$

$V_{GA} = V_E - \left(\frac{m_G + m_A'}{L}\right) = 6.18 - \left(\frac{45.47 + 0}{1.5}\right) = 24.13 kN$

$V_{AG} = (8.24 \times 1.5) - 24.13$

$= 11.77 kN$



Number 2.

Base design

N = 1200kN

Strength = 25-410N/mm²

Fb = 150kN/m²

Area of base req = $\frac{N \times 1.1}{R \times Fb}$ R = 1.46

$\frac{1200 \times 1.1}{1.46 \times 150} = 6.027 m^2$

$\sqrt{6.027} = 2.45 m \simeq 2.5$

Net pressure, $f_{net} = \frac{N \times 1.1}{B}$

$\frac{1200 \times 1.1}{2.5} - \frac{24 \times 0.666 \times 1.4}{2.5} = 505.824 kNm$

moment, $m = \frac{f_{net} l^2}{2}$ Where $l = \frac{1}{2}(B-h)$ 8 = depth of base = 660

$l = \frac{1}{2}(2.5 - 0.225) = 1.1375 \simeq 1.14 m$

$m = \frac{505.824 \times 1.14^2}{2}$ 328.68kNm

$d = h - Cover - \frac{1}{2}\phi$

$= 660 - 50 - 10 = 600mm$

$$K = \frac{m}{bd^2 f_{cu}} = \frac{328.68 \times 10^6}{1000 \times 600^2 \times 25} = 0.032$$

$$R_u = 0.5 + \sqrt{0.25 + \frac{0.032}{0.9}} = 0.96 > 0.95$$

$$Z = Rad = 0.95 \times 600 = 570\,mm$$

$$A_s = \frac{m}{0.95 f_y z} = \frac{328.68 \times 10^6}{0.95 \times 410 \times 570} = 1480.44\,mm^2$$

Provide Y25@ 300 c/c (1640)

Punching Shear

Column Size = 225 X 450 mm

f_{cu} f_y = 25. 410 mm

Area Footing = 6.027 m^2

Size of Footing = 2500 X 2500

q_s, Net pressure = 508.824 kN/m

Depth = 600

Critical Section d/2 = 300

300 + 300 + 225 = 825 mm

300 + 300 + 450 = 1050 mm

Shear Force $V_n = q_u \times [\text{Area of Footing} - (0.3 + d)^2]$

$= 505.824 [2.5 \times 2.5 - (0.3 + 0.6)^2]$

$V_n = 2751.68 kN$

Nomal Shear Stress $\tau_v = V_n / bd$

b = Peramter of Critical section

d = Effective Span/depth

$\tau_v = \frac{2751.68 \times 10^3}{((2 \times (825) + 2(1050)) \times 600)}$

$\tau_v = 1.223 N/mm^2$

Permissible Shear Stress

$\tau_c' = k_s \times \tau_c$

$k_s = (0.5 + \beta_c)$ but not Straater then 1

β_c = Ratio of Straater to lagger Side of Colums

$\tau_c = 0.25\sqrt{f_{ck}}$

$k_s = 1$

$\tau_c' = 0.25\sqrt{25} = 1.25 N/mm^2$

$\tau_v = 1.223 N/mm^2$

$\tau_v \leq \tau_v'$

Hence depth assumed is Ok

Cheching For Fb with actual size of Footing

Unit weight of Concrete = $24 kN/mm^2$

Unit weight of Soil = 1.091×10^{-6} kN/mm

Actual pressure footing below

$q = (1200 \times 2.5 \times 2.5) + (24 \times 0.666) + (1.091 \times 10^{-6} \times 0.66)$

$q = 214.94 \text{ kN/m}^2$