SEAA 4333- Reinforced Concrete Design

Bungalow Design Project

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Introduction

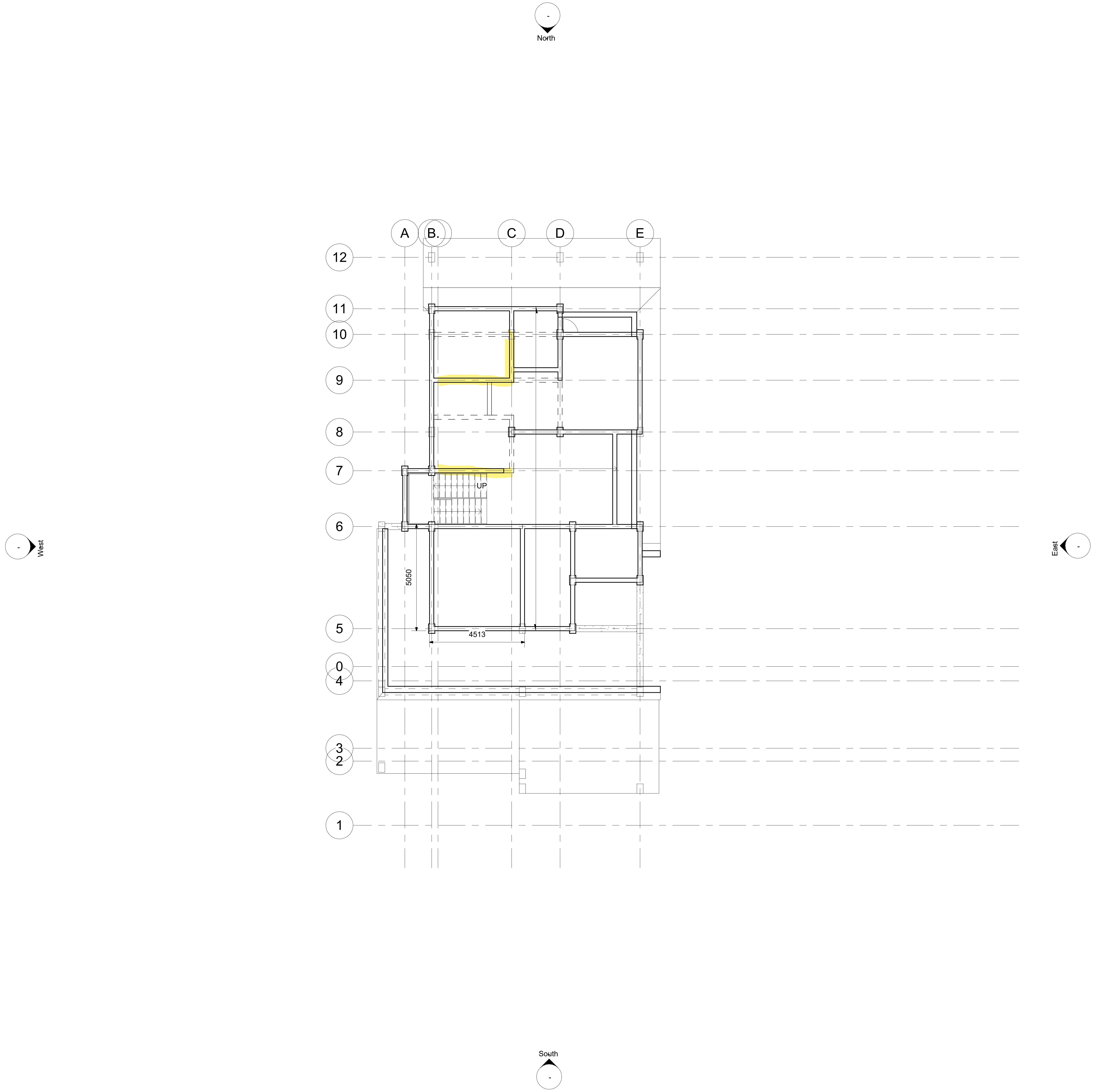
A two-story bungalow is proposed to be built. While the architectural drawing is provided, certain assumptions are made. This is to aid the calculation process as each individual designs a small part of the bungalow. The assumptions are as below:

C_{nom} is taken as 20mm for all components except foundation where 40mm is taken.

The loading on the beams supporting the roof is taken as 10kN/m

The rebar is not standardised, therefore must be referred to from each design.

There is shear walls present in the bungalow. The location is highlighted in the architectural drawing.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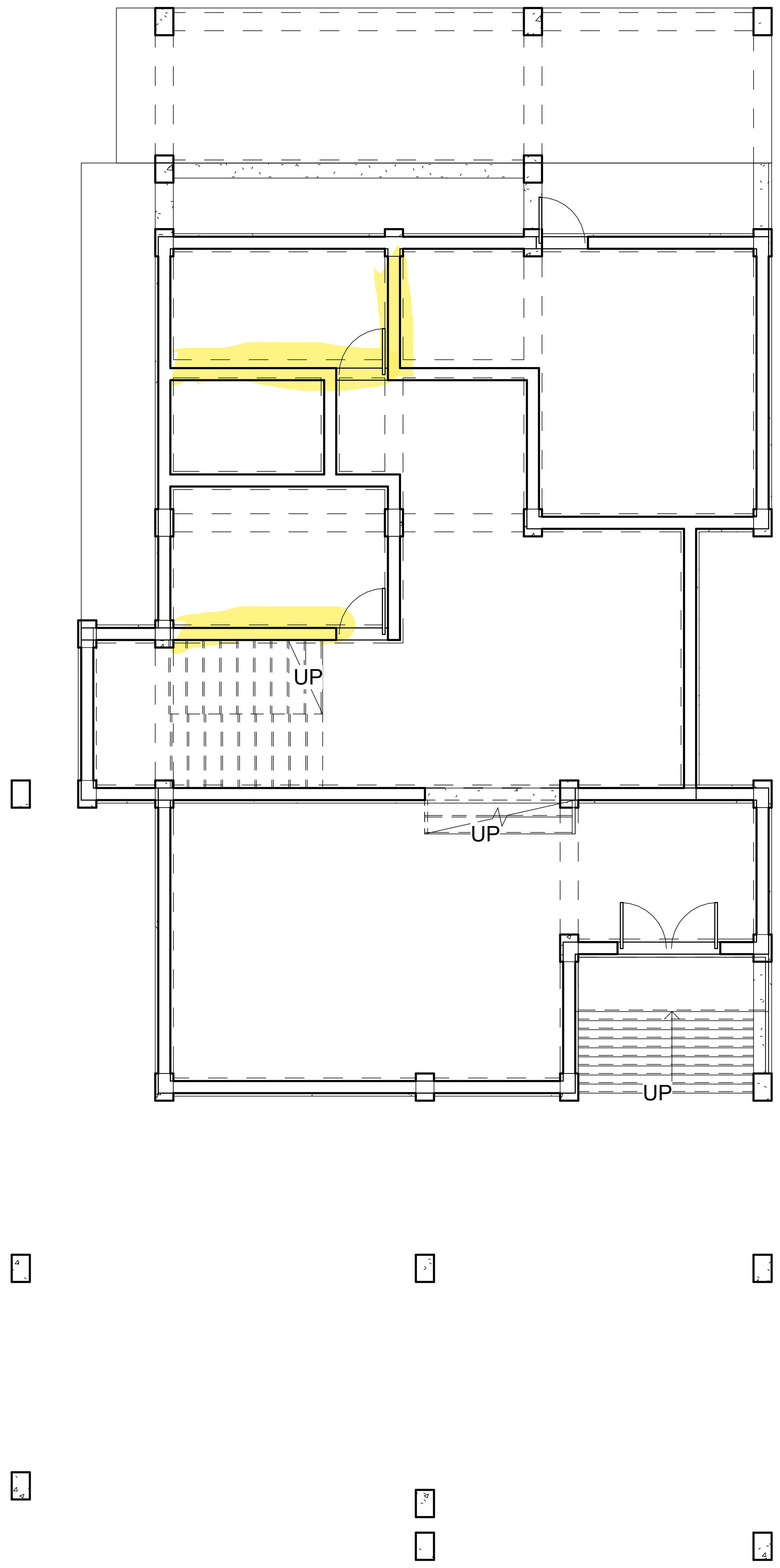
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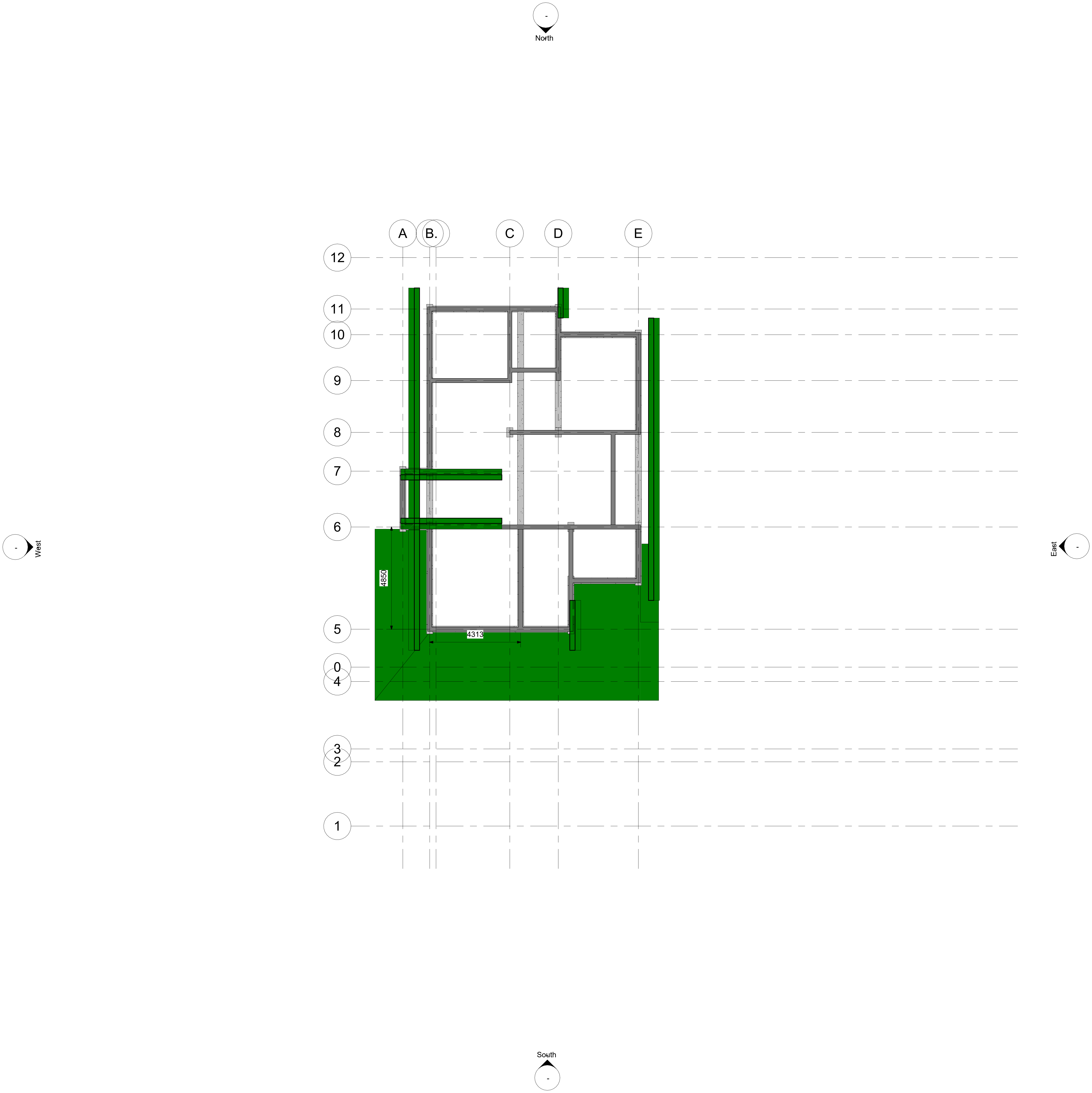

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MUHAMMAD AMIR ZAHIRUDDIN BIN ZAINAL ABIDIN

MUHAMMAD AMIR ZAHIRUDDIN BIN ZAINAL ABIDIN
A19EA0079
SEBA4333-02 (PROJECT RCD2)

STAIRCASE

CHOOSING THE MAIN STAIRCASE A6-C7

$h = 175$ $\quad f_{cu} = 30$
$G = 210$ $\quad f_{yk} = 500$
$R = 125$ $\quad \gamma_1 = 25$
$g_k = 1.5$
$q_k = 2.5$ $\quad \phi = 10$

Average thickness

$$y = h\left(\frac{\sqrt{G^2+R^2}}{G}\right) = 175\left(\frac{\sqrt{210^2+125^2}}{210}\right)$$

$$= 203.66\text{mm}$$

$$t = \frac{2(203.66)+125}{2} = 266.16\text{mm}$$

Action (Flight)

Slab selfweight $= 0.266 \times 25 = 6.65$ kN/m²
$g_k = 1.50$ kN/m²
$G_k = 8.15$ kN/m²

$q_k = 2.50$ kN/m²
$Q_k = 2.50$ kN/m²

$w = 1.35(8.15) + 1.5(2.50)$
$= 14.76$ kNm⁻²
$w = 14.76$ kNm⁻² $\times 1$m
$= 14.76$ kNm⁻¹

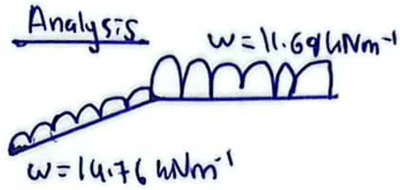
Landing

Slab selfweight $= 25 \times 0.175 = 4.38$ kN/m²
$g_k = 1.50$ kN/m²
$G_k = 5.88$ kN/m²
$q_k = 2.50$ kN/m²
$Q_k = 2.50$ kN/m²

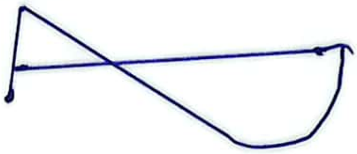
$w_l = (1.35(5.88) + 1.5(2.50)) \times 1$

$= 11.69$ kNm⁻¹

Analysis



FL/10



$L = 2.5 + 1.275 = 3.775$ m

$F = (14.76 \times 2.5) + (11.69 \times 1.275)$
$= 51.80$

$FL/10 = (51.80 \times 3.775)/10 = 19.55$ kNm

Main Reinforcement

$d = 175 - 20 - 0.5(10) = 150$

$$k = \frac{19.55 \times 10^6}{30 \times 1000 \times 150^2} = 0.029 < 0.167$$

No compression reinforced needed

$$z = d\left[0.5 + \sqrt{0.25 - \frac{0.029}{1.134}}\right] = 0.97d$$

$$A_s = \frac{19.55 \times 10^6}{0.87 \times 500 \times 0.95 \times 150} = 315.39\text{mm}^2$$

$A_{smin} = 0.0013(1000)(150) = 195.0$mm²

$A_{smax} = 0.04(1000)(175) = 7000$ mm²

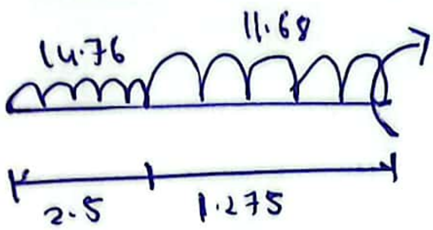
Provide H10-100 (786mm²)

Secondary reinforcement

$0.2A_s = 0.2(315.39) = 63.07$ mm²

provide H10-400 (196.5mm²)

Shear



$\circlearrowright \Sigma M_A = 0$

$$0 = 14.76\frac{(2.5^2)}{2} + 11.68\left(2.5 + \frac{1.275}{2}\right) + 19.55 - M_B(2.5 + 1.275)$$

$M_B = 29.77$ kN $\leftarrow V_{Ed}$

$\circlearrowright \Sigma M_B = 0$

$M_A = (14.76 \times 2.5) + (11.68 \times 1.275) - 29.77$

$= 22.02$ kN

$V_{Rdc} = 1000 \times 150 \times (0.12 \times 2 \times (100 \times 0.00524 \times 30)^{\frac{1}{3}} / 1000$

$= 90.18$ kN

$\rho_1 = \frac{786}{1000 \times 150} = 0.00524$ | $k = 1 + \sqrt{\frac{200}{150}} = 2.1572$

$V_{min} = [0.035 \times \sqrt{(2)^{\frac{3}{2}}} \times \sqrt{30}] \times 1000 \times 150$

$= 81.33$ kN

So $V_{Rdc} > V_{Ed}$

Deflection

$\rho = \frac{315.79}{1000 \times 150} = 0.0021$

$\rho_0 = \frac{\sqrt{30}}{1000} = 0.00548 > \rho$

$$\frac{l}{d} = 1.3\left[11 + 1.5\sqrt{30} \times \frac{548}{210} + 3.2\sqrt{30}\left(\frac{524}{210} - 1\right)^{\frac{3}{2}}\right]$$

$= 88.45$

$\frac{l}{d}$ allow $= 88.45 \times 1 \times \left(\frac{786}{315.79}\right)$

$= 88.45 \times 1 \times 1.5$

$= 132.68$

(l/d) actual $= \frac{3775}{150} = 25.17$ (ok!)

Cracking : h = 175 < 200

main bar : 3h = 3(175) = 525 > 400

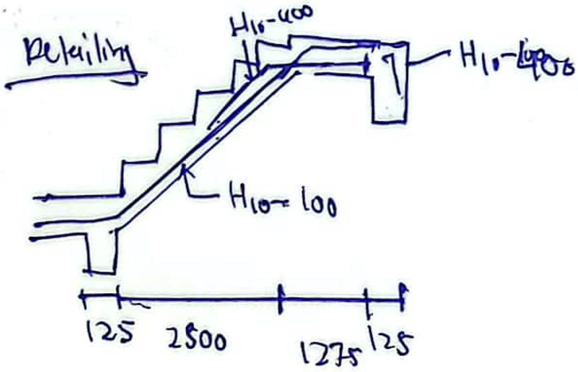
max bar spacing = 100 < 400 (ok!)

2nd bar : 3.5h = 612.5 > 450

max spacing = 450

max bar spacing = 400 < 450 (ok!)

Detailing



MIAMIR ZAHIRUDDIN BIN ZAINAL ABIDIN

A19EA0079 Column

Take Column A/7 Level 1

Size column $b \times h = 400 \times 400mm$

$$I = \frac{400 \times 400^3}{12} = 2.13 \times 10^9$$

$L = 2500$

Size beam $b \times h : 250 \times 500$

$$I = \frac{250 \times 500^3}{12} = 2.6 \times 10^9$$

$L_1 = 2650mm$

$L_2 = 1275mm$

Stiffness

$$\text{Column} = k_{zz} = \frac{2.13 \times 10^9}{2500} = 8.52 \times 10^5$$

$$\text{Beam} = k_{mb_1} = \frac{2.6 \times 10^9}{2650} = 9.83 \times 10^5$$

$$k_{mb_2} = \frac{2.6 \times 10^9}{1275} = 20.39 \times 10^5$$

Radius of Gyration

$$I_{zz} = \sqrt{\frac{2.13 \times 10^9}{400 \times 400}} = 115.38$$

Effective length

$$l_{oy} = l_{oz} = 0.75(2500) = 1875$$

$$\lambda_{zz} = \lambda_{yy} = \frac{1875}{115.38} = 16.25$$

Slenderness Limit

$$1.7 - \left(\frac{-0.5}{0.7}\right) \quad 1.7 - \left(\frac{-0.1}{0.13}\right)$$

$A = 0.7, B = 1.1 \quad C = 2.41$ and 2.47

$$n = \frac{31.89 \times 10^3 \times 1.5}{(400 \times 400) \times 30 \times 0.85} = 0.012$$

$$z\text{-axis } \lambda_{lim} = \frac{20 \times 0.7 \times 1.1 \times 2.41}{\sqrt{0.012}} = 339$$

$$\lambda_y = \frac{20 \times 0.7 \times 1.1 \times 2.47}{\sqrt{0.012}} = 347$$

(Both Slender)

Axial calculation

FSM

2nd Floor

$$\text{Selfweight} = 25 \times 0.25 \times 0.5 = 3.13 \text{ kN/m}$$

$$FEM_a = \frac{3.13 \times 1.275^2}{12} = 0.42kNm$$

$$FEM_b = \frac{3.13 \times 2.65^2}{12} = 1.83kNm$$

For main beam

$$M_{top} = \frac{1.83 \times 8.53}{8.53 + 8.53 + \frac{9.83}{2}} = 0.71kNm$$

$$M_{bottom} = \frac{1.83 \times 8.53}{8.53 + 8.53 + \frac{9.83}{2}} = 0.71kNm$$

$$N_{Ed} = \frac{3.125 \times 2.65}{2} = 4.14kN$$

For Secondary beam

$$M_{top} = \frac{0.42 \times 8.53}{8.53 + 8.53 + \frac{20.39}{2}} = 0.13kNm$$

$$M_{bottom} = \frac{0.42 \times 8.53}{8.53 + 8.53 + \frac{20.39}{2}} = 0.13kNm$$

$$N_{Ed} = \frac{3.125 \times 1.275}{2} = 1.99kN$$

Roof

Assume roof load = 10kN/m

$$FEM_a = \frac{10 \times 1.275^2}{12} = 1.35kNm$$

$$FEM_b = \frac{10 \times 2.65^2}{12} = 5.85kNm$$

For Main Beam

$$M = \frac{5.85 \times 8.53}{8.53 + \frac{9.83}{2}} = 3.71 kNm$$

$$N_{ED} = \frac{10 \times 2.65}{2} = 13.25 kN$$

Secondary Beam

$$M = \frac{1.35 \times 8.53}{8.53 + \frac{20.4}{2}} = 0.62 kNm$$

$$N_{Ed} = \frac{10 \times 1.275^2}{2} = 6.375 kN$$

Ground Floor

$w = 25 \times 0.25 \times 0.5 = 3.125 \, kNm^{-1}$

$$FEM_{sb} = \frac{3.125 \times 1.275^2}{12} = 0.42 kNm^{-1}$$

$$FEM_{mb} = \frac{3.125 \times 2.65^2}{12} = 1.83 \, kNm^{-1}$$

As w and FEM are same as 2nd Floor, the N_{Ed} is reused.

Main Bar

$$M_{top} = \frac{1.83 \times 8.53}{8.53 + 17.41 + \frac{9.83}{2}} = 0.51 kNm$$

$$M_{bot} = \frac{1.83 \times 17.41}{8.53 + 17.41 + \frac{9.83}{2}} = 1.03 kNm$$

$N_{Ed} = 4.14 kN$

Secondary beams

$$M_{top} = \frac{0.42 \times 8.53}{8.53 + 17.41 + \frac{20.4}{2}} = 0.1 kNm$$

$$M_{bot} = \frac{0.42 \times 17.41}{8.53 + 17.41 + \frac{20.4}{2}} = 0.2 kNm$$

$N_{Ed} = 1.99 kN$

$\Sigma N_{Ed} = 31.89 kN$

Design moment

$$M_{imp} = 31.89 \times \frac{2.5}{400} = 0.2 kNm$$

$M_{Ey} = 0.51 + 0.2 = 0.71$

$M_{Ez} = 0.71 + 0.2 = 0.91$

Biaxial Check

$$e_z = \frac{0.91 \times 10^6}{31.89 \times 10^3} = 29 mm$$

$$e_y = \frac{0.71 \times 10^6}{31.89 \times 10^3} = 22 mm$$

$$\left(\frac{e_y}{h}\right) / \left(\frac{e_z}{b}\right) = \left(\frac{22}{400}\right) / \left(\frac{29}{400}\right) = 0.76 > 0.2$$

$$\left(\frac{e_z}{b}\right) / \left(\frac{e_y}{h}\right) = \left(\frac{29}{400}\right) / \left(\frac{22}{400}\right) = 1.32 > 0.2$$

$$\frac{\lambda_y}{\lambda_z} \geq 1 < 2 \quad \text{(ignore biaxial bending)}$$

$\therefore$ Check biaxial bending

Reinforcement Design

$C_{nom} = 20$, $\phi_{bar} = 16$, $\phi_{link} = 8$

$$h' = 400 - 20 - 8 - \frac{16}{2} = 364$$

$b' = h' = 364$

$$\frac{M_{Edz}}{h'} = \frac{0.91 \times 10^6}{364} = 2.5 kN$$

$$\frac{M_{Edy}}{h'} = \frac{0.71 \times 10^6}{364} = 1.94 kN$$

$$\frac{M_{Edz}}{h'} > \frac{M_{Edy}}{h'}$$

$$\frac{N_{Ed}}{bhf_{ck}} = \frac{31.89 \times 10^3}{400 \times 400 \times 30} = 6.64 \times 10^{-3}$$

$\beta = 0.99$

$M'_{Ed z} = 0.91 + 0.99\left(\frac{364}{364}\right)0.71$

$= 1.014 Nm$

$d_2 = 20 + 8 + \frac{16}{2} = 36$

$d_2/h = 36/400 = 0.09$

$\frac{M}{bh^2 f_{ck}} = \frac{1.61 \times 10^6}{400 \times 400^2 \times 30} = 8.39 \times 10^{-4}$

$\frac{A_s f_{yk}}{bh f_{ck}} = 0$, $A_s = 0$

$A_{s\,min} = \frac{0.1 \times 31.89 \times 10^3}{0.87 \times 500} = 7.33 mm^2$

$= 0.0013 \times 400^2 = 208 mm^2$

$A_{s\,max} = 0.04 \times 400^2 = 6400 mm^2$

Provide 4H16 $A_s = 804 mm^2$

Link	Smax
- 0.25 × 16 = 4	20 × 16 = 320
- 6 → use 8	use 366

provide H8-300 and H8-175

as Smax = 0.6 × 300 = 180 mm

Check biaxial bending

Steel area

All : 4H16 + 0H20 = 804

z-z : 4H16 + 0H20 = 804

y-y : 4H16 + 0H20 = 804

$\frac{A_s f_{yk}}{bh f_{ck}} = \frac{804 \times 500}{400 \times 400 \times 30} = 0.08$

$\frac{M_{Rdz}}{bh^2 f_{ck}} = 0.03$

$M_{Rdz} = 57.64 Nm$

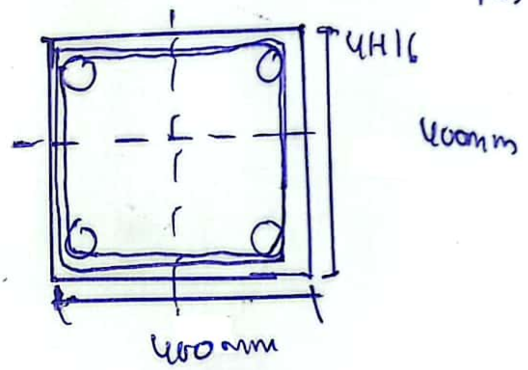
$N_{Rd} = (0.567 \times 25 \times 400^2) + (0.87 \times 500 \times 804)$

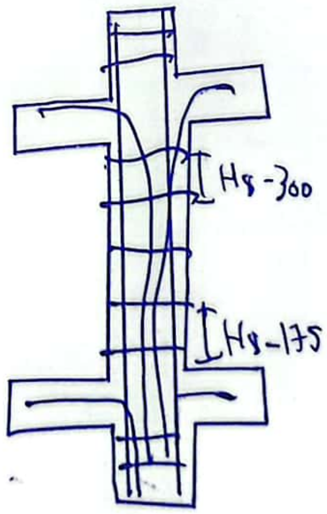
$= 2618 kN$

$\frac{N_{Ed}}{N_{Rd}} = \frac{31.89}{2618} = 0.012$, $a = 1.0$

$\left(\frac{0.91}{57.6}\right)^1 + \left(\frac{0.51}{57.6}\right)^1 = 0.024 < 1$

Pass





MUHAMMAD AMIER ZAHIRUDDIN BIN ZAINAL ABIDIN

A19EA0079 (FOUNDATION → A/7)

Dimension

B = 1.3 m

H = 1.3 m

h = 0.25 m

Service load = 70 kN

Cnom = 20 mm

Bar size = 8 mm

Allowable bearing capacity = 150 kN/mm² (loose gravel)

$SLS = \frac{31.89}{1.4} = 22.78\ kN$

Selfweight = 1.3 × 1.3 × 0.25 × 25

= 10.56 kN

Area required = $\frac{22.79 \times 1.1}{150} = 0.17\ m^2$

Area provided = 1.3 × 1.3 = 1.7 m² Ok!

Assume Footing selfweight = $\frac{(22.78 + 10.15)}{1.7}$

= 19.37 kN/m²

Analysis

Pressure at ULS = $\frac{31.89}{1.7}$ = 18.76 kN/m²

Side length = 450 mm

$M = \frac{18.76 \times \left(\frac{450}{1000}\right)^2}{2}$

= 1.9 kNm

d = 250 − 20 − 1.5(8) = 218 mm

$k = \frac{1.9 \times 10^6}{1300 \times 218^2 \times 30} = 0.001 < 0.167$

(No compression need)

$z = d\left[0.5 + \sqrt{0.25 - \frac{0.001}{1.134}}\right] = 0.99d$

= 0.95d

$As = \frac{1.9 \times 10^6}{0.87 \times 500 \times 0.95 \times 218}$

= 21.09 mm²

Asmin = 0.0013 × 1300 × 218 = 368.42 mm²

Asmax = 0.04 × 1300 × 250 = 13000 mm²

Use 8H8 (402 mm²)

Shear

Vertical shear

$V_{Ed} = 18.76 \times (0.25 - 0.218)$

= 0.6 kN

$k = 1 + \sqrt{\frac{200}{218}}$

= 1.95

~~Bar extend beyond critical section at~~

~~= 550 − 218 − 20~~

~~= 312~~

Asl = 402

$\rho_1 = \frac{402}{1300 \times 218} = 0.0014$

$V_{Rdc} = \left[0.12 \times 1.95 \sqrt[3]{(100 \times 0.0014 \times 30)}\right] \times 1300 \times 218$

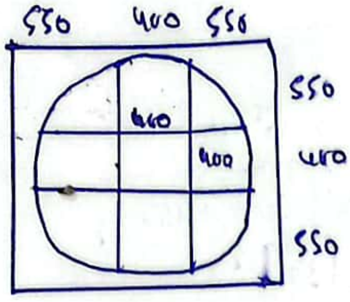
= 107 kN

$V_{min} = \left[0.035 \times 1.95^{\frac{3}{2}} \times \sqrt{30}\right] \times 1300 \times 218$

= 148 kN

$V_{min} = V_{Rdc} = 148\ kN > V_{Ed} = 0.6\ kN$

Ok!



$d = 250 - 30 - 8$
$= 212\text{mm}$
$2d = 424 \text{ mm}$

Perimeter $= 2 \times \pi \times 424 + 4 \times 400$
$= 4264\text{mm}$

area $= \pi \times 424^2 + 400^2 + 4(400)(424)$
$= 1403183 \text{ mm}^2$

$40\phi + d = 40(8) + 212 = 532$

Available length $= 450 - 219 = 232$

(Reinforce bar does not help with punching)

$V_{min} = (0.035 \times 1.95^{3/2} \times \sqrt{30})(212 \times 4264)$

$= 471.88 \text{ kN}$

punching $= \cancel{0.5(4264)(212)[0.6(1-30/250)] \times \frac{30}{1.5}}$

$= 18176(1.7-1.4)$

$= 5.63\text{kN} < V_{min}$ OK!

Shear at column perimeter

$= 0.5 \times 212 \times 4264 \times [0.6(1-30/250)] \times \frac{30}{1.5}$

$= 4773 \text{ kN}$

Axial force $= 31.99$ (ok!)

Cracking

$h = 250 \text{ mm} > 200\text{mm}$

$f_s = 0.6 \times \frac{402}{368.42} \times \frac{500}{1.15} = 284.65 \text{ kN}$

Taking for 320 N/mm^2

Using design crack 0.3mm

max spacing = 100mm

max bar size = 10mm

Current spacing $= \frac{(1300 - 2 \times 20 - 8)}{8}$

$= 157\text{mm} < 200 \text{ mm}$ ok!

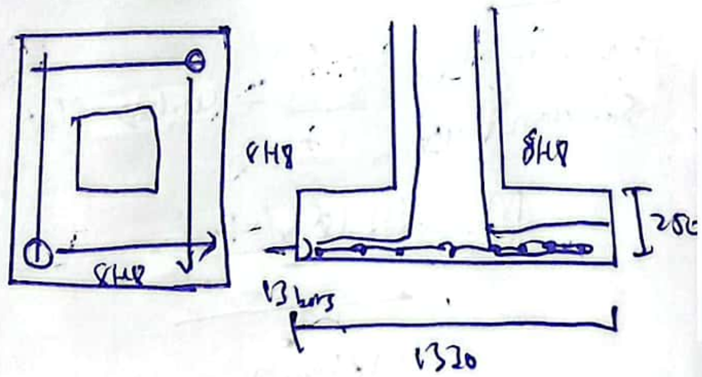
Number of reinforcement bar require $= \frac{(1.3 \times 1600 - 2 \times 20 - 8)}{160}$

$= 12.52 \text{ mm}$

Taking 13 bars

13 H8-100

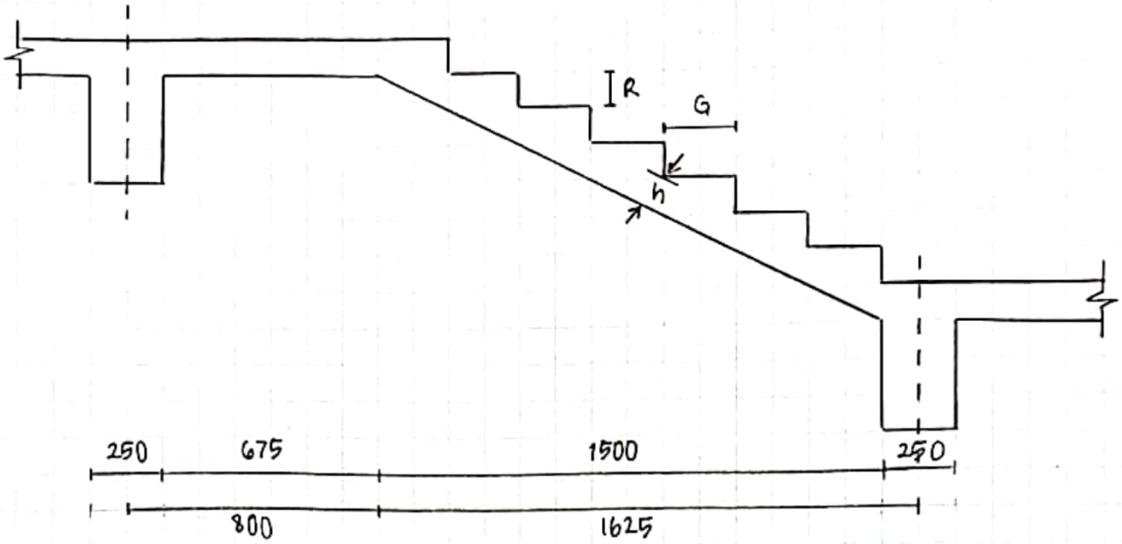
Detailing



NUR AVIKA AUDRIANA SAWALI

NAME : NUR ANIKA ANDRIANA SAWALI
MATRIC : A19EA3009

staircase



specification

dimension :

- $R = 167$ mm
- $G = 250$ mm
- $\frac{L}{d} \simeq 28$
- $\frac{2425}{28} = d \rightarrow d = 86.6 \text{ mm} \approx 87 \text{ mm}$
- $h = d + c + 0.5\varnothing bar$
  $= 87 + 2 + 0.5(10)$
  $= 112$ mm

characteristic actions :

- permanent, $g_k = 1.50 \text{ kN/m}^2$
- variable, $q_k = 2.5 \text{ kN/m}^2$

Materials :

- characteristic strength of concrete, $f_{ck} = 30 \text{ N/mm}^2$
- characteristic strength of steel, $f_{yk} = 500 \text{ N/mm}^2$
- Unit weight of reinforced concrete $= 25 \text{ kN/m}^3$

Assumed :
- concrete cover, $c = 20$ mm
- $\varnothing$ bar $= 10$ mm

Average thickness

- $y = h\left[\dfrac{\sqrt{G^2+R^2}}{G}\right]$

  $= 112\left[\dfrac{\sqrt{250^2+167^2}}{250}\right] \longrightarrow 135\ mm$

- $t = y + \dfrac{R}{2}$

  $= 135 + \dfrac{167}{2} \longrightarrow 218.5 \approx 219\ mm$

Actions

flight :

- slab selfweight $= 0.219 \times 25 = 5.48\ kN/m^2$
- permanent load (ex. selfweight) $= 1.50\ kN/m^2$ +
- char. permanent load, g_k $= 6.98\ kN/m^2$
- char. variable load, q_k $= 2.50\ kN/m^2$

- design action, $n_d = 1.35 g_k + 1.5 q_k$

  $= 1.35(6.98) + 1.5(2.5)$

  $= 13.17\ kN/m^2$ (consider 1 m width, $W_d = 13.17\ kN/m^2 \times 1\ m = 13.17\ kN/m$).

Landing :

- slab selfweight $= 0.112 \times 25 = 2.80\ kN/m^2$
- permanent load (ex. selfweight) $= 1.50\ kN/m^2$ +
- char. permanent load, g_k $= 4.30\ kN/m^2$
- char. variable load, q_k $= 2.50\ kN/m^2$

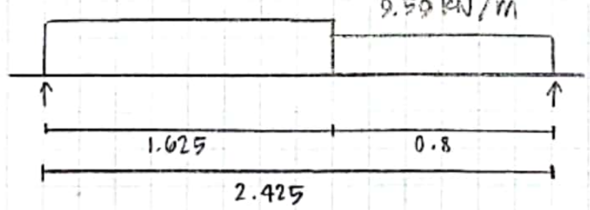
- design action, $n_d = 1.35 g_k + 1.5 q_k$

  $= 1.35(4.30) + 1.5(2.50)$

  $= 9.56\ kN/m^2$ (consider 1m width, $W_d = 9.56\ kN/m^2 \times 1\ m = 9.56\ kN/m$).

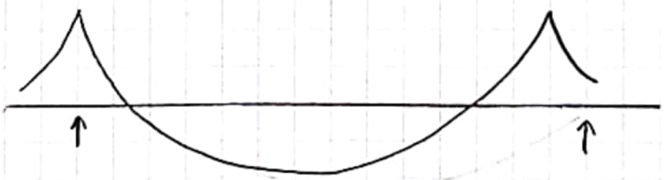
Analysis

consider 1m, width



$\Rightarrow F = (13.17 \times 1.625) + (9.59 \times 0.8)$

$= 29.07\ kN/m$



$\Rightarrow M = \dfrac{FL}{10}$

$= \dfrac{29.07 \times 2.425}{10}$

$= 7.05\ kNm/m$

Main Reinforcement

- Design moment, $M_{Ed} = 7.05$ kNm/m

- $K = \frac{M}{bd^2 f_{ck}}$

  $= \frac{7.05 \times 10^6}{1000 \times 87^2 \times 30} \rightarrow 0.031 \quad < K_{bal} = 0.167$

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

  $= d\left[0.5 + \sqrt{0.25 - \frac{0.031}{1.134}}\right] \rightarrow 0.97\ d > 0.95\ d$

- $A_s = \frac{M}{0.87 f_{yk} z}$

  $= \frac{7.05 \times 10^6}{0.87 \times 500 \times 0.95 \times 87} \rightarrow 196\ mm^2/m$

∴ Main bar : H10-250 ($314\ mm^2/m$)

minimum and maximum reinforcement area :

- $A_{s,min} = 0.26\left(\frac{f_{ctm}}{f_{yk}}\right) bd$

  $= 0.26\left(\frac{2.9}{500}\right) 1000 \times 87 \rightarrow 131.2\ mm^2/m.$

- $A_{s,max} = 0.04\,bh$

  $= 0.04 \times 1000 \times 112 \rightarrow 4480\ mm^2/m$

secondary reinforcement :

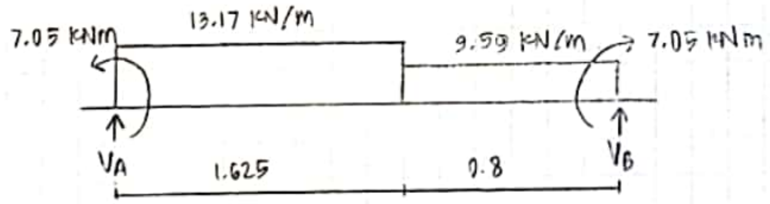
- $A_s = 20\%$ of reinforcement

  $= 20\% \times 196$

  $= 39.2\ mm^2/m$

∴ secondary bar : H10 - 300 ($262\ mm^2/m$).

shear



- $\Sigma M_B = 0$

  $V_A(2.425) - 13.17(1.625)(1.625/2 + 0.8) - 9.59(0.8)(0.8/2) = 0$

  $V_A = 15.50$ kN/m

- $\Sigma F_y = 0$

  $15.50 - 13.17(1.625) - 9.59(0.8) + V_B = 0$

  $V_B = 13.57$ kN/m

- design shear force, $V_{Ed} = 15.50$ kN/m

- $k = 1 + \left(\frac{200}{d}\right)^{1/2}$

  $= 1 + \left(\frac{200}{87}\right)^{1/2} \rightarrow 2.52 > 2.0$

  $\therefore k = 2.0$

- $\rho_1 = \frac{A_s}{bd} \leq 0.02$

  $= \frac{314}{1000 \times 87} \rightarrow 0.0036 \leq 0.02$

- $V_{Rd,c} = [0.12\,k\,(100\,\rho_1\,f_{ck})^{1/3}]\,bd$

  $= [0.12(2)(100 \times 0.0036 \times 30)^{1/3}]\,1000 \times 87$

  $= 46.2$ kN/m

- $V_{min} = (0.035\,k^{3/2}\,f_{ck}^{1/2})\,bd$

  $= (0.035\,(2)^{3/2}\,(30)^{1/2}) \times 1000 \times 87$

  $= 47.2$ kN/m.

  $V_{Rd,c} = 46.2$ kN/m $\geq V_{Ed} = 15.50$ kN/m

Deflection

- percentage of required tension reinforcement,

  $\rho = \frac{A_{s,req}}{bd}$

  $= \frac{196}{1000 \times 87} \rightarrow 0.002$

- Reference reinforcement ratio,

  $\rho_0 = \sqrt{f_{ck}} \times 10^{-3}$

  $= \sqrt{30} \times 10^{-3} \rightarrow 0.005$

  $\therefore \rho < \rho_0$.

- $K = 1.3$

- $l/d = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]$

  $= 1.3\left[11 + 1.5\sqrt{30}\left(\frac{0.005}{0.002}\right) + 3.2\sqrt{30}\left(\frac{0.005}{0.002} - 1\right)^{3/2}\right]$

  $= 82.86$

- Modification factor for span < 7m

  $= 1.0$

- Modification factor for steel area provide

  $= \frac{A_{s,req}}{A_{s,prov}} \rightarrow \frac{314}{196} = 1.6 < 1.5$

- $l/d_{allow} = 82.86 \times 1.0 \times 1.5$
  $= 124.29$

- $l/d_{act} = \frac{l}{d} \rightarrow \frac{2425}{87} = 27.87$

$\therefore l/d_{allow} > l/d_{act}$ (OK!).

Cracking

$h = 112$ mm < 200 mm

- Main bar:

$S_{max,slab} = 3h \leq 400$
$= 3(112) \leq 400$
$= 336$ mm ≤ 400 mm.

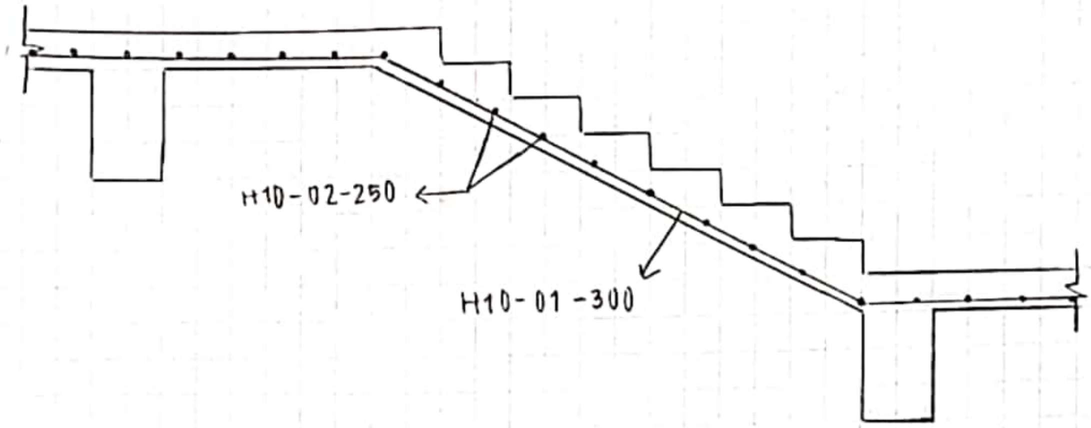
Max. bar spacing $= 250$ mm $< S_{max,slab}$.

- Secondary bar:

$S_{max,slab} = 3.5h \leq 450$
$= 3.5(112) \leq 450$
$= 392$ mm ≤ 450 mm

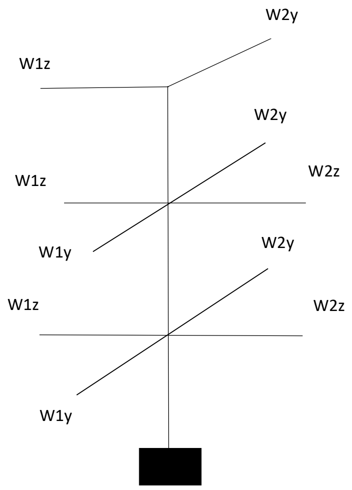
max. bar spacing $= 300$ mm $< S_{max,slab}$.

detailing



REF.	CALCULATION	OUTPUT

COLUMN 11/D



Specification

Fck =	30	N/mm²
Fyk =	500	N/mm²
Concrete Cover, C=	20	mm
Diameter Bar, Ø =	10	mm

Dimension

Slab Thickness, h_f =	150	mm

Characteristic Loads

Permanent Load (Excluding Selfweight)

Roof =	1.00	kN/m²
Others =	1.50	kN/m²

Variable Load

Roof =	0.25	kN/m²
Bedroom =	1.50	kN/m²
Bathroom =	1.50	kN/m²
Terrace =	1.50	kN/m²

Material

Unit weight of concrete, γ_c =	25	kN/m³

Dimension Size

Column, b x h =	300 x 350	mm
Roof Beam, b x h =	250 x 450	mm
First Floor Beam, b x h =	250 x 450	mm
Ground Floor Beam, b x h =	250 x 450	mm

Weight of Brickwork =	2.6	kN/m²

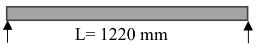
<u>ANALYSIS</u>

ROOF

Beam D/10-11

Selfweight =	1.88	kN/m
Characteristic Perm. Load, Gk =	1.88	kN/m

w= $1.35G_k$+$1.5Q_K$



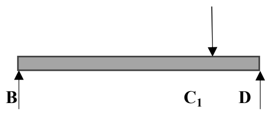
w= 2.53 kN/m

Q_K = 0

Beam 11/B-D

B-C_1

Selfweight =	1.88	kN/m
Characteristic Perm. Load, Gk =	1.88	kN/m



W_{B-C1} =	2.53	kN/m
W_{C1-D} =	2.53	kN/m
Point Load=	6.37	kN

C_1-D

Selfweight =	1.88	kN/m
Characteristic Perm. Load, Gk =	1.88	kN/m

Point Load (Beam C1/9-11)

Length of Secondary Beam =	3395	mm

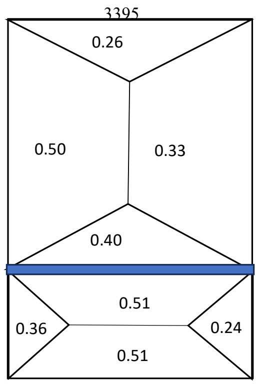
FIRST FLOOR

Beam 11/B_1 - D

Loads on Slab

Selfweight =	3.75	kN/m²
Permanent Load, g_k =	1.50	kN/m²
Characteristic Perm. Load, Gk =	5.25	kN/m²
Characteristic Variable Load, Qk =	1.50	kN/m²

L (Footing tc	2290	mm
L (1st Fl. To	3000	mm
L (1st Fl. To	3000	mm



4313

1788

Beam B_1 - C_1

Characteristic Permanent Load:

Selfweight =	1.88	kN/m
W_1 =	5.88	kN/m
Brickwall =	7.80	kN/m
Gk =	15.56	kN/m

Characteristic Variable Load:

W_1 =	1.68	kN/m
Characteristic Variable Load, Qk =	1.68	kN/m

w = 23.52 kN/m

Beam C_1-D

Characteristic Permanent Load:

Selfweight =	1.88	kN/m
W_1 =	2.25	kN/m
Brickwall =	7.80	kN/m
Gk =	11.93	kN/m

Characteristic Variable Load:

W_1 =	0.64	kN/m
Qk =	0.64	kN/m

w = 17.07 kN/m

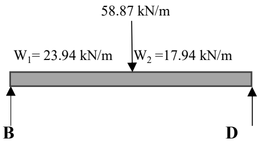
Point Load (Beam C1/9-11)

Characteristic Permanent Load:

Selfweight =	1.88	kN/m
W_1 =	7.13	kN/m

W_2 = 4.79 kN/m
Brickwall = 7.80 kN/m
Gk = 21.59 kN/m

Characteristic Variable Load:
W_1 = 2.04 kN/m
W_2 = 1.37 kN/m
Qk = 3.40 kN/m



Beam 11/B_1-D

w = 34.26 kN/m
Point Load = 58.15 kN

Beam 11/D-E
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
Brickwall = 0.00 kN/m
Gk = 1.88 kN/m
w = 2.53 kN/m

Beam D/10-11
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
W_1 = 4.79 kN/m
Brickwall = 7.8 kN/m
Gk = 14.46 kN/m

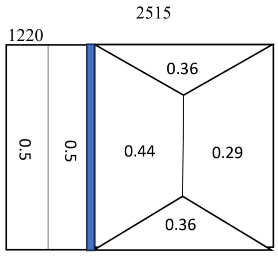
Characteristic Variable Load:
W_1 = 1.37 kN/m
Qk = 1.37 kN/m
w = 21.58 kN/m

Beam D/11-12
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
Gk = 1.88 kN/m
w = 2.53 kN/m

GROUND FLOOR
Beam 11/B_1 - D
Loads on Slab
Selfweight = 3.75 kN/m²
Permanent Load, g_k = 1.50 kN/m²
Characteristic Perm. Load, Gk = 5.25 kN/m²
Characteristic Variable Load, Qk = 0.00 kN/m²

Beam B_1 - D
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
W_1 = 3.20 kN/m
W_2 = 5.81 kN/m
Brickwall = 7.80 kN/m
Gk = 18.69 kN/m

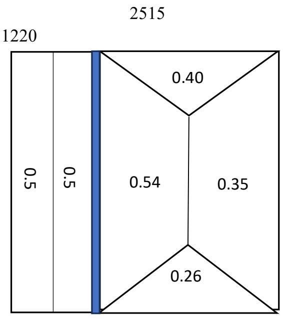
Characteristic Variable Load:
W_1 = 0.00 kN/m
Characteristic Variable Load, Qk = 0.00 kN/m



3040
w = 25.23 kN/m

Beam 11/D-E
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
W_1 = 3.20 kN/m
W_2 = 7.13 kN/m
Brickwall = 7.80 kN/m
Gk = 20.01 kN/m

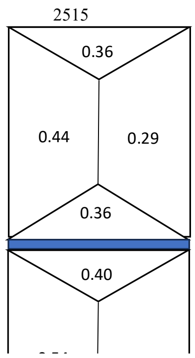
Characteristic Variable Load:
W_1 = 0.00 kN/m
Qk = 0.00 kN/m



3800
w = 27.01 kN/m

Beam 11-12/D
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
W_1 = 4.75 kN/m
W_2 = 5.28 kN/m
Brickwall = 7.80 kN/m
Gk = 19.71 kN/m

Characteristic Variable Load:
W_1 = 0.00 kN/m
Qk = 0.00 kN/m



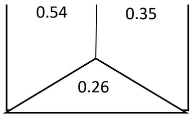
3040
w = 26.61 kN/m

Beam D/10-11
Characteristic Permanent Load:
Selfweight = 1.88 kN/m
Brickwall = 7.80 kN/m
Gk = 9.68 kN/m
3800

Characteristic Variable Load:

W_1 = 0.00 kN/m

Qk = 0.00 kN/m



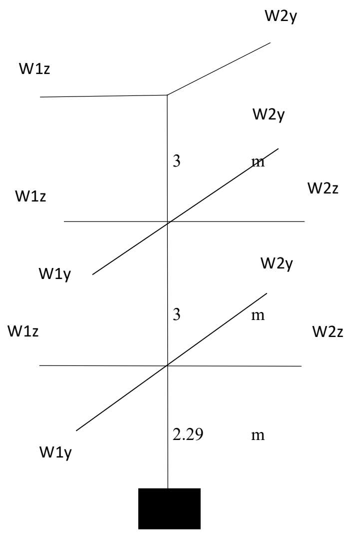
w = 13.06 kN/m

Axial Force

Roof	W_{1z}	W_{2z}	W_{1y}	W_{2y}	P_{2y}
G_k	1.88	0.00	0.00	3.75	6.37
Q_k	0.00	0.00	0.00	0.00	0.00
Max	2.53	0.00	0.00	5.06	8.59

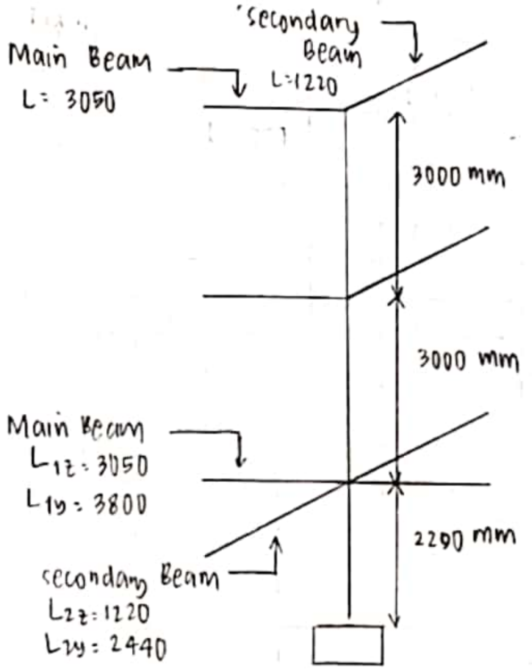
1st. Floor	W_{1z}	W_{2z}	W_{1y}	W_{2y}	P_{2y}
G_k	14.46	0.00	0.00	27.48	21.59
Q_k	1.37	0.00	0.00	2.32	3.40
Max	21.58	0.00	0.00	40.59	34.26

Gnd. Floor	W_{1z}	W_{2z}	W_{1y}	W_{2y}	P_{2y}
G_k	9.68	19.71	4.79	0.00	3.40
Q_k	0.00	0.00	0.00	9.38	1.88
Max	13.06	26.61	6.46	14.07	7.41



Axial Force:

Column : D/11
size: 300 X 350 mm



Roof	W_{1z}	W_{2y}	P_{2y}
Gk	1.88	3.75	6.37
Qk	0.0	0.0	0.0
Max	2.53	5.06	8.59

1st Floor	W_{1z}	W_{2y}	P_{2y}
GK	14.46	27.48	21.59
Qk	1.37	2.32	3.40
Max	21.58	40.59	34.26

Gnd. Floor	W_{1z}	W_{2z}	W_{1y}	W_{2y}	P_{2y}
GK	9.68	19.71	4.79	0.0	3.40
Qk	0.0	0.0	0.0	9.38	1.88
Max	13.06	26.61	6.46	14.07	7.41

1st Floor to Roof :

- Main beam : (2.53 x 3.05 x 0.35) = 2.70 kN
- sec. beam : (5.06 x 1.22 x 0.35) = 2.16 kN
- selfweight : 1.35 (0.3 x 0.35 x 3 x 25) = 10.63 kN +

$N_{1st\ fl-roof}$ = 15.49 kN

1st floor to Gnd. Floor:

- Main beam : (21.53 x 3.05 x 0.35) = 22.98 kN
- sec. beam : (40.59 x 1.22 x 0.35) = 17.33 kN
- selfweight : 1.35 (0.3 x 0.35 x 3 x 25) = 10.63 kN +

$N_{1st\ Flo-G.Flo}$ = 50.94 kN

Footing to Gnd. Floor:

- Main beam : (13.06 x 3.05 x 0.35) + (26.61 x 3.8 x 0.35) = 49.33 kN
- sec. beam : (6.46 x 1.22 x 0.05) + (14.07 x 2.44 x 0.35) = 14.77 kN
- selfweight : 1.35 (0.3 x 0.35 x 2.29 x 25) = 8.12 kN +

$N_{footing-G.Flo}$ = 72.22 kN

Bending Moment:

Z - Z

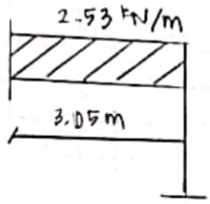
- Stiffness :

→ column 3.0 m = $\frac{I}{L} = \frac{bh^3}{12 \times L} \rightarrow \frac{300 \times 350^3}{12 \times 3000} = 3.6 \times 10^5$

Dipindai dengan CamScanner

- column 3.0 m $= \frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{300 \times 350^3}{12 \times 3000} = 3.6 \times 10^5$
- column 2.29 m $= \frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{300 \times 350^3}{12 \times 2290} = 4.7 \times 10^5$

- Beam 3.05 m $= \frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{250 \times 450^3}{12 \times 3050} = 6.2 \times 10^5$
- Beam 3.8 m $= \frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{250 \times 450^3}{12 \times 3800} = 5 \times 10^5$

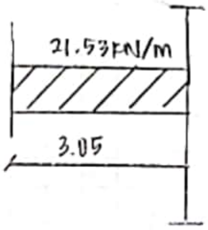


fixed end moment:

- $M_1 = \frac{WL^2}{12} \rightarrow \frac{2.53(3.05)^2}{12} = 1.96$ kNm

Moment in column:

- $M = \frac{\Delta M\, K_c}{K_c + K_b/2} \rightarrow \frac{1.96 \times 3.6 \times 10^5}{3.6 \times 10^5 + (6.2 \times 10^5/2)} = 1.1$ kNm.



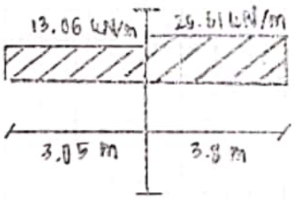
fixed end moment:

- $M_1 = \frac{WL^2}{12} \rightarrow \frac{21.53\,(3.05)^2}{12} = 16.69$ kNm

Moment in column:

- $M_u = \frac{\Delta M\, K_c}{K_c + K_b/2} \rightarrow \frac{16.69 \times 3.6 \times 10^5}{3.6 \times 10^5 + 3.6 \times 10^5 + (6.2 \times 10^5/2)} = 5.8$ kNm

- $M_l = \frac{\Delta M\, K_c}{K_c + K_b/2} \rightarrow \frac{16.69 \times 3.6 \times 10^5}{3.6 \times 10^5 + 3.6 \times 10^5 + (6.2 \times 10^5/2)} = 5.8$ kNm



fixed end moment:

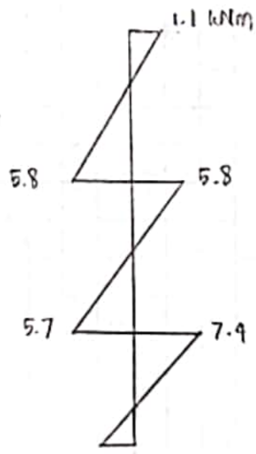
- $M_1 = \frac{WL^2}{12} \rightarrow \frac{13.06\,(3.05)^2}{12} = 10.12$ kNm
- $M_2 = \frac{WL^2}{12} \rightarrow \frac{26.61\,(3.8)^2}{12} = 32.02$ kNm.

- $\Delta M = M_2 - M_1 \rightarrow 32.02 - 10.12 = 21.9$ kNm.

Moment in column:

- $M_u = \frac{\Delta M\, K_c}{K_c + K_b/2} \rightarrow \frac{21.9 \times 3.6 \times 10^5}{3.6 \times 10^5 + 4.7 \times 10^5 + (6.2 \times 10^5/2) + (5 \times 10^5/2)}$
  $= 5.7$ kNm

- $M_l = \frac{\Delta M\, K_c}{K_c + K_b/2} \rightarrow \frac{21.9 \times 4.7 \times 10^5}{3.6 \times 10^5 + 4.7 \times 10^5 + (6.2 \times 10^5/2) + (5 \times 10^5/2)}$
  $= 7.4$ kNm



M_{2-2} (kNm)

y-y

- Stiffness :

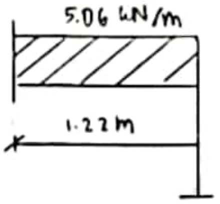
⇒ Column 3.0 m : $\frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{350 \times 300^3}{12 \times 3000} = 2.6 \times 10^5$

⇒ Column 3.0 m : $\frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{350 \times 300^3}{12 \times 3000} = 2.6 \times 10^5$

⇒ Column 2.29 m : $\frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{350 \times 300^3}{12 \times 2290} = 3.4 \times 10^5$

⇒ Beam 1.22 m : $\frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{250 \times 450^3}{12 \times 1220} = 15.6 \times 10^5$

⇒ Beam 2.44 m : $\frac{I}{L} = \frac{bh^3}{12L} \rightarrow \frac{250 \times 450^3}{12 \times 2440} = 7.8 \times 10^5$

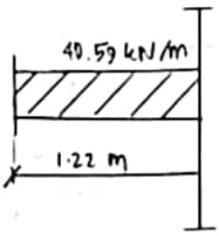


fixed end moment :

- $M_1 : \frac{WL^2}{12} \rightarrow \frac{5.06\,(1.22)^2}{12} = 0.63$ kNm

Moment in column :

- $M : \frac{\Delta M K_c}{K_c + K_b/2} \rightarrow \frac{0.63 \times 2.6 \times 10^5}{2.6 \times 10^5 + (15.6 \times 10^5/2)} = 0.2$ kNm .



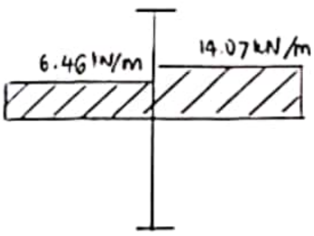
fixed end moment :

- $M_1 : \frac{WL^2}{12} \rightarrow \frac{40.59\,(1.22)^2}{12} = 5.03$ kNm

Moment in column :

- $M_u : \frac{\Delta M K_c}{K_c + K_b/2} \rightarrow \frac{5.03 \times 2.6 \times 10^5}{2.6 \times 10^5 + 2.6 \times 10^5 + (15.6 \times 10^5/2)} = 1$ kNm

- $M_l : \frac{\Delta M K_c}{K_c + K_b/2} \rightarrow \frac{5.03 \times 2.6 \times 10^5}{2.6 \times 10^5 + 2.6 \times 10^5 + (15.6 \times 10^5/2)} = 1$ kNm



fixed end moment :

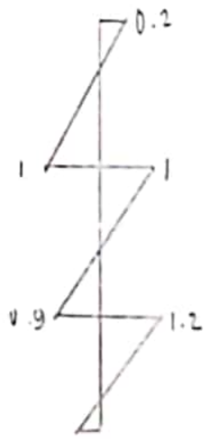
- $M_1 : \frac{WL^2}{12} \rightarrow \frac{6.46\,(1.22)^2}{12} = 0.80$ kNm
- $M_2 : \frac{WL^2}{12} \rightarrow \frac{14.07\,(2.44)^2}{12} = 6.98$ kNm

- $\Delta M : M_2 - M_1 \rightarrow 6.98 - 0.80 = 6.18$ kNm

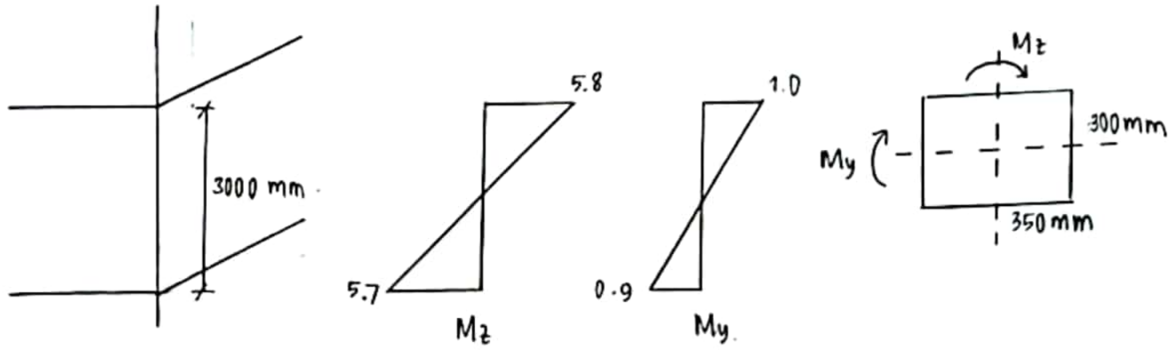
Moment in column :

- $M_u : \frac{\Delta M K_c}{K_c + K_b/2} \rightarrow \frac{6.18 \times 2.6 \times 10^5}{2.6 \times 10^5 + 3.4 \times 10^5 + (15.6 \times 10^5/2) + (7.8 \times 10^5/2)}$
  $= 0.9$ kNm

- $M_l : \frac{\Delta M K_c}{K_c + K_b/2} \rightarrow \frac{6.18 \times 3.4 \times 10^5}{2.6 \times 10^5 + 3.4 \times 10^5 + (15.6 \times 10^5/2) + (7.8 \times 10^5/2)}$
  $= 1.2$ kNm



$M_{y\text{-}y}$ (kNm)



Dimension and size:

Column: $b \times h = 300 \times 350$ mm

$L_z = 3000 - 450$

$= 2550$ mm

$L_y = 3000 - 450$

$= 2550$ mm

Beam:

- Main beam: $b \times h = 250 \times 450$ mm

  $L_1 = 3050$ mm

  $L_2 = 3800$ mm

- Sec. beam: $b \times h = 250 \times 450$ mm

  $L_1 = 1220$ mm

  $L_2 = 2440$ mm

Moment of Inertia:

Column:

- $I_{zz} = \frac{bh^3}{12} \rightarrow \frac{300 \times 350^3}{12} = 1.1 \times 10^9 \text{ mm}^4$
- $I_{yy} = \frac{bh^3}{12} \rightarrow \frac{350 \times 300^3}{12} = 0.8 \times 10^9 \text{ mm}^4$

Beam:

- $I_{MB} = \frac{bh^3}{12} \rightarrow \frac{250 \times 450^3}{12} = 1.9 \times 10^9 \text{ mm}^4$
- $I_{SB} = \frac{bh^3}{12} \rightarrow \frac{250 \times 450^3}{12} = 1.9 \times 10^9 \text{ mm}^4$

Stiffness:

Column:

- $K_{zz} = \frac{EI}{\ell} \rightarrow \frac{1.1 \times 10^9}{2250} = 4.9 \times 10^5 \text{ mm}^3$
- $K_{yy} = \frac{EI}{\ell} \rightarrow \frac{0.8 \times 10^9}{2250} = 3.6 \times 10^5 \text{ mm}^3$

Beam:

- Main, $K_{MB_1} = \frac{EI}{\ell} \rightarrow \frac{1.9 \times 10^9}{3050} = 6.2 \times 10^5 \text{ mm}^3$

  $K_{MB_2} = \frac{EI}{\ell} \rightarrow \frac{1.9 \times 10^9}{3800} = 5 \times 10^5 \text{ mm}^3$

- sec, $k_{sb1} = \frac{EI}{\ell} \rightarrow \frac{1.9 \times 10^9}{1220} = 15.6 \times 10^5 \ mm^5$

  $k_{sb2} = \frac{EI}{\ell} \rightarrow \frac{1.9 \times 10^9}{2440} = 7.8 \times 10^5 \ mm^5$

Relative column stiffness:

z - axis :

- Top end : $k_2 = \frac{k_c}{2(\Sigma k_b)} \rightarrow \frac{4.9 \times 10^5}{2(6.2 \times 10^5 + 5 \times 10^5)} = 0.2 \quad < 1.0$
- bot. end : $k_1 = \frac{k_c}{2(\Sigma k_b)} \rightarrow \frac{4.9 \times 10^5}{2(6.2 \times 10^5 + 5 \times 10^5)} = 0.2 \quad < 1.0$

y - axis :

- Top end : $k_2 = \frac{k_c}{2(\Sigma k_b)} \rightarrow \frac{3.6 \times 10^5}{2(15.6 \times 10^5 + 7.8 \times 10^5)} = 0.1 \quad < 1.0$
- bot. end : $k_1 = \frac{k_c}{2(\Sigma k_b)} \rightarrow \frac{3.6 \times 10^5}{2(15.6 \times 10^5 + 7.8 \times 10^5)} = 0.1 \quad < 1.0$

Effective length of column:

- $\ell_{0z} = 0.5 \ell_z \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right)\left(1 + \frac{k_2}{0.45 + k_2}\right)}$

  $= 0.5(2550)\sqrt{\left(1 + \frac{0.2}{0.45 + 0.2}\right)\left(1 + \frac{0.2}{0.45 + 0.2}\right)} \rightarrow 1471 \ mm$

- $\ell_{0y} : 0.5 \ell_y \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right)\left(1 + \frac{k_2}{0.45 + k_2}\right)}$

  $= 0.5(2550)\sqrt{\left(1 + \frac{0.1}{0.45 + 0.1}\right)\left(1 + \frac{0.1}{0.45 + 0.1}\right)} \rightarrow 1330 \ mm$

Radius of Gyration:

- $i_{zz} : \sqrt{\frac{I_{zz}}{A}} \rightarrow \sqrt{\frac{1.1 \times 10^9}{300 \times 350}} = 102$

- $i_{yy} : \sqrt{\frac{I_{yy}}{A}} \rightarrow \sqrt{\frac{0.8 \times 10^9}{300 \times 350}} = 87$

Slenderness Ratio:

- $\lambda_{zz} = \frac{\ell_{0z}}{i_{zz}} \rightarrow \frac{1471}{102} = 14.4$

- $\lambda_{yy} : \frac{\ell_{0y}}{i_{yy}} \rightarrow \frac{1330}{87} = 15.3$

Slenderness Limit:

- A : 0.7 (φ_{eff} not known)

- B : 1.1 (ω not known)

- $C = 1.7 - r_m$

  → z-axis : $r_m = \frac{M_{01}}{M_{02}} \rightarrow \frac{-5.8}{5.7} = -1.02$

  $C_z = 1.7 - (-1.02) = 2.72$

  → y-axis : $r_m = \frac{M_{01}}{M_{02}} \rightarrow \frac{-1.0}{0.9} = -1.11$

  $C_y = 1.7 - (-1.11) = 2.81$

- $f_{cd} = \frac{0.85 f_{ck}}{\gamma_c} \rightarrow \frac{0.85 \times 30}{1.5} = 17 \ N/mm^2$

- $n = \frac{N_{Ed}}{A_c f_{cd}} \rightarrow \frac{139 \times 10^3}{(300 \times 350)(17)} = 0.1$

- z-axis : $\lambda_{lim} = \frac{20ABC}{(n)^{1/2}} \rightarrow \frac{20 \times 0.7 \times 1.1 \times 2.72}{(0.1)^{1/2}} = 132.5 > 14.4$

- y-axis : $\lambda_{lim} = \frac{20ABC}{(n)^{1/2}} \rightarrow \frac{20 \times 0.7 \times 1.1 \times 2.81}{(0.1)^{1/2}} = 136.8 > 15.3$

The column is non-slender

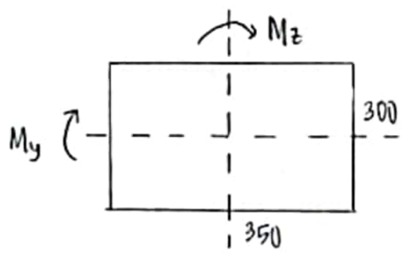
Specification:

Classification : short braced column

Axial load : 150 kN

Material:

- concrete, f_{ck} = 30 N/mm^2
- Reinforcement, f_{yk} = 500 N/mm^2
- size : b x h = 300 X 350 mm
- Effective length : l_{0z} = 1.471 m
  - l_{0y} = 1.330 m
- Slenderness Ratio : λ_z = 14.4
  - λ_y = 15.3
- Assumed : Ø link = 6 mm
  - Ø bar = 16 mm



M_z : 5.8 kNm
M_y : 1.0 kNm

Design Moment:

The imperfection moment,

- $M_{imp,z} = \frac{N_{Ed} \times l_{0z}}{400} \rightarrow \frac{150 \times 1.471}{400} = 0.55$
- $M_{imp,y} = \frac{N_{Ed} \times l_{0y}}{400} \rightarrow \frac{150 \times 1.330}{400} = 0.50$

The design moment including the effect of imperfection.

- $M_{Ed,z}$ = 5.8 + 0.55 → 6.35 kNm

- M_{Ey} : 1.0 + 0.50 → 1.50 kNm

Check biaxial bending:

- $e_z = \frac{M_{Edy}}{N_{Ed}} \rightarrow \frac{1.50 \times 10^6}{150 \times 10^3} = 10$ mm
- $e_y = \frac{M_{Edz}}{N_{Ed}} \rightarrow \frac{6.35 \times 10^6}{150 \times 10^3} = 42.3$ mm.
- $\left(\frac{e_y}{h}\right) / \left(\frac{e_z}{b}\right) = \left(\frac{42.3}{350}\right) / \left(\frac{10}{300}\right) = 3.63 \quad > 0.2$
- $\left(\frac{e_z}{b}\right) / \left(\frac{e_y}{h}\right) = \left(\frac{10}{300}\right) / \left(\frac{42.3}{350}\right) = 0.28 \quad > 0.2$

∴ not pass!

- $\frac{\lambda_y}{\lambda_z} = \frac{15.3}{14.4} \rightarrow 1.06 < 2.0$
- $\frac{\lambda_z}{\lambda_y} = \frac{14.4}{15.3} \rightarrow 0.94 < 2.0$

∴ pass!

∴ since, there is one of two requirement unsatisfied. So, we have to check biaxial loading.

Reinforcement Design:

Effective depth, $d = h - C_{nom} - \varnothing_{link} - 0.5\varnothing_{bar}$
- $h' = 350 - 20 - 6 - (16/2) \rightarrow 316$ mm
- $b' = 300 - 20 - 6 - (16/2) \rightarrow 266$ mm

- $\frac{M_{Edz}}{h'} \rightarrow \frac{6.35 \times 10^6}{316} = 20095$
- $\frac{M_{Edy}}{b'} \rightarrow \frac{1.50 \times 10^6}{266} = 5639$

∴ $M_{Ed,z}/h' > M_{Ed,y}/b'$

- use: $M'_z = M_z + \beta\,(h'/b')\,M_y$.
- $\frac{N}{bhf_{ck}} \rightarrow \frac{150 \times 3}{(300 \times 350) \times 30} = 0.05$
- $\beta = 1 - \frac{N}{bhf_{ck}} \rightarrow 1 - 0.05 = 0.95 \quad \geq 0.3$
- $M'_z = 6.35 + 0.95(316/266) \times 1.50$
  $= 8.3$ kNm.
- $\frac{d}{h} \rightarrow \frac{266}{350} = 0.76$
- $\frac{N}{bhf_{ck}} = 0.05$
- $\frac{M}{bh^2 f_{ck}} = \frac{8.3 \times 10^6}{(300 \times 350^2) \times 30} = 0.01$

- $\frac{A_s f_{yk}}{bh f_{ck}} = 0.1$

- $A_s = \frac{0.10\, bh f_{ck}}{f_{yk}} \rightarrow \frac{0.10 \times 300 \times 350 \times 30}{500} = 630 \text{ mm}^2$ $\therefore$ 4H16 (804 mm^2)

- $A_{s,min} = \frac{0.1 N_{Ed}}{f_{yd}} = \frac{0.1 N_{Ed}}{0.87 f_{yk}} \rightarrow \frac{0.1 \times 150 \times 10^3}{0.87 \times 500} = 34.5 \text{ mm}^2$

- $A_{s,max} = 0.04 A_c = 0.04\, bh$
  $= 0.04(300 \times 350) \rightarrow 4200 \text{ mm}^2$

- Links, $\varnothing_{min} = 0.25 \times 30$
  $= 7.5 \text{ mm}$ $\geq 6.0 \text{ mm}$

  S_v max : the lesser of $= 20 \times 16$
  $= 320$ mm or 300 mm or 400 mm

  use: H6 − 300

  At section: 400 mm below and above beam and at lapped joints, S_v max $= 0.6 \times 300 = 180$ mm

  use: H6 −180

Check biaxial bending

Steel Area,

All : 4H16 $A_s = 804 \text{ mm}^2$
z-z : 4H16 $A_s = 804 \text{ mm}^2$
y-y : 4H16 $A_s = 804 \text{ mm}^2$

→ $\frac{d_z}{h} \rightarrow \frac{316}{350} = 0.9$

→ $\frac{d_y}{b} \rightarrow \frac{266}{300} = 0.9$

- $\frac{N}{bh f_{ck}} \rightarrow \frac{150 \times 10^3}{300 \times 350 \times 30} = 0.05$

- $\frac{A_{sz} f_{yk}}{bh^2 f_{ck}} \rightarrow \frac{804 \times 500}{300 \times 350^2 \times 30} = 0.13$

  → $M/bh^2 f_{ck} = 0.08$
  $M_{Rdz} = 0.08 \times 300 \times 350^2 \times 30$
  $= 88.2 \text{ kNm}$

- $\frac{A_{sy} f_{yk}}{bh^2 f_{ck}} \rightarrow \frac{804 \times 500}{300 \times 350^2 \times 30} = 0.13$

  → $M/bh^2 f_{ck} = 0.08$
  $M_{Rdy} = 0.08 \times 300 \times 350^2 \times 30$
  $= 88.2 \text{ kNm}$

- $N_{Rd} = 0.567 f_{ck} A_c + 0.87 f_{yk} A_s$
  $= 0.567(30)(350 \times 300) + 0.87(500)(804)$
  $= 2136 \text{ kN}$

- $\frac{N_{ed}}{N_{Rd}} \rightarrow \frac{150}{2136} = 0.07 \approx 0.1$ $\therefore a = 1.0$

- $M_{ed,z} = 6.35$ kNm
- $M_{ed,y} = 1.50$ kNm

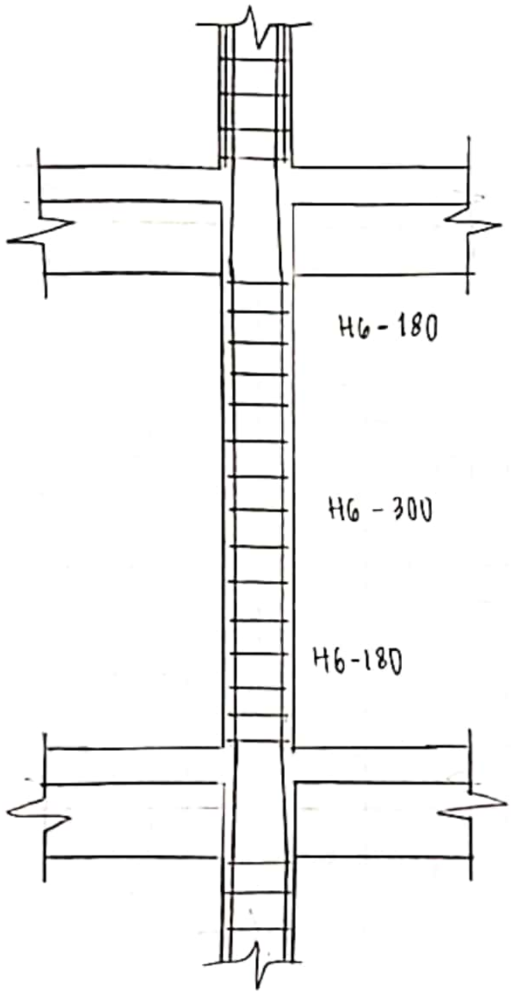
$$\left(\frac{M_{ed\,z}}{M_{Rd\,z}}\right)^{a} + \left(\frac{M_{ed\,y}}{M_{Rd\,y}}\right)^{a} \leq 1.0$$

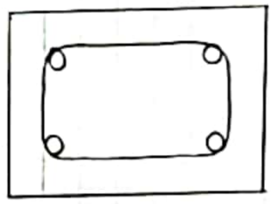
$$\left(\frac{6.35}{88.2}\right)^{1.0} + \left(\frac{1.50}{88.2}\right)^{1.0} \leq 1.0$$

$$0.09 \leq 1.0$$

OK !

detailing.



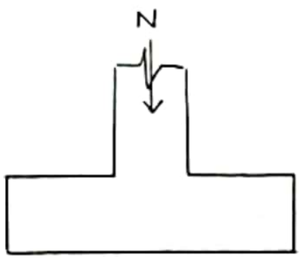


use: 4H16
($804 mm^2$).

NAME : NUR AVIFA AUDRIANA JAWALI
MATRIC : A19EA3009

pad footing

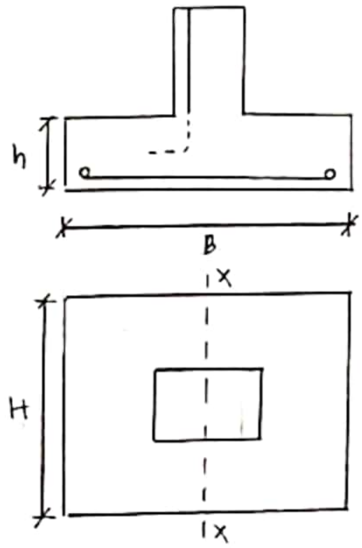
specification:



Axial Force, N
- N_{Ed} = 300 kN

Material :
- concrete, f_{ck} = 30 N/mm²
- Reinforcement, f_{yk} = 500 N/mm²
- unit weight of conc. = 25 kN/m³
- soil bearing capacity = 150 kN/m³
- Assumed : Ø bar = 10 mm
  Cnom = 20 mm

size :



- service action, N = 300 kN
- Assumed footing selfweight 10% of service load, W = 30 kN
- Area of footing required,
  $A_f = \frac{N+W}{\text{soil bearing cap.}}$
  $= \frac{300+30}{150} \rightarrow 2.2\ m^2$
- Try size : B x H x h
  = 1.5 x 2 x 0.2
  A = 3 m² > Area req = 2.2 m²
- selfweight = 25 x 3 x 0.5
  = 15 kN

- service soil pressure,
  $= \frac{N+W}{A} \rightarrow \frac{300+30}{3} = 110\ kN/m^2 \leq \sigma_{safe} = 150\ kN/m^2$

∴ size OK !

Analysis :

- Ultimate axial force :
  $N_{ult} = \frac{300}{1.4} \rightarrow 214\ kN$
- soil pressure at ultimate load,
  $P = \frac{N_{Ed}}{A} \rightarrow \frac{214}{3} = 71.3\ kN/m^2$
- soil pressure per m length,
  W = 71.3 x 2
  = 143 kN/m

Design shear resistance,

- $K = 1 + \left(\frac{200}{d}\right)^{1/2} \rightarrow 1 + \left(\frac{200}{165}\right)^{1/2} = 2.1 \geq 2.0$

Note: Bar extend beyond critical section at

$410 - 20 > 36\phi + d$

$410 - 20 > 36(10) + 165$

$390 < 525$

$\therefore$ so, As not required

- $V_{min} = V_{Rd} = (0.035\,k^{3/2} f_{ck}^{1/2})\,bd$

$= (0.035(2)^{3/2}(30)^{1/2})(1500 \times 165)$

$= 134200$ N

$= 134$ kN

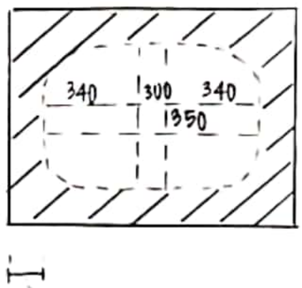
so, $V_{Rd} = 134$ kN $> V_{Ed} = 59$ kN $\therefore$ OK!

- $FOS = \frac{V_{Rd}}{V_{Ed}} \rightarrow \frac{134}{59} = 2.3$

(ii) Punching shear at perimeter 2d from column face

- Average $d = 200 - 20 - 10 \rightarrow 170$ mm

$2d = 2(170) \rightarrow 340$ mm



- control perimeter,

$u = 2(300 + 350) + (2 \times \pi \times 340)$

$= 3436$ mm

- Area within perimeter

$A = (0.35 \times 0.3) + (2 \times 0.3 \times 0.34) + (2 \times 0.35 \times 0.34) + (\pi \times 0.34^2)$

$= 0.91\ m^2$

$l = 575 - 340 - 20 < 36(10) + 170$

$= 215 > 530$

$\therefore$ so, As not required

Punching shear resistance due to concrete

- $V_{Rdc} = V_{min} = (0.035\,k^{3/2} f_{ck}^{1/2})\,ud$

$= (0.035(2)^{3/2}(30)^{1/2}) \times 3436 \times 170$

$= 316720$ N

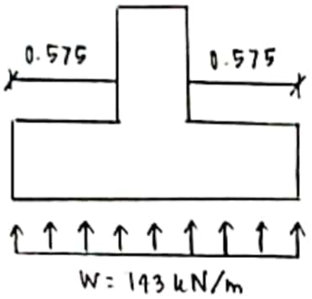
$= 317$ kN

Punching shear force,

- $V_{Ed} = 71.3((1.5 \times 2) - 0.91)$

$= 149$ kN $< V_{Rdc} = 317$ kN

- $FOS = \frac{V_{Rdc}}{V_{Ed}} \rightarrow \frac{317}{149} = 2.1$



- Max. moment column,

$$M = \frac{WL^2}{2} = \frac{143 \times (0.575)^2}{2} \rightarrow 23.6\ kNm$$

Main Reinforcement:

- Effective depth,

$d = h - C_{nom} - \varnothing bar - 0.5 \varnothing bar$

$= 200 - 20 - 10 - 0.5(10) \rightarrow 165\ mm$

- Bending Moment, $M = 23.6\ kNm$

- $K = \frac{M}{bd^2 f_{ck}} \rightarrow \frac{23.6 \times 10^6}{1500 \times 165^2 \times 30} = 0.02 < K_{bal} = 0.167$

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

$= d\left(0.5 + \sqrt{0.25 - \frac{0.02}{1.134}}\right) \rightarrow 0.98d \geq 0.95d$

- $As = \frac{M}{0.87 f_{yk} z}$

$= \frac{23.6 \times 10^6}{0.87 \times 500 \times 0.95(165)} \rightarrow 346\ mm^2$

Main bar : 6H10 ($471\ mm^2$)

Maximum and Minimum reinforcement Area,

- $As,min = 0.26\left(\frac{f_{ctm}}{f_{yk}}\right) bd$

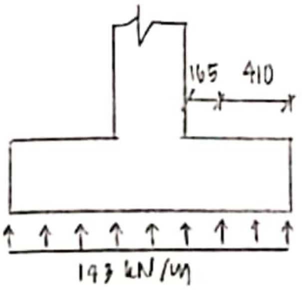
$= 0.26\left(\frac{2.9}{500}\right)(1500 \times 165) \rightarrow 373\ mm^2$

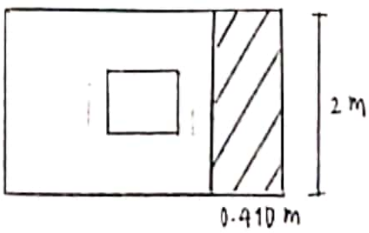
- $As,max = 0.04\ Ac = 0.04 \times b \times h$

$= 0.04 \times 1500 \times 200 \rightarrow 12000\ mm^2$

Shear:

(i) vertical shear : critical at 1.0 d from column face.





- Design shear force, $V_{Ed} = 143 \times 0.410$

$= 59\ kN/m$

(iii) Maximum punching shear at column perimeter,
Maximum shear resistance,

- $V_{Rd,max} = 0.5\,ud\,[0.6(1-f_{ck}/250)]\,f_{ck}/1.5$

$$= 0.5\,(2(350+300))\,(170)\left[0.6\left(1-\frac{30}{250}\right)\right]\left(\frac{30}{1.5}\right)$$

$$= 1167 \text{ kN} \qquad > V_{Ed,max} : 300 \text{ kN}$$

$\therefore$ OK!

<u>Cracking:</u>

Assume steel stress under quasi-permanent loading,

$$= 0.6\left(\frac{f_{yk}}{1.15}\right)\left(\frac{A_{s,req}}{A_{s,prov}}\right) \rightarrow 0.6\left(\frac{500}{1.15}\right)\left(\frac{346}{471}\right) = 192 \text{ N/mm}^2$$

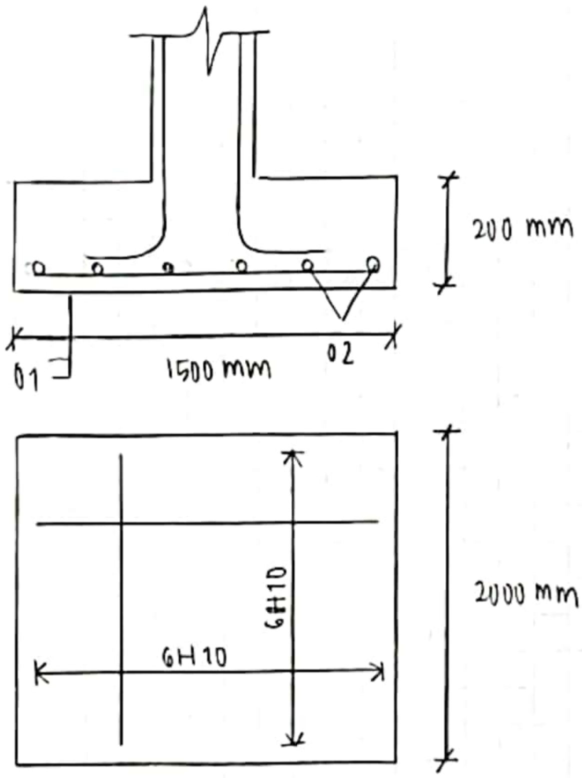
For design crack width = 0.3 m

$\therefore$ • Max. allowable bar spacing = 260 mm

• Max bar spacing = (1500 − 2(20) − 10)/6

= 242 mm < 260 mm $\therefore$ OK!

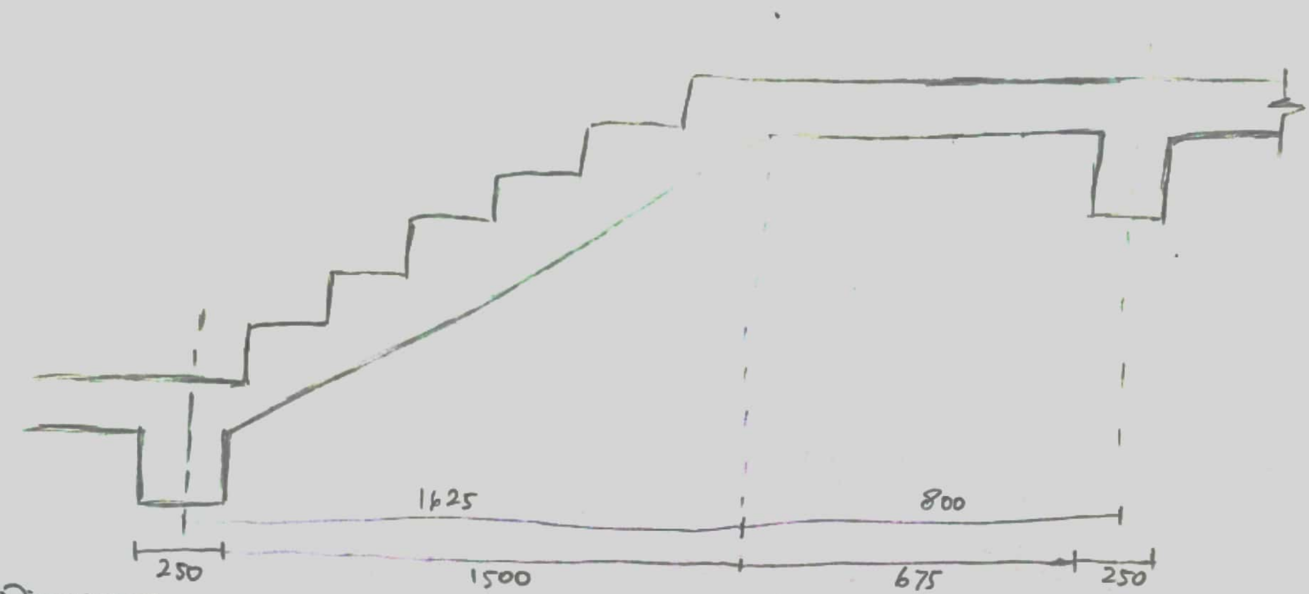
detailing.



DWI RAHAYU PUTRI SISWATI

Name : Dwi Rahayu Putri Siswati

Matric No.: A19EA3006



Dimension

- $R = 167$ mm
- $G = 250$ m
- $\frac{L}{d} \simeq 29$ -> can choose between 26 -30
- $\frac{2425}{29} = d$; $d = 84$mm
- $h = d + c + 0.5\,\phi bar$
  $= 84 + 20 + 0.5(10)$
  $= 109$ mm ≈ 110 mm

Actions

- Permanent charac, $g_k = 1.5$ kN/m²
- Variable charac, $q_k = 2.5$ kN/m²
- F_{ck} 30 N/mm²
- $F_{yk} = 500$ N/mm²
- Unit weight of concrete = 25 N/mm³
- Concrete cover, $C = 20$mm
  $\phi bar = 10$mm

average thickness

$$\Rightarrow y = h\left[\frac{\sqrt{G^2+R^2}}{G}\right] = 110\left[\frac{\sqrt{250^2+167^2}}{250}\right] = 132\text{mm}$$

$\Rightarrow$ average thickness

$$t = y + \frac{R}{2} = 132 + \frac{167}{2} = 216 \text{ mm}$$

Action & Analysis

Flight

Slab selfweight : 0.216 × 25 = 5.4 kN/m²

Permanent load (excluding self weight) = 1.5 kN/m² +

$\Rightarrow g_k = 6.9$ kN/m²

$\Rightarrow q_k = 2.5$ kN/m²

Design Action, $n_d = 1.35(g_k) + 1.5(q_k) = 13.1$ kN/m²

Consider 1m width, $w_d = 13.1$ kN/m² × 1m = 13.1 kN/m

Landing

Slab selfweight $= 0.11 \times 25 = 2.75 \ kN/m^2$

Permanent load (exclude selfweight) $= 1.5 \ kN/m^2$

$g_k = 4.25 \ kN/m^2$

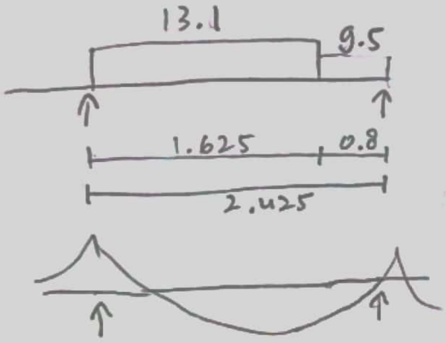
$q_k = 2.5 \ kN/m^2$

Design Action, $n_d = 1.35(g_k) + 1.5(q_k) = 9.5 \ kN/m^2$

Consider 1m width, $w_d = 9.5 \times 1 = 9.5 \ kN/m$

Analysis

Consider 1m width,



Total Action,

$F = (13.1 \times 1.625) + (9.5 \times 0.8)$

$= 28.9 \ kN/m$

Moment,

$M = \frac{FL}{10} = \frac{28.9 \times 2.425}{10} = 7 \ kN/m$

Main Reinforcement

- Design moment, $M_{ed} = 7 \ kNm/m$
- $k = \frac{7 \times 10^6}{1000 \times 84^2 \times 30} = 0.03 < k_{bal} = 0.167$

$\therefore$ compression reinforcement isn't required

- $z = d\left(0.5 + \sqrt{0.25 - \frac{k}{1.134}}\right) = d\left(0.5 + \sqrt{0.25 - \frac{0.03}{1.134}}\right)$

$= 0.97d > 0.95d$

- $A_s = \frac{M}{0.87 f_{yk} z} = \frac{7 \times 10^6}{0.87(500)(0.95)(84)} = 201.65 \ mm^2/m \rightarrow$

Main bar: H10 - 250 ($314 \ mm^2/m$)

- Minimum & Maximum reinforcement area,

$A_{s,min} = 0.26\left(\frac{f_{ctm}}{f_{yk}}\right)bd = 0.26\left(\frac{2.9}{500}\right)1000(84) = 126.7 \ mm^2/m$

$A_{s,max} = 0.04 \ bh = 0.04 \times 1000 \times 110 = 4400 \ mm^2/m$

- secondary reinforcement,

$A_s = 20\%$ of reinforcement

$= 0.2 \times 201.65 = 40.33 \ mm^2/m \rightarrow$

Secondary bar: H10 - 300 ($262 \ mm^2/m$)

Deflection

Percentage of required tension reinforcement,

$$\rho = \frac{A_{s,req}}{bd} = \frac{202}{1000 \times 84} = 0.0024$$

Reference reinforcement ratio,

$$\rho_o = (f_{ck})^{1/2} \times 10^{-3} = (30)^{1/2} \times 10^{-3} = 0.0055$$

Factor for structural system, $k = 1.3$

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right)$$

$$= 1.3\left(11 + 1.5\sqrt{30}\,\frac{0.0055}{0.0024} + 3.2\sqrt{30}\left(\frac{0.0055}{0.0024} - 1\right)^{3/2}\right)$$

$$= 1.3\,(11 + 18.8 + 25.73)$$

$$= 72.189$$

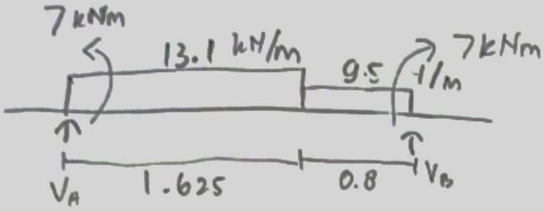
- Modification factor for span $< 7m = 1.0$
- Modification factor for steel area provided $= \frac{314}{202} = 1.55 > 1.5$
- Therefore, allowable span-effective depth ratio,

$$\left(\frac{l}{d}\right)_{allowable} = 72.189 \times 1 \times 1.4 = 101.064$$

Actual span-effective depth,

$$\left(\frac{l}{d}\right)_{actual} = \frac{2425}{84} = 28.87 < \left(\frac{l}{d}\right)_{allowable} \quad \text{ok!}$$

Shear



$\Sigma M_B = 0$

$V_A (2.425) - 13.1 (1.625)(1.61) - 9.5 (0.8)(0.4) = 0$

$V_A = 15.39 \text{ kN/m}$

$V_B = 9.5(0.8) + (13.1 \times 1.625) - 15.35$

$= 13.54 \text{ kN/m}$

Design shear force, $V_{Ed} = 15.39 \text{ kN/m}$

$V_{Rd,c} = (0.12 k (100 \rho_1 f_{ck})^{1/3}) bd$

$k = 1 + \left(\frac{200}{d}\right)^{1/2} \leq 2.0$

$= 1 + \left(\frac{200}{84}\right)^{1/2} > 2.0$

$= 2.54 > 2.0$

$k = 2.0$

$\rho_1 = \frac{A_{s1}}{bd} \leq 0.02 = \frac{314}{1000 \times 84} \leq 0.02$

$= 0.0037 \leq 0.002$

$V_{Rd,c} = (0.12 (2)(100 \times 0.0037 \times 30)^{1/3}) \times 1000 \times 84$

$= 44.97 \text{ kN/m}$

$V_{min} = (0.035 k^{3/2} f_{ck}^{1/2}) bd$

$= (0.035 (2)^{3/2} (30)^{1/2}) \times 1000 \times 84$

$= 45546 \text{ N/mm} = 45.5 \text{ kN/m}$

so, $V_{Rd,c} = 45.5 > V_{Ed} = 15.39 \text{ kN/m}$ ok!

Cracking

$h = 110\ mm < 200\ mm$

Main bar

$S_{max,slab} = 3h \leq 400 = 330$

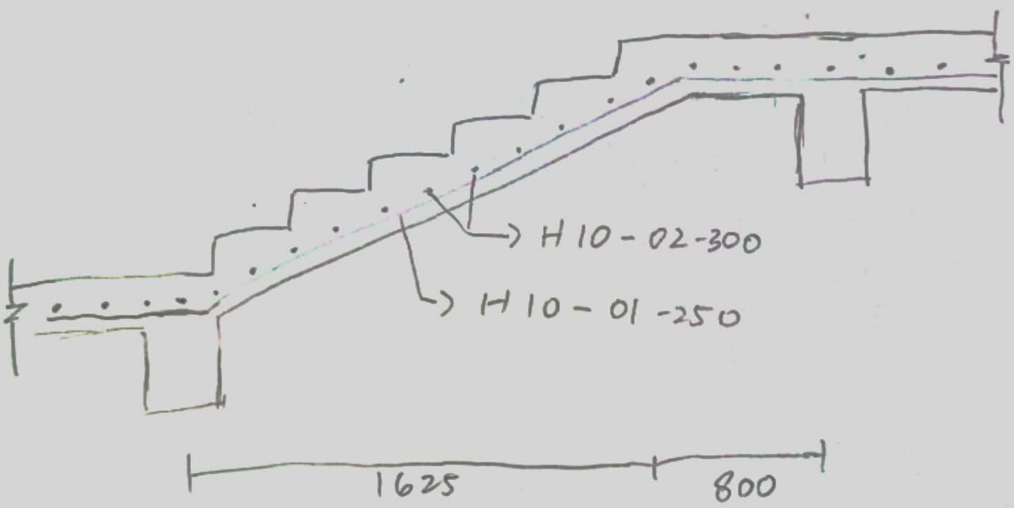
Max bar spacing = 250 mm < $S_{max, slab}$

secondary bar

$S_{max, slab} = 3.5h \leq 450 = 385$

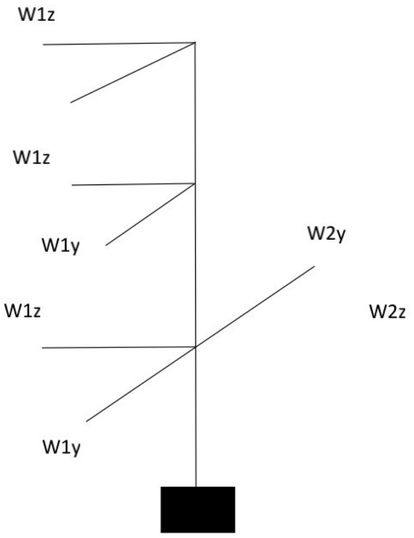
Max bar spacing = 300 < S_{max} slab.

Detailing



CALCULATION

COLUMN 11/D



Specification

Fck =	30	N/mm²
Fyk =	500	N/mm²
Concrete Cover, C=	20	mm
Diameter Bar, Ø =	16	mm

Dimension

Slab Thickness, h_f =	150	mm

Characteristic Loads

Permanent Load (Excluding Selfweight)

Roof =	1,00	kN/m²
Others =	1,50	kN/m²
Variable Load		
Roof =	0,25	kN/m²
Bedroom =	1,50	kN/m²
Kitchen =	1,50	kN/m²
Terrace =	1,50	kN/m²

Material

Unit weight of concrete, γ_c =	25	kN/m³

Dimension Size

Column, b x h =	400 x 400	mm
Roof Beam, b x h =	250 x 500	mm
First Floor Beam, b x h =	250 x 500	mm
Ground Floor Beam, b x h =	250 x 500	mm

Weight of Brickwork =	2,6	kN/m²

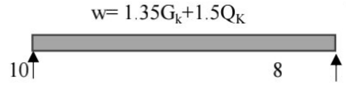
ANALYSIS

ROOF

Beam E/10-8

Selfweight =	2,19	kN/m
Characteristic Perm. Load, Gk =	2,19	kN/m

$w = 1.35G_k + 1.5Q_K$ W= 2,95 kN/m



Beam 11/E-D

B-C_1

Selfweight =	2,19	kN/m
Characteristic Perm. Load, Gk =	2,19	kN/m

W = 2,95 kN/m

FIRST FLOOR

Beam 11/B_1 - D

Loads on Slab

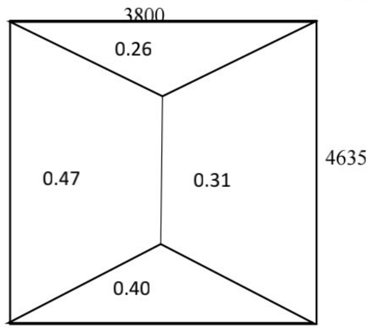
Selfweight =	3,75	kN/m²
Permanent Load, g_k =	1,50	kN/m²
Characteristic Perm. Load, Gk =	5,25	kN/m²
Characteristic Variable Load, Qk =	1,50	kN/m²

L (Footing tc	2290	mm
L (1st Fl. To	3000	mm
L (1st Fl. To	3000	mm

Beam E/10-8

Characteristic Permanent Load:

Selfweight =	2,19	kN/m
W_1 =	6,18	kN/m
Brickwall =	7,80	kN/m
Gk =	16,17	kN/m



w = 24,48 kN/m

Characteristic Variable Load:

W_1 =	1,77	kN/m
Characteristic Variable Load, Qk =	1,77	kN/m

Beam 10/E-D

Characteristic Permanent Load:

Selfweight =	2,19	kN/m
W_1 =	5,19	kN/m
Brickwall =	7,80	kN/m
Gk =	15,17	kN/m

Characteristic Variable Load:

W_1 = 1,48 kN/m

Qk = 1,48 kN/m

w = 22,71 kN/m

GROUND FLOOR

Loads on Slab

Selfweight = 3,75 kN/m²

Permanent Load, g_k = 1,50 kN/m²

Characteristic Perm. Load, Gk = 5,25 kN/m²

Characteristic Variable Load, Qk = 1,50 kN/m²

Beam 11/D-E

Characteristic Permanent Load:

Selfweight = 2,19 kN/m

W_1 = 7,17 kN/m

W_2 = 3,20 kN/m

Brickwall = 7,80 kN/m

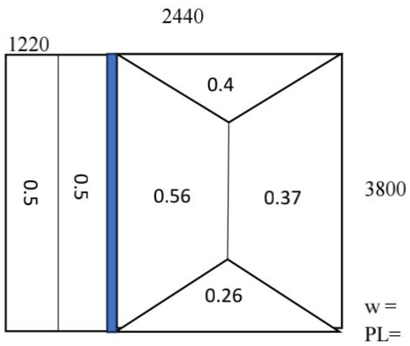
Gk = 20,36 kN/m

Characteristic Variable Load:

W_1 = 2,05 kN/m

W2 = 0,92

Characteristic Variable Load, Qk = 2,96 kN/m



w = 31,94 kN/m

PL= 60,68 kN

Beam 10/12-11

Characteristic Permanent Load:

Selfweight = 2,19 kN/m

W_1 = 3,33 kN/m

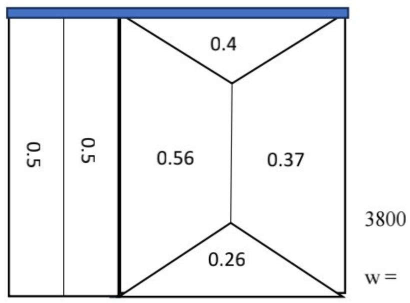
Brickwall = 7,80 kN/m

Gk = 13,32 kN/m

Characteristic Variable Load:

W_1 = 0,95 kN/m

Qk = 0,95 kN/m



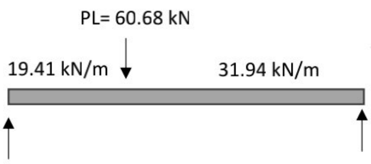
w = 19,41 kN/m

Beam E/11-10

Characteristic Permanent Load:

Selfweight = 2,19 kN/m

Gk = 2,19 kN/m



w= 2,95 kN/m

Beam E/10-8

Characteristic Permanent Load:

Selfweight = 2,19 kN/m

W1 = 6,18 kN/m

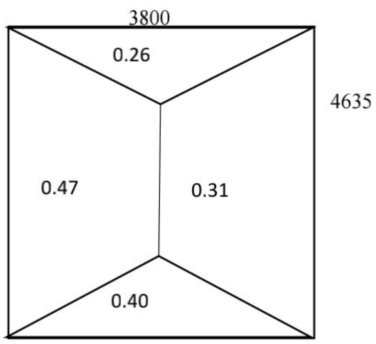
Brickwall = 7,80 kN/m

Gk = 16,17 kN/m

Characteristic Variable Load:

W1 = 1,77 kN/m

Characteristic Variable Load, Qk = 1,77 kN/m



w = 24,48 kN/m

Beam 10/E-D

Characteristic Permanent Load:

Selfweight = 2,19 kN/m

W1 = 5,19 kN/m

Brickwall = 7,80 kN/m

Gk = 15,17 kN/m

Characteristic Variable Load:

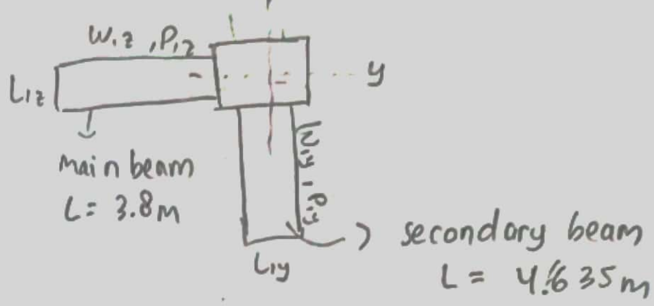
W1 = 1,48 kN/m

Qk = 1,48 kN/m

w = 22,71 kN/m

Roof	W_{1y}	W_{1z}
Gk	2.19	2.19
Qk	0	0
Max	2.95	2.95

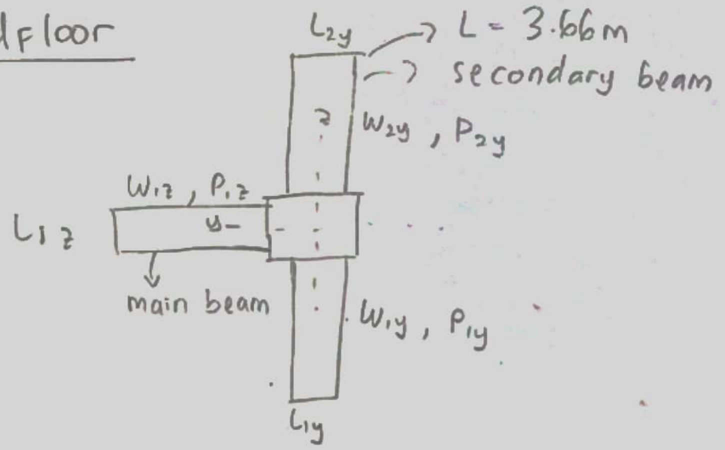
Roof and First Floor



Firstfloor	W_{1y}	W_{1z}
Gk	16.17	15.17
Qk	1.77	1.48
Max	24.48	22.71

Groundfloor	W_{1y}	W_{1z}	W_{2y}	P_{2y}
Gk	16.17	15.7	15.51	22.47
Qk	1.77	1.48	0.95	20.23
Max	24.48	22.71	22.36	60.68

Groundfloor



First Floor to Roof

Main beam = $(2.95 \times 3.8 \times 0.4) = 4.484$ kN

secondary beam = $(2.95 \times 4.635 \times 0.4) = 5.47$ kN

selfweight = $(1.35 \times 0.4 \times 0.4 \times 3 \times 25) = 16.2$ kN +

N 1st Floor-Floor = 26.154 kN

First floor to Ground Floor

load from above

Main beam = $(22.71 \times 3.8 \times 0.4) = 34.52$ kN

Secondary beam = $(24.48 \times 4.635 \times 0.4) = 45.386$ kN

selfweight = $(1.35 \times 0.4 \times 0.4 \times 3 \times 25) = 16.2$ kN +

N gnd.FL – 1st.FL = 96.106 kN

Footing to Gnd. FL

Load from above

Main beam = $(22.71 \times 3.8 \times 0.4) = 34.52$ kN

Sec beam = $(24.48 \times 4.635 \times 0.4) + (51.35 \times 3.660 \times 0.4) = 120.56$ kN

Selfweight = $1.35\,(0.4 \times 0.4 \times 2.29 \times 25) = 12.366$ +

$N_{footing - Gnd.FL} = 167.446$ kN

Bending Moments

z-z :

stiffness, $K = I/L = (bh^3/12)/L$

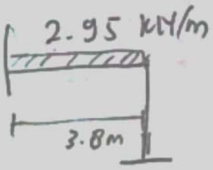
Column 3.0m = $(400 \times 400^3 / (12 \times 3000) = 7.1 \times 10^5$

3.0 m = 7.1×10^5

2.29 m = 9.3×10^5

Beam 4.635 = $250 \times 500^3 / (12 \times 4635) = 5.62 \times 10^5$

3.8 = 6.85×10^5



Fixed End Moment,

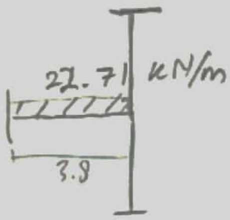
$M_1 = wL^2/12$

$= \dfrac{2.95\,(3.8)^2}{12} = 3.55$ kNm

Moment in column

$M = \Delta M \times K_c / (K_c + K_b/2)$

$= \dfrac{3.55 \times 7.1}{7.1 + 6.85/2} = 2.395$



Fixed End Moment,

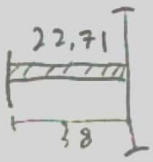
$M_1 = wL^2/12$

$= \dfrac{22.71 \times (3.8)^2}{12} = 27.3$ kNm

Moment in column

$M_u = \dfrac{27.3 \times 7.1}{7.1 + 7.1 + 6.85/2} = 10.998$

$M_L = \dfrac{27.3 \times 7.1}{7.1 + 7.1 + 6.85/2} = 10.998$



Fixed End Moment,

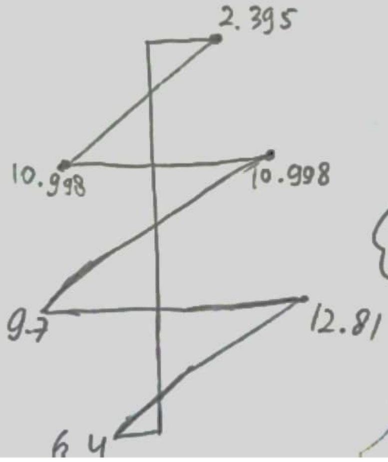
$M_1 = wL^2/12$

$= \dfrac{22.71 \times (3.8)^2}{12} = 27.3$ kNm

Moment in Column

$M_L = \dfrac{27.3 \times 9.3}{9.3 + 7.1 + 6.85/2} = 12.81$

$M_u = \dfrac{27.3 \times 7.1}{9.3 + 7.1 + 6.85/2} = 9.77$



M_{z-z}

y-y axis :

Stiffness , $k = I/L = (bh^3/12)L$

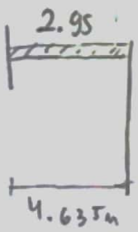
Column : 3m $= 7.1 \times 10^5$

3m $= 7.1 \times 10^5$

2.29m $= 9.3 \times 10^5$

Beam : 4.635 $= 250 \times 500^3 / 12 \times 4635 = 5.62 \times 10^5$

3.66 $= 250 \times 500^3 / 12 \times 3.660 = 7.12 \times 10^5$



→ Fixed end moment

$M_1 = wL^2/12 = 2.95(4.62)^2/12 = 5.247$

→ Moment in column

$$M = \frac{5.247 \times 7.1}{7.1 + 5.62/2} = 3.76 \text{ kNm}$$

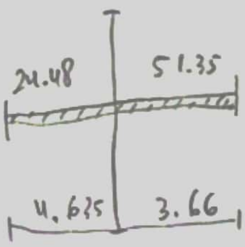
→ Fixed end moment

$M_1 = 24.48 (4.62)^2/12 = 43.54$

• Moment in column

$$M_U = \frac{4347 \times 7.1}{7.1 + 7.1 + 5.62/2} = 11.409 \text{ kNm}$$

$$M_L = \frac{43.47 \times 7.1}{7.1 + 7.1 + 5.62/2} = 11.409 \text{ kNm}$$



→ Fixed end moment

$M_1 = wL^2/12 = 24.48 (4.635^2)/12 = 43.826$

$$M_2 = wL^2/12 + \frac{Pa^2b}{9} = \frac{51.35(3.6)^2}{12} + \frac{60.68(2.44)^2(1.22)}{9}$$

$= 60.85$

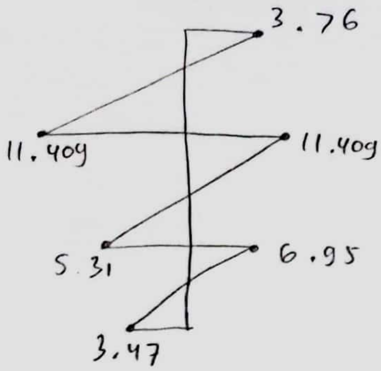
$\Delta m = 60.85 - 43.826$

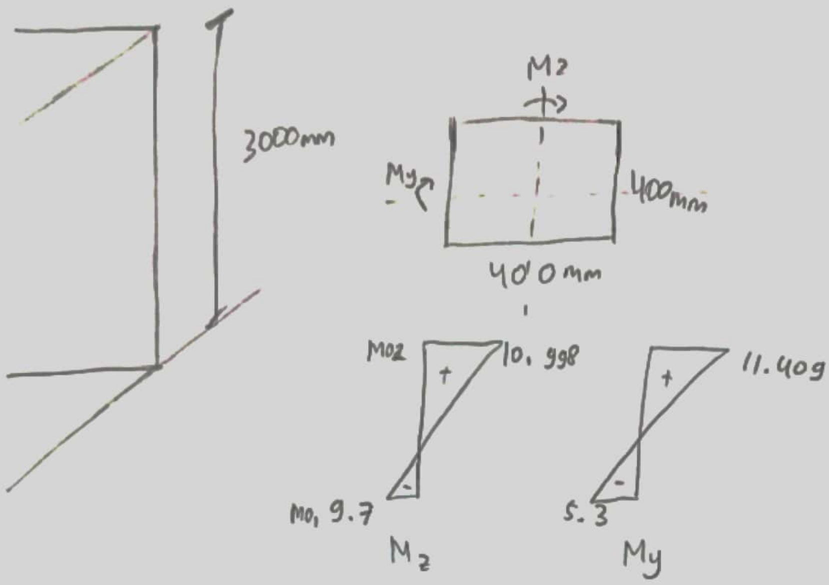
$= 17.027$

→ Moment in column

$$M_u = \frac{17.027 \times 7.1}{7.1 + 9.3 + 5.62/2 + 7.12/2} = 5.31$$

$$M_L = \frac{17.027 \times 9.3}{9.3 + 7.1 + 5.62/2 + 7.12/2} = 6.95$$


3.76
11.409
11.409
5.31
6.95
3.47



Dimension and Size

Column : $b \times h = 400 \times 400$

$L_z = 3000 - 400$
$= 2600$ mm

$L_y = 3000 - 400$
$= 2600$ mm

Beam : $b \times h = 250 \times 500$ mm
$L = 3800$ mm } main beam

$b \times h = 250 \times 500$
$L = 4635$ mm $L = 3660$ mm

Moment of inertia $I = bh^3/12$

Column : $I_{zz} = 400 \times 400^3/12 = 2.13 \times 10^9$
$I_{yy} = 400 \times 400^3/12 = 2.13 \times 10^9$

Beam : main beam, $I = 250 \times 500^3/12 = 2.6 \times 10^9$
secondary beam, $I = 250 \times 500^3/12 = 2.6 \times 10^9$

Specification

Classification : short braced column

Material :

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

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

Size $b \times h$ = 400 × 400 mm

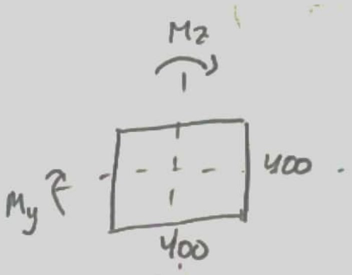
Effective length, $l_{oz} = 2.304\ m$

$l_{oy} = 2.254\ m$

Slenderness ratio, $\lambda_z = 20.28$

$\lambda_y = 19.54$

Axial Load = 150 kN



$M_z = 10.988\ kNm$

$M_y = 11.409\ kNm$

Assumed , ϕ link = 6mm

ϕ bar = 16 mm

Design Moment

The imperfection moment,

$M_{edz} = 10.988 + (150 \times (2.304 / 400)) = 11.85\ kNm$

$M_{edy} = 11.409 + (150 \times (2.254 / 400)) = 12.27\ kNm$

Check Biaxial Bending

$e_z = M_{edy}/N_{ed} = 13.04 \times 10^6 / 150 \times 10^3 = 86.93$

$e_y = M_{edz}/N_{ed} = 12.654 \times 10^6 / 150 \times 10^3 = 84.36$

① $(e_y/h)/(e_z/b) = (84.36/400)/(86.93/400) = 0.97 > 0.2$ not pass

② $\lambda_y/\lambda_z = 19.54/20.28 = 0.96 < 2$ Pass!

$\lambda_z/\lambda_y = 20.28/19.54 = 1.037 < 2$

Since , there is one of the two requirement unsatisfied so we have to check biaxial bending

Stiffness $k = EI/l$

Column : $k_{zz} = 2.13 \times 10^9 / 2600 = 8.2 \times 10^5\ mm^4$

$k_{yy} = 2.13 \times 10^9 / 2600 = 8.2 \times 10^5\ mm^4$

Beam : main $k_{mb} = 2.6 \times 10^9 / 3800 = 6.8 \times 10^5\ mm^5$

secondary $k_{sb_1} = 2.6 \times 10^9 / 4635 = 5.6 \times 10^5\ mm^5$

$k_{sb_2} = 2.6 \times 10^9 / 3660 = 7.1 \times 10^5\ mm^5$

Relative column stiffness, $k = k_{col} / 2(\Sigma k_{beam})$

z axis : Top end : $k_2 = 8.2 / 2(6.8) = 0.6 > 0.1$

Bottom end = $k_1 = 8.2 / 2(6.8) = 0.6 > 0.1$

y axis = Top end = $k_2 = 8.2 / 2(5.6) = 0.7 > 0.1$

Bottom end = $k_2 = 8.2 / 2(5.6 + 7.1) = 0.3 > 0.1$

Effective length of Column

$$l_0 = 0.5\, l \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) + \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

$$l_{0z} = 0.5 . 2600 \sqrt{\left(1 + \frac{0.6}{0.45 + 0.6}\right) + \left(1 + \frac{0.6}{0.45 + 0.6}\right)}$$

$$= 2304\ mm$$

$$l_{0y} = 0.5\ 2600 \sqrt{\left(1 + \frac{0.6}{0.45 + 0.6}\right) + \left(1 + \frac{0.3}{0.45 + 0.3}\right)}$$

$$= 2254.926$$

slenderness Ratio, $L = l_0 / i$

$$\lambda_z = 2304 / (2.13 \times 10^9 / 160000)^{1/2} = 20.28$$

$$\lambda_y = 2254.926 / (213 \times 10^9 / 160000)^{1/2} = 19.54$$

Slenderness limit

$A = 0.7$, $B = 1.1$, $C_z = 2.58$, $C_y = 2.165$, $n = 290 \times 10^3 / 160000(0.567 \times 30)$
$= 0.2$

z-axis : $\lambda_{lim} = 20 \times 0.7 \times 1.1 \times 2.58 / (0.2)^{0.5} = 88.8 > 20.28$

y-axis = $\lambda_{lim} = 20 \times 0.7 \times 1.1 \times 2.165 / (0.2)^{0.5} = 74.6 > 19.54$

Column is non-slender

Reinforcement Design

$h' = 400 - 20 - 6 - 8 = 366$ mm
$b' = 400 - 20 - 6 - 8 = 366$ mm

$Med_z / h' = 11.85 \times 10^6 / 366 = 32377$
$Med_y / b' = 12.27 \times 10^6 / 366 = 33524$

$Med_z / h' < Med_y / b'$

use => $M'_y = 12.27 + \beta (b'/h')\ 11.9$

- $N / bh F_{ck} = 150 \times 10^3 / (400 \times 400 \times 30) = 0.03$
- $\beta = 1 - N / bh F_{ck} = 1 - 0.03$
$= 0.1$

> $M'_y = 12.27 + 0.1 (366 / 366)\ 11.9$
$= 14$ kNm

> $d/h = 366 / 400 = 0.9$

$N / bh F_{ck} = 0.1$
$M / bh^2 F_{ck} = 14 \times 10^6 / (400 \times 400^2 \times 30) = 0.01$

$A_s F_{yk} / bh F_{ck} = 0.1$

$A_s = 1 (bh F_{ck} / F_{yk})$
$= \frac{400 \times 400 \times 30}{500} = 960\ mm^2$ => use 5H16
(1005 mm²)

$A_{S,min} = 0.1\ N_{Ed} F_{yd} = 0.1 N_{Ed} / 0.87 F_{yk})$
$= 34.48\ mm^2$

$A_{s max} = 0.04\ (400 \times 400) = 6400\ mm^2$

•> link, $\phi_{min} = 0.25 \times 30 = 7.5$ mm $\geq$ 6mm

$S_{v max}$ = the lesser of
$= 20 \times 16 = 320$ or 300mm or 400mm
use H6 - 300mm

•> At section, 400 mm below and above beam and at lapped joints, $S_{v max} = 0.6 \times 300 = 180$ mm

use : H6 - 180

Check Biaxial Bending

All : 5H16 $\quad A_s = 1005\ mm^2$

z-z : 5H16 $\quad A_s = 1005\ mm^2$

y-y : 5H16 $\quad A_s = 1005\ mm^2$

$d/h = 366/400 = 0.9$

$N/bhF_{ck} = 150 \times 10^3 / (400 \times 400 \times 30)$
$= 0.03$

$A_s F_{yk}/bhF_{ck} = 1005 \times 500 / (400 \times 400 \times 30) = 0.1$

$M/bh^2F_{ck} = 0.06$

$\Rightarrow \quad M_{Rdz} = 0.06 \times 400 \times 400^2 \times 30$
$= 134\ kNm$

$A_s F_{yk}/bhF_{ck} = 1005 \times 500 / (400 \times 400 \times 30)\ 0.1$

$M/bh^2F_{ck} = 0.06$

$M_{Rdz} = 0.06 \times 400 \times 400^2 \times 30$
$= 134\ kNm$

$N_{Rd} = 0.567\ F_{yk}\ A + 0.87\ F_{yk}\ A_s$
$= 0.567\ (500)(400 \times 400) + 0.87\ (500)(1005)$
$= 3159\ kN$

$N_{Ed}/N_{Rd} = 150/3159$
$= 0.05 \leq 0.1 \Rightarrow \quad a = 1$

$\Rightarrow M_{edz} = 12.654\ kNm$

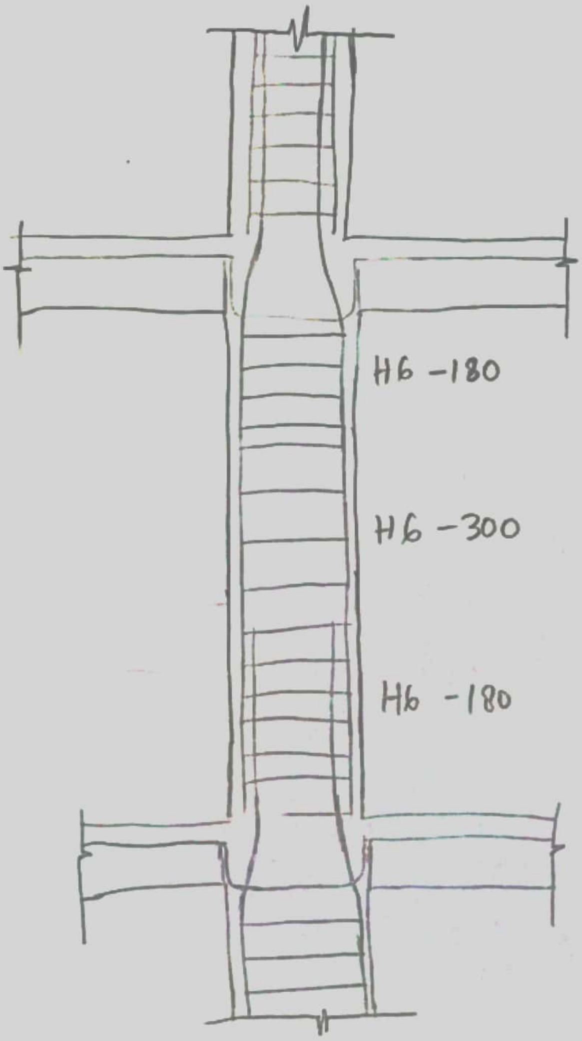
$M_{edy} = 13.04\ kNm$

$(M_{Edz}/M_{Rdz})^a + (M_{edy}/M_{Rdy})^a \leq 1.0$

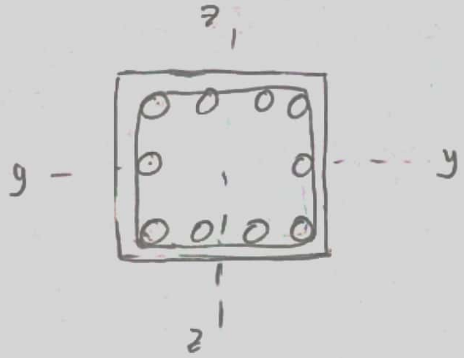
$(12.654/134)^1 + (13.04/134)^1 \leq 1.0$

$0.2 \leq 1.0$

Ok !



Use = 5H16 ($1005 mm^2$)

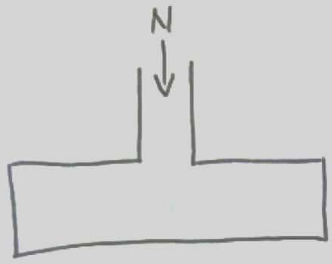


Column :
Size = 400 x 400

Concrete cover = 20 mm

Pad Footing

Specification



Column size :
400 mm x 400mm

Axial Force, N
$N_{ed} = 300$ kN

Material :
Concrete, $F_{ck} = 30\ N/mm^2$
Reinforcement, $f_{yk} = 500N/mm^2$
Unit weight of concrete $= 25\ kN/m^3$
Soil bearing Capacity $= 150\ kN/m^2$
Assumed : ϕ bar $= 10$ mm
Cnom $= 40$

Size

- Service load, $N = 300$ kN
- Assume footing selfweight 10% of service load, $w = 30$ kN
- Area of footing required,
  $= (N+w)$ / soil bearing capacity
  $= (300 + 30) / 150 = 2.2\ m^2$

Try square footing,
$B \times H \times h = 1.5 \times 1.5 \times 0.2$ => area : $B \times h = 2.25 >$ area req $= 2.2$
$= 0.9\ m^2$

Selfweight $= 0.45 \times 25$
$= 12$ kN

Service soil pressure

$= (N + w) / A$
$= (300 + 12) / 2.25$
$= 138.6\ kN/m^2 < 150\ kN/m^2$

Size ok!

Analysis

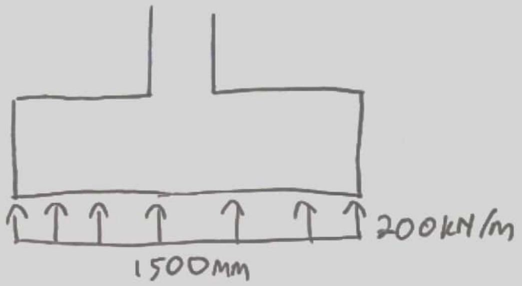
Ultimate axial force,

$N_{ed,ult} = 300/1.4 = 215$ kN

Soil pressure at ultimate load,

$P = N_{ed}/A = 215/2.25 = 95.5$ kN/m²

$w = 100 \times 2 = 200$ kN/m



Max moment at column,

$M = wL^2/2$

$= \dfrac{200(0.6)^2}{2}$

$= 37$ kNm

Main Reinforcement

Effective depth,

$d = h - c_{nom} - \phi bar - 0.5\phi bar$

$= 200 - 20 - 10 - 5 = 165$

Bending Moment, $M = 37$ kNm

$K = M/bd^2 F_{ck}$

$= 37 \times 10^6 / (1500 \times 165^2 \times 30)$

$= 0.03 < k_{bal} = 0.167$

$z = d\left(0.5 + \sqrt{0.25 - \dfrac{0.03}{1.134}}\right) = 0.97d \leq 0.95d$

$As = \dfrac{37 \times 10^6}{0.87(500)(0.95)(165)} = 542\ mm^2$

Main bar : 8H10 (628 mm²)

minimum & maximum reinforcement area,

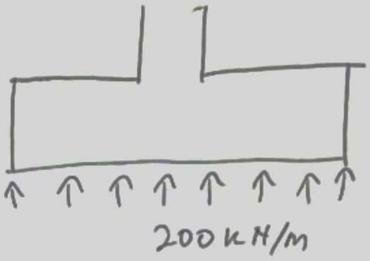
$As_{min} = 0.26\,(f_{ctm}/f_{yk})\,bd = 0.26 \times (2.9/500)\,2000 \times 365$

$= 373.3\ mm^2$

$As_{max} = 0.04 \times 1500 \times 165 = 9900\ mm^2$

Shear

1) Vertical shear : critical at 1.0d



Design shear force,

$V_{ed} = 200 \times 0.385$
$= 77$ kN

$25 = 40\phi$
$30 = 36\phi$

Design shear resistance

$385 - 20 = 365 < (lb + d) = 36\phi + d$
$= 36(10) + 165$

so, as not required

$V_{min} = V_{Rdc} = (0.035 k^{3/2} F_{ck}^{1/2})\, bd \rightarrow k = 1 + (200/d)^{0.5}$
$= 1 + (200/165)^{0.5}$
$= 2$

$= 0.035(2)^{3/2}(30)^{1/2}$
$= 134198$ N $= 134$ kN

$V_{Rd} = 134 > V_{ed} = 77 \Rightarrow$ ok!

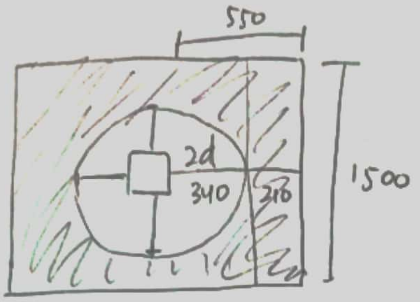
$FOS = \dfrac{V_{Rd}}{V_{ed}} = \dfrac{134}{77} = 1.7$

ii) Punching shear

$d = 200 - 20 - 10 = 170\,mm$

$2d = 2 \times 170 = 340\,mm$

$V_{ed} : 200\left((1.5)^2 - 1.067\right)$
$= 236.6\ kN$



Achorage provided

$(550 - 340) - 20 = 190$

$(l_{bd} + d) = 36(10) + 170$
$= 550$

$190 < 550$, so A_s not provided

Area within perimeter

$\therefore A = \left(4(0.4 \times 0.34)\right) + (0.4)^2 + (\pi \times 0.34^2)$
$= 1.067\ m^2$

control perimeter

$u = (4 \times 150) + (2 \times \pi \times 340) = 2736\ mm$

Punching shear resistance due to concrete

$V_{Rdc} = V_{min} = (0.035 \times 2^{3/2} \times 30^{0.5} \times 2736 \times 340)$
$= 504\ kN$

$V_{min} = 504\,kN > V_{ed} = 236.6\,kN$

$FOS = \dfrac{504}{236.6} = 2.1$

Dipindai dengan CamScanner

iii) Punching Shear around the column

Punching shear Force

$V_{ed} = 200\left((1.5)^2 - 0.4^2\right)$

$= 418 \text{ kN} < V_{Rdc} = 504$

$V_{rd,max} = 0.5\,(2736)(170)\left(0.6\left(1-\frac{30}{250}\right)\right)(30/1.5)$

$= 2455 \text{ kN} > V_{ed,max} = 300 \text{ kN}$ Ok !

Cracking

assume steel stress under quasi permanent loading,

$= 0.6\,(f_{yk}/1.15)\,(A_{s,req}/A_{s,prov})$

$= 0.6\,(500/1.15)\,(542/628)$

$= 225 \text{ N/mm}^2$

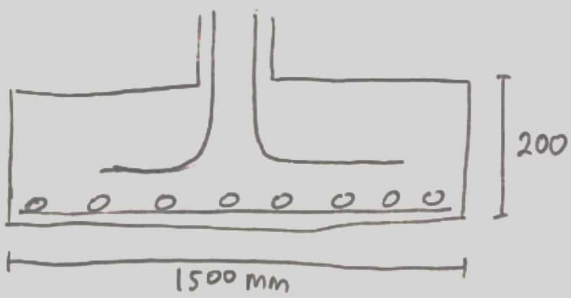
For design crack width = 0.3mm

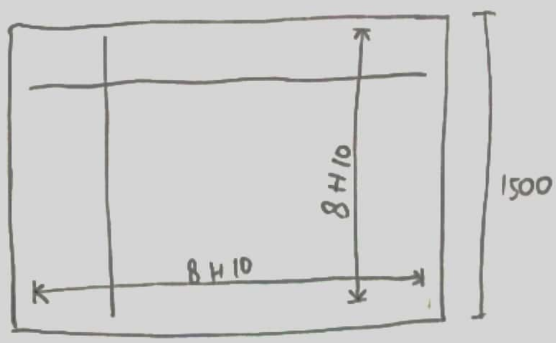
max allowable bar spacing = 200mm

max bar spacing = $(1500 - 2(20) - 10)/8$

$= 182 < 200$ Ok !

Detailing





cover = 20mm

RUHAN PERIYANNAN

No: .... Date: ....

Main staircase

Taking $h = 175mm$
$G = 210mm$
$R = 125mm$

Loading
$G = 1.5\ N/mm^2$
$Qk = 2.5\ N/mm^2$

$f_{ck} = 30$
$f_{yk} = 500$

$C_{nom} = 20mm$
$\phi = 10mm$

Average thickness

$$y = 175 \times \frac{\sqrt{210^2 + 125^2}}{210} = 203.7mm$$

$$t = 203.7 + \frac{125}{2} = 266.16mm$$

Action on stairs.

self weight $= 0.266 \times 25 = 6.65\ kN/m^2$
$Gk = 1.5$
$\Sigma Gk = 8.15$

$Qk = 2.5$

Design Action $= 1.35Gk + 1.5Qk$
$= 14.76\ kN/m^2$

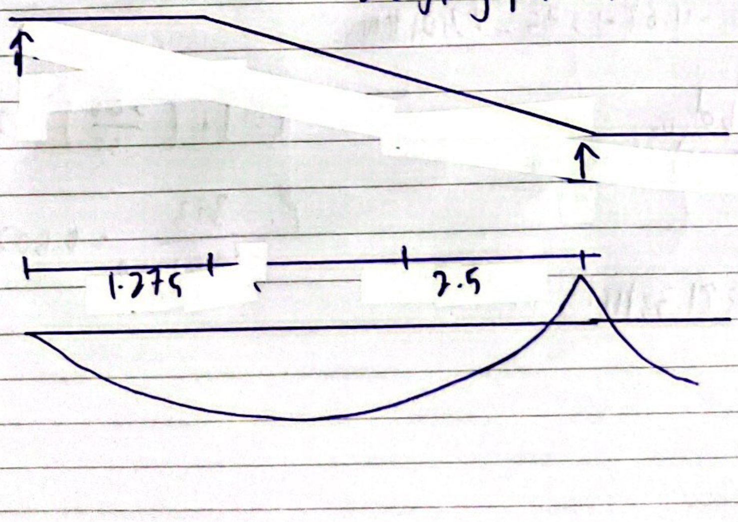
Action on Landing.

self weight $= 0.175 \times 25 = 4.375$
$Gk = 1.5$
$\Sigma Gk = 5.875$

$Qk = 2.5$

Design Action $= 1.35Gk + 1.5Qk$
$= 11.68\ kN/m^2$

Designing per m.



$$M = \frac{FL}{10} = \frac{(14.76 \times 3.775 + 11.68 \times 1.275) \times 3.775}{10}$$
$= 19.55kNm$

No:

$d = 175 - 20 - \frac{10}{2} = 150mm.$

$k = \frac{19.55\times10^6}{1000\times150^2\times30} = 0.029 < 0.167$ ok!

$z = d(0.5 + \sqrt{0.25 - \frac{0.029}{1.134}}) = 0.973d > 0.95d \rightarrow$ Taking 0.95d

$As = \frac{19.54\times10^6}{0.87\times500\times0.95\times150} = 315.39\,mm^2/m.$

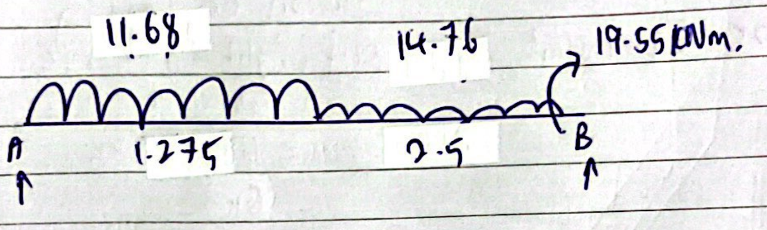
main bar = H10-200 ($393\,mm^2/m$).

$As_{min} = 0.0013\times1000\times150 = 199.68\,mm^2/m$

secondary bar = H10-300 ($262\,mm^2/m$).

$As_{max} = 0.04\times1000\times175 = 7000\,mm^2/m.$

Secondary bar $= 0.2\times315.39 = 63.08\,mm^2/m.$



$\Sigma M_A = 0$

$19.55 + 11.68\times\frac{1.275^2}{2} + 14.76\times2.5\times(\frac{2.5}{2} + 1.275) - 3.775F_B = 0.$

$F_B = 32.37$

$F_A = 32.37 - 14.76\times2.5 - 11.68\times1.275 = 19.42\,kN.$

$V_{Rd,c} = 0.12k(100\rho_1 f_{ck})^{1/3}bd$
$= 0.12\times2\times(100\times0.00262\times30)^{1/3}\times1000\times150$
$= 71.58\,kN.$

$k = 1 + (\frac{200}{150})^{0.5} = 2.15$
$\hookrightarrow$ Take $k = 2$

$\rho_1 = \frac{393}{1000\times150} = 0.00262.$

$V_{min} = 0.035k^{1.5}f_{ck}^{0.5}bd = 81.33\,kN.$ ok!

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Deflection

$\rho = \frac{315.59}{1000 \times 150} = 0.0021$ $\rho_0 = \sqrt{30} \times 10^{-3} = 0.00548.$

End pan $K = 1.3.$

$\frac{l}{d} = 1.3 \left(11 + 1.5\sqrt{30} \frac{0.0054}{0.0021} + 3.2\sqrt{30} \left(\frac{0.0054}{0.0021} - 1 \right)^{1.5} \right)$

$= 110.72$

$\frac{l}{d}_{actual} = \frac{3775}{150} = 25.17$, $L < 7m$, no need to modify for $\frac{As_{prov}}{As_{req}}$

Cracking

$h < 200$ ok!

main bar.

$3(175) = 525 < 400$.

max spacing 400.

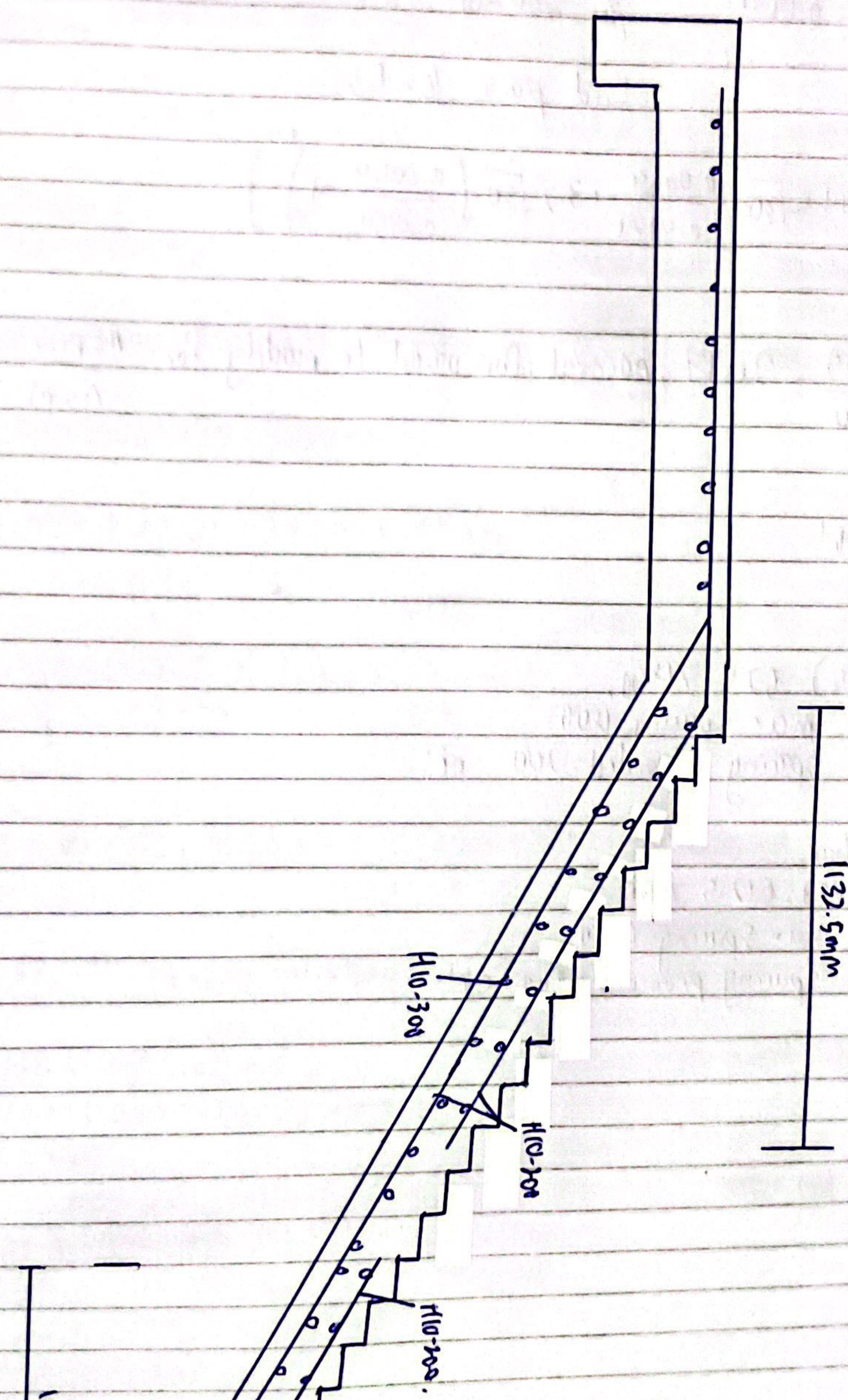
spacing provided = 200 ok!

Secondary bar.

$3.5(175) = 612.5 > 450$

max Spacing 450.

spacing provided = 300 ok!


H10-300
H10-300
H10-300.
132.5mm
132.5mm

No: .... Date: ....

Column. B/5

Top floor.

Column
$b = 400$
$h = 400$
$I = \frac{bh^3}{12} = 2.13\times10^9$
$L = 2500mm$
$k = \frac{I}{L} = 8.5\times10^5$

Beam
$b = 250$
$h = 500$
$I = 2.6\times10^9$
$L = 4313$
$k = 6.04\times10^5$
$0.5k = 3.02\times10^5$

Taking loading on beam = 10 kN.
$w = 10 kN/m.$
$FEM = 10\times\frac{4.313^2}{12} = 15.5 kNm.$
$moment = 15.5\times\frac{8.5\times10^5}{8.5\times10^5+3.02\times10^5} = 11.45 kNm.$
$N_{Ed} = 10\times\frac{4.313}{2} = 21.57 kN.$

Column
$L = 2500$
$k = 8.5\times10^5$

Beam.
$L = 4850$
$I = 2.6\times10^9$
$k = 5.36\times10^5$
$0.5k = 2.68\times10^5$

$w = 10 kN/m.$
$FEM = 19.6 kNm.$
$moment = 14.91$
$N_{Ed} = 24.25 kN.$

2nd floor.

Slab 1. $\frac{L_y}{L_x} = 1.118 \rightarrow 1.2.$

Gk
Self weight $= 0.15\times25 = 3.75$
Gk = 1.5
$\Sigma Gk = 5.25.$ Qk = 1.5.
$w = 1.35(5.25)+1.5(1.5) = 15.95 kN/m^2.$
$\approx 16 kN.$
V on 5.05m $= \beta_n l_x = 16\times0.31\times4.513 = 22.38$
V on 4.513 m $= 16\times4.513\times0.26 = 18.77 kN/m.$
Beam self weight $= (0.5-0.15)\times0.25\times25 = 2.188.$
$1.35(2.188) = 2.95.$
V on 5.05 = 25.33 kN/m V on 4.513 = 21.72 kN/m

2^{nd} floor.

$\bar{w} = 21.72$ kN/m
FEM = 36.86 kNm.

Column top	column bottom	Beam
L = 2500	L = 2500	L = 4513
$k = 8.5 \times 10^5$	$k = 8.5 \times 10^5$	$k = 5.77 \times 10^5$
$I = 2.13 \times 10^9$	$I = 2.13 \times 10^9$	$0.5k = 2.88 \times 10^5$

moment Top: $\frac{8.5 \times 10^5}{8.5 \times 10^5 + 8.5 \times 10^5 + 2.88 \times 10^5} \times 36.86 = 15.77$ kNm.

moment bottom = 15.77 kNm.

$N_{Ed} = 49.01$ kN.

Column Top	column bottom	Beam
L = 2500	L = 2500	L = 5050
$k = 8.5 \times 10^5$	$k = 8.5 \times 10^5$	$k = 5.16 \times 10^5$
$I = 2.13 \times 10^9$	$I = 2.13 \times 10^9$	$0.5k = 2.57 \times 10^5$

w = 25.33
FEM = 53.83 kNm.

moment top = 23.38 kNm

$N_{Ed} = 63.96$ kN.

moment bottom = 23.38 kNm.

Base floor.

Slab 1. $\frac{L_y}{L_x} = 1.44 \approx 1.5$.

Gk

self weight = $0.15 \times 25 = 3.75$ Qk = 1.5

Gk : 1.5

$w = 1.35 G_k + 1.5 Q_k = 15.95 \approx 16$.

V on 7000 = $16 \times 4.85 \times 0.35 = 27.16$

V on 4850 = $16 \times 4.85 \times 0.26 = 20.176$.

Beam self weight = 2.95.

V on 7000 = 30.11 kN/m,   V on 4850 = 23.126 kN/m.

No: .... Date: ....

Base floor.

Column top
$L = 2500$
$k = 8.9 \times 10^5$
$I = 2.13 \times 10^9$

Column bottom
$L = 1225$
$k = 1.74 \times 10^5$
$I = 2.13 \times 10^9$

Beam.
$L = 7000$
$k = 3.72 \times 10^5$
$0.5k = 1.86 \times 10^5$

$W = 27.16$ kN/m.

FEm = 110.9 kNm.

moment Top = 34.03 kNm.
moment bottom = 69.45 kNm.
$N_{Ed} = 95.06$ kN

Base floor.

Column Top
$L = 2500$
$k = 8.9 \times 10^5$
$I = 2.13 \times 10^9$

Column bottom
$L = 1225$
$k = 1.74 \times 10^5$
$I = 2.13 \times 10^9$

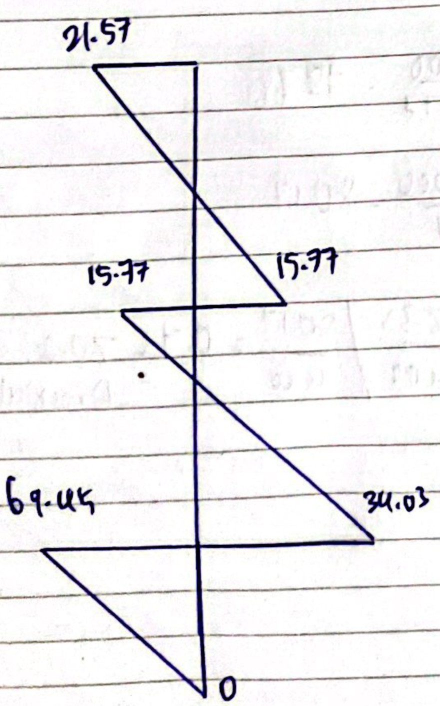
Beam
$L = 4850$
$k = 5.37 \times 10^5$
$0.5k = 2.68 \times 10^5$

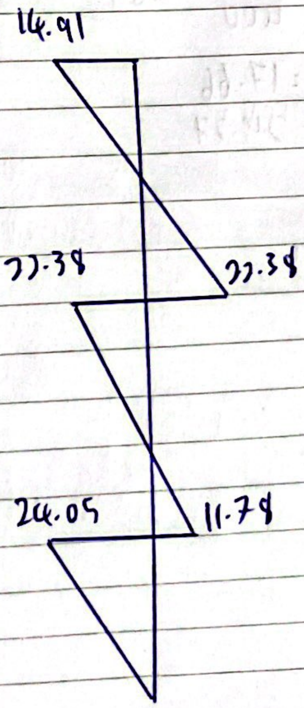
$W = 20.176$ kN/m

FEm = 39.55 kNm.

moment Top = 11.79 kNm.
moment bottom = 24.05 kN/m.
$N_{Ed} = 48.93$ kN.

Total Axial = 302.77 kN.





Date:

No:

Designing for base floor to 2^{nd} floor.

$L = 3000$

$L_{actual} = 3000 - 500 = 2500.$

Both fixed end.

$L_o = 2500 \times 0.75 = 1875mm.$

$M_{02} = 34.03$

$M_{01} = -15.76.$

$A = 0.7$

$B = 1.1$

$C = 1.4 - \left(\frac{-15.76}{34.03}\right) = 1.86.$

$$n = \frac{302.77 \times 10^3}{400^2 \times \frac{0.85 \times 30}{1.5}} = 0.11$$

$$\lambda_{lim} = 20 \times 0.7 \times 1.1 \times \frac{1.86}{\sqrt{0.11}} = 85.99.$$

$M_{02} = 22.38$

$M_{01} = -11.78$

$A = 0.7$

$B = 1.1$

$C = 1.4 - \left(\frac{-11.78}{22.38}\right) = 1.93.$

$\lambda_{lim} = 88.92.$

$$\lambda_{actual} = \frac{1875}{\sqrt{\frac{2.13 \times 10^9}{400^2}}} = 16.24$$ Non slender. Column.

Since bending on both axis obtained, Biaxial check.

$$M_{imp} = 302.77 \times \frac{2.5}{400} = 1.89.$$

$M_y = 15.76 + 1.89 = 17.66$

$M_z = 22.38 + 1.89 = 24.27$

$$e_y = \frac{17.66 \times 1000}{302.77} = 58.33$$

$$e_z = \frac{24.27 \times 1000}{302.77} = 80.17.$$

$$\frac{e_y}{h} \Big/ \frac{e_z}{b} = \frac{58.33}{400} \Big/ \frac{80.17}{400} = 0.72 > 0.2$$

Biaxial considered.

Scanned with CamScanner

No: ....

Date: ....

$C_{nom} = 20mm.$
$\phi = 16$
$\phi = 8.$

$h' = 400 - 20 - 8 - \frac{16}{2} = 364$
$b' = 364.$

$d_2 = 20 + \frac{16}{2} + 8 = 36.$

$\frac{M_z}{h'} = \frac{24.27 \times 10^6}{364} = 6.67 \times 10^4$ $\frac{M_y}{b'} = 4.85 \times 10^4$ $d_2/h = 0.09 \approx 0.1$

$\beta = 1 - \frac{302.77 \times 1000}{400^2 \times 30} = 0.936.$

$M'_z = 24.27 \times 10^6 + 0.936 \times 17.66 = 40.82 kNm.$

using chart,

$N/bh f_{cu} = \frac{302.77 \times 10^3}{400^2 \times 30} = 0.063$

$M/bh^2 f_{cu} = \frac{40.82 \times 10^6}{400^3 \times 30} = 0.021$

From chart, 0. → Taking 0.05

$As_{req} = \frac{0.05 \times 400^2 \times 30}{500} = 480 mm^2$ 4H16 ($804 mm^2$).

$As_{min} = 0.1 \times \frac{302.77 \times 10^3}{0.87 \times 500} = 69.6 mm^2$

$As_{max} = 0.04 \times 400^2 = 6400 mm^2.$

As_x & $As_y = 804 mm^2$
$\frac{d_{2x}}{h}$ & $\frac{d_{2y}}{b} = \frac{36}{400} = 0.09.$
$\frac{As f_{yk}}{bh f_{ck}} = 0.084$
$\frac{N}{bh f_{cu}} = 0.063.$
$\frac{M}{bh^2 f_{ck}} = 0.58$

$N_{Rd} = 0.567 \times 30 \times 400^2 + 0.87 \times 500 \times 804 = 3071 kN$
$N_{Ed}/N_{Rd} = 302.77 / 3071 = 0.0986.$
$\alpha = 1.$

Bending check

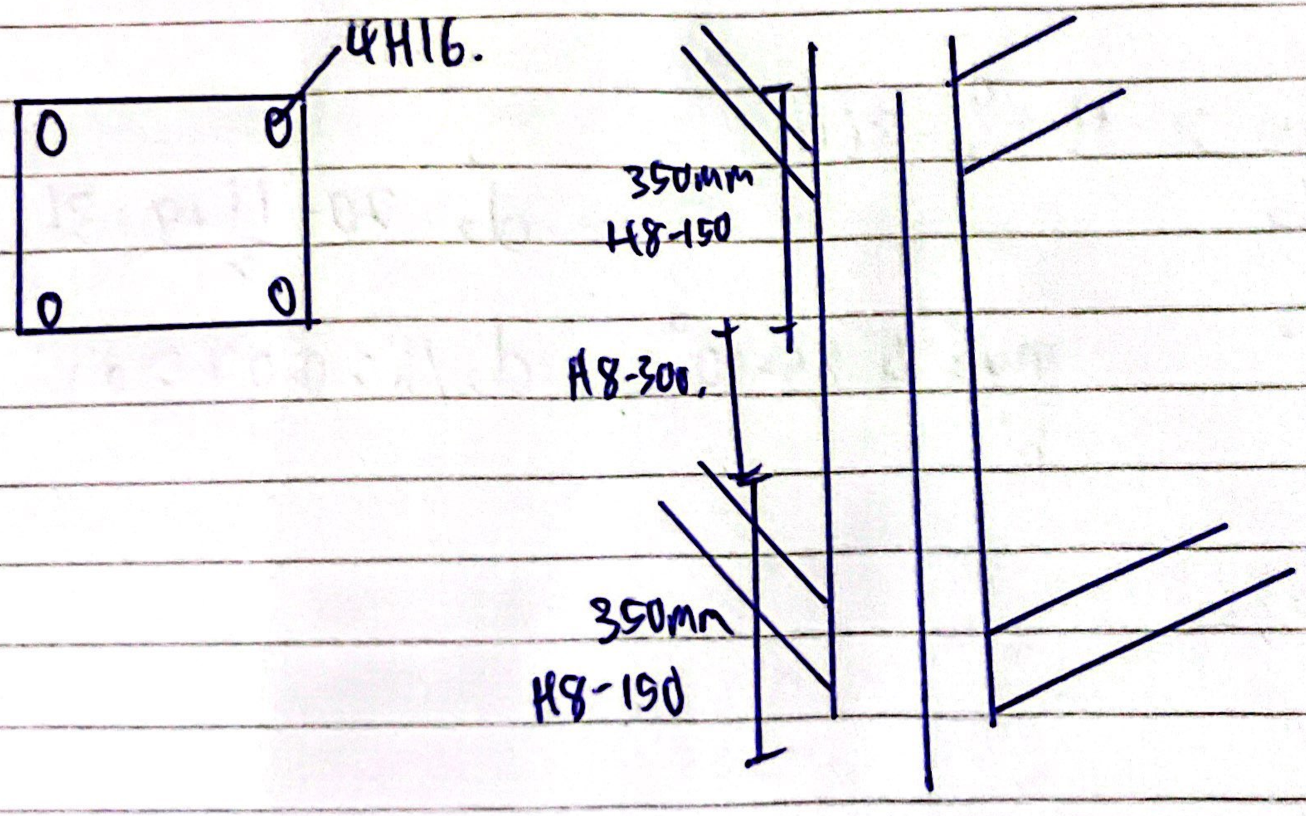
$\left(\frac{24.27}{111.36}\right)^1 + \left(\frac{17.66}{111.36}\right)^1 = 0.377 < 1$ ok!

$M_{resistance} = 111.36 kNm.$

Link $= 0.25(16) = 4$
↳ 6.
provided 8 ok!

spacing = 400
→ 400
$\geq 20 \times 16 = 320.$

near Joint $> 0.6(300) = 180.$
H8-300
H8-150 @ 350mm from


4H16.
350mm
H8-150
H8-300.
350mm
H8-150

No: Date:

Foundation

$N_{ult} = 302.77kN$ $\sigma = 150 N/mm^2$ (Loose gravel).
$N = 216.27kN$. $C_{nom} = 40$.
$\varnothing = 8$.

Dimensions
$B = 1.3m$
$D = 1.3m$
$h = 0.25m$.

Footing weight $= 1.3 \times 1.3 \times 0.25 \times 25 = 10.56kN$.

Area req $= \frac{216.27 + 10.56}{150} = 1.51m^2$

$d = 250 - 40 - 1.5(8) = 198mm$.

Area provided $= 1.69m^2$

At ULS,

$\sigma = \frac{302.77}{1.69} = 179.15$

moment at face: $179.5 \times \frac{0.45^2}{2} = 18.14kNm$.

$K = \frac{18.14 \times 10^6}{1000 \times 198^2 \times 30} = 0.012 < 0.167$.

$z = d(0.5 + \sqrt{0.25 - \frac{0.012}{1.134}}) = 0.989d > 0.95d$.

$A_s = \frac{18.14 \times 10^6}{0.87 \times 500 \times 0.95 \times 198} = 221.67 mm^2/m$.

$A_{s,min} = 0.0013 \times 1000 \times 198 = 334.62 mm^2/m$ H8-125 ($402 mm^2/m$).
$A_{s,max} = 0.04 \times 1000 \times 250 = 10000 mm^2/m$.

Verticle shear.

$V_{Ed} = 179.15 \times (\frac{450-198}{1000}) = 45.15kN$.

$k = 1 + \sqrt{\frac{200}{198}} = 2.005 \rightarrow 2$.

$450 - 40 < 40(8) + 198$ $A_{sl} = 402$.

$\rho_1 = \frac{402}{1000 \times 198} = 2.03 \times 10^{-3}$

$V_{Rd,c} = 0.12 \times 2(100 \times 2.03 \times 10^{-3} \times 30)^{1/3} 1000 \times 198 = 144kN$.

$V_{min} = 0.035 \times 2^{1.5} \times 30^{0.5} \times 1000 \times 198 = 19.6kN$. ok!

Scanned with CamScanner

Punching shear.

$2d = 396mm$

perimeter $= 2\pi(396) + 4\times400 = 4088$ mm
Area $= 400^2 + \pi(396)^2 + 4(400)(396) = 1296252\,mm^2$.

$400 + d = 518$.
$L = 252$ Reinforcement doesnt help.

$V_{min} = 0.035 \times 2^{1.5} \times 30^{0.5} \times 4088 \times 198 = 438\,kN$.

Punching shear $= 179.15 \times (1.69 - 1.286) = 72.33\,kN$ ok!

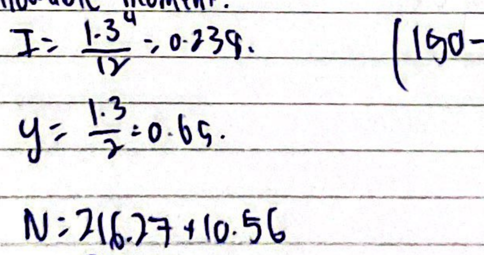
Shear at column perimeter.

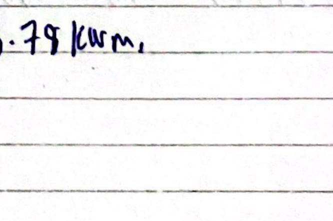
$V = 0.5 \times 4088 \times 198(0.6(1 - \frac{30}{250})(30/1.5) = 4273\,kN$.
Axial force $= 302\,kN$.

Cracking
$h = 250mm$.
$f_s = 0.6 \times \frac{384}{402} \times \frac{500}{1.15} = 313.39 \rightarrow$ Taking 320.
max bar size 10 → ok!
max spacing = 100 mm
no of bar req $= \frac{(1300 - 2(40) - 8)}{100} = 12.12$
≈ 13 bars.

13H8-100

Detailing





Allowable moment.
$I = \frac{1.3^4}{12} = 0.238$.

$y = \frac{1.3}{2} = 0.65$.

$N = 216.27 + 10.56$
$= 226.83\,kN$.

$\left(190 - \frac{226.83}{1.69}\right)\frac{0.238}{0.65} = 5.78\,kNm$.

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RCD Project

Staircase

Taking main staircase A6-C7

Data

$h = 175$, $G = 210$, $R = 125$

$g_k = 1.5$, $q_k = 2.5$

$f_{ck} = 30$, $f_{yk} = 500$, $\gamma_c = 25$

$c = 20$, $\phi = 10$

Average Thickness

$$y = h\left(\frac{\sqrt{G^2+R^2}}{G}\right)$$

$$= 175\left(\frac{\sqrt{210^2+125^2}}{210}\right)$$

$$= 203.66\text{mm}$$

$$t = \frac{2(203.66)+125}{2} = 266.16\text{mm}$$

Action

Slab SW $= 25 \times 0.266 = 6.65$

g_k $g_k = 1.5$

$G_k = 8.15$

$q_k = 2.5$

$Q_k = 2.5$

$w = 1.35(8.15) + 1.5(2.5) = 14.76\,\text{kNm}^{-2}$

Landing SW $= 25 \times 0.175 = 4.375$

$g_k = 1.5$

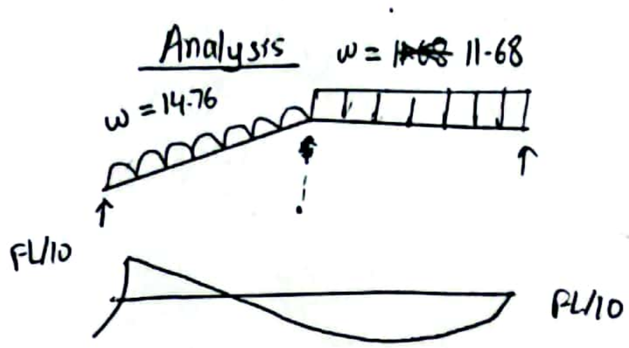
5.875

$q_k = 2.5$

$Q_k = 2.5$

$w = 1.35(5.875) + 1.5(2.5) = 11.68\,\text{kNm}^{-2}$

Analysis



$L = 2.5 + 1.275 = 3.775$

$F = (14.76 \times 2.5) + (11.68 \times 1.275) = 51.79$

$FL/10 = (51.79 \times 3.775)/10 = 19.55\text{ kNm}$

Main Reinforcement

$d = 175 - 20 - 0.5(10) = 150$

$$k = \frac{19.55 \times 10^6}{30 \times 1000 \times 150^2} = 0.029 < 0.167$$

No compression reinforcement needed

$$z = d\left[0.5 + \sqrt{0.25 - \frac{0.029}{1.134}}\right] = 0.97d$$

$$A_s = \frac{19.55 \times 10^6}{0.87 \times 500 \times 0.97 \times 150} = 315.39\,\text{mm}^2/\text{m}$$

$$A_{s\,min} = 0.26\left(\frac{2.56}{500}\right) \times 1000 \times 150 = 200\,\text{mm}^2/\text{m}$$

$$A_{s\,max} = 0.04 \times 1000 \times 150 = 6000\,\text{mm}^2/\text{m}$$

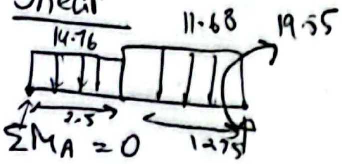
Provide H10-100 ($786\text{mm}^2/\text{m}$)

Secondary Reinforcement

$0.2A_s = 0.2 \times 315.39 = 63.07\,\text{mm}^2/\text{m}$

Provide H10-400 ($196.5\text{mm}^2/\text{m}$)

Shear



$\Sigma M_A = 0$

$$14.76\frac{(2.5)^2}{2} + 11.68\left(2.5+\frac{1.275}{2}\right) + 19.55 = M_B(2.5+1.275)$$

$M_B = 29.77\,kN = V_{Ed}$

$M_A = (14.76\times2.5) + (11.68\times1.275) - 29.77$

$= 22.02\,kN$

$$V_{Rdc} = 1000\times150\times\left(0.12\times2\times(100\times0.00524\times30)^{1/3}\right)/1000$$

$= 90.18\,kN$

$$k = 1+\sqrt{\frac{200}{150}} = 2.1572,\ \text{take } k=2$$

$$P_1 = \frac{786}{1000\times150} = 0.00524$$

$$P = \frac{315.39}{1000\times150} = 0.00210$$

$P_0 = \sqrt{30}/1000 = 0.005477 > P$

$$l/d = 1.3\left[11+1.5\sqrt{30}\times\frac{548}{210} + 3.2\sqrt{30}\left(\frac{548}{210}-1\right)^{3/2}\right]$$

$= 88.45$

$$l/d_{\text{mod allow}} = 88.45\times1\times\left(\frac{315.39}{786}\right)^{-1} = 132.68$$

$$l/d_{actual} = \frac{3775}{150} = 25.17 \quad (\text{ok!})$$

Cracking : $h = 175 < 200$

Main bar : $3h = 525 > 400$

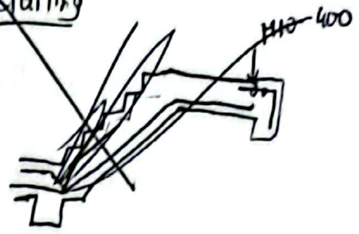
Max spacing = 100 < 400 (ok!)

2nd bar : $3.5h = 612.5 > 450$

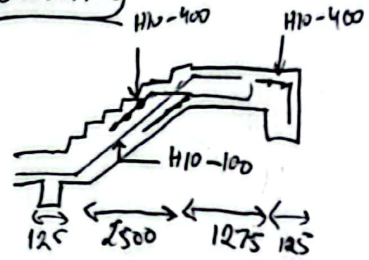
Max spacing = 450

Max bar spacing = 400 < 450 (ok!)

Detailing



Detailing



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Column

Taking Column A/B Level 1

Dimension & Size

Column:

$b \times h = 400 \times 400mm$

$I = \frac{400 \times 400^3}{12} = 2.13 \times 10^9$

Main Beam:

$b \times h = 250 \times 500 mm$

$I = \frac{250 \times 500^3}{12} = 2.6 \times 10^9$

$L_1 = 2650mm$

$L_2 = 1275mm$

Stiffness

Column: $k_{zz} = \frac{2.13 \times 10^9}{2,500} = 8.52 \times 10^5$

Beam: $k_{mb_1} = \frac{2.6 \times 10^9}{2650} = 9.83 \times 10^5$

$k_{mb_2} = \frac{2.6 \times 10^9}{1275} = 20.39 \times 10^5$

Radius of Gyration

$i_{zz} = \sqrt{\frac{\cancel{8}2.13 \times 10^9}{400 \times 400}} = 115.38$

Effective Length

$l_{oy} = L_{oz} = 0.75(2500 - \cancel{400})$
$= 1875$

$\lambda_{zz} = \lambda_{yy} = \frac{1875}{115.38} = 16.25$

Slenderness Limit

$A = 0.7,\ B = 1.1,\ C = \cancel{2.7}\ \cancel{1.53}\ 2.41/2.47$

$n = \frac{31.89 \times 10^3 \times 1.5}{(400 \times 400) \times 0.85 \times 30} = \cancel{11.72}\ 0.012$

$\lambda = \frac{20 \times 0.7 \times 1.1 \times 2.41}{\sqrt{0.012}} = \cancel{10.84}\ 339$

$\lambda = \frac{20 \times 0.7 \times 1.1 \times 2.47}{\cancel{\sqrt{11.72}}\sqrt{0.012}} = \cancel{11.11}\ 347$

Both non slender

$r_{main} = \frac{-0.5}{0.7} = -0.71,\ C = 2.41$

$r_{sec} = \frac{-0.1}{0.13} = -0.77,\ C = 2.47$

Axial Calculation

FEM

2nd Floor

$SW = 25 \times 0.25 \times 0.5 = 3.125 kN/m$

$FEM_{sb} = 3.125 \times 1.275^2/12 = 0.423 kNm$

$FEM_{mb} = 3.125 \times 2.65^2/12 = 1.828 kNm$

For main beam

$M_{top} = \frac{1.83 \times 8.52}{8.53 + 8.53 + \frac{9.83}{2}} = 0.71 kNm$

$M_{bottom} = \frac{1.83 \times 8.53}{8.53 + 8.53 + \frac{9.83}{2}} = 0.71 kNm$

$N_{Ed} = \frac{3.125 \times 2.65}{2} = 4.141$

For secondary beam

$M_{top} = \frac{0.42 \times 8.53}{8.53 + 8.53 + \frac{20.39}{2}} = 0.13 kNm$

$M_{bottom} = \frac{0.42 \times 8.53}{8.53 + 8.53 + \frac{20.39}{2}} = 0.13 kNm$

$N_{Ed} = \frac{3.125 \times 1.275}{2} = 1.99 kN$

~~$r_m = -1,\ C = 1.7 - (-1) = 2.7$~~

Roof

Assume roof load = 10kN/m

$FEM_{sb} = 10 \times 1.275^2/12 = 1.355kN$

$FEM_{mb} = 10 \times 2.65^2/12 = 5.852kN$

For main beam

$M_{top} = \frac{5.85 \times 8.53}{8.53 + \frac{9.83}{2}} = 3.71 kNm$

$N_{Ed} = \frac{10 \times 2.65}{2} = 13.25kN$

~~For secondary beam~~

~~$M_{bot} = 5.85 \times 1.275 \times 10 / 2 =$~~

Secondary Beam

$$M_{bot} = \frac{1.35 \times 8.53}{8.53 + \frac{20.4}{2}} = 0.62\,kNm$$

$$N_{Ed} = \frac{10 \times 1.275}{2} = 6.375\,kN$$

Grand Floor

$w = SW = 25 \times 0.25 \times 0.5 = 3.125\,kNm^{-1}$

$FEM_{slab} = 3.125 \times 1.275^2/12 = \cancel{1.83}\ 0.42\,kNm^{-1}$

$FEM_{mb} = 3.125 \times 2.65^2/12 = 1.83\,kNm^{-1}$

As w and FEM are same as 2nd floor, reuse same N_{Ed}

For main beam

$$M_{top} = \frac{1.83 \times 8.53}{8.53 + 17.41 + \frac{9.83}{2}} = 0.51\,kNm$$

$$M_{bot} = \frac{1.83 \times 17.41}{8.53 + 17.41 + \frac{9.83}{2}} = 1.03\,kNm$$

$N_{Ed} = 4.14\,kN$

For secondary beam

$$M_{top} = \frac{0.42 \times 8.53}{8.53 + 17.41 + \frac{20.4}{2}} = 0.10\,kNm$$

$$M_{bot} = \frac{0.42 \times 17.41}{8.53 + 17.41 + \frac{20.4}{2}} = 0.20\,kNm$$

$N_{Ed} = 1.99\,kN$

$\Sigma N_{Ed} = 31.89\,kN$

Design Moment

$$M_{imp} = N_{Ed}\left(\frac{l_0}{400}\right)$$

$$= 31.89 \times \frac{2.5}{400} = 0.2\,kNm$$

$M_{ey} = 0.51 + 0.2 = 0.71$

$M_{ez} = 0.71 + 0.2 = 0.91$

Biaxial check

$$e_z = \frac{0.91 \times 10^6}{31.89 \times 10^3} = 29\,mm$$

$$e_y = \frac{0.71 \times 10^6}{31.89 \times 10^3} = 22\,mm$$

$$\left(\frac{e_y}{h}\right)\Big/\left(\frac{e_z}{b}\right) = \left(\frac{22}{400}\right)\Big/\left(\frac{29}{400}\right) = 0.76 > 0.2$$

$$\left(\frac{e_z}{b}\right)\Big/\left(\frac{e_y}{h}\right) = 0.76^{-1} = 1.32 > 0.2$$

Check biaxial bending

$\lambda_y/\lambda_z = 1 < 2$ ignore biaxial bending

$\therefore$ Check biaxial bending

Reinforcement Design

$c_{nom} = 20$, $\varnothing_{bar} = 16$, $\varnothing_{link} = 8$

$h' = 400 - 20 - 8 - \frac{16}{2} = 364$

$b' = 400 - 20 - 8 - \frac{16}{2} = 364$

$$\frac{M_{Edz}}{h'} = \frac{0.91 \times 10^6}{364} = 2.5\,kN$$

$$\frac{M_{Edy}}{h'} = \frac{0.71 \times 10^6}{364} = 1.95\,kN$$

$$\frac{M_{Edz}}{h'} > \frac{M_{Edy}}{h'} \therefore M'_{Edz} = M_{Edz} + B\left(\frac{h'}{b'}\right)M_{Edy}$$

$$\frac{N_{Ed}}{bhf_{ck}} = \frac{31.89 \times 10^3}{400 \times 400 \times 30} = 6.64 \times 10^{-3}$$

$B = 0.99$

$$M'_{Edz} = 0.91 + 0.99\left(\frac{364}{364}\right)0.71 = 1.61\,kNm$$

$d_2 = 20 + 8 + \frac{16}{2} = 36$

$d_2/h = 36/400 = 0.09$

$\frac{M}{bh^2 f_{ck}} = \frac{1.61 \times 10^6}{400 \times 400^2 \times 30} = 8.39 \times 10^{-4} \simeq 0$

$\frac{A_s f_{yk}}{bh f_{ck}} = 0$, $A_s = 0$

$A_{s,min} = \frac{0.1 \times 31.89 \times 10^3}{0.87 \times 500} = 7.33 mm^2$

$0.002 \times 400^2 = 320 mm^2$

$A_{s,max} = 0.04 \times 400^2 = 6400 mm^2$

Provide ~~2~~4H16 + ~~2H16~~ (~~A_s = 803 mm²~~ 402) $A_s = 804 mm^2$

Link

✓ $0.25 \times$ ~~16~~ $= \cancel{6.3}$ 4

✓ 6

Use 8

S_{max}

✓ $20 \times 16 = 320$

· 400

· 400

Use 320

Provide H8-300

and H8-175

as $S_{max} = 0.6 \times 300 = 180 mm$

Check Biaxial Bending

Steel Area:

All : 4H16 + 0H20 = 804

z-z : 4H16 + 0H20 = 804

y-y : 4H16 + 0H20 = 804

$\frac{A_s f_{yk}}{bh f_{ck}} = \frac{804 \times 500}{400 \times 400 \times 30} = 0.08$

$\therefore \frac{M_{Rdz}}{bh^2 f_{ck}} = 0.03$

$M_{Rd z} = 57.6 kNm$

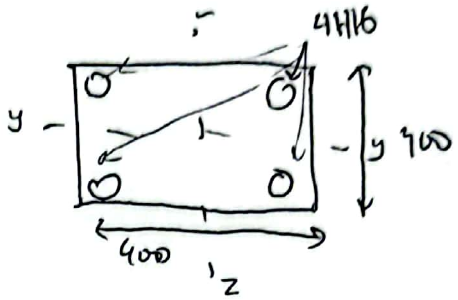
$N_{Rd} = (0.567 \times 25 \times 400^2) + (0.87 \times 500 \times 804)$

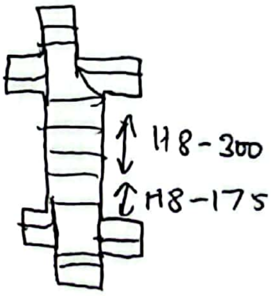
$= 2618 kN$

$\frac{N_{Ed}}{N_{Rd}} = \frac{31.89}{2618} = 0.012$, $K = 1.00$

$\left(\frac{0.91}{57.6}\right)^1 + \left(\frac{0.51}{57.6}\right)^1 = 0.02 < 1$

(pass)





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Pad Footing

$N_{Ed} = 31.89\,kN$, $\gamma = 150$

$N = \frac{31.89}{1.4} = 22.77\,kN$

Area of footing required $= \frac{1.1(22.77)}{150} = 0.17\,m^2$

Try $1.3 \times 1.3 \times 0.25$

Area $= 1.3 \times 1.3 = 1.69\,m^2$ (ok!)

SW $= 1.69 \times 0.25 \times 25 = 10.56\,kN$

SSBC Check

$\frac{22.77 + 10.56}{1.69}$ ~~$= 19.72 < 150$ (ok!)~~

$= \frac{33.33}{1.69} = 19.72 < 150$ (ok!)

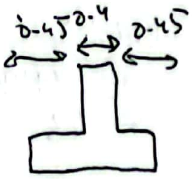
Analysis

$N_{Ed} = 31.89\,kN$

$P = \frac{31.89}{1.69} = 18.87\,kNm^{-2}$

$w = 18.87 \times 1.3 = 24.5\,kNm^{-1}$

$M_{Ed} = 24.5 \times \frac{0.45^2}{2} = 2.48\,kNm$



$d = \cancel{400}\,250 - 20 - 1.5(8)$

$= 218\,mm$

$k = \frac{2.48 \times 10^6}{30 \times 1300 \times 218^2} = 1.34 \times 10^{-3} < 0.167$

No compressive reinforcement needed

$z = d\left[0.5 + \sqrt{0.25 - \frac{1.34 \times 10^{-3}}{1.134}}\right] = 1.00 > 0.95d$

$As_{req} = \frac{2.48 \times 10^6}{0.87 \times 500 \times 0.95 \times 218} = 27.5\,mm^2$

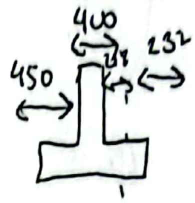
$As_{min} = 0.013 \times 1300 \times 218 = 368.42$

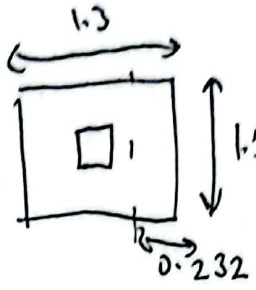
- $As_{max} = 0.04 \times 1300 \times 250$
$= 13000\,mm^2$

Take $As_{min} = 368.42\,mm^2$

Provide 8H8 $- 402\,mm^2$

i) Vertical Shear





$V_{Ed} = 24.5 \times 0.232 = 5.68\,kN$

$k = 1 + \sqrt{\frac{200}{218}} = 1.96 < 2.0$

Anchorage

$232 - 20 \qquad 36 \times 8 + 218$

$212 < 506$

$A_{sl} = 0,\ P_l = 0$

$V_{Rd,c} = V_{min} = 0.035\,(1.96)^{3/2}\sqrt{30} \times 1.3 \times 218$

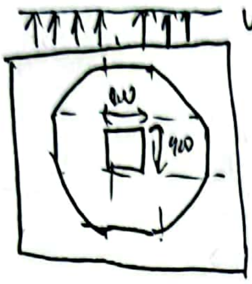
$= 149\,kN > 5.68\,kN$ (ok!)

6

Punching Shear

$d_{avg} = 250 - 30 - 8 = 222\,mm$

$2d = 444\,mm$



$u = (4 \times 400) + 2\pi \times 444$

$= 4390\,mm$

$A = (0.4)^2 + (4 \times 0.4 \times 0.444) + (\pi \times 0.444^2)$

$= 1.49\,m^2$

Anchorage

$450 - 444 < (36 \times 12) + 218$

Reinforcement doesn't contribute to punching shear

Punching Shear Force:

$V_{Ed} = 18.87\,(1.3^2 - 1.49)$

$= 3.77\,kN$

Punching Shear Resistance:

$V_{Rdc} = V_{min} = 0.035\,(1.95)^{3/2}\sqrt{30} \times 4390 \times 222$

$= 508.7 > 3.77$ (ok!)

M allowable

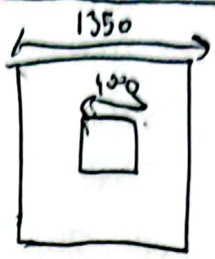
$I = (1.3)^4/12 = 0.238 \times 10^9$

~~Ma = 150~~ $y = 1.3/2 = 0.65$

$M_a = \left(150 - \frac{33.33}{1.69}\right) \times \left(\frac{0.238}{0.65}\right)$

$= 47.7\,kNm$

iii) Maximum Punching Shear at Column Perimeter



1875

$V_{Ed,max} = 18.87\,(1.69 - 0.16) = 28.87\,kN$

$V_{Rd,max} = 0.5\,(1600)(222)\left[0.6\left(1 - \frac{30}{250}\right)\right]\left(\frac{30}{1.5}\right)$

$= 1875\,kN > 28.87$ (ok!)

Cracking

$h = \cancel{450}\ 250 > 200$

$f_s = 0.6\left(\frac{368.42}{402}\right)\left(\frac{500}{1.15}\right) = 239\,N/mm^2$

↳ Take 240

For design crack width 0.3mm:

Max allowable bar spacing = 200mm

Actual spacing = $\frac{|1300 - 2(20) - 8|}{16} = 78 < 100$ ~~18.9~~ (ok!)

Max bar size = 16 > 8 (ok!)

No. of bars = 1300/100 = ~~16.67 ≈ 17~~ ~~13.5~~ 13

Take 13 bars of 8H8-100

Detailing