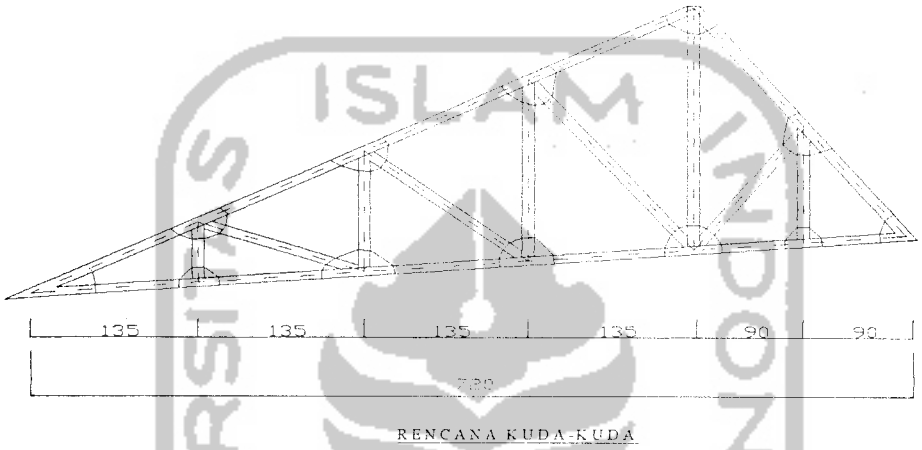
# BAB IV

# PERHITUNGAN STRUKTUR

## 4.1. Perencanaan Kuda – kuda




**Gambar 4.1 rencana kuda – kuda**

### 4.1.1 Perhitungan Gording

Jarak Gording = 1,49 m (kiri) ; 1,27 m (kanan)

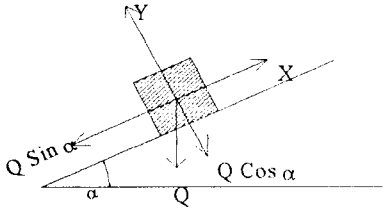
Jarak Kuda – kuda = 3,0 m

Berat sendiri penutup atap = 50 Kg / $m^2$

Beban Hidup ( P ) = 100 Kg

Dicoba Gording ukuran 8/15, Kayu Bengkirai Klas I Mutu A

BD = 0,93 gr/$cm^3$ ; E = 125.000 Kg/$cm^2$



### 4.1.2 Beban yang bekerja pada gording

**1. <u>Beban Angin Kiri ( $\alpha = 24{,}7^o$ )</u>**

Konstruksi dengan beban angin kiri

Koefisien angin ( c ), menurut PPI. 1983

Untuk $\alpha = 24{,}7^o < 65^o$

$C_1 = (0{,}02\alpha - 0{,}4) = 0{,}094$

$C_2 = -0{,}4$

$Q_{angin1} = C_1 . Q . L_1 = 0{,}049.25.1{,}49 = 3{,}5$ kg/m'

$Q_{angin2} = C_2 . Q . L_2 = 0{,}040.25.1{,}27 = 12{,}7$ kg/m'

**2. <u>Beban Angin Kanan ( $\alpha = 45^o$ )</u>**

Konstruksi dengan beban angin kanan

Koefisien angin ( c ), menurut PPI. 1983

Untuk $\alpha = 45^o < 65^o$

$C_1 = (0{,}02\alpha - 0{,}4) = 0{,}50$

$C_2 = -0{,}40$

$Q_{angin1} = C_1 . Q . L_1 = 0{,}50.25.1{,}27 = 15{,}87$ kg/m'

$Q_{angin2} = C_2 . Q . L_2 = 0{,}040.25.1{,}49 = 14{,}90$ kg/m'

**3. Beban Atap**

Digunakan jenis atap " Genteng ", Q = 50 kg/m$^2$ – PPI - 83

Q atap kiri = 50 . 1,49 = 74,50 kg/m'

$Q_y$ = $Q_{atap.}$ Cos α

= 74,5 cos 24,7° = 67,68 kg/m'

$Q_x$ = $Q_{atap.}$ sin α

= 74,5 sin 24,7° = 31,13 kg/m'

Q atap kanan = 50 . 1,27 = 63,50 kg/m'

$Q_y$ = $Q_{atap.}$ Cos α

= 63,5 cos 45° = 44,90 kg/m'

$Q_x$ = $Q_{atap.}$ sin α

= 63,5 sin 45° = 44,90 kg/m'

**4. Beban Pekerja**

Menurut PPI – 1983, beban pekerja / orang ( P ) = 100 kg

$P_{y\ kiri}$ = P cos α

= 100 cos 24,7° = 90,85 kg/m'

$Px_{kiri}$ = P sin α

= 100 sin 24,7° = 41,79 kg/m'

$P_{y\ kanan}$ = P cos α

= 100 cos 45° = 70,71 kg/m'

$P_{x\ kanan}$ = P sin α

= 100 sin 45° = 70,71 kg/m'

**5. Berat sendiri Gording**

$Q_{gording} = \rho \,.\, b \,.\, h$

$= 930 \,.\, 0{,}08 \,.\, 0{,}15 = 11{,}16$ kg

$Q_{y\ kiri} = Q_{gording} \cos \alpha$

$= 11{,}16 \cos 24{,}7° = 10{,}14$ kg

$Q_{x\ kiri} = Q_{gording} \sin \alpha$

$= 11{,}16 \sin 24{,}7° = 4{,}66$ kg

$Q_{y\ kanan} = Q_{gording} \cos \alpha$

$= 11{,}16 \cos 45° = 7{,}89$ kg

$Q_{x\ kanan} = Q_{gording} \sin \alpha$

$= 11{,}16 \sin 45° = 7{,}89$ kg

### 4.1.3 Jenis gording

Gording digunakan kayu Bengkirai Klas I mutu A

Kayu klas kuat I, mutu A, menurut PPKI – 1961

**Untuk beban tetap**

$\overline{\sigma}_{lt} = 150$ kg/cm$^2$

$\overline{\sigma}_{tk\parallel} = \overline{\sigma}_{tr\parallel} = 130$ kg/cm$^2$

$\overline{\sigma}_{tk\perp} = 40$ kg/cm$^2$

$\overline{\tau}_{\parallel} = 20$ kg/cm$^2$

**Untuk beban sementara digandakan dengan faktor pengali 5/4**

$\overline{\sigma}_{lt}$ = 150 . 5/4 = 187,50 kg/cm$^2$

$\overline{\sigma}_{tk//}$ = $\overline{\sigma}_{tr//}$ = 130 . 5/4 = 162,50 kg/cm$^2$

$\overline{\sigma}_{tk\perp}$ = 40 . 5/4 = 50 kg/cm$^2$

$\overline{\tau}_{//}$ = 20 . 5/4 = 25 kg/cm$^2$

Ix = 1/12 . 8 . 15$^3$ = 2250 cm$^4$

Iy = 1/12 . 8$^3$ .15 = 640 cm$^4$

Wx = 1/6 . 8 . 15$^2$ = 300 cm$^3$

Wy = 1/6 . 8$^2$ .15 = 160 cm$^3$

### 4.1.4 Kontrol Terhadap Tegangan

1. tegangan Tetap

$Q_{y\ tetap}$ = $Q_{y\ atap}$ + $Q_{y\ gording}$

= 67,68 + + 10,13 = 77, 82 kg/m' ( Ki )

= 44,90 + 7,89 = 52,79 kg/m' ( Ka )

$Q_{x\ tetap}$ = 31,13 + 4,66 = 35,79 kg/m' ( Ki )

= 44,90 + 7,89 = 52,79 kg/m' ( Ka )

$P_{y\ tetap}$ = $P_{y\ orang}$ = 90,85 kg ( Ki )

= 70,71 kg ( Ka )

$P_{x\ tetap}$ = $P_{x\ orang}$ = 41,79 kg ( Ki )

= 70,71 kg ( Ka )

Momen yang terjadi

$M_{x\ kiri} = 1/8 \cdot Q_{y\ tetap} \cdot L^2 + ¼ \cdot P'_{y\ tetap} \cdot L$

$= 1/8 \cdot 77{,}82 \cdot 3{,}0^2 + ¼ \cdot 90{,}85 \cdot 3{,}0$

$= 87{,}55 + 68{,}14$

$= 155{,}69$ kg.m

$M_{y\ kiri} = 1/8 \cdot Q_{x\ tetap} \cdot L^2 + ¼ \cdot P_{x\ tetap} \cdot L$

$= 1/8 \cdot 35{,}79 \cdot 3{,}0^2 + ¼ \cdot 70{,}71 \cdot 3{,}0$

$= 40{,}26 + 53{,}03$

$= 93{,}29$ kg.m

$\sigma_{lentur} = \frac{Mx}{Wx} + \frac{My}{Wy} < \overline{\sigma}_{lt} = 150$ kg/cm$^2$

$= \frac{155{,}69.10^2}{300} + \frac{93{,}29.10^2}{160}$

$= 51{,}90 + 58{,}31$

$= 110{,}206$ kg/cm$^2$ $< \overline{\sigma}_{lt} = 150$ kg/cm$^2$ **OK!**

2. tegangan Sementara

$Q_{y\ sementara} = Q_{y\ tetap}$

$= 77{,}82$ kg/m'

$Q_{x\ sementara} = Q_{x\ tetap} + Q_{angin}$

$= 35{,}79 + 3{,}50$

$= 39{,}24$ kg/m'

$P_{y\ tetap} = P_{y\ orang} = 90{,}85$ kg ( Ki )

$P_{x\ tetap} = P_{x\ orang} = 41{,}79$ kg ( Ki )

Momen yang terjadi

$M_{x\,kiri} = 1/8 \,.\, Q_{y\,sementara} \,.\, L^2 + ¼ \,.\, P_{y\,tetap} \,.\, L$

$= 1/8 \,.\, 77{,}82 \,.\, 3{,}0^2 + ¼ \,.\, 90{,}85 \,.\, 3{,}0$

$= 87{,}55 + 68{,}14$

$= 155{,}69$ kg.m

$M_{y\,kiri} = 1/8 \,.\, Q_{x\,sementara} \,.\, L^2 + ¼ \,.\, P_{x\,tetap} \,.\, L$

$= 1/8 \,.\, 39{,}24 \,.\, 3{,}0^2 + ¼ \,.\, 70{,}71 \,.\, 3{,}0$

$= 44{,}15 + 53{,}03$

$= 97{,}18$ kg.m

$$\sigma_{lentur} = \frac{Mx}{Wx} + \frac{My}{Wy} < \overline{\sigma}_{lt\,sementara} = 187{,}50 \text{ kg/cm}^2$$

$$= \frac{155{,}69.10^2}{300} + \frac{97{,}18.10^2}{160}$$

$= 51{,}90 + 60{,}73$

$= 112{,}63 \text{ kg/cm}^2 < \overline{\sigma}_{lt\,sementara} = 150 \text{ kg/cm}^2$ **OK!**

### 4.1.5 Kontrol terhadap defleksi akibat beban tetap

$$f = \frac{5}{384} \,.\, \frac{QL^4}{EI} + \frac{1}{48} \,.\, \frac{P \,.\, L^3}{EI} \leq f_{maks} = 1/360 \,.\, L$$

$$f_x = \frac{5}{384} \,.\, \frac{Q_{x\,tetap} \,.\, L^4}{EI_y} + \frac{1}{48} \,.\, \frac{P_{x\,tetap} \,.\, L^3}{EI_y}$$

$$= \frac{5}{384} \,.\, \frac{52{,}79.10^{-2}.\, 300^4}{1{,}25.10^5.\, 640} + \frac{1}{48} \,.\, \frac{70{,}71 \,.\, 300^3}{1{,}25.10^5.\, 640}$$

$= 0{,}03 + 0{,}85$

$= 0{,}87$ cm $< 2{,}0$ cm **OK!**

$f_y = \frac{5}{384} \cdot \frac{Q_{y\,tetap} \cdot L^4}{EI_x} + \frac{1}{48} \cdot \frac{P_{y\,tetap} \cdot L^3}{EI_x}$

$= \frac{5}{384} \cdot \frac{77{,}82.10^{-2} \cdot 300^4}{1{,}25.10^5 \cdot 2250} + \frac{1}{48} \cdot \frac{90{,}85 \cdot 300^3}{1{,}25.10^5 \cdot 2250}$

$= 0{,}61 + 0{,}31$

$= 0{,}92$ cm $< 2{,}0$ cm **OK!**

$f_{xy} = \sqrt{f_x^2 + f_y^2}$

$= \sqrt{0{,}87^2 + 0{,}92^2}$

$= \sqrt{0{,}76 + 0{,}84}$

$= 1{,}26$ cm $< 2{,}0$ cm **OK!**

### 4.1.6 Perhitungan struktur kuda – kuda

Perhitungan struktur kuda – kuda dalam perencanaan ulang/*redesign* ini dihitung dengan menggunakan program SAP 2000.

**Beban – beban yang bekerja pada kuda – kuda**

**1. Beban Tetap**

Terdiri dari

- Beban Atap

  Jenis atap "Genteng Keramik" Q = 50 kg/m$^2$

  $P_{ki} = K_{ki} \cdot Q_{atap} \cdot L$

  $= 1{,}49 \cdot 50 \cdot 3{,}0$

  $= 223{,}5$ kg

  $P_{ka} = K_{ka} \cdot Q_{atap} \cdot L$

  $= 1{,}27 \cdot 50 \cdot 3{,}0$

  $= 190{,}5$ kg

  $P_1 = P_{ki} = 223{,}5$ kg

  $P_2 = \frac{1}{2} P_{ki} + \frac{1}{2} P_{ka}$

  $= \frac{1}{2} \cdot 223{,}5 + \frac{1}{2} \cdot 190{,}5$

  $= 197{,}0$ kg

  $P_3 = P_{ka} = 190{,}5$ kg

- Beban Gording

  $P_{gording} = \rho \cdot b \cdot h \cdot L$

  $= 930 \cdot 0{,}08 \cdot 0{,}15 \cdot 3{,}0$

  $= 33{,}48$ kg

- Muatan akibat beban tetap = Beban atap + beban Gording

  $P_1$ = 223,5 + 33,48 = 256,98 kg

  $P_2$ = 197,0 + 33,48 = 230,48 kg

  $P_3$ = 190,5 + 33,48 = 223,98 kg

- Beban Langit – langit

  Terdiri atas beban penggantung + beban eternit

  Q = ( 7 + 11 ) kg/m$^2$ . 3.0 m

  = 54,0 kg/m

  $P_4$ = ½ . 1,35 . 54,0 = 36,45 kg

  $P_5$ = 1,35 . 54,0 = 72,90 kg

  $P_8$ = ½ . 0,90 . 54,0 = 24,30 kg

  $P_7$ = 0,90 . 54,0 = 48,60 kg

  $P_6$ = ½$P_5$ + ½$P_7$ = 60,75 kg

**2. Beban Hidup**

Beban Hidup = $P_{orang}$ = 100 kg

**3. Beban sementara**

Beban Angin

Muatan angin ( Wu ) = 25 kg/m$^2$

- Angin dari kiri ke kanan

  Untuk $\alpha = 24,7^o$

  $C_1$ = ( 0,02$\alpha$ - 0,4 ) = 0,094

  $C_2$ = - 0,04

Tekan

$Q = Wu \cdot C_1 = 25 \cdot 0{,}094 = 2{,}35 \text{ kg/m}^2$

$P = Q \cdot 1 \cdot L$

$= 2{,}35 \cdot 1{,}49 \cdot 3{,}0 = 10{,}505 \text{ kg}$

$P_1 = 10{,}505 \text{ kg}$

$P_2 = ½ \cdot P_1 = ½ \cdot 10{,}505 = 5{,}253 \text{ kg}$

Hisap

$Q = Wu \cdot C_2 = 25 \cdot 0{,}4 = 10 \text{ kg}$

$P = Q \cdot 1 \cdot L$

$= 10 \cdot 1{,}27 \cdot 3{,}0 = 38{,}10 \text{ kg}$

$P_3 = 38{,}10 \text{ kg}$

$P_2' = ½ \cdot P_3 = ½ \cdot 38{,}10 = 19{,}05 \text{ kg}$

- Angin dari kiri ke kanan

Untuk $\alpha = 45^\circ$

$C_1 = ( 0{,}02\alpha - 0{,}4 ) = 0{,}50$

$C_2 = - 0{,}04$

Tekan

$Q = Wu \cdot C_1 = 25 \cdot 0{,}50 = 12{,}50 \text{ kg/m}^2$

$P = Q \cdot 1 \cdot L$

$= 12{,}50 \cdot 1{,}27 \cdot 3{,}0 = 47{,}625 \text{ kg}$

$P_1 = 47{,}625 \text{ kg}$

$P_2 = ½ \cdot P_1 = ½ \cdot 47{,}625 = 23{,}813 \text{ kg}$

Hisap

$Q = Wu \cdot C_2 = 25 \cdot 0,4 = 10$ kg

$P = Q \cdot 1 \cdot L$

$= 10 \cdot 1,49 \cdot 3,0 = 44,70$ kg

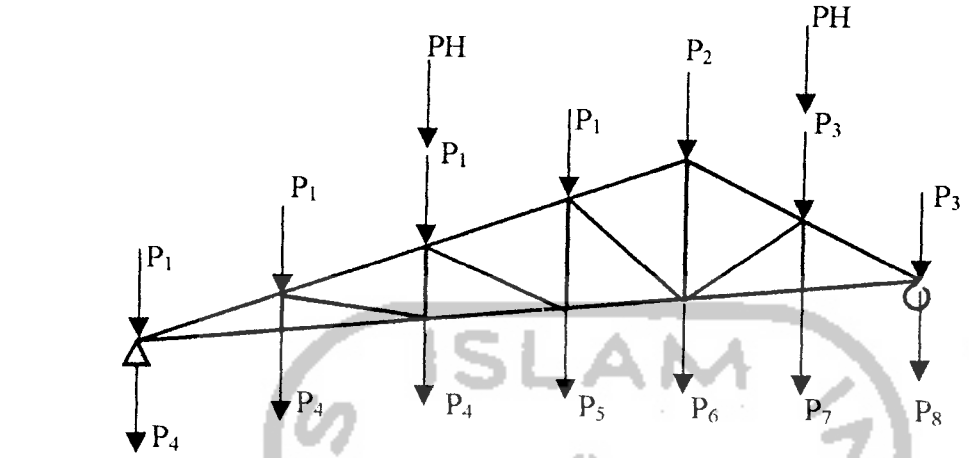
$P_3 = 44,70$ kg

$P_2' = ½ \cdot P_3 = ½ \cdot 44,70 = 22,35$ kg

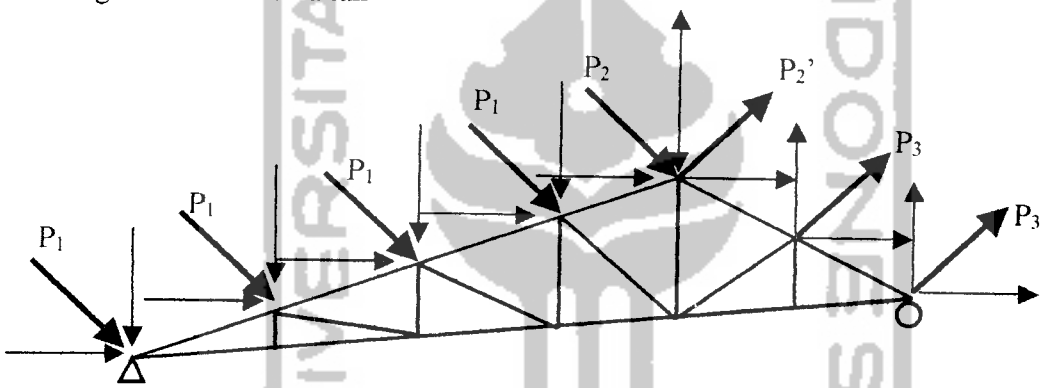
**Tabel 4.1 beban angin untuk setiap joint**

| Angin dari kiri ke kanan | Angin dari kanan ke kiri |
|---|---|
| $P_1\cos\alpha_1 = 10,505\cos 24,7^o = 9,54$ kg | $P_1\cos\alpha_1 = 47,625\cos 24,7^o = 43,26$ kg |
| $P_1\sin\alpha_1 = 10,505\sin 24,7^o = 4,39$ kg | $P_1\sin\alpha_1 = 47,625\sin 24,7^o = 19,90$ kg |
| $P_2\cos\alpha_1 = 5,253\cos 24,7^o = 4,77$ kg | $P_2'\cos\alpha_1 = 22,35\cos 24,7^o = 20,305$ kg |
| $P_2\sin\alpha_1 = 5,253\sin 24,7^o = 2,19$ kg | $P_2'\sin\alpha_1 = 22,35\sin 24,7^o = 9,33$ kg |
| $P_2'\cos\alpha_2 = 19,05\cos 45^o = 13,47$ kg | $P_2\cos\alpha_2 = 23,813\cos 45^o = 16,83$ kg |
| $P_2'\sin\alpha_2 = 19,05\sin 45^o = 13,47$ kg | $P_2\sin\alpha_2 = 23,813\sin 45^o = 16,83$ kg |
| $P_3\cos\alpha_2 = 38,10\cos 45^o = 26,94$ kg | $P_3\cos\alpha_2 = 44,70\cos 45^o = 31,608$ kg |
| $P_3\sin\alpha_2 = 38,10\sin 45^o = 26,94$ kg | $P_3\sin\alpha_2 = 44,70\sin 45^o = 31,608$ kg |

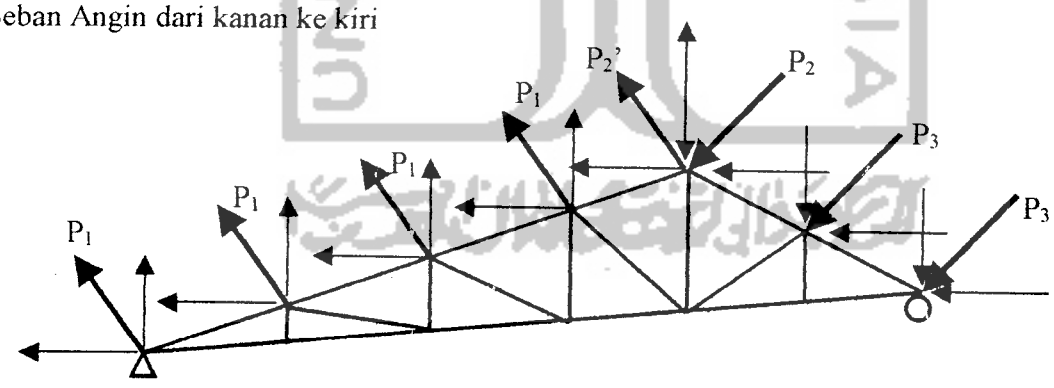
Beban Tetap + beban Hidup




Beban Angin dari kiri ke kanan

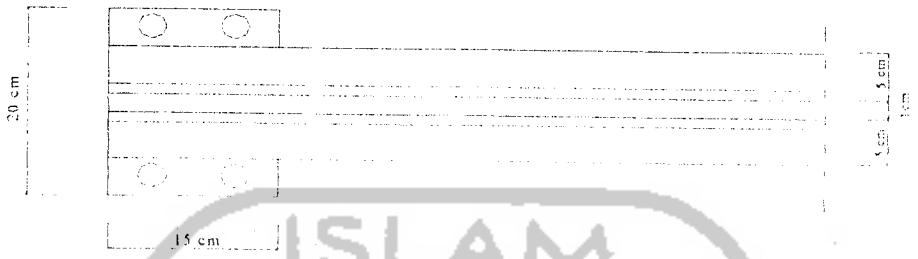



Beban Angin dari kanan ke kiri




Untuk perhitungan struktur tangga selanjutnya menggunakan Program SAP200, data hasil perhitungan dapat dilihat pada lampiran 1.

### 4.1.7 Perencanaan pelat kuda - kuda




PELAT KUDA-KUDA

Beban P diambil dari reaksi dukungan perhitungan SAP2000 :

$P_{maks}$ = 2191,45 kg

F'c = 25 Mpa = 250 kg/cm$^2$

$$A_{perlu} = \frac{P}{0{,}33 f'c} = \frac{2191{,}45}{0{,}33.250} = 26{,}5630 \text{ cm}^2$$

Dipakai ukuran pelat = 15 cm x 20 cm = 300 cm$^2$ > $A_{perlu}$

$$Q = \frac{P}{B \times L} = \frac{2191{,}45}{300} = 7{,}375 \text{ kg/cm}$$

$$X = \frac{20 - (5 + 1 + 5)}{2} = 4{,}5 \text{ cm}$$

$$M = \tfrac{1}{2} . q . x^2 = \tfrac{1}{2} . 7{,}375 . 4{,}5^2 = 74{,}6719 \text{ kgcm}$$

Syarat :

$$0{,}6 Fy = \frac{M}{1/6 . 1 . tp}$$

$$tp = \sqrt{\frac{10 M}{Fy}} = \sqrt{\frac{10 . 74{,}6719}{2531}} = 0{,}5431 \text{ cm} \approx 1 \text{ cm}$$

Pelat kuda – kuda berukuran : 15 x 20 x 1 cm$^3$

### 4.1.8 Perencanaan sambungan

- Tebal pelat sambung = 1 cm
- $D_{baut}$ = ½ " = 1,27 cm
- Mutu baja profil
- Tegangan leleh (fy) = 36 Ksi = 2531 kg/cm$^2$
- Mutu baut A325X ( Non Full Draat )
- Tegangan tarik (ft ) = 44 Ksi = 3093 kg/cm$^2$
- Tegangan geser (Fv) = 30 Ksi = 2109 kg/cm$^2$

Tinjauan tegangan geser 1 baut :

$P_{geser}$ = ¼ . π . $D^2_{baut}$ x Fv x jumlah bidang geser (n)

= ¼ . π . $1{,}27^2$ . 2109 . 2

= 5343,762 kg

Tinjauan tegangan tumpu 1 baut :

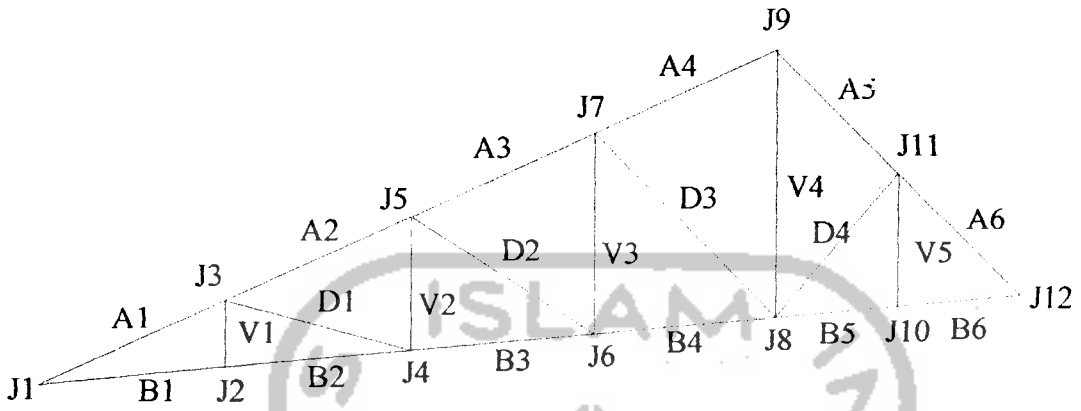
$P_{tumpu}$ = 1,2 . Fu . $D_{baut}$ . t . jumlah tumpuan (n)

= 1,2 . 4077 . 1,27 .1 .1

= 6213 kg

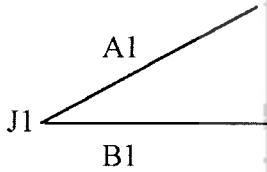
jadi P 1 baut dipakai $P_{geser}$ = 5343 kg

jumlah baut (N) = $\frac{P_{terjadi}}{P_{1\ baut}}$




RANGKA KUDA-KUDA

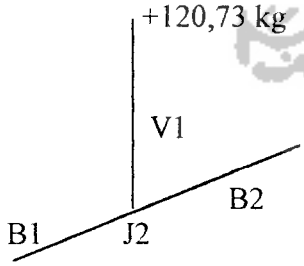
Joint 1 ( J1 )




-3859,3 kg  $nA1 = \frac{3859,3}{5343} = 0,72 \approx 2$ buah

+ 4045,73 kg  $nB1 = \frac{4045,72}{5343} = 0,75 \approx 2$ buah

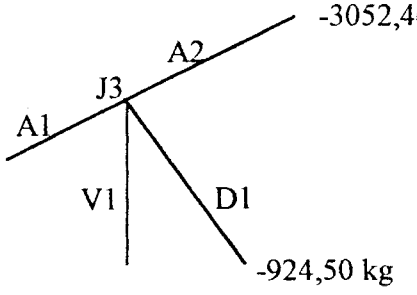
Joint 2 ( J2 )




+120,73 kg  $nV1 = \frac{120,73}{5343} = 0,02 \approx 2$ buah

+4042,34 kg  $nB2 = \frac{4042,34}{5343} = 0,76 \approx 2$ buah

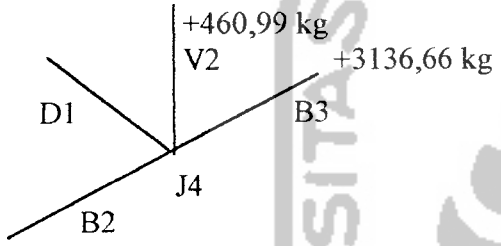
Joint 3 ( J3 )




$nA2 = \frac{3052,46}{5343} = 0,57 \approx 2$ buah

$nD1 = \frac{924,50}{5343} = 0,17 \approx 2$ buah

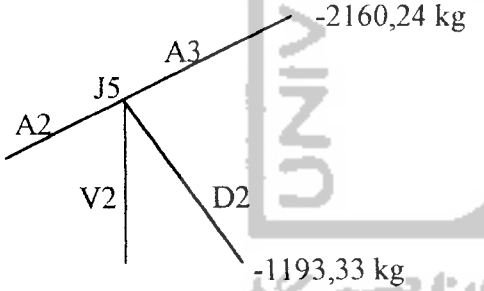
Joint 4 ( J4 )




$nV2 = \frac{460,99}{5343} = 0,08 \approx 2$ buah

$nB3 = \frac{3136,66}{5343} = 0,5 \approx 2$ buah

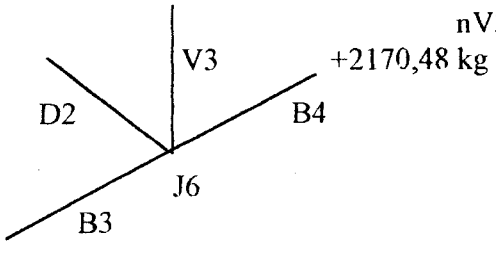
Joint 5 ( J5 )




$nA3 = \frac{2160,24}{5343} = 0,40 \approx 2$ buah

$nD2 = \frac{1193,33}{5343} = 0,22 \approx 2$ buah

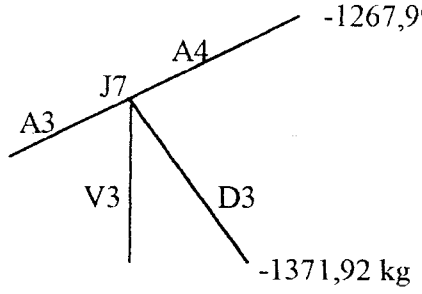
Joint 6 ( J6 )




$nV3 = \frac{824,25}{5343} = 0,15 \approx 2$ buah

$nB4 = \frac{2170,48}{5343} = 0,40 \approx 2$ buah

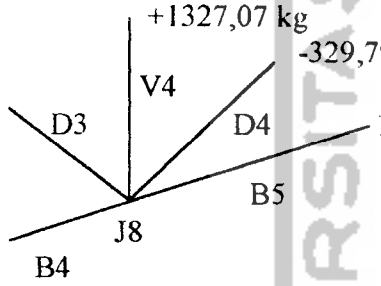
Joint 7 ( J7 )




$nA4 = \frac{1267,99}{5343} = 0,24 \approx 2$ buah

$nD3 = \frac{1371,92}{5343} = 0,26 \approx 2$ buah

Joint 8 ( J8 )

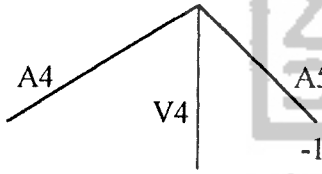



$nV4 = \frac{1327,07}{5343} = 0,25 \approx 2$ buah

$nD4 = \frac{329,79}{5343} = 0,06 \approx 2$ buah

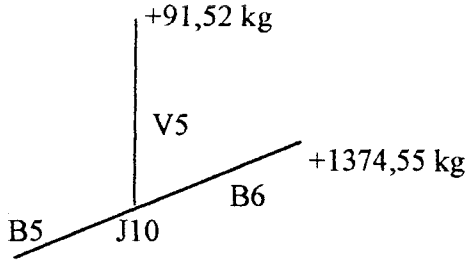
$nB5 = \frac{1372,07}{5343} = 0,26 \approx 2$ buah

Joint 9 ( J9 )




$nA5 = \frac{1634,97}{5343} = 0,30 \approx 2$ buah

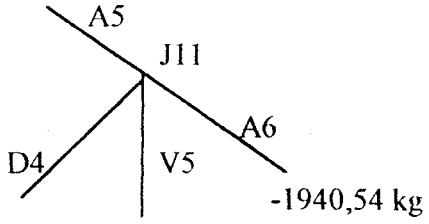
Joint 10 ( J10 )




$nV5 = \frac{91,52}{5343} = 0,02 \approx 2$ buah

$nB6 = \frac{1374,55}{5343} = 0,26 \approx 2$ buah

Joint 11 ( J11 )




$$nA6 = \frac{1940,54}{5343} = 0,36 \approx 2 \text{ buah}$$

**Tabel. Jumlah baut yang diperlukan**

| Batang | Jumlah baut ( buah ) |
|---|---|
| Atas<br>A1, A2, A3, A4, A5, A6 | 2 |
| Bawah<br>B1, B2, B3, B4, B5, B6 | 2 |
| Vertikal<br>V1, V2, V3, V4, V5 | 2 |
| Diagonal<br>D1, D2, D3, D4 | 2 |

## 4.2. Perencanaan Pelat

### 4.2.1 Pelat basement

Beban yang bekerja :

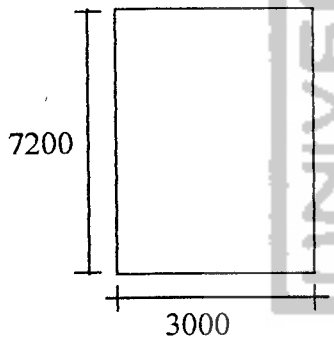
- Beban mati ( DL )

| | | |
|---|---|---|
| Berat sendiri pelat | : 0,14 x 2400 | = 336 kg/m$^2$ |
| Berat lapisan pasir bawah lantai | : 0,10 x 1600 | = 160 kg/m$^2$ |
| Berat spesi + ( finishing ) | : 0,05 x 2200 | = 110 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 606 kg/m$^2$ |
| | | = 5,945 KN/m |

- Beban hidup ( LL )

Beban hidup lantai parkir ( $q_L$ ) = 800 kg/m$^2$ = 7,85 KN/m

**1. Pelat basement**




Lebar balok = 250 mm

Tumpuan dianggap jepit - jepit

$L_{ny}$ = 7200 – 250 = 6950 mm

$L_{nx}$ = 3000 – 250 = 2750 mm

$\frac{Lny}{Lnx} = 2,5$

$$h_{min} = \frac{Lny \,.\, ( 0,8 + fy / 1500 )}{36 + 9 \,.\, \beta}$$

$$= \frac{6950 \,( 0,8 + 240/1500 )}{36 + 9 \,.\, 2,5}$$

= 113,52 mm

Diambil tebal pelat 140 mm

∅ pokok = 10 mm ⟶ A∅10 = 78,54 $mm^2$

∅ bagi = 8 mm ⟶ A∅8 = 50,265 $mm^2$

Penutup beton (pb) = 20 mm

Tebal efektif pelat

- Arah – x = 140 – 20 – 5 = 115 mm
- Arah – y = 140 – 20 – 10 – 5 = 105 mm

Beban merata terfaktor :

$q_u$ = 1,2 . $q_d$ + 1,6 . $q_l$

= 1,2 . 5,945 + 1,6 . 7,85

= 19,694 KN/m

Kontrol terhadap tegangan geser

Vu = 1,5 . 0,5 . $q_u$ . Lx

= 1,5. 0,5 . 19,694 . 2,75

= 40,62 KN

Vc = 1/6 $\sqrt{f'c}$ . b . d

= 1/6 . $\sqrt{25}$ . 1000. 115

= 95,83 KN

∅Vc = 0,6 . Vc > Vu

= 0,6 . 95,83

= 57,50 KN > Vu ..... **aman terhadap geser!**

Momen – momen yang terjadi

$\frac{Lny}{Lnx} = 2,5 \longrightarrow$ menurut tabel PBI – 1971

$M = \pm\ 0,001 \cdot q_u \cdot Lx^2 \cdot X$

$Mlx = 0,001 \cdot 19,694 \cdot 2,75^2 \cdot 42 = 6,2553$ KN.m

$Mly = 0,001 \cdot 19,694 \cdot 2,75^2 \cdot 10 = 1,4983$ KN.m

$Mtx = 0,001 \cdot 19,694 \cdot 2,75^2 \cdot 83 = 12,3617$ KN.m

$Mty = 0,001 \cdot 19,694 \cdot 2,75^2 \cdot 57 = 8,4893$ KN.m

Syarat – syarat batas

$\rho_{min} = \frac{1,4}{fy}$

$= \frac{1,4}{240} = 0,0058$

$\rho_b = \frac{0,85 \cdot f'c}{fy} \cdot \beta \cdot \frac{600}{600 + fy}$

$= \frac{0,85 \cdot 25}{240} \cdot 0,85 \cdot \frac{600}{600 + 240}$

$= 0,0537$

$\rho_{maks} = 0,75 \cdot \rho_b$

$= 0,75 \cdot 0,0537$

$= 0,0403$

$m = \frac{fy}{0,85 \cdot f'c}$

$= \frac{240}{0,85 \cdot 25}$

$= 11,2941$

Penulangan lapangan arah x

Mlx/∅ = 6,2553 / 0,8 = 7,8191 KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{7{,}8191 \cdot 10^6}{1000 \cdot 115^2}$$

= 0,591 Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}}\right)$$

$$= \frac{1}{11{,}2941} \cdot \left(1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}591}{240}}\right)$$

$= 0{,}00249 < \rho_{min}$

dipakai $\rho_{min}$ = 0,0058

$As_{perlu} = \rho \cdot b \cdot d$

$= 0{,}0058 \cdot 1000 \cdot 115$

$= 670{,}45\ mm^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{607{,}45} \leq 117{,}14\ mm$$

Dipakai tulangan pokok ∅10 – 110

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{110}$$

$= 714\ mm^2$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{714 \cdot 240}{0{,}85 . 25 . 1000}$$

$$= 8{,}064 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 714 \cdot 240 \cdot (115 - 8{,}064/2)$$

$$= 19{,}015 \text{ KN.m}$$

$$Mu = \varnothing \cdot Mn$$

$$= 0{,}8 \cdot 19{,}015$$

$$= 15{,}21 \text{ KN.m} \geq Mlx = 6{,}2553 \text{ KN.} \quad \textbf{OK!}$$

Penulangan tumpuan arah – x

Mtx/∅ = 12,3617 / 0,8 = 15,452 KN.m

$$Rn = \frac{Mtx/\varnothing}{b \cdot d^2}$$

$$= \frac{15{,}452 \cdot 10^6}{1000 \cdot 115^2}$$

= 1,1684 Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 1{,}1684}{240}} \right)$$

$= 0{,}0050 < \rho_{min}$

dipakai $\rho_{min} = 0{,}0058$

$As_{perlu} = \rho \cdot b \cdot d$

$= 0{,}0058 \cdot 1000 \cdot 115$

$= 670{,}45 \text{ mm}^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{607{,}45} \leq 117{,}14 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 110

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{110}$$

$= 714 \text{ mm}^2$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{714 \cdot 240}{0{,}85 . 25 . 1000}$$

$$= 8{,}064 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot ( d \quad a/2 )$$

$$= 714 \cdot 240 \cdot ( 115 - 8{,}064/2 )$$

$$= 19{,}015 \text{ KN.m}$$

$$Mu = \varnothing \cdot Mn$$

$$= 0{,}8 \cdot 19{,}015$$

$$= 15{,}21 \text{ KN.m} \geq Mtx = 12{,}3617 \text{ KN.m}$$ **OK!**

Tulangan bagi tumpuan

$$As = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 140$$

$$= 280 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$$

$$\leq \frac{50{,}265 \cdot 1000}{280}$$

$$\leq 179{,}51 \text{ mm}$$

Dipakai tulangan bagi ∅8 – 170

Penulangan lapangan arah – y

Mly/∅ = 1,4893/0,8 = 1,8616 KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{1{,}8616 \cdot 10^6}{1000 \cdot 105^2}$$

$$= 0{,}1688 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}1688}{240}} \right)$$

$$= 0{,}0009 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0058$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0058 \cdot 1000 \cdot 105$$

$$= 612{,}15 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{612{,}15} \leq 128{,}30 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 120

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{120}$$

$$= 654{,}5 \text{ mm}^2$$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{654{,}5 \cdot 240}{0{,}85.25.1000}$$

$$= 10{,}83 \text{ mm}$$

$Mn = As_{ada} \cdot fy \cdot ( d \quad a/2 )$

$= 654{,}5 \cdot 240 \cdot ( 105 - 10{,}83/2 )$

$= 15{,}6452$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 15{,}6452$

$= 12{,}514$ KN.m $\geq$ Mly = 1,4893 KN.m **OK!**

Penulangan tumpuan arah – y

Mty/∅ = 8,4893 / 0,8 = 10,611 KN.m

$$Rn = \frac{Mty/\varnothing}{b \cdot d^2}$$

$$= \frac{10{,}611 \cdot 10^6}{1000 \cdot 105^2}$$

$$= 0{,}9625 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}9625}{240}} \right)$$

$$= 0{,}0041 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0058$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0058 \cdot 1000 \cdot 105$$

$$= 612{,}15 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{612{,}5} \leq 128{,}30 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 120

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{120}$$

$$= 654{,}5 \text{ mm}^2$$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{654{,}5 \cdot 240}{0{,}85.25.1000}$$

$$= 10{,}83 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 654{,}5 \cdot 240 \cdot (105 - 10{,}83/2)$$

$$= 15{,}6452 \text{ KN.m}$$

$$Mu = \varnothing \cdot Mn$$

$$= 0{,}8 \cdot 15{,}6452$$

$$= 112{,}514 \text{ KN.m} \geq Mty = 10{,}611 \text{ KN.m}$$ **OK!**

Tulangan bagi tumpuan

$$As = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 140$$

$$= 280 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$$

$$\leq \frac{50{,}265 \cdot 1000}{280}$$

$$\leq 179{,}51 \text{ mm}$$

Dipakai tulangan bagi $\varnothing$8 – 170

Untuk perhitungan Pelat basement type berikutnya hasilnya dapat dilihat pada tabel 4.3 hal 91.

### 4.2.2 Pelat Lantai

Beban yang bekerja :

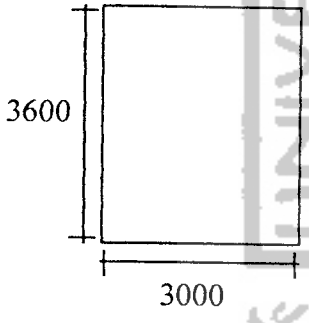
- Beban mati ( DL )

| | | |
|---|---|---|
| Berat sendiri pelat | : 0,12 x 2400 | = 388 kg/m$^2$ |
| Berat lapisan pasir bawah lantai | : 0,10 x 1600 | = 160 kg/m$^2$ |
| Berat spesi + ( finishing ) | : 0,05 x 2200 | = 110 kg/m$^2$ |
| Berat plafaon + rangka | : ( 11 + 7 ) | = 18 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 576 kg/m$^2$ |
| | | = 5,65 KN/m |

- Beban hidup ( LL )

Beban hidup pelat lantai ( $q_L$ ) = 250 kg/m$^2$ ≈ 2,50 KN/m

**1. Pelat lantai**




Lebar balok = 300 mm

Tumpuan dianggap jepit - jepit

$L_{ny}$ = 3600 – 300 = 3300 mm

$L_{nx}$ = 3000 – 300 = 2700 mm

$\frac{Lny}{Lnx} = 1,2$

$$h_{min} = \frac{Lny \,.\, ( 0,8 + fy / 1500 )}{36 + 9 \,.\, \beta}$$

$$= \frac{3300 \,( 0,8 + 240/1500 )}{36 + 9 \,.\, 1,2}$$

= 67,69 mm

Diambil tebal pelat 120 mm

∅ pokok = 10 mm ⟶ A∅10 = 78,54 $mm^2$

∅ bagi = 8 mm ⟶ A∅8 = 50,265 $mm^2$

Penutup beton (pb) = 20 mm

Tebal efektif pelat

- Arah – x = 120 – 20 – 5 = 95 mm
- Arah – y = 120 – 20 – 10 – 5 = 85 mm

Beban merata terfaktor :

$q_u$ = 1,2 . $q_d$ + 1,6 . $q_l$

= 1,2 . 5,65 + 1,6 . 2,5

= 10,78 KN/m

Kontrol terhadap tegangan geser

Vu = 1,5 . 0,5 . $q_u$ . Lx

= 1,5. 0,5 . 10,78 . 2,70

= 21,83 KN

Vc = 1/6 $\sqrt{f'c}$ . b . d

= 1/6 . √25 . 1000. 95

= 79 KN

∅Vc = 0,6 . Vc > Vu

= 0,6 . 79

= 47,70 KN > Vu ..... **aman terhadap geser!**

Momen – momen yang terjadi

$\frac{Lny}{Lnx} = 1,2 \longrightarrow$ menurut tabel PBI – 1971

$M = \pm\ 0,001 \cdot q_u \cdot Lx^2 \cdot X$

$Mlx = 0,001 \cdot 19,694 \cdot 2,70^2 \cdot 28 = 2,2004$ KN.m

$Mly = 0,001 \cdot 19,694 \cdot 2,70^2 \cdot 20 = 1,5717$ KN.m

$Mtx = 0,001 \cdot 19,694 \cdot 2,70^2 \cdot 64 = 5,0295$ KN.m

$Mty = 0,001 \cdot 19,694 \cdot 2,70^2 \cdot 56 = 4,4008$ KN.m

Syarat – syarat batas

$\rho_{min} = \frac{1,4}{fy}$

$= \frac{1,4}{240} = 0,0058$

$\rho_b = \frac{0,85 \cdot f'c}{fy} \cdot \beta \cdot \frac{600}{600 + fy}$

$= \frac{0,85 \cdot 25}{240} \cdot 0,85 \cdot \frac{600}{600 + 240}$

$= 0,0537$

$\rho_{maks} = 0,75 \cdot \rho_b$

$= 0,75 \cdot 0,0537$

$= 0,0403$

$m = \frac{fy}{0,85 \cdot f'c}$

$= \frac{240}{0,85 \cdot 25}$

$= 11,2941$

Penulangan lapangan arah – x

Mlx/∅ = 2,2004 / 0,8 = 2,7505 KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{2{,}7505 \cdot 10^6}{1000 \cdot 95^2}$$

$$= 0{,}3047 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}3047}{240}} \right)$$

$$= 0{,}00127 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0058$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0058 \cdot 1000 \cdot 95$$

$$= 553{,}85 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{553{,}85} \leq 141{,}80 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 140

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{140}$$

$$= 561{,}00 \text{ mm}^2$$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{561{,}00 \cdot 240}{0{,}85 . 25 . 1000}$$

$= 6{,}33$ mm

$Mn = As_{ada} \cdot fy \cdot ( d \quad a/2 )$

$= 561 \cdot 240 \cdot ( 115 - 6{,}33/2 )$

$= 12{,}36$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 12{,}36$

$= 9{,}88$ KN.m $\geq$ Mlx $= 2{,}2004$ KN.m **OK!**

Penulangan tumpuan arah – x

Mtx/∅ = 5,0295 / 0,8 = 6,286 KN.m

$$Rn = \frac{Mtx/\varnothing}{b \cdot d^2}$$

$$= \frac{6{,}286 \cdot 10^6}{1000 \cdot 95^2}$$

$$= 0{,}6965 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}6965}{240}} \right)$$

$$= 0{,}00295 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0058$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0058 \cdot 1000 \cdot 95$$

$$= 553{,}85 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{553{,}85} \leq 141{,}80 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 140

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{140}$$

$$= 561{,}00 \text{ mm}^2$$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{561{,}00 \cdot 240}{0{,}85 . 25 . 1000}$$

$= 6{,}33$ mm

$Mn = As_{ada} \cdot fy \cdot (d - a/2)$

$= 561 \cdot 240 \cdot (115 - 6{,}33/2)$

$= 12{,}36$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 12{,}36$

$= 9{,}88$ KN.m $\geq$ Mtx $= 5{,}0295$ KN.m **OK!**

Tulangan bagi tumpuan

$As = 0{,}002 \cdot b \cdot h$

$= 0{,}002 \cdot 1000 \cdot 120$

$= 240$ mm$^2$

$$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$$

$$\leq \frac{50{,}265 \cdot 1000}{240}$$

$\leq 209{,}43$ mm

$\leq 2$ ht $= 2 \cdot 80 = 160$ mm

Dipakai tulangan bagi ∅8 – 160

Penulangan lapangan arah – y

Mly/∅ = 1,5717 / 0,8 = 1,9646 KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{1{,}9646 \cdot 10^6}{1000 \cdot 85^2}$$

= 0,2719 Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2 \cdot 11{,}2941 \cdot 0{,}2719}{240}} \right)$$

$$= 0{,}0011 < \rho_{min}$$

dipakai $\rho_{min}$ = 0,0058

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0058 \cdot 1000 \cdot 85$$

$$= 495{,}55 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{495{,}55} \leq 158{,}70 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 150

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{150}$$

$$= 523{,}60 \text{ mm}^2$$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{523{,}60 \cdot 240}{0{,}85.25.1000}$$

$$= 5{,}9163 \text{ mm}$$

$Mn = As_{ada} \cdot fy \cdot ( d - a/2 )$

$= 523{,}60 \cdot 240 \cdot ( 105 - 5{,}9163/2 )$

$= 10{,}3098$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 10{,}3098$

$= 8{,}2479$ KN.m $\geq$ Mly $= 1{,}5717$ KN.m **OK!**

Penulangan tumpuan arah – y

Mty/∅ = 4,4008 / 0,8 = 5,501 KN.m

$$Rn = \frac{Mty/\varnothing}{b \cdot d^2}$$

$$= \frac{5{,}501 \cdot 10^6}{1000 \cdot 85^2}$$

= 0,7614 Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 0{,}7614}{240}} \right)$$

$= 0{,}0032 < \rho_{min}$

dipakai $\rho_{min}$ = 0,0058

$As_{perlu} = \rho \cdot b \cdot d$

$= 0{,}0058 \cdot 1000 \cdot 85$

$= 495{,}55 \text{ mm}^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{495{,}55} \leq 158{,}70 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 150

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{150}$$

$= 523{,}60 \text{ mm}^2$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{523{,}60 \cdot 240}{0{,}85 . 25 . 1000}$$

$$= 5{,}9163 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 523{,}60 \cdot 240 \cdot (105 - 5{,}9163/2)$$

$$= 10{,}3098 \text{ KN.m}$$

$$Mu = \varnothing \cdot Mn$$

$$= 0{,}8 \cdot 10{,}3098$$

$$= 8{,}2479 \text{ KN.m} \geq Mty = 4{,}4008 \text{ KN.m}$$ **OK!**

Tulangan bagi tumpuan

$$As = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 120$$

$$= 240 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$$

$$\leq \frac{50{,}265 \cdot 1000}{240}$$

$$\leq 209{,}43 \text{ mm}$$

$$\leq 2\,ht = 2 \cdot 80 = 160 \text{ mm}$$

Dipakai tulangan bagi ∅8 – 160

Untuk perhitungan Pelat lantai type berikutnya hasilnya dapat dilihat pada tabel 4.4 hal 91.

### 4.2.3 Pelat Atap

Beban yang bekerja :

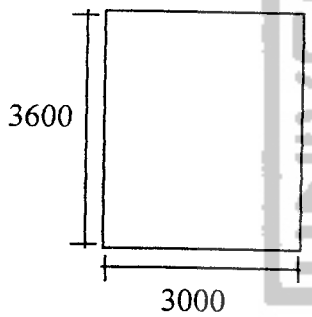
- Beban mati ( DL )

| | | |
|---|---|---|
| Berat sendiri pelat | : 0,08 x 2400 | = 192 kg/m$^2$ |
| Berat Finishing | : | = 66 kg/m$^2$ |
| Berat air hujan | : 0,05 x 1000 | = 50 kg/m$^2$ |
| Berat langit – langit + penggantung ( 11 + 7 ) | | = 18 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 326 kg/m$^2$ |
| | | = 3,20 KN/m |

- Beban hidup ( LL )

Beban hidup pelat atap ( $q_L$ ) = 100 kg/m$^2$ ≈ 1,00 KN/m

**1. Pelat Atap**




Lebar balok = 250 mm

Tumpuan dianggap jepit - jepit

$L_{ny}$ = 3600 – 250 = 3350 mm

$L_{nx}$ = 3000 – 250 = 2750 mm

$\frac{Lny}{Lnx} = 1,2$

$$h_{min} = \frac{Lny \,.\, ( 0,8 + fy / 1500 )}{36 + 9 \,.\, \beta}$$

$$= \frac{3350 \,( 0,8 + 240/1500 )}{36 + 9 \,.\, 2,5}$$

= 67,69 mm

Diambil tebal pelat 80 mm

∅ pokok = 10 mm ⟶ A∅10 = 78,54 $mm^2$

∅ bagi = 8 mm ⟶ A∅8 = 50,265 $mm^2$

Penutup beton (pb) = 20 mm

Tebal efektif pelat

- Arah – x = 80 – 20 – 5 = 55 mm
- Arah – y 80 20 10 5 45 mm

Beban merata terfaktor :

$q_u$ = 1,2 . $q_d$ + 1,6 . $q_l$

= 1,2 . 3,20 + 1,6 . 1,00

= 5,44 KN/m

Kontrol terhadap tegangan geser

Vu = 1,5 . 0,5 . $q_u$ . Lx

= 1,5. 0,5 . 5,44 . 2,75

= 11,22 KN

Vc = 1/6 $\sqrt{f'c}$ . b . d

= 1/6 . √25 . 1000. 55

= 46,67 KN

∅Vc = 0,6 . Vc > Vu

= 0,6 . 46,67

= 28,00 KN > Vu ..... **aman terhadap geser!**

Momen – momen yang terjadi

$\frac{Lny}{Lnx} = 2,5 \longrightarrow$ menurut tabel PBI – 1971

$M = \pm\ 0,001 \cdot q_u \cdot Lx^2 \cdot X$

$Mlx = 0,001 \cdot 5,44 \cdot 2,75^2 \cdot 28 = 1,1519$ KN.m

$Mly = 0,001 \cdot 5,44 \cdot 2,75^2 \cdot 20 = 0,8228$ KN.m

$Mtx = 0,001 \cdot 5,44 \cdot 2,75^2 \cdot 64 = 2,6329$ KN.m

$Mty = 0,001 \cdot 5,44 \cdot 2,75^2 \cdot 56 = 2,3038$ KN.m

Syarat – syarat batas

$\rho_{min} = \frac{1,4}{fy}$

$= \frac{1,4}{240} = 0,0058$

$\rho_b = \frac{0,85 \cdot f'c}{fy} \cdot \beta \cdot \frac{600}{600 + fy}$

$= \frac{0,85 \cdot 25}{240} \cdot 0,85 \cdot \frac{600}{600 + 240}$

$= 0,0537$

$\rho_{maks} = 0,75 \cdot \rho_b$

$= 0,75 \cdot 0,0537$

$= 0,0403$

$m = \frac{fy}{0,85 \cdot f'c}$

$= \frac{240}{0,85 \cdot 25}$

$= 11,2941$

Penulangan lapangan arah – x

$Mlx/\varnothing = 1,1519 / 0,8 = 1,4398$ KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{1,4398 \cdot 10^6}{1000 \cdot 55^2}$$

$$= 0,4759 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11,2941} \cdot \left( 1 - \sqrt{1 - \frac{2 \cdot 11,2941 \cdot 0,4759}{240}} \right)$$

$$= 0,0020 < \rho_{min}$$

dipakai $\rho_{min} = 0,0058$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0,0058 \cdot 1000 \cdot 55 = 319,0 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78,54 \cdot 1000}{319,0} \leq 246,20 \text{ mm}$$

$$\leq 2\,ht \leq 2.80 = 160 \text{ mm}$$

Dipakai tulangan pokok ∅10 – 160

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{160}$$

$$= 490,875 \text{ mm}^2$$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{490{,}875 \cdot 240}{0{,}85.25.1000}$$

$$= 5{,}544 \text{ mm}$$

$Mn = As_{ada} \cdot fy \cdot ( d \quad a/2 )$

$= 490{,}875 \cdot 240 \cdot ( 55 - 5.544/2 )$

$= 6{,}1530$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 6{,}1530$

$= 4{,}9223$ KN.m $\geq$ Mlx $= 1{,}1519$ KN.m **OK!**

Penulangan tumpuan arah – x

$Mtx/\varnothing = 2{,}6329 / 0{,}8 = 3{,}2911$ KN.m

$$Rn = \frac{Mtx/\varnothing}{b \cdot d^2}$$

$$= \frac{3{,}2911 \cdot 10^6}{1000 \cdot 55^2}$$

$= 1{,}0879$ Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11{,}2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 1{,}0879}{240}} \right)$$

$= 0{,}0046 < \rho_{min}$

dipakai $\rho_{min} = 0{,}0058$

$As_{perlu} = \rho \cdot b \cdot d$

$= 0{,}0058 \cdot 1000 \cdot 55 = 319{,}0$ mm$^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{319{,}0} \leq 246{,}20 \text{ mm}$$

$\leq 2\,ht \leq 2.80 = 160$ mm

Dipakai tulangan pokok $\varnothing 10 - 160$

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{160}$$

$= 490{,}875$ mm$^2$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{490{,}875 \cdot 240}{0{,}85 . 25 . 1000}$$

$$= 5{,}544 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 490{,}875 \cdot 240 \cdot (55 - 5.544/2)$$

$$= 6{,}1530 \text{ KN.m}$$

$$Mu = \varnothing \cdot Mn$$

$$= 0{,}8 \cdot 6{,}1530$$

$$= 4{,}9223 \text{ KN.m} \geq Mtx = 2{,}6329 \text{ KN.m}$$ **OK!**

Tulangan bagi tumpuan

$$As = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 80$$

$$= 160 \text{ mm}^2$$

$$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$$

$$\leq \frac{50{,}265 \cdot 1000}{160} \leq 314 \text{ mm}$$

$$\leq 2 \text{ ht} = 2 \cdot 80 = 160 \text{ mm}$$

Dipakai tulangan bagi ∅8 – 160

Penulangan lapangan arah – y

$Mly/\varnothing = 0,8228 / 0,8 = 1,0283$ KN.m

$$Rn = \frac{Mlx/\varnothing}{b \cdot d^2}$$

$$= \frac{1,0283 \cdot 10^6}{1000 \cdot 45^2}$$

$= 0,5079$ Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{11,2941} \cdot \left( 1 - \sqrt{1 - \frac{2.11,2941 \cdot 0,5079}{240}} \right)$$

$= 0,00214 < \rho_{min}$

dipakai $\rho_{min} = 0,0058$

$As_{perlu} = \rho \cdot b \cdot d$

$= 0,0058 \cdot 1000 \cdot 45 = 261$ mm$^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78,54 \cdot 1000}{261} \leq 300,90 \text{ mm}$$

$\leq 2\,ht \leq 2.80 = 160$ mm

Dipakai tulangan pokok $\varnothing 10 - 160$

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{160}$$

$= 490,875$ mm$^2$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{490{,}875 \cdot 240}{0{,}85 \cdot 25 \cdot 1000}$$

$$= 5{,}544 \text{ mm}$$

$Mn = As_{ada} \cdot fy \cdot ( d \quad a/2 )$

$= 490{,}875 \cdot 240 \cdot ( 45 - 5{,}544/2 )$

$= 4{,}9748$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 4{,}9748$

$= 3{,}9799$ KN.m $\geq$ Mly $= 0{,}8228$ KN.m **OK!**

Penulangan tumpuan arah – y

Mty/∅ = 2,3038 / 0,8 = 2,8797 KN.m

$$Rn = \frac{Mty/\varnothing}{b \cdot d^2}$$

$$= \frac{2{,}8797 \cdot 10^6}{1000 \cdot 45^2}$$

= 1,422 Mpa

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}}\right)$$

$$= \frac{1}{11{,}2941} \cdot \left(1 - \sqrt{1 - \frac{2.11{,}2941 \cdot 1{,}422}{240}}\right)$$

$= 0{,}0061 < \rho_{maks}$

dipakai $\rho_{perlu} = 0{,}0061$

$As_{perlu} = \rho \cdot b \cdot d$

$= 0{,}0061 \cdot 1000 \cdot 45 = 276{,}21\ mm^2$

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{78{,}54 \cdot 1000}{276{,}21} \leq 284{,}36\ mm$$

$\leq 2\,ht \leq 2.80 = 160$ mm

Dipakai tulangan pokok ∅10 – 160

$$As_{ada} = \frac{A\varnothing 10 \cdot 1000}{160}$$

$= 490{,}875\ mm^2$

Kontrol kapasitas tampang

$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$

$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$

$= \frac{490{,}875 \cdot 240}{0{,}85.25.1000}$

$= 5{,}544$ mm

$Mn = As_{ada} \cdot fy \cdot (d - a/2)$

$= 490{,}875 \cdot 240 \cdot (45 - 5{,}544/2)$

$= 4{,}9748$ KN.m

$Mu = \varnothing \cdot Mn$

$= 0{,}8 \cdot 4{,}9748$

$= 3{,}9799$ KN.m $\geq$ Mty = 2,3038 KN.m **OK!**

Tulangan bagi tumpuan

$As = 0{,}002 \cdot b \cdot h$

$= 0{,}002 \cdot 1000 \cdot 80$

$= 160$ mm$^2$

$S \leq \frac{A\varnothing 8 \cdot 1000}{As}$

$\leq \frac{50{,}265 \cdot 1000}{160} \leq 314$ mm

$\leq 2\,ht = 2 \cdot 80 = 160$ mm

Dipakai tulangan bagi ∅8 – 160

Untuk perhitungan Pelat atap type berikutnya hasilnya dapat dilihat pada tabel 4.5 hal. 91

**Tabel 4.3 Rekapitulasi Penulangan Pelat Basement**

| Type | Ly x Lx ( mm ) | Tulangan Pokok ( mm ) | | | | Tulangan Bagi ( mm ) |
|---|---|---|---|---|---|---|
| | | lx | tx | ly | ty | |
| A | 7200 x 3000 | ∅10 – 110 | ∅10 – 75 | ∅10 – 110 | ∅10 – 75 | ∅8 - 170 |
| B | 3000 x 3000 | ∅10 – 110 | ∅10 – 75 | ∅10 – 110 | ∅10 – 75 | ∅8 - 170 |

**Tabel 4.4 Rekapitulasi Penulangan Pelat Lantai**

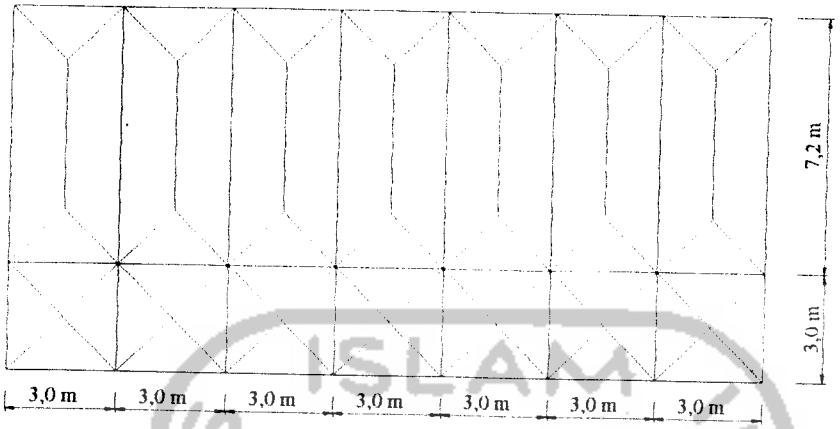
| Type | Ly x Lx ( mm ) | Tulangan Pokok ( mm ) | | | | Tulangan Bagi ( mm ) |
|---|---|---|---|---|---|---|
| | | lx | tx | ly | ty | |
| A | 3000 x 2500 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅8 – 280 |
| B | 3000 x 3000 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅8 – 280 |
| C | 3600 x 3000 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅10 – 140 | ∅8 – 280 |

**Tabel 4.5 Rekapitulasi Penulangan Pelat Atap**

| Type | Ly x Lx ( mm ) | Tulangan Pokok ( mm ) | | | | Tulangan susut |
|---|---|---|---|---|---|---|
| | | lx | tx | ly | ty | |
| I | 3600 x 3000 | ∅10 - 160 | ∅10 – 320 | ∅10 – 160 | ∅10 – 320 | ∅10 – 320 |
| II | 3000 x 3000 | ∅10 – 160 | ∅10 – 320 | ∅10 – 160 | ∅10 – 320 | ∅10 – 320 |

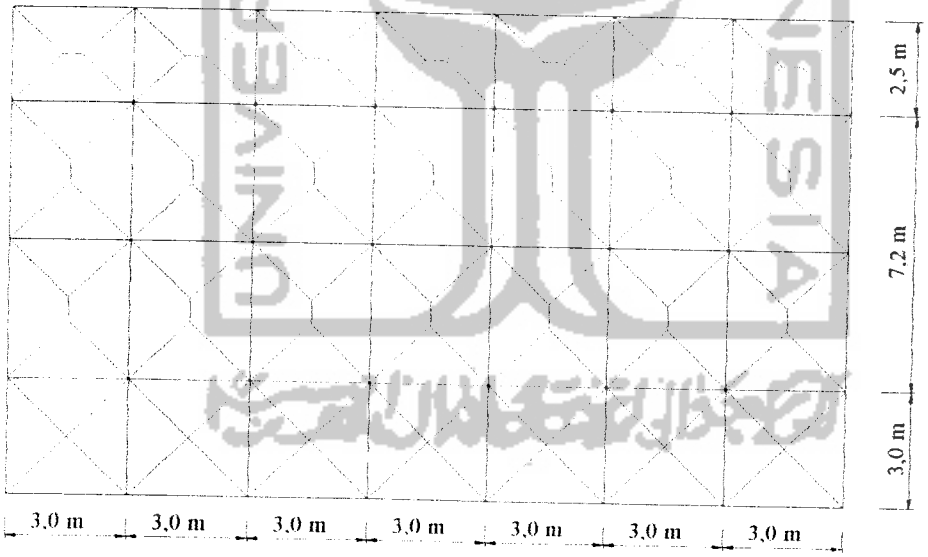
## 4.3. Distribusi Beban Pelat

### 4.3.1 Distribusi Beban Pelat Basement ( Dasar )



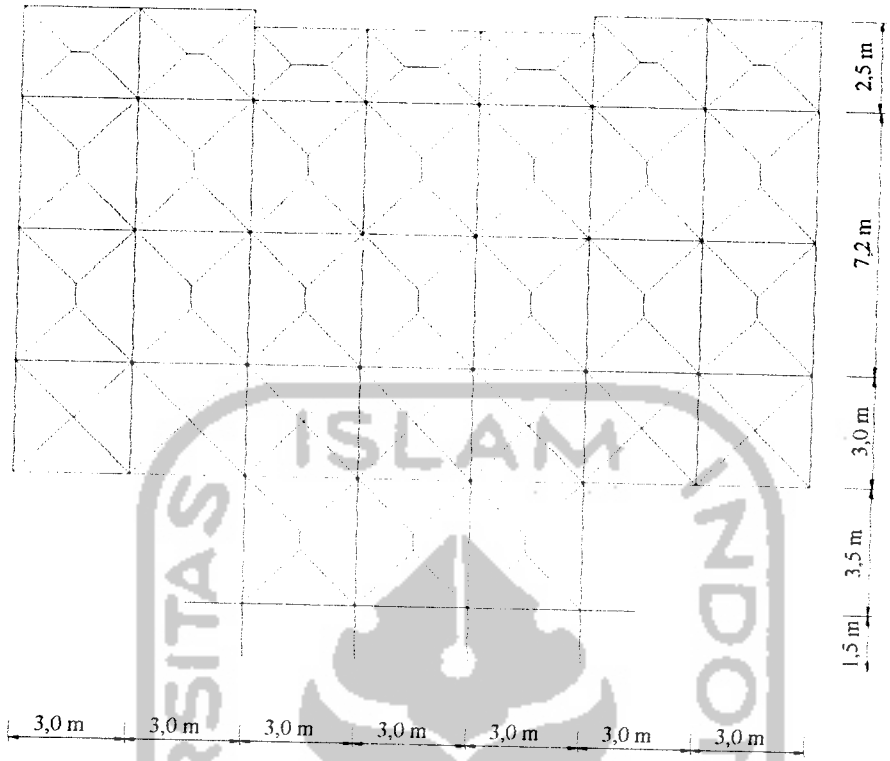

**Gambar 4.4 Distribusi beban pelat basement**

### 4.3.2 Distribusi Beban Pelat Lantai Satu



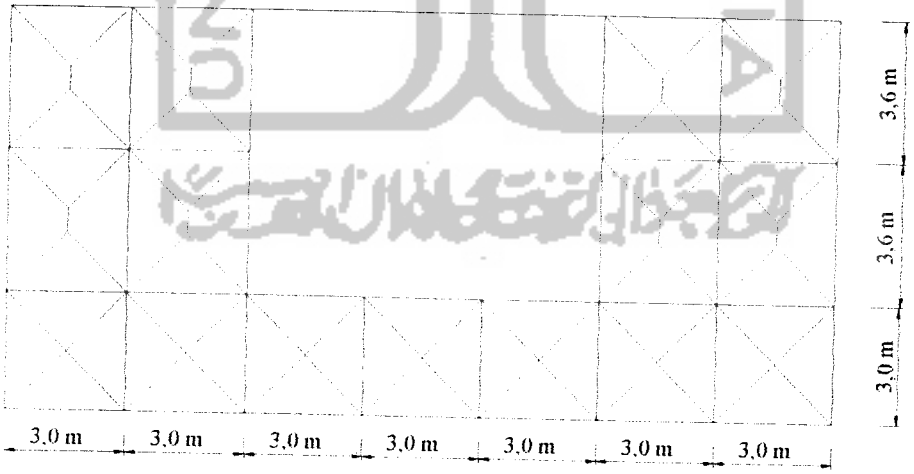

**Gambar 4.5 Distribusi beban pelat lantai satu**

### 4.3.3 Distribusi Beban Pelat Lantai Dua




**Gambar 4.6 Distribusi beban pelat lantai dua**

### 4.3.4 Distribusi Beban Pelat Atap




**Gambar 4.7 Distribusi beban pelat atap**

## 4.4 Analisis Pembebanan Pada Pelat

### 4.4.1 Pelat Basement

1. Beban mati ( DL )

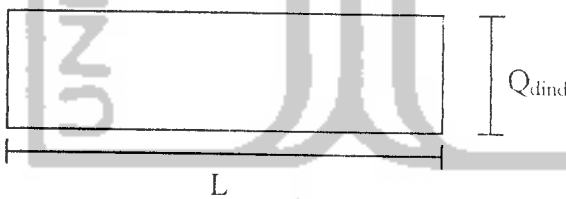
| | | |
|---|---|---|
| Berat sendiri pelat | : 0,14 x 2400 | = 336 kg/m$^2$ |
| Berat lapisan pasir bawah lantai | : 0,10 x 1600 | = 160 kg/m$^2$ |
| Berat spesi + ( finishing ) | : 0,05 x 2200 | = 110 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 606 kg/m$^2$ |

2. Beban hidup ( LL )

Beban hidup lantai parkir ( $q_L$ ) = 800 kg/m$^2$ = 7,85 KN/m

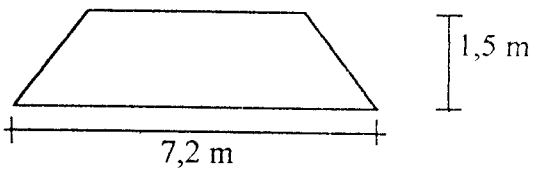
Distribusi beban pelat basement pada balok berupa beban segitiga dan beban trapesium dengan tipe beban sebagai berikut.

Beban Dinding




$Q_{dinding}$ = 2,8 . 250 = 700 kg/m

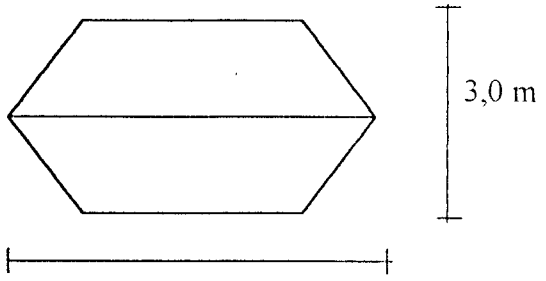
Tipe I




$QD_I$ = 1,5 . 606 = 909 kg/m

$QL_I$ = 1,5 . 800 = 1200 kg/m

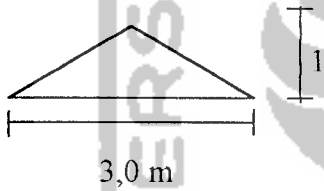
Tipe II




$QD_{II} = 2 . QD_{I} = 2 . 909 = 1818$ kg/m

$QL_{II} = 2 . QL_{I} = 2 .1200 = 2400$ kg/m

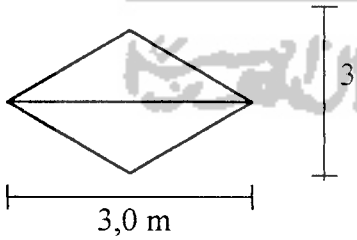
Tipe III




$QD_{III} = 1,5 . 606 = 909$ kg/m

$QL_{III} = 1,5 . 800 = 1200$ kg/m

Tipe IV




$QD_{IV} = 2 . QD_{III} = 2 . 909 = 1818$ kg/m

$QL_{IV} = 2 . QL_{III} = 2 .1200 = 2400$ kg/m

### 4.4.2 Pelat Lantai

1. Beban mati ( DL )

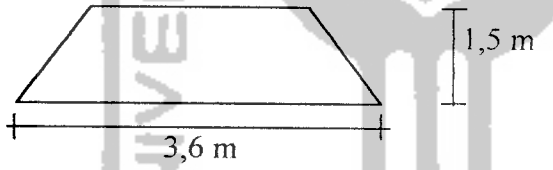
| | | |
|---|---|---|
| Berat sendiri pelat | : 0,12 x 2400 | = 388 kg/m$^2$ |
| Berat lapisan pasir bawah lantai | : 0,10 x 1600 | = 160 kg/m$^2$ |
| Berat spesi + ( finishing ) | : 0,05 x 2200 | = 110 kg/m$^2$ |
| Berat plafaon + rangka | : ( 11 + 7 ) | = 18 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 576 kg/m$^2$ |

2. Beban hidup ( LL )

Beban hidup pelat lantai ( $q_L$ ) = 250 kg/m$^2$

Distribusi beban pada balok

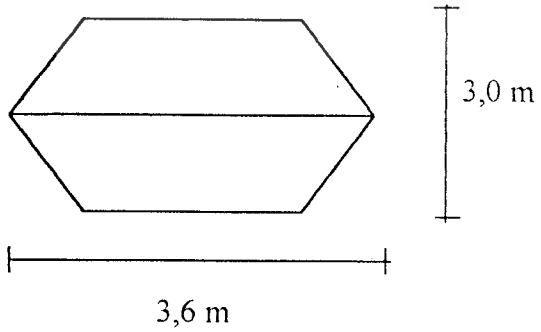
Tipe I




$QD_I$ = 1,5 . 576 = 864 kg/m

$QL_I$ = 1,5 . 250 = 375 kg / m

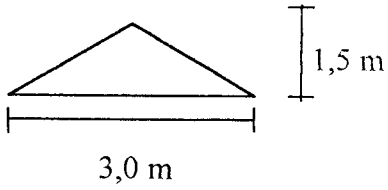
Tipe II




$QD_{II}$ = 2 . $QD_I$ = 2 . 864 = 1728 kg/m

$QL_{II}$ = 2 . $QL_I$ = 2 . 375 = 750 kg/m

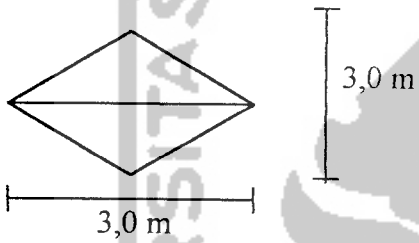
Tipe III




$QD_{III}$ = 1,5 . 576 = 864 kg/m

$QL_{III}$ = 1,5 . 250 = 375 kg / m

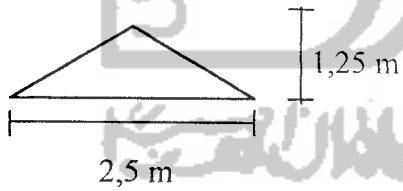
Tipe IV




$QD_{IV}$ = 2 . $QD_{III}$ = 2 . 864 = 1728 kg/m

$QL_{IV}$ = 2 . $QL_{III}$ = 2 . 375 = 750 kg/m

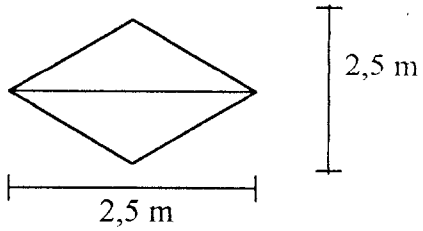
Tipe V




$QD_V$ = 1,25 . 576 = 720 kg/m

$QL_V$ = 1,25 . 250 = 312,5 kg/m

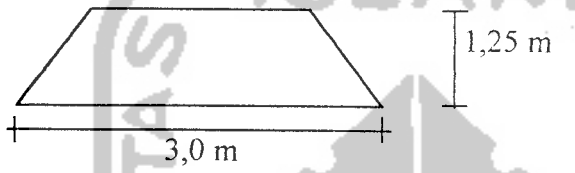
Tipe VI




$QD_{IV} = 2 . QD_V = 2 . 720 = 1440$ kg/m

$QL_{IV} = 2 . QL_V = 2 . 313 = 626$ kg/m

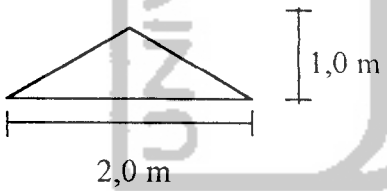
Tipe VII




$QD_{VII} = 1,25 . 576 = 720$ kg/m

$QL_{VII} = 1,25 . 250 = 312,5$ kg/m

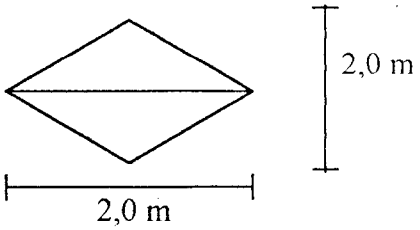
Tipe VIII




$QD_{VIII} = 1,0 . 576 = 576$ kg/m

$QL_{VIII} = 1,0 . 250 = 250$ kg/m

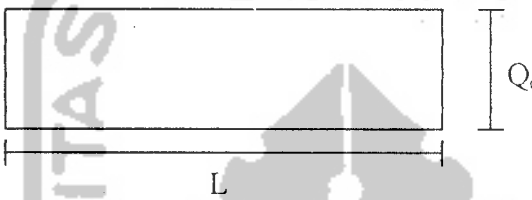
Tipe IX




$QD_{IX} = 2 . QD_{VIII} = 2 . 576 = 1152$ kg/m

$QL_{IX} = 2 . QL_{VIII} = 2 . 250 = 500$ kg/m

Beban Dinding




$Q_{dinding} = 5,10 . 250 = 1275$ kg/m

### 4.4.3 Pelat Atap

1. Beban mati ( DL )

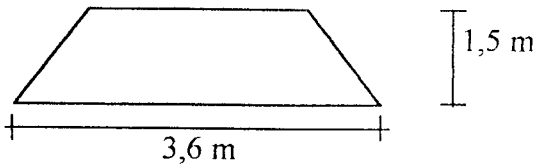
| | | |
|---|---|---|
| Berat sendiri pelat | : 0,08 x 2400 | = 192 kg/m$^2$ |
| Berat Finishing | : | = 66 kg/m$^2$ |
| Berat air hujan | : 0,05 x 1000 | = 50 kg/m$^2$ |
| Berat langit – langit + penggantung ( 11 + 7 ) | | = 18 kg/m$^2$ |
| | Beban mati ( $q_D$ ) | = 326 kg/m$^2$ |

2. Beban hidup ( LL )

Beban hidup pelat atap ( $q_L$ ) = 100 kg/m$^2$

Distribusi beban pada balok

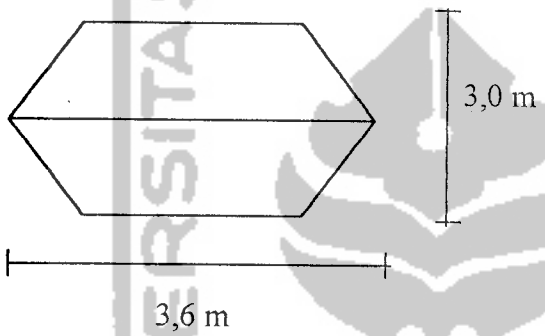
Tipe I




$QD_I = 1,5 \cdot 326 = 489$ kg/m

$QL_I = 1,5 \cdot 100 = 150$ kg/m

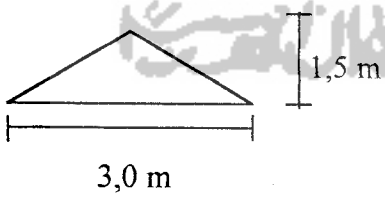
Tipe II




$QD_{II} = 2 \cdot QD_I = 2 \cdot 489 = 978$ kg/m

$QL_{II} = 2 \cdot QL_I = 2 \cdot 150 = 300$ kg/m

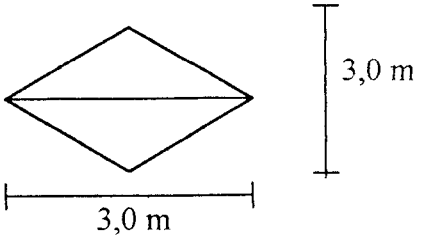
Tipe III




$QD_{III} = 1,5 \cdot 326 = 489$ kg/m

$QL_{III} = 1,5 \cdot 100 = 150$ kg/m

Tipe IV




$QD_{IV} = 2 . QD_{III} = 2 . 489 = 978$ kg/m

$QL_{IV} = 2 . QL_{III} = 2 . 150 = 300$ kg/m

## 4.5 Perhitungan Gaya Geser dan Gaya Horisontal Total Akibat Gempa

### 4.5.1 Berat bangunan total ( Wt )

A. Berat lantai atap

Beban mati

Pelat = ((21x10,2) – (9x7,2)) x 0,08 x 2400 = 28684,8 kg

Kolom = 24 x 2,075 x 0,50 x 0,60 x 2400 = 35856,0 kg

Dinding = 2,075 x 64,8 x 250 = 33615,0 kg +

$W_{mati}$ = 98155,8 kg

Beban hidup

Qh atap = 100 kg/m$^2$

Koefisien reduksi = 0,3

Wh = 0,3 ((10 x 10,2) – (9 x 7,2)) x 100

= 4482 kg

Beban total (Watap) = $W_{mati}$ + Wh

= 98155,8 + 4482

= 102637,8 kg

B. Berat Lantai III

Beban mati

Pelat = ((21x12,7) – (9x7,2))x 0,12 x 2400 = 58147,2 kg

Kolom = 24 x 4,20 x 0,50 x 0,60 x 2400 = 72576,0 kg

Dinding = 4,20 x 85,8 x 250 = 90090,0 kg

Spesi = 201,9 x 21 = 4239,9 kg

Tegel = 201,9 x 24 = 4845,6 kg +

$W_{mati}$ = 229898,7 kg

Beban hidup

Qh lantai = 250 kg/m$^2$

Koefisien reduksi = 0,3

Wh = 0,3 x 201,9 x 250

= 15142,5 kg

Beban total (WIII) = $W_{mati}$ + Wh

= 229898,7 + 15142,5

= 245041,2 kg

C. Berat Lantai II

Beban mati

| | | | |
|---|---|---|---|
| Pelat | = ((21x12,7) – (9x7,2))x 0,12 x 2400 | = | 58147,2 kg |
| Kolom | = 24 x 4,675 x 0,50 x 0,70 x 2400 | = | 94248,0 kg |
| Dinding | = 4,675 x 65,4 x 250 | = | 76436,25 kg |
| Spesi | = 201,9 x 21 | = | 4239,9 kg |
| Tegel | = 201,9 x 24 | = | 4845,6 kg + |
| | | $W_{mati}$ = | 237916,95 kg |

Beban hidup

Qh lantai = 250 kg/m$^2$

Koefisien reduksi = 0,3

Wh = 0,3 x 201,9 x 250

= 15142,5 kg

Beban total (WII) = $W_{mati}$ + Wh

= 23791,95 + 15142,5

= 253059,45 kg

D. Berat Lantai I

Beban mati

| | | | |
|---|---|---|---|
| Pelat | = (21 x 12,7) x 0,12 x 2400 | = | 76809,6 kg |
| Kolom | = 24 x 3,95 x 0,50 x 0,70 x 2400 | = | 76632,0 kg |
| Dinding | = 3,95 x 52,2 x 250 | = | 51547,5 kg |
| Spesi | = 266,7 x 21 | = | 5600,7 kg |
| Tegel | = 266,7 x 24 | = | 6400,8 kg + |
| | | $W_{mati}$ = | 219990,6 kg |

Beban hidup

Qh lantai = 250 kg/m$^2$

Koefisien reduksi = 0,3

Wh = 0,3 x 201,9 x 250

= 15142,5 kg

Beban total (WI) = $W_{mati}$ + Wh

= 219990,6 + 15142,5

= 235133,1 kg

E. Berat Lantai Basement

Beban mati

| | | | |
|---|---|---|---|
| Pelat | = (21 x 10,2) x 0,14 x 2400 | = | 71971,2 kg |
| Kolom | = 24 x 5,9 x 0,50 x 0,70 x 2400 | = | 79632,0 kg |
| Dinding | = 1,4 x 62,4 x 250 | = | 21840,0 kg |
| Spesi | = 214,2 x 21 | = | 4498,2 kg |
| Tegel | = 214,2 x 24 | = | 5140,8 kg + |
| | | $W_{mati}$ = | 222394,2 kg |

Beban hidup

Qh parkir = 800 kg/$m^2$

Koefisien reduksi = 0,3

Wh = 0,3 x 214,2 x 800

= 5140,8 kg

Beban total (Wbasement) = $W_{mati}$ + Wh

= 222394,2 + 5140,8

= 273802,2 kg

Wtotal bangunan = Watap + WIII + WII + WI + Wbasement

= 1109673,75 kg

**4.5.2** Waktu getar bangunan

Waktu getar bangunan ( T )

Dengan rumus empiris

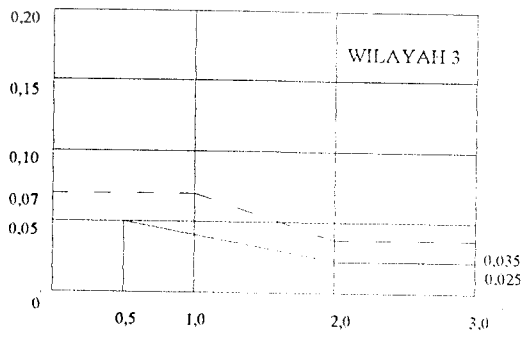
Tx = Ty = 0,06 $H^{3/4}$

= 0,06 x $20,8^{3/4}$

= 0,584 detik

**4.5.3** Koefisien gempa dasar

C diperoleh dari gambar grafik dibawah ini :




UntukTx = Ty = 0,584 detik, zone 3 dan jenis tanah keras diperoleh C = 0,05

**4.5.4** Faktor keutamaan I dan faktor jenis struktur K

Dari buku Desain struktur rangka beton bertulang di daerah rawan gempa, karangan Gideon Kesuma diperoleh I = 1,0 dan K = 1,0 untuk bangunan kantor yang menggunakan struktur rangka beton bertulang dengan daktilitas penuh.

**4.5.5** Gaya geser horisontal total akibat gempa

Vx = Vy = C I K Wtotal bangunan

= 0,05 x 1,0 x 1,0 x 1109673,75 kg

= 55483,68 kg

**4.5.6** Distribusi gaya geser horisontal total akibat gempa

a. arah x ( lihat tabel 4.1 )

H/A = 20,8 / 21

= 0,9 < 3

$$Fi,x = \frac{Wi \cdot hi}{\Sigma Wi \cdot hi} \times Vx$$

b. arah y ( lihat tabel 4.1 )

H/A = 20,8 / 12,7

= 1,6 < 3

$$Fi,x = \frac{Wi \cdot hi}{\Sigma Wi \cdot hi} \times Vy$$

dengan :

Fi = Gaya geser horisontal akibat gempa pada lantai ke i

Hi = Tinggi lantai ke i tehadap lantai dasar

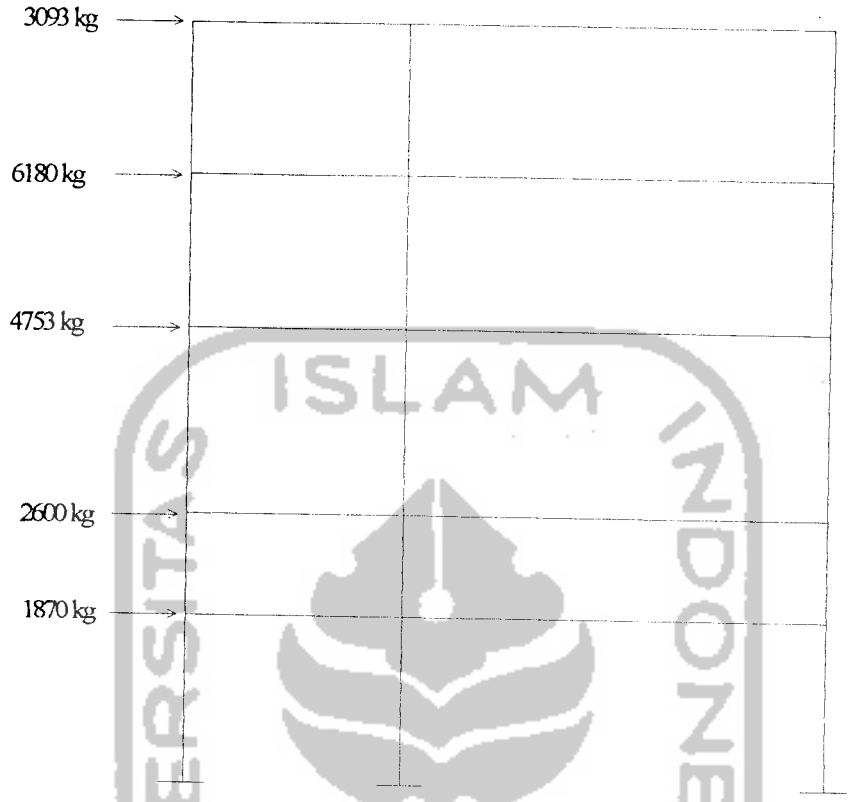
Vx,y = Gaya geser horisontal total akibat gempa arah x dan y

A, B = Panjang sisi bangunan dalam arah x dan y

**Tabel 4.1** Distribusi gaya geser dasar horisontal total akibat gempa ke sepanjang tinggi gedung dalam arah x dan y untuk tiap portal

| Tingkat | hi | Wi | Wi . hi | Fix, y | Untuk tiap portal | |
|---|---|---|---|---|---|---|
| | ( m ) | ( t ) | ( tm ) | Total ( t ) | 1/8 fix( t ) | 1/3 fiy( t ) |
| Atap | 20,8 | 98,16 | 2041,73 | 9,278 | 1,160 | 3,093 |
| III | 16,65 | 245,04 | 4079,92 | 18,541 | 2,318 | 6,180 |
| II | 12,4 | 253,06 | 3137,95 | 14,260 | 1,783 | 4,753 |
| I | 7,3 | 235,14 | 1716,52 | 7,800 | 0,975 | 2,600 |
| Basement | 4,5 | 273,80 | 1232,1 | 5,599 | 0,700 | 1,870 |
| | | Σ | 12208,22 | 55,478 | 6,936 | 18,496 |


Gambar 4.8 Distribusi beban gempa untuk portal arah x




Gambar 4.9 Distribusi beban gempa untuk portal arah y

Untuk perhitungan struktur portal akibat beban gempa selanjutnya menggunakan program SAP2000, data hasil perhitungan dapat dilihat pada lampiran 2.

**4.5.7** Waktu getar struktur dengan cara T Rayleigh

Dengan program analisa struktur SAP 2000 dapat dihitung besarnya deformasi lateral total akibat beban gempa tadi untuk portal arah x dan arah y.

Waktu getar struktur yang sebenarnya untuk tiap arah dapat dihitung berdasarkan besar simpangan tadi dengan rumus T rayleigh :

$$Tx = 6{,}3 \sqrt{\frac{\Sigma Wi \,.\, dix^2}{g\Sigma Fix.dix}} \quad \text{untuk portal arah x}$$

$$Ty = 6{,}3 \sqrt{\frac{\Sigma Wi \,.\, diy^2}{g\Sigma Fiy.diy}} \quad \text{untuk portal arah y}$$

Langkah ini ditabelkan seperti pada tabel 4.2 dan 4.3

**Tabel 4.2** Waktu getar bangunan dalam arah x (Tx)

| Tingkat | Wi ( t ) | dix ( cm ) | $dix^2$ ( $cm^2$ ) | Fix ( t ) | $Wi.dix^2$ ( $tcm^2$ ) | Fix.dix ( tcm ) |
|---|---|---|---|---|---|---|
| Atap | 98,16 | 0,3 | 0,09 | 9,278 | 8,834 | 2,783 |
| III | 245,04 | 0,25 | 0,0625 | 18,541 | 15,315 | 4,635 |
| II | 253,06 | 0,2 | 0,04 | 14,260 | 10,122 | 2,852 |
| I | 235,14 | 0,07 | 0,0049 | 7,80 | 1,152 | 0,546 |
| Base | 273,80 | 0,02 | 0,0004 | 5,599 | 0,109 | 0,112 |
| | | | | Σ | 35,532 | 10,883 |

$$Tx = 6{,}3 \sqrt{\frac{35{,}532}{981 \,.\, 10{,}883}} \quad \text{untuk portal arah x}$$

$$= 0{,}36 \text{ detik}$$

**Tabel 4.3** Waktu getar bangunan dalam arah y (Ty)

| Tingkat | Wi ( t ) | diy ( cm ) | $diy^2$ ( $cm^2$ ) | Fiy ( t ) | $Wi.diy^2$ ( $tcm^2$ ) | Fiy.diy ( tcm ) |
|---|---|---|---|---|---|---|
| Atap | 98,16 | 1,2 | 1,44 | 9,278 | 141,350 | 11,134 |
| III | 245,04 | 1,6 | 2,56 | 18,541 | 627,302 | 29,666 |
| II | 253,06 | 1,2 | 1,44 | 14,260 | 364,406 | 17,112 |
| I | 235,14 | 0,045 | 0,2025 | 7,80 | 47,616 | 3,51 |
| Base | 273,80 | 0,14 | 0,0196 | 5,599 | 5,366 | 0,784 |
| | | | | Σ | 1186,04 | 62,206 |

$$Ty = 6,3 \sqrt{\frac{1186,04}{981 \cdot 62,206}} \text{ untuk portal arah y}$$

$$= 0,88 \text{ detik}$$

**4.5.8** Distribusi akhir gaya geser dasar horisontal akibat gempa

Dengan cara T rayleigh diperoleh

Tx = 0,36 detik

Ty = 0,88 detik

Untuk Tx = 0,36 detik, zone 3 dan jenis tanah keras diperoleh C = 0,05

UntukTy = 0,88 detik, zone 3 jenis tanah keras diperoleh C = 0,039

Karena koefisien gempa dasar C untuk perhitungan periode bangunan dengan cara empiris berbeda dengan cara T rayleigh, maka perhitungan diulang dengan menggunakan koefisien gempa dasar C = 0,039.

Gaya geser akibat gempa :

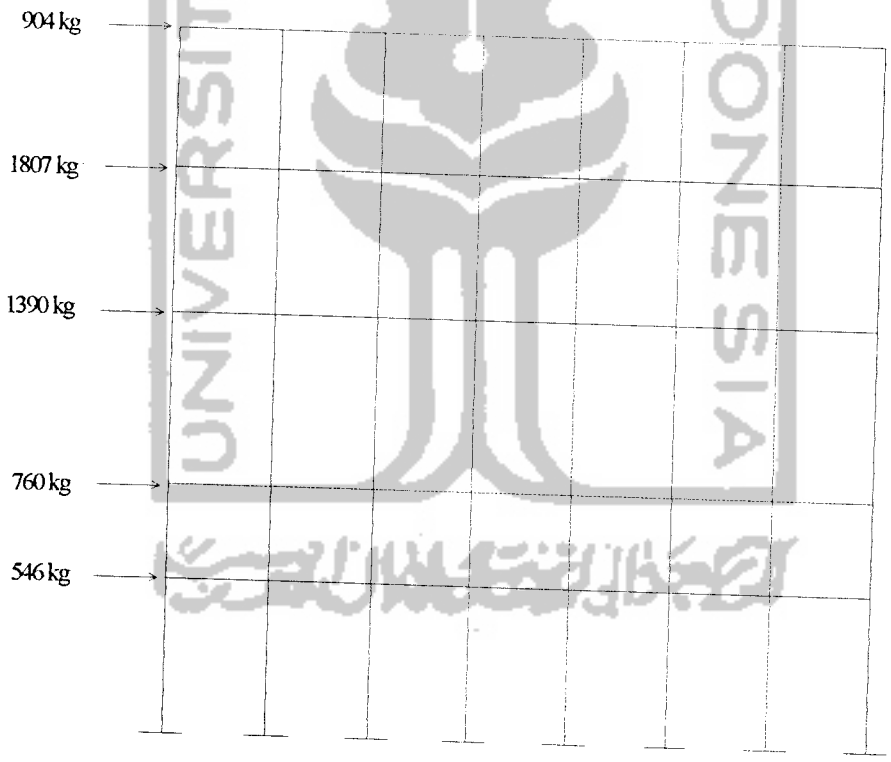
Vx = Vy = CIK.Wt

= 0,039 . 1,0 . 1,0 . 1109673,75 kg

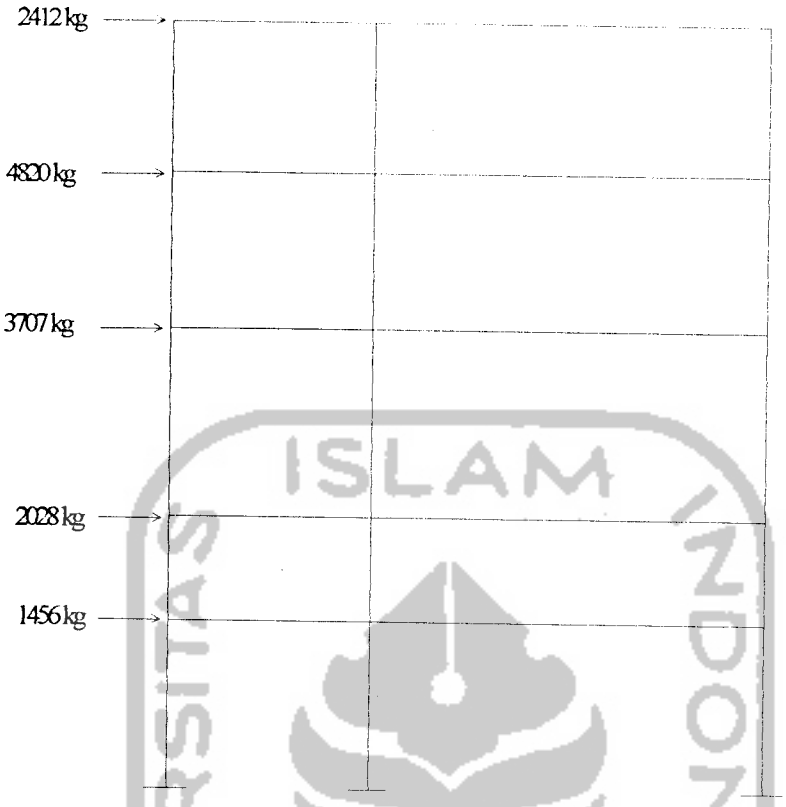
= 43277,276 kg

**Tabel 4.4** Distribusi gaya geser dasar horisontal total akibat gempa ke sepanjang tinggi gedung dalam arah x dan y untuk tiap portal

| Tingkat | hi ( m ) | Wi ( t ) | Wi . hi ( tm ) | Fix, y Total ( t ) | Untuk tiap portal | |
|---|---|---|---|---|---|---|
| | | | | | 1/8 fix( t ) | 1/3 fiy( t ) |
| Atap | 20,8 | 98,16 | 2041,73 | 7,237 | 0,904 | 2,412 |
| III | 16,65 | 245,04 | 4079,92 | 14,462 | 1,807 | 4,820 |
| II | 12,4 | 253,06 | 3137,95 | 11,123 | 1,390 | 3,707 |
| I | 7,3 | 235,14 | 1716,52 | 6,084 | 0,760 | 2,028 |
| Basement | 4,5 | 273,80 | 1232,1 | 4,367 | 0,546 | |
| | | Σ | 12208,22 | | | |




Gambar 4.10 Distribusi beban gempa untuk portal arah x



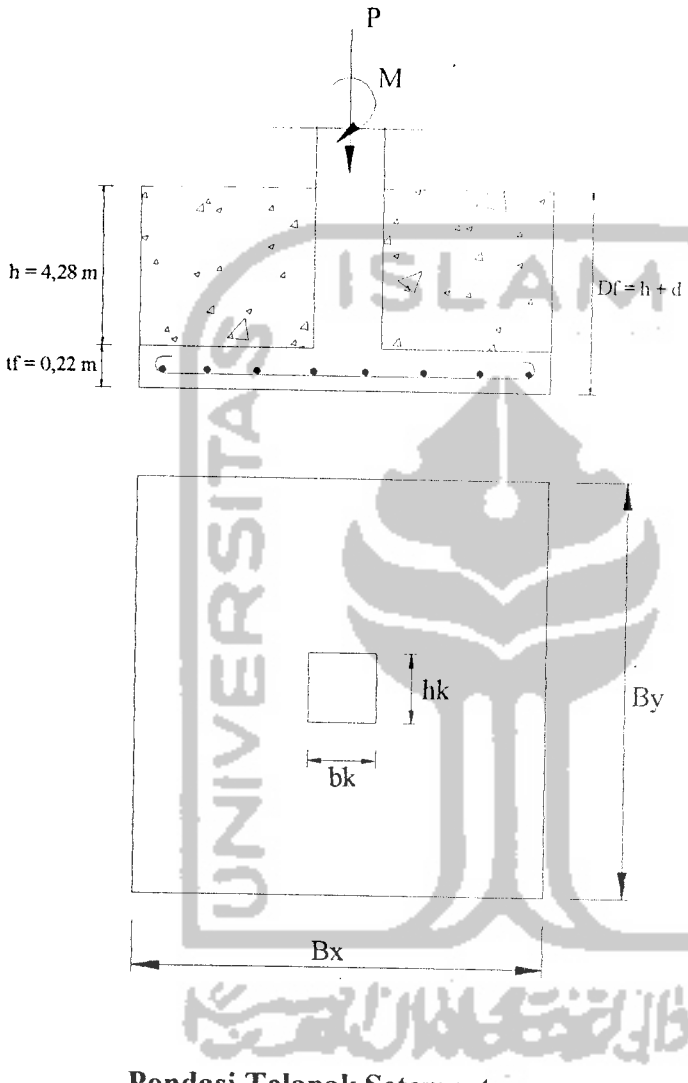

Gambar 4.11 Distribusi beban gempa untuk portal arah y

Untuk perhitungan struktur portal akibat beban gempa selanjutnya menggunakan program SAP2000, data hasil perhitungan dapat dilihat pada lampiran 2.

## 4.6 Perencanaan Pondasi

### 4.6.1 Perencanaan Pondasi Telapak Setempat ( As – A )

#### A. Perencanaan Dimensi Pondasi




**Pondasi Telapak Setempat**

Data tanah dan pondasi

$\sigma_{tanah}$ = 686,7 KN / m$^2$

f'c = 25 Mpa

$\gamma b_{tanah}$ = 17,44 KN / m$^2$

$\gamma b_{beton}$ = 24 KN / m$^2$

P = 519,25 KN

Mx = 7,92 KNm

My = 1.15 KNm

Asumsi tebal pelat ( tf ) = 220 mm = 0,22 m

Ukuran Kolom = 0,5 m x 0,6 m

$\sigma_{\text{netto tanah}} = \sigma_{\text{tanah}} - \Sigma ( h . \gamma_{\text{beton}} ) - \Sigma ( h . \gamma_{\text{tanah}} )$

$= 686,7 - ( 0,5 . 24 ) - ( 4,28 . 17,44 )$

$= 600,057 \text{ KN / m}^2$

Digunakan pondasi penampang bujur sangkar

Dicoba dengan ukuran Bx = By = 1,00 m

Luas penampang pelat pondasi :

A = Bx x By

= 1,0 x 1,0 = 1,0 $m^2$

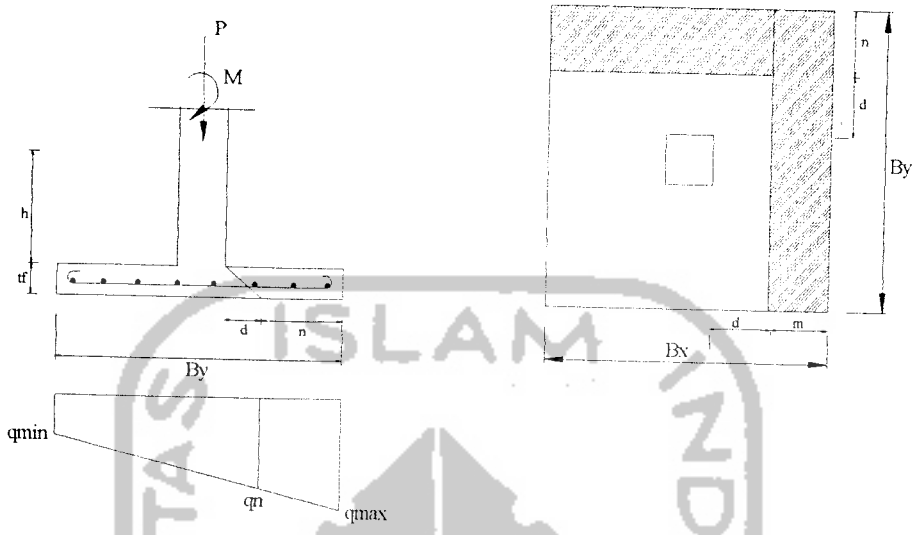
Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A} \pm \frac{6My}{Bx^2.By} \pm \frac{6Mx}{By^2.Bx}$$

$$= \frac{519,25}{1,0} + \frac{6 . 1,15}{1,0^3} + \frac{6 . 7,92}{1,0^3}$$

$= 573,67 \text{ KN/m}^2 < \sigma_{\text{netto tanah}} = 600,057 \text{ KN/m}^2$ **.....Ok!**

## B. Perencanaan Geser Satu Arah




**Pondasi dengan geser satu arah**

P = 519,25 KN

Mx = 7,92 KNm

My = 1,15 KNm

d = tf – Pb – ½ ∅ pokok

= 220 – 50 – 8 = 162 mm = 0,162 m

$m = \frac{Bx - bk - 2.d}{2} = \frac{1 - 0{,}5 - 2.\ 0{,}162}{2}$

= 0,088 m

$n = \frac{Bx - hk - 2.d}{2} = \frac{1 - 0{,}6 - 2.\ 0{,}162}{2}$

= 0,038 m

Tegangan kontak yang terjadi :

$$q_{terjadi} = \frac{P}{A} \pm \frac{6 . Mx}{By^2 . Bx}$$

$$= \frac{519,25}{1} \pm \frac{6 . 7,92}{1^3}$$

$q_{max} = 566,77 \text{ KN/m}^2$

$q_{min} = 471,73 \text{ KN/m}^2$

$$q_{tjd\ n} = \frac{(q_{max} - q_{min}) . (By - n)}{By} + q_{min}$$

$$= \frac{(566,77 - 471,73) . (1 - 0,038)}{1} + 471,73$$

$= 563,158 \text{ KN/m}^2$

$q_{tjd\ n} = \frac{1}{2} (q_{max} + q_{min})$

$= \frac{1}{2} (566,77 + 471,73) = 519,25 \text{ KN/m}^2$

diambil yang terbesar $q_{tjd\ n} = 563,158 \text{ KN/m}^2$

Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$Vu = q_{tjd\ n} . n . Bx$

$= 563,158 . 0,076 . 1 = 42,80 \text{ KN}$

$Vu/\Phi = 42,80 / 0,6 = 71,33 \text{ KN}$

Kekuatan beton menahan gaya geser

$Vc = 1/6 . \sqrt{f'c} . Bx . d$

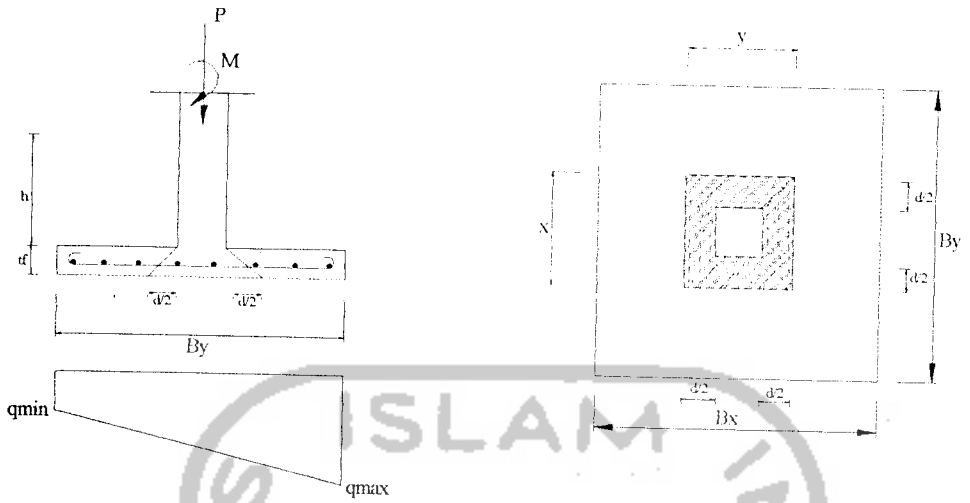
$= 1/6 . \sqrt{25} . 1 . 0,162 . 10^3$

$= 135 \text{ KN}$

Kontrol gaya geser

$Vc = 135 \text{ KN} \geq Vu/\Phi = 71,33 \text{ KN}$**.....Ok!**

## C. Perencanaan Geser Dua Arah




**Pondasi dengan geser dua arah**

X = bk + d = 500 + 162 = 662 mm = 0,662 m

Y = hk + d = 600 + 162 = 762 mm = 0,762 m

Tegangan kontak yang terjadi :

$$q_{tjdn} = \frac{P}{A} \pm \frac{6 \,.\, My}{Bx^2 . By} \pm \frac{6 \,.\, Mx}{By^2 . Bx}$$

$$= \frac{519,25}{1^2} \pm \frac{6 \,.\, 1,15}{1^3} \pm \frac{6 \,.\, 7,92}{1^3}$$

$q_{max} = 573,67 \text{ KN/m}^2$

$q_{min} = 464,83 \text{ KN/m}^2$

$q_{pakai} = ½ \, (q_{max} + q_{min})$

$= 519,25 \text{ KN/m}^2$

Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$$Vu = q_{pakai} \left[ (Bx \,.\, By) - (x \,.\, y) \right]$$

$$= 519,25 \,. \left[ (1.1) - (0,662 \,.\, 0,762) \right] = 257,317 \text{ KN}$$

$Vu/\Phi = 257,317 / 0,6 = 428,862 \text{ KN}$

Kekuatan beton menahan gaya geser :

$\beta c = \frac{\text{sisi panjang}}{\text{sisi pendek}} = \frac{y}{x} = \frac{0,762}{0,662} = 1,15$

$bo = 2 ( x + y ) = 2 ( 0,662 + 0,762 ) = 2,848 = 2848$ mm

$Vc_1 = ( 1 + 2/\beta c ) . ( 2 . \sqrt{f'c} ) . bo . d$

$= ( 1 + 2/1,15 ) . ( 2.5 ) . 2848 .162 . 10^{-3} = 12,637$ KN

$Vc_2 = 4 . \sqrt{f'c} . bo . d$

$= 4 . 5 . 2848 . 162 . 10^{-3} = 9227,52$ KN

Kontrol gaya geser :

$Vc = 9227,52$ KN $\geq Vu/\Phi = 428,862$ KN **.....Ok!**

**D. Kuat Tumpuan Pondasi**

Kuat tumpuan pondasi:

$\Phi.Pn = \Phi . ( 0,85 . f'c . A_1 . \sqrt{A_2/A_1} )$

$A_1$ = Luas penampang kolom = bk . hk = 0,50 x 0,60 = 0,30 $m^2$

$A_2$ = Luas penampang pondasi = Bx . By = 1,0 x 1,0 = 1,00 $m^2$

$\sqrt{A_2/A_1} = \sqrt{1,0/0,3} = 3,33 > 2$ ( dipakai nilai 2 )

$\Phi.Pn = \Phi . ( 0,85 . f'c . A_1 . 2 )$

$= 0,7 ( 0,85 . 25 . 0,3 . 2 ) . 10^3 = 8925$ KN

Kuat tumpuan kolom

$\Phi.Pn = \Phi . ( 0,85 . f'c . A_1 )$

$= 0,7 ( 0,85 . 25 . 0,3 ) . 10^3 = 4462,5$ KN

Kontrol kuat tumpuan

$\Phi Pn_{pondasi} > \Phi Pn_{kolom}$

8925 KN > 4462,5 KN **.....Ok!**

## E. Perencanaan Tulangan Lentur Pondasi

Karena penampang pondasi berbentuk bujur sangkar, sehingga arah x dan arah y sama panjang, maka perencanaan tulangan lenturnya dianggap sama.

$L = \frac{Bx - bk}{2} = \frac{1,0 - 0,5}{2} = 0,25$ m

$q_{terjadi} = 573,67$ KN/m$^2$

$Mu = 0,5\ q\ .\ L^2 = 0,5\ .\ 573,67\ .\ 0,25^2 = 17,927$ KNm

$Mu/\Phi = 17,927/0,8 = 22,408$ KNm

Digunakan tulangan pokok D16 mm, maka AD16 = 201,1 mm$^2$

Tebal pelat pondasi ( tf ) = 220 mm, selimut beton = 50 mm

$d = 220 - 50 - \frac{1}{2}.16 = 162$ mm

Syarat – syarat batas

$\rho_{min} = \frac{1,4}{fy}$

$= \frac{1,4}{390} = 0,0035$

$\rho_b = \frac{0,85\ .\ f'c}{fy}\ .\ \beta\ .\ \frac{600}{600 + fy}$

$= \frac{0,85\ .\ 25}{390}\ .\ 0,85\ .\ \frac{600}{600 + 390}$

$= 0,0280$

$\rho_{maks} = 0,75\ .\ \rho_b$

$= 0,75\ .\ 0,0280$

$= 0,0210$

$m = \frac{fy}{0,85\ .\ f'c}$

$= \frac{390}{0,85\ .\ 25}$

$= 19,5$

$$Rn = \frac{Mn/\varnothing}{b \cdot d^2}$$

$$= \frac{22{,}408 \cdot 10^6}{1000 \cdot 162^2}$$

$$= 0{,}853 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{19{,}5} \cdot \left( 1 - \sqrt{1 - \frac{2.19{,}5 \cdot 0{,}853}{390}} \right)$$

$$= 0{,}002 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0035$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0035 \cdot 1000 \cdot 162$$

$$= 567 \text{ mm}^2$$

$0{,}002 \cdot b \cdot h = 0{,}002 \cdot 1000 \cdot 600 = 1200 \text{ mm}^2$

Dipakai As $= 1200 \text{ mm}^2$

$$S \leq \frac{A\varnothing 16 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{201{,}1 \cdot 1000}{1200} \leq 167{,}58 \text{ mm}$$

Dipakai tulangan pokok $\varnothing 16 - 160$

$$As_{ada} = \frac{A\varnothing 16 \cdot 1000}{160}$$

$$= 1256 \text{ mm}^2$$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{1256 \cdot 390}{0{,}85 . 25 . 1000}$$

$$= 23{,}067 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 1256 \cdot 390 \cdot (162 - 23{,}067/2)$$

$$= 73{,}70 \text{ KN.m} > Mu/\Phi = 22{,}408 \text{ KNm} \ldots\ldots\ldots \textbf{Ok!}$$

**F. Perencanaan Tulangan Susut Pondasi**

$$As_{susut} = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 600$$

$$= 1200 \text{ mm}$$

Dipakai tulangan ∅10 mm, A∅10 = 78,5 $mm^2$

Jarak antar tulangan

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{susut}}$$

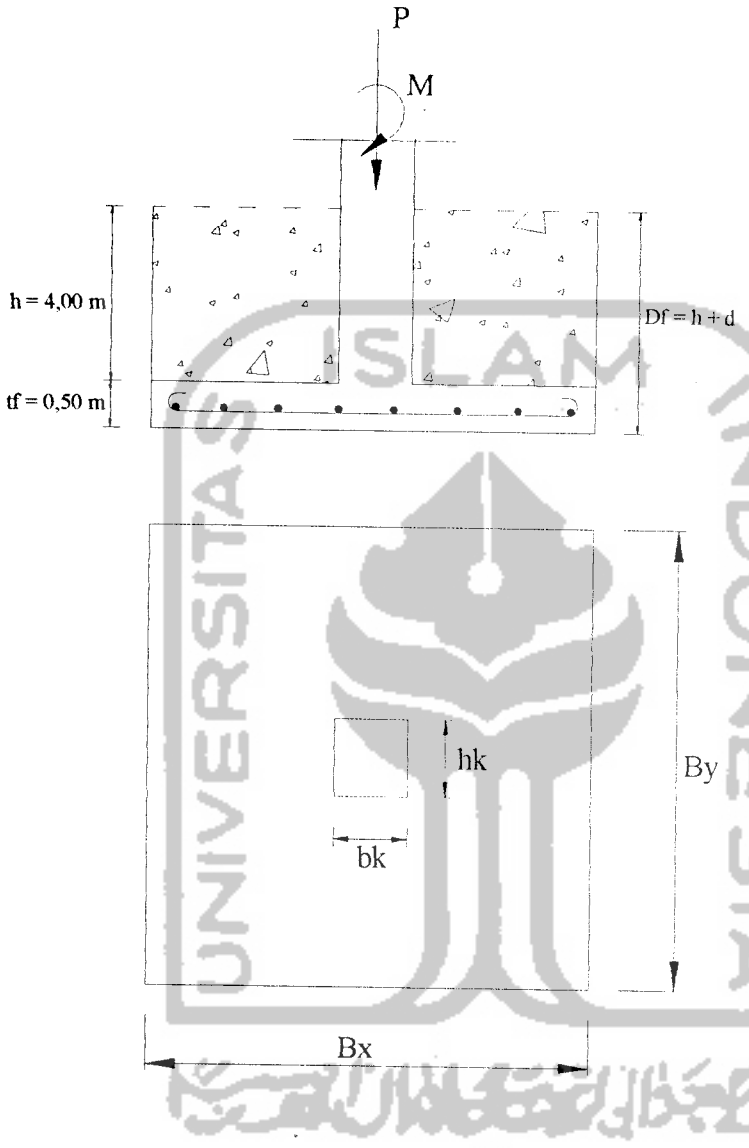
$$\leq \frac{78{,}50 \cdot 1000}{1200}$$

$$\leq 65{,}4 \text{ mm}$$

Dipakai tulangan susut ∅10 - 60

### 4.6.2 Perencanaan Pondasi Telapak Setempat ( As – C )

#### A. Perencanaan Dimensi Pondasi




**Pondasi Telapak Setempat**

Data tanah dan pondasi

$\sigma_{tanah}$ = 686,7 KN / $m^2$

f'c = 25 Mpa

$\gamma b_{tanah}$ = 17,44 KN / $m^2$

$\gamma b_{beton}$ = 24 KN / $m^2$

P = 1451,25 KN

Mx = 11,7 KNm

My = 1.51 KNm

Asumsi tebal pelat ( tf ) = 500 mm = 0,50 m

Ukuran Kolom = 0,5 m x 0,7 m

$\sigma_{\text{netto tanah}} = \sigma_{\text{tanah}} - \Sigma ( h . \gamma_{\text{beton}} ) - \Sigma ( h . \gamma_{\text{tanah}} )$

$= 686,7 - ( 0,5 . 24 ) - ( 4,50 . 17,44 )$

$= 596,22 \text{ KN / m}^2$

Digunakan pondasi penampang bujur sangkar

Dicoba dengan ukuran Bx = By = 2,00 m

Luas penampang pelat pondasi :

A = Bx x By

= 2,0 x 2,0 = 4,0 $\text{m}^2$

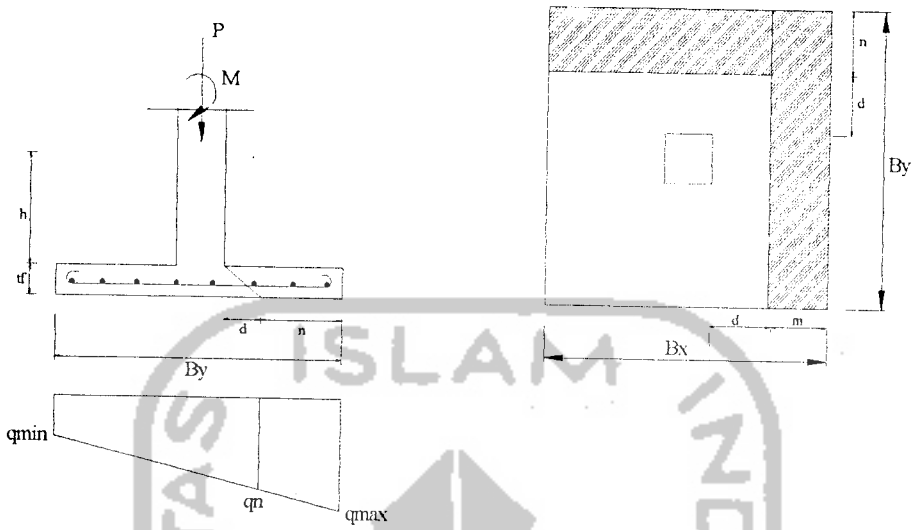
Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A} \pm \frac{6My}{Bx^2.By} \pm \frac{6Mx}{By^2.Bx}$$

$$= \frac{1451,52}{4,0} + \frac{6 . 1,51}{8,0} + \frac{6 . 11,7}{8,0}$$

$= 352,97 \text{ KN/m}^2 < \sigma_{\text{netto tanah}} = 596,22 \text{ KN/m}^2$ **.....Ok!**

## B. Perencanaan Geser Satu Arah




**Pondasi dengan geser satu arah**

P = 1451,25 KN

Mx = 11,71 KNm

My = 1,51 KNm

d = tf – Pb – ½ ∅ pokok

= 500 – 50 – 8 = 442 mm = 0,442 m

$m = \frac{Bx - bk - 2.d}{2} = \frac{1 - 0,5 - 2.0,442}{2}$

= 0,308 m

$n = \frac{Bx - hk - 2.d}{2} = \frac{1 - 0,7 - 2.0,442}{2}$

= 0,208 m

Tegangan kontak yang terjadi :

$$q_{terjadi} = \frac{P}{A} \pm \frac{6 \cdot Mx}{By^2 \cdot Bx}$$

$$= \frac{1451}{4} \pm \frac{6 \cdot 11,71}{2^3}$$

$q_{max} = 371,525 \text{ KN/m}^2$

$q_{min} = 353,975 \text{ KN/m}^2$

$$q_{tjd\,n} = \frac{(q_{max} - q_{min}) \cdot (By - n)}{By} + q_{min}$$

$$= \frac{(371,525 - 353,975) \cdot (2 - 0,208)}{2} + 353,975$$

$= 430,975 \text{ KN/m}^2$

$q_{tjd\,n} = \frac{1}{2} (q_{max} + q_{min})$

$= \frac{1}{2} (371,525 + 353,975) = 362,75 \text{ KN/m}^2$

diambil yang terbesar $q_{tjd\,n} = 430,975 \text{ KN/m}^2$

Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$Vu = q_{tjd\,n} \cdot n \cdot Bx$

$= 430,975 \cdot 0,208 \cdot 1 = 89,64 \text{ KN}$

$Vu/\Phi = 89,64 / 0,6 = 149,40 \text{ KN}$

Kekuatan beton menahan gaya geser

$Vc = 1/6 \cdot \sqrt{f'c} \cdot Bx \cdot d$

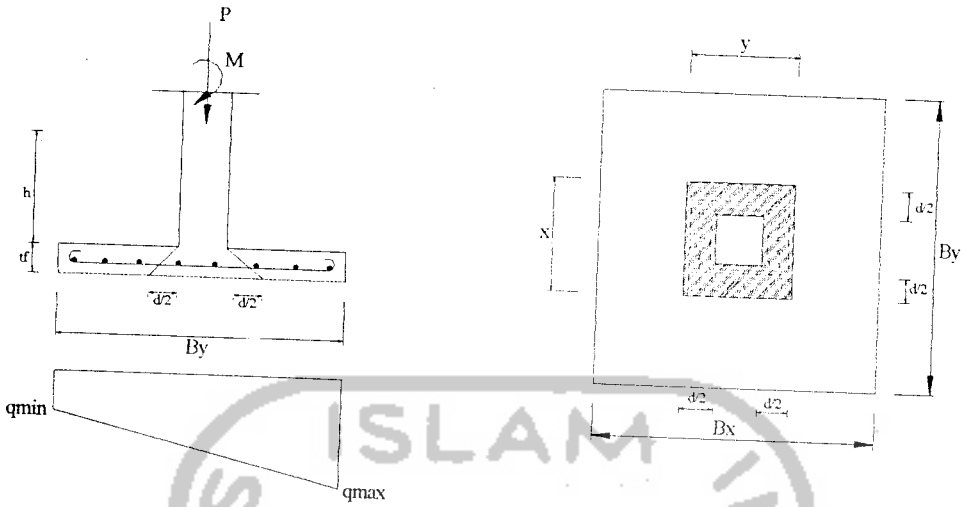
$= 1/6 \cdot \sqrt{25} \cdot 2 \cdot 0,442 \cdot 10^3$

$= 736,67 \text{ KN}$

Kontrol gaya geser

$Vc = 736,67 \text{ KN} \geq Vu/\Phi = 149,40 \text{ KN}$**.....Ok!**

## C. Perencanaan Geser Dua Arah




**Pondasi dengan geser dua arah**

X = bk + d = 500 + 442 = 942mm = 0,942 m

Y = hk + d = 700 + 442 = 1142 m = 1,142 m

Tegangan kontak yang terjadi :

$$q_{tjd\,n} = \frac{P}{A} \pm \frac{6\,.\,My}{Bx^2.By} \pm \frac{6\,.\,Mx}{By^2.Bx}$$

$$= \frac{1451,25}{2^2} \pm \frac{6\,.\,1,51}{2^3} \pm \frac{6\,.\,11,71}{2^3}$$

$q_{max}$ = 372,72 KN/m$^2$

$q_{min}$ = 352,89 KN/m$^2$

$q_{pakai} = \frac{1}{2}\,(q_{max} + q_{min})$

= 362,81 KN/m$^2$

Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$$Vu = q_{pakai}\left[(Bx\,.\,By) - (x\,.\,y)\right]$$

$$= 1451,25\left[(2.2) - (0,942\,.\,1,142)\right] = 1060,94KN$$

$Vu/\Phi$ = 1060,94/ 0,6 = 1768,236 KN

Kekuatan beton menahan gaya geser :

$\beta c = \frac{\text{sisi panjang}}{\text{sisi pendek}} = \frac{y}{x} = \frac{1,142}{0,942} = 1,21$

$bo = 2 ( x + y ) = 2 ( 0,942 + 1,142 ) = 4,168 = 4168$ mm

$Vc_1 = ( 1 + 2/\beta c ) . ( 2 . \sqrt{f'c} ) . bo . d$

$= ( 1 + 2/1,21 ) . ( 2.5 ) . 4168 .442 . 10^{-3} = 48873,07$ KN

$Vc_2 = 4 . \sqrt{f'c} . bo . d$

$= 4 . 5 . 4168 . 442 . 10^{-3} = 36845,12$ KN

Kontrol gaya geser :

$Vc = 36845,12$ KN $\geq Vu/\Phi = 1768,236$ KN .....**Ok!**

**D. Kuat Tumpuan Pondasi**

Kuat tumpuan pondasi:

$\Phi.Pn = \Phi. ( 0,85 . f'c . A_1 . \sqrt{A_2/A_1} )$

$A_1$ = Luas penampang kolom = bk . hk = 0,50 x 0,70 = 0,35 $m^2$

$A_2$ = Luas penampang pondasi = Bx . By = 2,0 x 2,0 = 4,00 $m^2$

$\sqrt{A_2/A_1} = \sqrt{4,0/0,35} = 3,38 > 2$ ( dipakai nilai 2 )

$\Phi.Pn = \Phi. ( 0,85 . f'c . A_1 . 2 )$

$= 0,7 ( 0,85 . 25 . 0,35 . 2 ) . 10^3 = 10412,5$ KN

Kuat tumpuan kolom

$\Phi.Pn = \Phi. ( 0,85 . f'c . A_1 )$

$= 0,7 ( 0,85 . 25 . 0,35 ) . 10^3 = 5206,25$ KN

Kontrol kuat tumpuan

$\Phi Pn_{pondasi} > \Phi Pn_{kolom}$

10412,5 KN > 5206,25 KN .....**Ok!**

## E. Perencanaan Tulangan Lentur Pondasi

Karena penampang pondasi berbentuk bujur sangkar, sehingga arah x dan arah y sama panjang, maka perencanaan tulangan lenturnya dianggap sama.

$L = \frac{Bx - bk}{3} = \frac{2,0 - 0,5}{2} = 0,75$ m

$q_{terjadi} = 372,725$ KN/m$^2$

$Mu = 0,5\ q\ .\ L^2 = 0,5\ .\ 372,725\ .\ 0,75^2 = 104,83$ KNm

$Mu/\Phi = 104,725/0,8 = 131,036$ KNm

Digunakan tulangan pokok D16 mm, maka AD16 = 201,1 mm$^2$

Tebal pelat pondasi ( tf ) = 500 mm, selimut beton = 50 mm

$d = 500 - 50 - \frac{1}{2}.16 = 442$ mm

Syarat – syarat batas

$\rho_{min} = \frac{1,4}{fy}$

$= \frac{1,4}{390} = 0,0035$

$\rho_b = \frac{0,85\ .\ f'c}{fy}\ .\ \beta\ .\ \frac{600}{600 + fy}$

$= \frac{0,85\ .\ 25}{390}\ .\ 0,85\ .\ \frac{600}{600 + 390}$

$= 0,0280$

$\rho_{maks} = 0,75\ .\ \rho_b$

$= 0,75\ .\ 0,0280$

$= 0,0210$

$m = \frac{fy}{0,85\ .\ f'c}$

$= \frac{390}{0,85\ .\ 25}$

$= 19,5$

$$Rn = \frac{Mn/\varnothing}{b \cdot d^2}$$

$$= \frac{131{,}036 \cdot 10^6}{1000 \cdot 162^2}$$

$$= 0{,}6707 \text{ Mpa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left( 1 - \sqrt{1 - \frac{2m \cdot Rn}{fy}} \right)$$

$$= \frac{1}{19{,}5} \cdot \left( 1 - \sqrt{1 - \frac{2.19{,}5 \cdot 0{,}6707}{390}} \right)$$

$$= 0{,}0017 < \rho_{min}$$

dipakai $\rho_{min} = 0{,}0035$

$$As_{perlu} = \rho \cdot b \cdot d$$

$$= 0{,}0035 \cdot 1000 \cdot 442$$

$$= 1547 \text{ mm}^2$$

$0{,}002 \cdot b \cdot h = 0{,}002 \cdot 1000 \cdot 700 = 1400 \text{ mm}^2$

Dipakai As $= 1547 \text{ mm}^2$

$$S \leq \frac{A\varnothing 16 \cdot 1000}{As_{perlu}}$$

$$\leq \frac{201{,}1 \cdot 1000}{1547} \leq 129{,}9 \text{ mm}$$

Dipakai tulangan pokok ∅16 – 120

$$As_{ada} = \frac{A\varnothing 16 \cdot 1000}{120}$$

$$= 1675{,}833 \text{ mm}^2$$

Kontrol kapasitas tampang

$$0{,}85 \cdot f'c \cdot b \cdot a = As_{ada} \cdot fy$$

$$a = \frac{As_{ada} \cdot fy}{0{,}85 f'c \cdot b}$$

$$= \frac{1675{,}833 \cdot 390}{0{,}85 . 25 . 1000}$$

$$= 30{,}75 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot (d - a/2)$$

$$= 1675{,}8333 \cdot 390 \cdot (442 - 30{,}75/2)$$

$$= 278{,}83 \text{ KN.m} > Mu/\Phi = 131{,}036 \text{ KNm}\textbf{.....Ok!}$$

**F. Perencanaan Tulangan Susut Pondasi**

$$As_{susut} = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 700$$

$$= 1400 \text{ mm}$$

Dipakai tulangan ∅10 mm, A∅10 = 78,5 $mm^2$

Jarak antar tulangan

$$S \leq \frac{A\varnothing 10 \cdot 1000}{As_{susut}}$$

$$\leq \frac{78{,}50 \cdot 1000}{1400}$$

$$\leq 56{,}07 \text{mm}$$

Dipakai tulangan susut ∅10 - 50

## 4.7 Perencanaan Tulangan Geser

Balok Sloof ( 250 x 500 )

d = h – pb – Ds – ½ ∅ pokok

= 500 – 20 – 10 – ½ . 16

= 462 mm

$Vu_{maks}$ = 183,63 KN

$Vu/\phi$ = 183,63 / 0,6

= 306, 05 KN

Vc = 1/6 . $\sqrt{f'c}$ . bw . d

= 1/6 . 5 . 250 . 462

= 96,25 KN

$Vu / \phi$ > Vc ..... perlu tulangan geser !

$Vs_{perlu}$ = $Vu/\phi$ - Vc

= 306,05 – 96,25

= 209,8 KN

Dipakai sengkang ∅10mm, Av = 100,6 $mm^2$

Jarak sengkang :

$$S \leq \frac{Av \cdot fy \cdot d}{Vs}$$

$$\leq \frac{100,6 \cdot 240 \cdot 462 \cdot 10^{-3}}{209,8}$$

≤ 82,97 mm

Dipakai sengkang ∅10 – 80

Balok B1 ( 300 x 700 )

d = h – pb – Ds – ½ ∅ pokok

= 700 – 20 – 10 – ½ . 16

= 662 mm

$Vu_{maks}$ = 141,76 KN

Vu/ϕ = 141,76 / 0,6

= 236,27 KN

Vc = 1/6 . √f'c . bw . d

= 1/6 . 5 . 300 . 664

= 166 KN

Vu / ϕ > Vc ..... perlu tulangan geser !

$Vs_{perlu}$ = Vu/ϕ - Vc

= 236,27 – 166

= 70,26 KN

Dipakai sengkang ∅10mm, Av = 157 $mm^2$

Jarak sengkang :

$$S \leq \frac{Av \cdot fy \cdot d}{Vs}$$

$$\leq \frac{157 \cdot 240 \cdot 664 \cdot 10^{-3}}{70,26}$$

≤ 355,02 mm

Dipakai sengkang ∅10 – 300

Balok B2 ( 300 x 600 )

d = h – pb – Ds – ½ ∅ pokok

= 600 – 20 – 10 – ½ . 16

= 562 mm

$Vu_{maks}$ = 126,32 KN

$Vu/\phi$ = 126,32 / 0,6

= 210,53 KN

Vc = 1/6 . $\sqrt{f'c}$ . bw . d

= 1/6 . 5 . 300 . 564

= 141 KN

$Vu / \phi$ > Vc ..... perlu tulangan geser !

$Vs_{perlu}$ = $Vu/\phi$ - Vc

= 210,53 – 141

= 69,53 KN

Dipakai sengkang ∅10mm, Av = 157 $mm^2$

Jarak sengkang :

$$S \leq \frac{Av \cdot fy \cdot d}{Vs}$$

$$\leq \frac{157 \cdot 240 \cdot 564 \cdot 10^{-3}}{69,53} \leq 304,56 \text{ mm}$$

≤ 304,56 mm

≤ ½.d

≤ ½.562 = 281 mm

Dipakai sengkang ∅10 – 250

Balok B3 ( 300 x 500 )

d = h – pb – Ds – ½ ∅ pokok

= 500 – 20 – 10 – ½ . 16

= 462 mm

$Vu_{maks}$ = 74,64 KN

Vu/ϕ = 74,64 / 0,6

= 124,4 KN

Vc = 1/6 . √f'c . bw . d

= 1/6 . 5 . 300 . 464

= 116 KN

Vu / ϕ > Vc ..... perlu tulangan geser !

$Vs_{perlu}$ = Vu/ϕ - Vc

= 124,4 – 116

= 8,4 KN

Dipakai sengkang ∅10mm, Av = 157 $mm^2$

Jarak sengkang :

$$S \leq \frac{Av \cdot fy \cdot d}{Vs}$$

$$\leq \frac{157 \cdot 240 \cdot 464 \cdot 10^{-3}}{8,4}$$

≤ 2081,37 mm

≤ d/2 = 462 /2

≤ 231 mm

Dipakai sengkang ∅8 – 200

Balok B4 ( 300 x 400 )

d = h – pb – Ds – ½ ∅ pokok

= 400 – 20 – 10 – ½ . 16

= 362 mm

$Vu_{maks}$ = 44,21 KN

Vu/ϕ = 44,21 / 0,6

= 73,68 KN

Vc = 1/6 . √f'c . bw . d

= 1/6 . 5 . 300 . 364

= 91 KN

Vu / ϕ < Vc ..... tidak perlu tulangan geser !

Dipakai sengkang minimum

Dipakai sengkang ∅10mm, Av = 157 $mm^2$

Jarak sengkang :

$$S \leq \frac{Av \,.\, fy \,.\, 3}{bw}$$

$$\leq \frac{157 \,.\, 240 \,.\, 3.}{300}$$

≤ 376,8 mm

≤ d/2 = 364 /2

≤ 182 mm

Dipakai sengkang ∅10 – 150

**Tabel. Rekapitulasi Tulangan Balok Terpasang**

| Portal | Lantai | Balok | Dimensi | Tumpuan | | | Lapangan | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Tul. Atas | Tul. Bawah | Geser | Tul. Atas | Tul. Bawah | Geser |
| As - 1 | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10 -150 | 2D16 | 2D16 | ∅10-300 |
| | | S1 | 250 x 250 | 5D16 | 3D16 | ∅10-200 | 2D16 | 3D16 | ∅10-400 |
| | Lantai 1 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 3D16 | 3D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 2 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 3D16 | 2D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 3 | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 4D16 | 3D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Atap | R1 | 250 x 500 | 3D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-300 |
| As - 2 | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-300 |
| | | S3 | 250 x 250 | 2D16+6D19 | 5D16 | ∅10-200 | 3D16 | 5D16 | ∅10-400 |
| | Lantai 1 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 4D16 | 2D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 2 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 4D16 | 2D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 3 | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | B2 | 300 x 600 | 5D16 | 4D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Atap | R1 | 250 x 500 | 3D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-400 |
| | | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-300 |

| Portal | Lantai | Balok | Dimensi | Tumpuan | | | Lapangan | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Tul. Atas | Tul. Bawah | Geser | Tul. Atas | Tul. Bawah | Geser |
| As - 3 | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-80 | 2D16 | 2D16 | ∅10-150 |
| | | S3 | 250 x 500 | 2D16+6D19 | 5D16 | ∅10-80 | 3D16 | 5D16 | ∅10-150 |
| | Lantai 1 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B2 | 300 x 600 | 4D16 | 2D16 | ∅10-250 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 2 | B3 | 300 x 500 | 3D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B1 | 300 x 700 | 4D16 | 2D16 | ∅10-300 | 2D16 | 2D16 | ∅10-350 |
| | | B8 | 200X900 | 4D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-300 |
| | | KS | 200X300 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-150 |
| | Lantai 3 | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B1 | 300 x 700 | 4D16 | 3D16 | ∅10-300 | 2D16 | 2D16 | ∅10-350 |
| | Atap | R1 | 250 x 500 | 3D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-250 |
| | | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-200 |
| As - 4 | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-80 | 2D16 | 2D16 | ∅10-150 |
| | | S3 | 250 x 500 | 2D16+6D19 | 5D16 | ∅10-80 | 3D16 | 5D16 | ∅10-150 |
| | Lantai 1 | B3 | 300 x 500 | 2D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B1 | 300 x 700 | 4D16 | 2D16 | ∅10-300 | 2D16 | 2D16 | ∅10-350 |
| | Lantai 2 | B3 | 300 x 500 | 3D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B8 | 200 x 900 | 5D16 | 3D16 | ∅10-200 | 2D16 | 2D16 | ∅10-300 |
| | | KS | 200X300 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-150 |
| | Lantai 3 | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | | B1 | 300 x 700 | 5D16 | 3D16 | ∅10-300 | 2D16 | 2D16 | ∅10-350 |

| Portal | Lantai | Balok | Dimensi | Tumpuan | | | Lapangan | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Tul. Atas | Tul. Bawah | Geser | Tul. Atas | Tul. Bawah | Geser |
| As - A | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-80 | 2D16 | 2D16 | ∅10-150 |
| | Lantai 1 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Lantai 2 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Lantai 3 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Atap | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-200 |
| As - B | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-80 | 2D16 | 2D16 | ∅10-150 |
| | Lantai 1 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Lantai 2 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Lantai 3 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Atap | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-200 |
| As – C | Lantai 1 | B5 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-200 |
| | Lantai 2 | B5 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-200 |
| | Lantai 3 | B5 | 200 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-200 |
| | Atap | R3 | 250 x 400 | 2D16 | 2D16 | ∅10-150 | 2D16 | 2D16 | ∅10-200 |
| As – D | Basement | S2 | 250 x 400 | 2D16 | 2D16 | ∅10-80 | 2D16 | 2D16 | ∅10-150 |
| | Lantai 1 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | Lantai 2 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | Lantai 3 | B4 | 300 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-120 |
| | | B3 | 300 x 500 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-230 |
| | Atap | R2 | 250 x 400 | 2D16 | 2D16 | ∅10-100 | 2D16 | 2D16 | ∅10-200 |
| As - E | Lantai 1 | B6 | 150 x 600 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 2 | B6 | 150 x 600 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-300 |
| | Lantai 3 | B6 | 150 x 600 | 2D16 | 2D16 | ∅10-200 | 2D16 | 2D16 | ∅10-300 |

**Rekapitulasi Tulangan Kolom Terpasang**

| Kolom | Dimensi (mm) | Tulangan Pokok (mm) | Sengkang (mm) |
|---|---|---|---|
| K1 – 0 | 500 x 600 | 7D25 | ∅10 - 400 |
| K1 – 1 | 500 x 600 | 7D25 | ∅10 - 400 |
| K1 – 2 | 500 x 600 | 7D25 | ∅10 - 400 |
| K1 – 3 | 500 x 500 | 6D25 | ∅10 - 400 |
| K1 – 4 | 500 x 500 | 6D25 | ∅10 - 400 |
| K2 – 0 | 500 x 700 | 8D25 | ∅10 - 400 |
| K2 – 1 | 500 x 700 | 8D25 | ∅10 - 400 |
| K2 – 2 | 500 x 700 | 8D25 | ∅10 - 400 |
| K2 – 3 | 500 x 600 | 6D25 | ∅10 - 400 |
| K2 – 4 | 500 x 600 | 6D25 | ∅10 - 400 |

**Rekapitulasi Tulangan Pondasi Telapak Setempat**

| Pondasi | Dimensi (mm) | Tulangan Terpasang | | | |
|---|---|---|---|---|---|
| | | Arah - x | | Arah - y | |
| | | Tul Pokok | Tul Susut | Tul Pokok | Tul susut |
| As – A | 1000 x 1000 x 220 | D16 - 160 | ∅10 - 60 | D16 - 160 | ∅10 - 60 |
| As – B | 2000 x 2000 x 500 | D16 - 120 | ∅10 – 50 | D16 - 120 | ∅10 – 50 |
| As – E | 2000 x 2000 x 500 | D16 - 120 | ∅10 - 50 | D16 - 120 | ∅10 - 50 |

## 4.8 Perencanaan Tangga

1. Spesifikasi tangga

   a. Tinggi lantai ( h ) = 5,10 m

   b. Lebar bordes ( lb ) = 2,00 m

   c. Tinggi optrede diambil = 17,0 cm

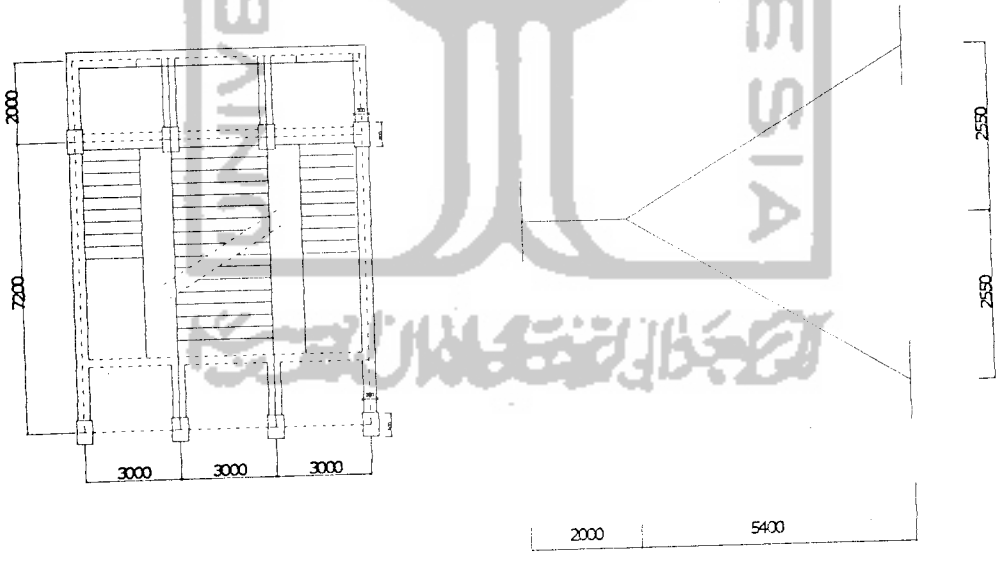
   d. Jumlah optrede = 510 / 17 ≈ 30 buah

   e. Dipakai lebar antrede = 30 cm

2. Dimensi Tangga

   - Tebal pelat tangga = 120 cm
   - Sudut miring tangga $= tg\alpha = \frac{5,1 / 2}{5,4} = 0,47$

     $\alpha = 25,17^0$




**Gambar. Dimensi Tangga**

3. Analisis Pembebanan pada tangga

Beban mati

- Berat ubin, spesi, pasir = 365 kg / $m^2$
- Berat railling tangga = 50 kg / $m^2$ +

Qd = 415 kg / $m^2$

Beban hidup pada tangga Ql = 300 kg / $m^2$

Untuk perhitungan struktur tangga selanjutnya menggunakan program SAP2000, data hasil perhitungan dapat dilihat pada lampiran 3.