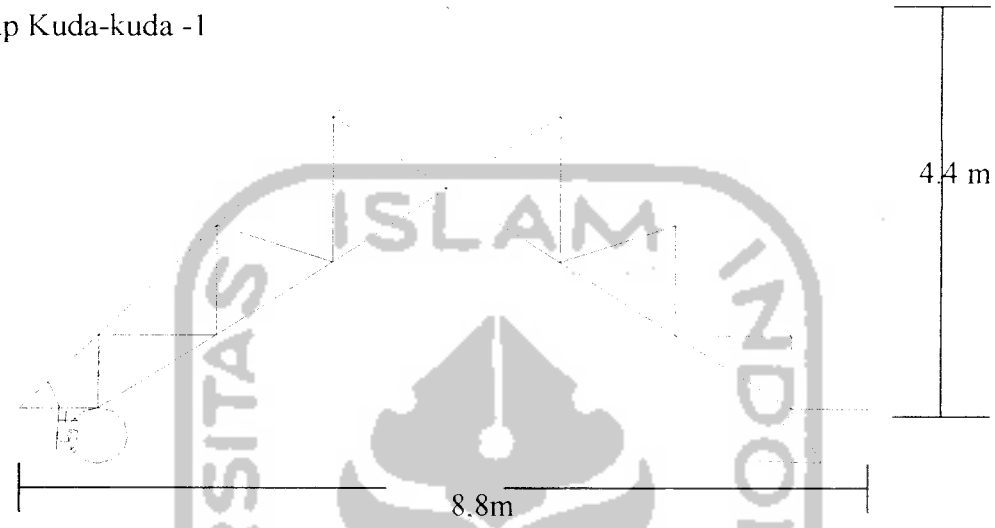
BAB IV

PERHITUNGAN KONSTRUKSI

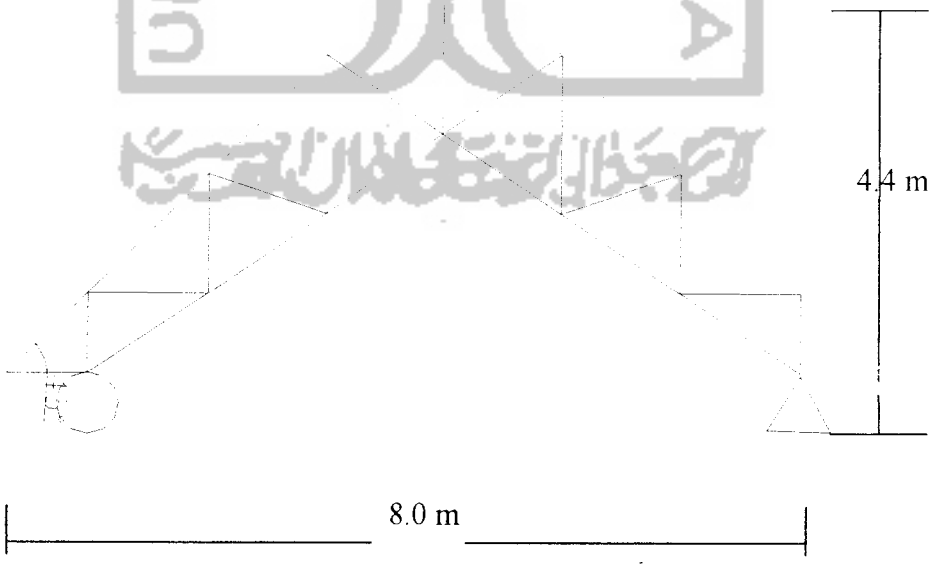
4.1 RENCANA KUDA-KUDA BAJA

1. Atap Kuda-kuda -1



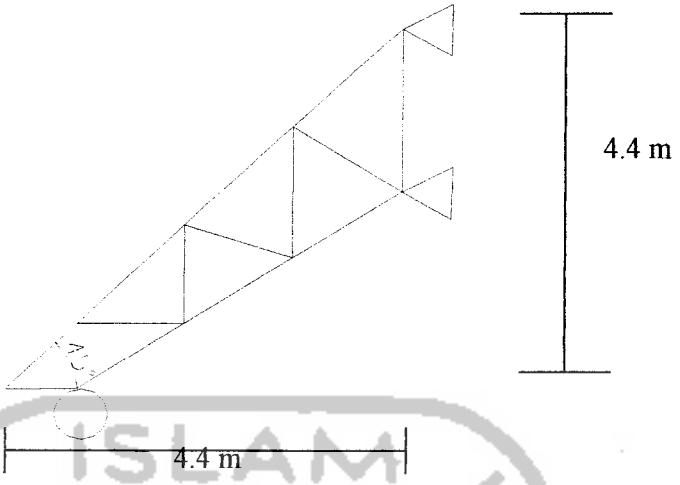
Gambar 4.1 Rencana atap kuda-kuda -1

2. Atap Kuda-kuda 2



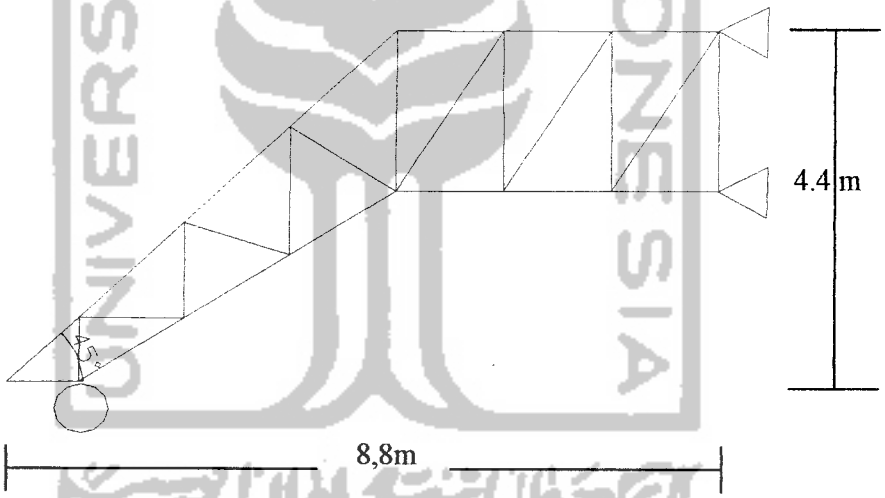
Gambar 4.2 Rencana atap kuda-kuda -2

3. Atap Kuda-kuda 3



Gambar 4.3 Rencana atap kuda-kuda -3

3. Atap Kuda-kuda 4



Gambar 4.4 Rencana atap kuda-kuda -4

4.1.1 Data-data

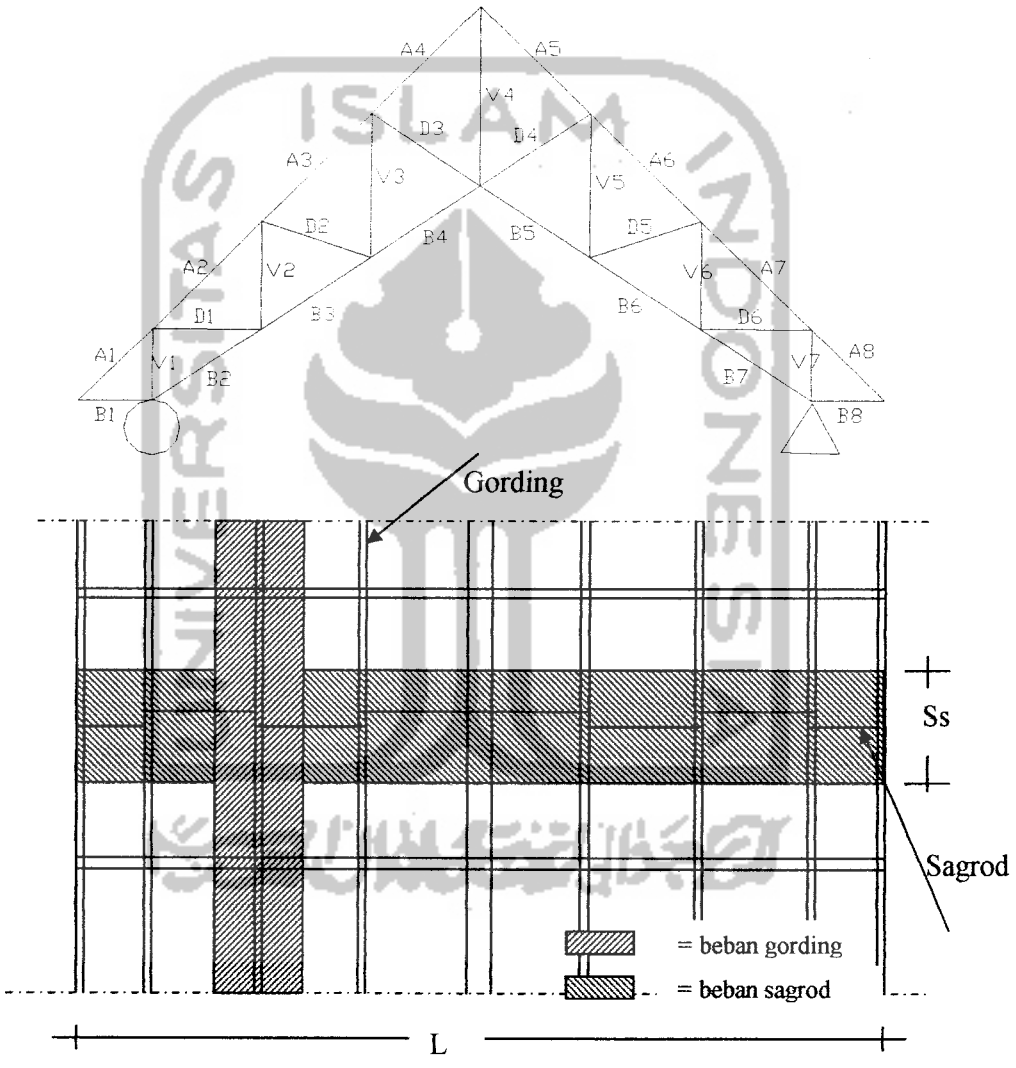
- Jarak antar kuda-kuda = 3,6 m
- Mutu baja profil fy = 2500 kg/cm^2
- Kuat tarik fu = 3700 kg/cm^2

- Mutu baut non full drat dari AISC A_{325x}

  Fu = 8250 kg/cm^2

  Fv = 2050 kg/cm^2

- Direncanakan terhadap bangunan di darat.
- Panjang batang, diberikan contoh perencanaan Kuda-kuda 1



Gambar 4.5 Pembebanan atap

Tabel 4.1 Dimensi Batang kuda-kuda -1

Batang	Panjang	Batang	Panjang	Batang	Panjang
A1	1,131	V3	1,600	D6	1,200
A2	1,697	V4	2,000	B1	0,800
A3	1,697	V5	1,600	B2	1,442
A4	1,697	V6	1,200	B3	1,442
A5	1,697	V7	0,800	B4	1,442
A6	1,697	D1	1,200	B5	1,442
A7	1,697	D2	1,265	B6	1,442
A8	1,131	D3	1,442	B7	1,442
V1	0,800	D4	1,442	B8	0,800
V2	1,200	D5	1,265		

4.1.2 Perencanaan Gording

a. Pembebanan Gording

1. Beban Tetap

- Berat penutup atap

  Berat penutup atap yang berupa genting dalam PPIUG 1983 tabel 2.1 hal 11 adalah 50 kg/m^2

  = 50kg/m^2 x 1.70 m = 85 kg/m

- Beban hidup

  Beban hidup yang bekerja pada atap berupa beban air hujan. Menurut PPIUG 1983 pada pasal 3.2.2 besar beban air hujan = (40 - 0,8α), dimana α adalah sudut kemiringan atap

  = (40 – 0,8 . 45) x 1,70 = 6,8 kg/m

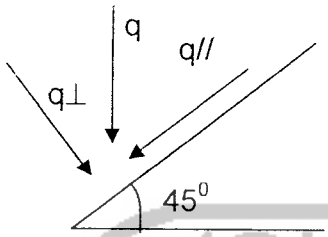
- Berat gording sendiri (perkiraan) = 10 kg/m

  q = 101,8 kg/m

$q\perp = q \cdot \cos \alpha$

$= 101{,}8 \cdot \cos 45 = 72{,}125$ kg/m

$q_{//} = 101{,}8 \cdot \sin 45 = 72{,}125$ kg/m



Gambar 4.6 Arah gaya akibat beban tetap

2. Beban Angin

W angin (dalam PPIUG 1983, pasal 4.2.1) = 25 kg/m^2

a. Angin Tekan (Wt), untuk $\alpha < 65^0$, diketahui $\alpha = 45°$

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

$= 0{,}02 \cdot 45 - 0{,}4 = -0{,}5$

- Wt = C_1 x W x jarak gording

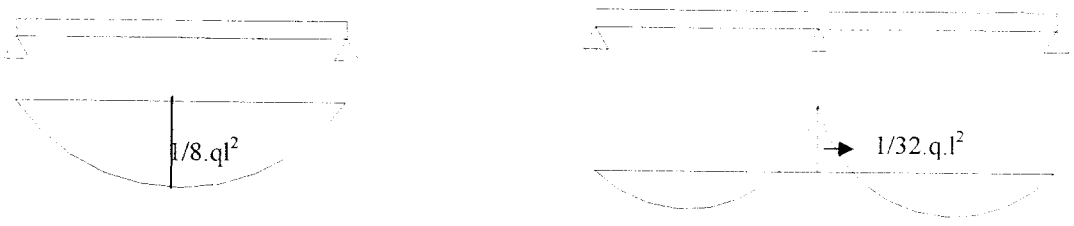
  = 0,5 x 25 x 1.70 = 21,25 kg/m

b. Angin hisap (Wh)

$C_2 = -0{,}4$

- Wh = -0,4 x 25 x 1,70 = -17 kg/m

b. Momen yang terjadi



Gambar 4.7 BMD Gording

- Akibat beban tetap

$M_{\perp}max = 1/8 . q\perp . b^2$

$= 1/8 . 72,125 . 3,6^2 = 116,842$ kgm

$M_{//}max = 1/32 . q// . b^2$

$= 1/32 . 72,125 . 3,6^2 = 29,211$ kgm

- Akibat beban angin

$M_{\perp}max = 1/8 . Wt . b^2$

$= 1/8 . 21,25 . 3,6^2 = 34,425$ kgm

c. Penentuan profil baja :

Dicoba profil C 150 x 50 x 20 x 2,3

Sx = 1,709 in^3 = 28 cm^3

Sy = 0,513 in^3 = 6,33 cm^3

Ix = 5,045 in^4 = 210 cm^4

Iy = 0,526 in^4 = 21,9 cm^4

W = 3,33 lb/ft = 4,96 kg/m^2

d. Kontrol Tegangan

$$fbx = \frac{M\perp_{max}}{Sx}$$

$$= \frac{(116,842 + 34,425)100}{28} = 540,241 \text{ kg/cm}^2$$

$$fby = \frac{M_{//max}}{Sy}$$

$$= \frac{29,211.100}{6,33} = 461,469 \text{ kg/cm}^2$$

Berat penutup atap yang berupa genting dalam PPIUG 1983 tabel 2.1 hal 11 adalah sama dengan 50 kg/m^2

Berat penutup atap = 50 x (1/2 . 3,6/cos45) = 127,279 kg/m

- Beban hidup

  Beban hidup yang bekerja pada atap berupa beban air hujan. Menurut PPIUG 1983 pada pasal 3.2.2 besar beban air hujan = (40 - 0,8α), dimana α adalah sudut kemiringan atap.

  Beban hidup = (40 – 0,8. 45) x (1/2 . 3,6/cos α) = 10,182 kg/m

- Beban gording = 5 x 4,96 kg/m = 24,800 kg/m

P = 162,262 kg/m

$P_{//}$ = P . sin α . Ss

= 162,262 . sin 45 . 1,8 = 206,526 kg/m

$$A_{sagrod} = \frac{P_{//}}{0{,}33.Fu} = \tfrac{1}{4} . \pi . D^2_{sagrod}$$

$$D = \sqrt{\frac{P_{//}.4}{0{,}33.Fu.\pi}} = \sqrt{\frac{206{,}526x4}{0{,}33.3700.\pi}} = 0{,}464 \text{ cm}$$

Sagrod = D + 3 =4,64 + 3 = 7,64 mm, Dipakai = 8 mm

b. Perencanaan Tierod

Beban tierod = T = $P_{//}$. cos α . 2

= 206,526 . cos 45 . 2 = 292,072 kg/cm

$$A \text{ tierod} = \frac{T}{0{,}33.Fu} = \tfrac{1}{4} . \pi . D^2_{tierod}$$

$$D = \sqrt{\frac{T.4}{0{,}33.Fu.\pi}} = \sqrt{\frac{292{,}072x4}{0{,}33.3700.\pi}} = 0{,}552 \text{ cm}$$

Tierod = 5,52 + 3 = 8,52 mm, Dipakai = 10 mm

- $\frac{fbx}{0,66.Fy} + \frac{fby}{0,75.Fy} \leq 1,0$

$$\frac{540,241}{0,66.2500} + \frac{461,469}{0,75.2500} \leq 1,0$$

$$0,597 \leq 1,0$$

e. Kontrol Lendutan

$$\delta_{\perp} = \frac{5}{384} \cdot \frac{q \perp .L^4}{E.Ix} \leq \frac{L}{360}$$

$$= \frac{5}{384} \cdot \frac{72,125x10^{-2}.360^4}{2,1.10^6.210} \leq \frac{360}{360}$$

$$= 0,358 < 1 \quad \textbf{...OK}$$

$$\delta_{//} = \frac{5}{384} \cdot \frac{q_{//} \cdot \left(L/(a+1)\right)^4}{E.Iy} \leq \frac{L}{360}$$

$$= \frac{5}{384} \cdot \frac{72,125.10^{-2}\left(360/(1+1)\right)^4}{2,1.10^6.21,9} \leq \frac{360}{360}$$

$$= 0,214 < 1 \quad \textbf{.....OK}$$

- $\delta = \sqrt{\delta_{\perp}^{\,2} + \delta_{//}^{\,2}}$

$$= \sqrt{0,358^2 + 0,214^2} = 0,417 < 1$$

Profil *Light lip channel* 150 x 50 x 20 x 2,3 dapat digunakan.

4.1.3. Perencanaan Sagrod dan Tierod

a. Perencanaan Sagrod

Beban Sagrod dan Tierod :

- Berat penutup atap

4.1.4. Perencanaan Kuda-Kuda

1. Pembebanan dan Gaya Batang Rencana Kuda-kuda

a. Kuda-kuda I

1. Beban tetap

- Berat gording = 4,96 kg/m
- Berat eternit = 11 kg/m^2
- Berat penutup atap = 50 kg/m^2
- Beban hidup (PPIUG '83) = (40 – 0,8 . 45) = 4 kg/m^2
- Berat kuda-kuda (taksiran) :

$$W = \left(10 \pm \left(\frac{L-12}{3}\right).5\right). \text{jarak kuda-kuda}$$

$$= \left(10 \pm \left(\frac{8{,}8-12}{3}\right).5\right).3{,}6 = 55{,}2 \text{ kg/m'}$$

Beban-beban pada joint :

a) $P_1 = P_9$

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x ½ 1,131	= 101,79 kg
Beban hidup	= 4 x 3,6 x ½ 1,131	= 8,143 kg
		= 127,789 kg

b) $P_2 = P_8$

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x ½ (1,131+1,697)	= 254,52 kg
Beban hidup	= 4 x 3,6 x ½ (1,131+1,697)	= 20,362 kg

= 292,738 kg

c) $P_3 = P_7 = P_4 = P_6$

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x 1,697	= 305,46 kg
Beban hidup	= 4 x 3,6 x 1,697)	= 24,437 kg
		= 347,753 kg

d) P_5

Berat gording	= 2 (4,96 x 3,6)	= 35,712 kg
Berat penutup atap	= 50 x 3,6 x 1,697	= 305,46 kg
Beban hidup	= 4 x 3,6 x 1,697	= 24,437 kg
		= 365,609 kg

e) $P'_1 = P'_9$

Berat eternit	= 11 x 3,6 x ½ 0,8	= 15,84 kg
Berat kuda-kuda	= 55,2 x ½ 0,8	= 22,08 kg
		= 37,92 kg

f) $P'_2 = P'_8$

Berat eternit	= 11 x 3,6 x ½ (0,8+1,442)	= 44,392 kg
Berat kuda-kuda	= 55,2 x ½ (0,8+1,442)	= 61,879 kg
		= 106,271 kg

g) $P'_3 = P'_4 = P'_5 = P'_6 = P'_7$

Berat eternit	= 11 x 3,6 x 1,442	= 57,103 kg
Berat kuda-kuda	= 55,2 x 1,442	= 79,598 kg
		= 136,701 kg

b. Kuda-kuda II

1.Beban tetap

- Berat gording = 4,96 kg/m
- Berat eternit = 11 kg/m^2
- Berat penutup atap = 50 kg/m^2
- Beban hidup (PPIUG '83) = (40 – 0,8 . 45) = 4 kg/m^2
- Berat kuda-kuda (taksiran) :

$$W = \left(10 \pm \left(\frac{8-12}{3}\right).5\right). 3{,}6 = 60 \text{ kg/m'}$$

Beban-beban pada joint :

a. P_1

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x ½ 1,131	= 101,79 kg
Beban hidup	= 4 x 3,6 x ½ 1,131	= 8,143 kg
		= 127,789 kg

b. P_2

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x ½ (1,131+1,697)	= 254,52 kg
Beban hidup	= 4 x 3,6 x ½ (1,131+1,697)	= 20,362 kg
		= 292,738 kg

c. $P_3 = P_4 = P_6 = P_7$

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x 1,697	= 305,46 kg

2. Beban Angin

W angin (PPIUG 1983 pasal 4.2.1) = 25 kg/m^2

Koefisien angin :

- Angin Tekan (Wt)

  C_1 = 0,02 α - 0,4

  = 0,02 . 45 – 0,4 = -0,5

- Angin hisap (Wh)

  C_2 = -0,4

Beban-beban angin

- Angin tekan (Wt) = 0,5 x 25 = 12,5 kg/m^2
- Angin hisap (Wh) =-0,4 x 25 = -10 kg/m^2

a) Angin kiri

Wt_1 = 12,5 x 3,6 x ½ 1,13 = 25,448 kg

Wt_2 = 12,5 x 3,6 x ½ (1,131 + 1,697) = 63,63 kg

$Wt_3 = Wt_4$ = 12,5 x 3,6 x 1,697 = 76,365 kg

Wt_5 = 12,5 x 3,6 x ½ 1,697 = 38,183 kg

Wh_1 = -10 x 3,6 x ½ 1,131 = -20,358 kg

Wh_2 = -10 x 3,6 x ½ (1,131 + 1,697) = -50,904 kg

$Wh_3 = Wt_4$ = -10 x 3,6 x 1,697 = -61,092 kg

Wh_5 = -10 x 3,6 x ½ 1,697 = -30,546 kg

b) Angin kanan

besar angin kanan sama dengan besar angin kiri.

Beban hidup	= 4 x 3,6 x 1,697)	= 24,437 kg
		= 347,753 kg

d. P_5

Berat gording	= 2 (4,96 x 3,6)	= 35,712 kg
Berat penutup atap	= 50 x 3,6 x 1,697	= 305,46 kg
Beban hidup	= 4 x 3,6 x 1,697	= 24,437 kg
		= 365,609 kg

e. P_8

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x ½ . 1,697)	= 152,73 kg
Beban hidup	= 4 x 3,6 x ½ .1,697)	= 12,218 kg
		= 182,804 kg

f. P'_1

Berat eternit	= 11 x 3,6 x ½ 0,8	= 15,84 kg
Berat kuda-kuda	= 60 x ½ 0,8	= 24 kg
		= 39,84 kg

g. P'_2

Berat eternit	= 11 x 3,6 x ½ (0,8+1,442)	= 44,39 kg
Berat kuda-kuda	= 60 x ½.(0,8+1,442)	= 67,26 kg
		= 111,65 kg

h. $P'_3 = P'_4 = P'_5 = P'_6 = P'_7$

Berat eternit	= 11 x 3,6 x 1,442	= 57,103 kg
Berat kuda-kuda	= 60 x 1,442	= 86,52 kg

= 143,623 kg

i. P'_8

Berat eternit	= 11 x 3,6 x ½ .1,442)	= 28,551 kg
Berat kuda-kuda	= 60 x ½..1,442)	= 43,26 kg
		= 71,811 kg

2.Beban Angin

W angin (PPIUG 1983 pasal 4.2.1) = 25 kg/m^2

Koefisien angin :

- Angin Tekan (Wt)

  C_1 = 0,02 α - 0,4

  = 0,02 . 45 – 0,4 = -0,5

- Angin hisap (Wh)

  C_2 = -0,4

Beban-beban angin

- Angin tekan (Wt) = 0,5 x 25 = 12,5 kg/m^2
- Angin hisap (Wh) =-0,4 x 25 = -10 kg/m^2

a) Angin kiri

Wt_1 = 12,5 x 3,6 x ½ 1,13	= 25,448 kg
Wt_2 = 12,5 x 3,6 x ½ (1,131 + 1,697)	= 63,63 kg
$Wt_3 = Wt_4$ = 12,5 x 3,6 x 1,697	= 76,365 kg
Wt_5 = 12,5 x 3,6 x ½ 1,697	= 38,183 kg
$Wh_2 = Wh_5$ = -10 x 3,6 x ½ .1,697)	= -30,546 kg
$Wh_3 = Wt_4$ = -10 x 3,6 x 1,697	= -61,092 kg

b) Angin kanan

besar angin kanan sama dengan besar angin kiri.

Wt_1 tidak ada

Wt_2 = 12,5 x 3,6 x ½ 1,697 = 38,183 kg

Wh_1 = -10 x 3,6 x ½ 1,13 = 20,340 kg

Wh_2 = -10 x 3,6 x ½ (1,131 + 1,697) = 50,904 kg

c. Kuda-kuda III

1.Beban tetap

- Berat gording = 4,96 kg/m
- Berat eternit = 11 kg/m^2
- Berat penutup atap = 50 kg/m^2
- Beban hidup (PPIUG '83) = (40 – 0,8 . 45) = 4 kg/m^2
- Berat kuda-kuda (taksiran) :

$$W = \left(10 \pm \left(\frac{4,4-12}{3}\right).5\right).3,6 = 81.6 \text{ kg/m'}$$

Beban-beban pada joint :

a. P_1

Berat gording = 4,96 x 3,6 = 17,856 kg

Berat penutup atap = 50 x 3,6 x ½ 1,131 = 101,79 kg

Beban hidup = 4 x 3,6 x ½ 1,131 = 8,143 kg

= 127,789 kg

b P_2

Berat gording = 4,96 x 3,6 = 17,856 kg

Berat penutup atap = 50 x 3,6 x ½ (1,131+1,697) = 254,52 kg

Beban hidup = 4 x 3,6 x ½ (1,131+1,697) = 20,362 kg

= 292,738 kg

c. $P_3 = P_4$

Berat gording = 4,96 x 3,6 = 17,856 kg

Berat penutup atap = 50 x 3,6 x 1,697 = 305,46 kg

Beban hidup = 4 x 3,6 x 1,697) = 24,437 kg

= 347,753 kg

d. P_5

Berat gording = 2 (4,96 x 3,6) = 35,712 kg

Berat penutup atap = 50 x 3,6 x 1,697 x 1/2 = 152,73 kg

Beban hidup = 4 x 3,6 x 1,697 x 1/2 = 12,218 kg

= 200,66 kg

e. P'_1

Berat eternit = 11 x 3,6 x ½ 0,8 = 15,84 kg

Berat kuda-kuda = 81,6 x ½ 0,8 = 32,64 kg

= 48,48 kg

f. P'_2

Berat eternit = 11 x 3,6 x ½ (0,8+1,442) = 44,392 kg

Berat kuda-kuda = 81,6 x ½ (0,8+1,442) = 91,474 kg

= 135,866 kg

g. $P'_3 = P'_4$

Berat eternit = 11 x 3,6 x 1,442 = 57,103 kg

Berat kuda-kuda = 81,6 x 1,442 = 117,667 kg

= 174,77 kg

h. P'_5

Berat eternit = 11 x 3,6 x ½ 1,442 = 28,552 kg

Berat kuda-kuda = 81,6 x ½ 1,442 = 58,834 kg

= 87,386 kg

2.Beban Angin

W angin (PPIUG 1983 pasal 4.2.1) = 25 kg/m^2

Koefisien angin :

- Angin Tekan (Wt)

  C_1 = 0,02 α - 0,4

  = 0,02 . 45 – 0,4 = -0,5

- Angin hisap (Wh)

  C_2 = -0,4

Beban-beban angin

- Angin tekan (Wt) = 0,5 x 25 = 12,5 kg/m^2
- Angin hisap (Wh) =-0,4 x 25 = -10 kg/m^2

a) Angin kiri

Wt_1 = 12,5 x 3,6 x ½ 1,13 = 25,448 kg

Wt_2 = 12,5 x 3,6 x ½ (1,131 + 1,697) = 63,63 kg

Wt_3 = 12,5 x 3,6 x 1,697 = 76,365 kg

Wt_5 = 12,5 x 3,6 x ½ 1,697 = 38,183 kg

b) Angin kanan

Wh_1 = -10 x 3,6 x ½ 1,131 = -20,358 kg

Wh_2 = -10 x 3,6 x ½ (1,131 + 1,697) = -50,904 kg

$Wh_3 = Wt_4$ = -10 x 3,6 x 1,697 = -61,092 kg

Wh_5 = -10 x 3,6 x ½ 1,697 = -30,546 kg

d. Kuda-kuda IV

1.Beban tetap

- Berat gording = 4,96 kg/m
- Berat eternit = 11 kg/m^2
- Berat penutup atap = 50 kg/m^2
- Beban hidup (PPIUG '83) = (40 – 0,8 . 45) = 4 kg/m^2
- Berat kuda-kuda (taksiran) :

$$W = \left(10 \pm \left(\frac{8-12}{3}\right).5\right).3,6 = 60 \text{ kg/m'}$$

Beban-beban pada joint :

a. P_1

Berat gording = 4,96 x 3,6 = 17,856 kg

Berat penutup atap = 50 x 3,6 x ½ 1,131 = 101,79 kg

Beban hidup = 4 x 3,6 x ½ 1,131 = 8,143 kg

= 127,789 kg

b. P_2

Berat gording = 4,96 x 3,6 = 17,856 kg

Berat penutup atap = 50 x 3,6 x ½ (1,131+1,697) = 254,52 kg

Beban hidup	= 4 x 3,6 x ½ (1,131+1,697)	= 20,362 kg
		= 292,738 kg

c. $P_3 = P_4$

Berat gording	= 4,96 x 3,6	= 17,856 kg
Berat penutup atap	= 50 x 3,6 x 1,697	= 305,46 kg
Beban hidup	= 4 x 3,6 x 1,697)	= 24,437 kg
		= 347,753 kg

d. P_5

Berat gording	= 2 (4,96 x 3,6)	= 35,712 kg
Berat penutup atap	= 50 x 3,6 x ½ (1,697 +1,2)	= 260,73 kg
Beban hidup	= 4 x 3,6 x ½ (1,697 +1,2)	= 20,858 kg
		= 263,300 kg

e.. $P_6 = P_7$

Berat gording	= 2.(4,96 x 3,6)	= 35,712 kg
Berat penutup atap	= 50 x 3,6 x 1,2	= 216 kg
Beban hidup	= 4 x 3,6 x 1,2	= 17,28 kg
		= 268,992 kg

f. P_8

Berat gording	= 2.(4,96 x 3,6)	= 35,712 kg
Berat penutup atap	= 50 x 3,6 x ½ 1,2	= 108,000 kg
Beban hidup	= 4 x 3,6 x ½ 1,2	= 8,640 kg
		= 152,352 kg

= 59,76 kg

2. Beban Angin

W angin (PPIUG 1983 pasal 4.2.1) = 25 kg/m^2

Koefisien angin :

- Angin Tekan (Wt)

  C_1 = 0,02 α - 0,4

  = 0,02 . 45 – 0,4 = -0,5

- Angin hisap (Wh)

  C_2 = -0,4

Beban-beban angin

- Angin tekan (Wt) = 0,5 x 25 = 12,5 kg/m^2
- Angin hisap (Wh) =-0,4 x 25 = -10 kg/m^2

a) Angin kiri

Wt_1 = 12,5 x 3,6 x ½ 1,13 = 25,448 kg

Wt_2 = 12,5 x 3,6 x ½ (1,131 + 1,697) = 63,63 kg

$Wt_3 = Wt_4$ = 12,5 x 3,6 x 1,697 = 76,365 kg

Wt_5 = 12,5 x 3,6 x ½ 1,697 = 38,183 kg

b) Angin kanan

Wh_1 = -10 x 3,6 x ½ 1,131 = -20,358 kg

Wh_2 = -10 x 3,6 x ½ (1,131 + 1,697) = -50,904 kg

$Wh_3 = Wt_4$ = -10 x 3,6 x 1,697 = -61,092 kg

Wh_5 = -10 x 3,6 x ½ 1,697 = -30,546 kg

g. P'_1

Berat eternit	= 11 x 3,6 x ½ 0,8	= 15,84 kg
Berat kuda-kuda	= 60 x ½ 0,8	= 24 kg
		= 39,84 kg

h. P'_2

Berat eternit	= 11 x 3,6 x ½ (0,8+1,442)	= 44,392 kg
Berat kuda-kuda	= 60 x ½ (0,8+1,442)	= 67,26 kg
		= 111,652 kg

i. $P'_3 = P'_4$

Berat eternit	= 11 x 3,6 x 1,442	= 57,103 kg
Berat kuda-kuda	= 60 x 1,442	= 86,52 kg
		= 143,623 kg

j. P'_5

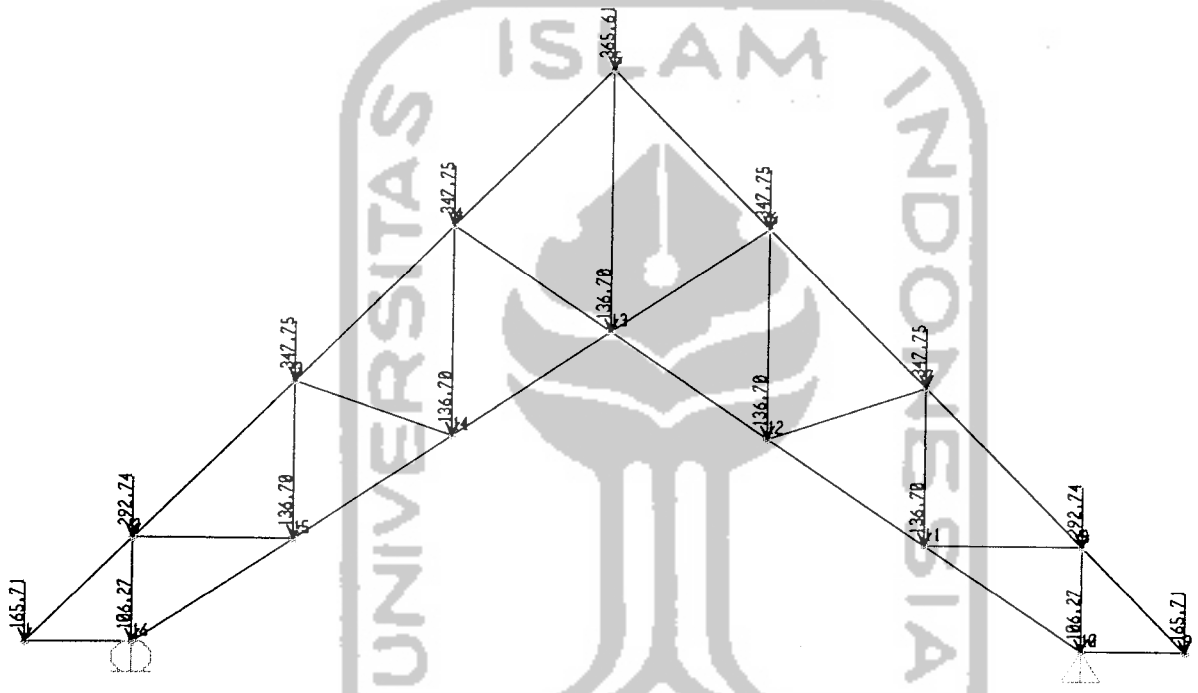
Berat eternit	= 11 x 3,6 x ½ (1,442+1, 2)	= 52,312 kg
Berat kuda-kuda	= 60 x ½ (1,442+1, 2)	= 79,26 kg
		= 131,572 kg

k. $P'_6 = P'_7$

Berat eternit	= 11 x 3,6 x 1, 2	= 47,520 kg
Berat kuda-kuda	= 60 x 1, 2	= 72 kg
		= 119,52 kg

l. P'_8

Berat eternit	= 11 x 3,6 x ½ .1, 2	= 23,760 kg
Berat kuda-kuda	= 60 x ½ .1, 2	= 36 kg


365.61
342.75
342.75
342.75
342.75
136.78
136.78
136.78
136.78
136.78
292.24
292.24
165.71
165.71
186.27
186.27
1
2
3
4
5
6
7
8
9
10

Analisa rangka menggunakan SAP 2000 dan gambar beban rencana Kuda-kuda dapat dilihat dalam lampiran.

Tabel 4.2 Beban Rencana Kuda-kuda -1

No	Letak	Batang	Gaya Batang (Kg)			Gaya Batang (Kg)			
			Bebab Tetap	B. Angin Kiri	B. Angin Kanan	1.3 B.Tetap	B Tetap+ B A. Kiri	B Tetap + BA Kanan	B Rencana
1	Bawah	9	-165.71	28.8	-35.98	-215.423	-136.91	-201.69	-165.71
2	Bawah	10	-199.16	-393.69	385.06	-258.908	-592.85	185.9	-199.16
3	Bawah	11	1333.56	-545.83	557.4	1733.628	787.73	1890.96	1333.56
4	Bawah	12	1663.24	-544.03	546.22	2162.212	1119.21	2209.46	1663.24
5	Bawah	13	1663.24	-310.39	312.58	2162.212	1352.85	1975.82	1663.24
6	Bawah	14	1333.56	-156.44	168.01	1733.628	1177.12	1501.57	1333.56
7	Bawah	15	-166.16	-43.24	34.61	-216.008	-209.4	-131.55	-166.16
8	Bawah	16	-165.71	-35.98	28.8	-215.423	-201.69	-136.91	-165.71
9	Atas	1	234.35	25.44	-20.36	304.655	259.79	213.99	234.35
10	Atas	2	-1569.2	95.01	-126.43	-2039.96	-1474.19	-1695.63	-1569.2
11	Atas	3	-1957.13	199.8	-235.46	-2544.269	-1757.33	-2192.59	-1957.13
12	Atas	4	-1778.82	323.76	-349.76	-2312.466	-1455.06	-2128.58	-1778.82
13	Atas	5	-1778.82	255.03	-281.02	-2312.466	-1523.79	-2059.84	-1778.82
14	Atas	6	-1957.13	268.53	-304.19	-2544.269	-1688.6	-2261.32	-1957.13
15	Atas	7	-1569.2	209.56	-240.97	-2039.96	-1359.64	-1810.17	-1569.2
16	Atas	8	234.35	-20.36	25.44	304.655	213.99	259.79	234.35
17	Vertikal	17	-1568.04	4.21	-39.01	-2038.452	-1563.83	-1607.05	-1568.04
18	Vertikal	19	-713.5	62.79	-73.99	-927.55	-650.71	-787.49	-713.5
19	Vertikal	21	-137.61	128.09	-120.29	-178.893	-9.52	-257.9	-257.9
20	Vertikal	23	2150.01	-414.67	440.62	2795.013	1735.34	2590.63	2150.01
21	Vertikal	25	-137.61	-1.5	9.3	-178.893	-139.11	-128.31	-139.11
22	Vertikal	27	-713.5	84.39	-95.6	-927.55	-629.11	-809.1	-713.5
23	Vertikal	29	-1568.04	198.57	-233.37	-2038.452	-1369.47	-1801.41	-1568.04
24	Diagonal	18	1275.3	-94.19	110.99	1657.89	1181.11	1386.29	1275.3
25	Diagonal	20	289.15	-135.02	126.8	375.895	154.13	415.95	289.15
26	Diagonal	22	-151.54	-170.25	149.05	-197.002	-321.79	-2.49	-151.54
27	Diagonal	24	-151.54	63.39	-84.59	-197.002	-88.15	-236.13	-151.54
28	Diagonal	26	289.15	1.59	-9.81	375.895	290.74	279.34	289.15
29	Diagonal	28	1275.3	-126.59	143.39	1657.89	1148.71	1418.69	1275.3

Syarat :

a. 30 % beban tetap > beban angin (angin kanan + angin kiri)

→ Beban rencana = beban tetap.

b. 30 % beban tetap < beban angin (angin kanan + angin kiri)

→ Beban rencana = beban tetap + beban angin

2. Perencanaan Profil

a. Kuda-kuda 1

1. Batang Bawah

- **Batang Tarik**

  - Gaya tarik (P) maksimal = 2209.459 kg
  - Panjang batang (L) = 1,442 m = 144,2 cm

    Fy = 2500 Kg/cm^2 Fu = 3700 Kg/cm^2
  - Syarat batang tarik :

$$\frac{L}{r} \leq 240 \text{ s/d } 300 \quad \Rightarrow \quad r\min = \frac{L}{240} = \frac{144,2}{240} = 0,601 \text{ cm}$$

  - Luas tampang perlu :

$$Ag_1 = \frac{T}{0,6.Fy} = \frac{2209.459}{0,6.2500} = 1.473 \text{ cm}^2$$

$$Ag_2 = \frac{T}{0,5.Fu.0,85} + \left(\frac{1}{8}'' + \phi baut\right).tp.n$$

$$= \frac{2209.459}{0,5.3700.0,85} + \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8".2 = 2.596 \text{ cm}^2$$

⇒ Dicoba Profil 2L 50x50x5

A = 4,80 cm^2 W = 3,77 Kg/m

r = 1,51 cm

Abruto = 2 x 4,80 cm^2 = 9,60 cm^2

$$\text{Alubang} = \left(\frac{1}{8}'' + \phi baut\right).tp.n = \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8".2 = 0{,}469" = 1{,}1906 \text{ cm}^2$$

Anetto = Abruto – Alubang = 9,60 cm^2 - 1,1906 cm^2

= 8,4094 cm^2

Aeffektif = 0,85.Anetto = 0,85 x 8,4054 = 7,148 cm^2

Kontrol tegangan :

° $\frac{T}{Aprofil} \le 0{,}6.Fy \Rightarrow \frac{2209.459}{9{,}60} \le 0{,}6.2500$

230.152 Kg/cm^2 ≤ 1500 Kg/cm^2 **.....Ok**

° $\frac{T}{Aeffektif} \le 0{,}5.Fu \Rightarrow \frac{2209.459}{7{,}148} \le 0{,}5.3700$

309.102 kg/cm^2 ≤ 1850 kg/cm^2 **.....Ok**

⇒ Profil yang digunakan 2L 50x50x5

- **Batang Tekan**
  - Gaya tekan (P) maksimal = -592.847 Kg
  - Panjang batang (L) = 1,442 m =144,2 cm

    Fy = 2500 Kg/cm^2 Fu = 3700 Kg/cm^2

    E = 2,1 x 10^6 Mpa K = 1 (sendi – sendi)

  - Syarat batang tekan :

$$\frac{KL}{r} \le 200 \Rightarrow \text{r min} = \frac{KL}{200} = \frac{144{,}2}{200} = 0{,}721 \text{ cm}$$

⇒ Dicoba Profil 2L 50x50x5

A = 4,80 cm^2 Atotal = 2 x 4,8 = 9,60 cm^2

r = 1,51 cm

W = 3,77 Kg/m

Ix = Iy =11,0 cm^4 ix = iy = 1,51 cm e = 1,40

x = e + ½.tp = 1,40 + ½.1 = 1,90 cm

Ix.gab = 2 x 11,0 = 22 cm^4

Iy.gab = 2.Iy + 2 $A.x^2$ = 22 + 2 . 4,8 . $1,9^2$ = 40,24 cm^4

$$ix.gab = \sqrt{\frac{Ix.gab}{2A}} = \sqrt{\frac{22}{9,6}} = 1,51\,cm$$

$$iy.gab = \sqrt{\frac{Iy.gab}{2A}} = \sqrt{\frac{40,24}{9,60}} = 2,05\,cm$$

r = 1,51 cm ≥ r min = 0,721 cm → dipakai r = ix.gab = 1,51 cm

Syarat :

$$\frac{KL}{r} \le Cc = \sqrt{\frac{2\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}} \Rightarrow \frac{1.144,2}{1,51} \le \frac{6400}{\sqrt{2500}}$$

$$95,497 \le 128$$

sehingga digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3}{8}.\frac{KL/r}{Cc} - \frac{1}{8}.\frac{(KL/r)^3}{Cc^3}$$

$$= \frac{5}{3} + \frac{3}{8}.\frac{95,497}{128} - \frac{1}{8}.\frac{(95,497)^3}{(128)^3} = 1,894$$

$$Fa = \frac{Fy}{Fs}\left(1 - \frac{(KL/r)^2}{2.Cc^2}\right)$$

$$= \frac{2500}{1,894}\left(1 - \frac{(95,497)^2}{2.(128)^2}\right) = 952,334\ kg/cm^2$$

Kontrol kapasitas :

P = Fa . Atotal > P terjadi

= 952,334 . 9,60 > 592.847 kg

= 9144,96 kg > 592.847 kg.....**Ok**

⇒ Profil yang digunakan 2L 50x50x5

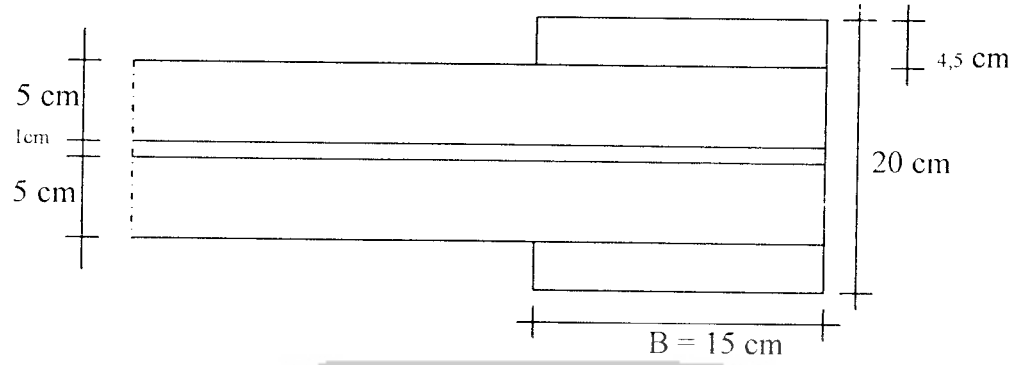
Tabel 4.3 Perencanaan dimensi batang tarik kuda-kuda -1

Batang Tarik	**Btg Atas**	**Btg Bawah**	**Btg Vertikal**	**Btg Diagonal**
Gaya tarik maksimal (P) (kg)	234.348	2209.459	2150.014	1275.3
Panjang Batang Maks (cm)	113.1	144.2	200	120
Fy (kg/m^2)	2500	2500	2500	2500
Fu (kg/m^2)	3700	3700	3700	3700
r min (cm)	0.47125	0.6008333	0.83333333	0.5
Alubang (cm^2)	1.1906	1.1906	1.1906	1.1906
Ag1 (cm^2)	0.156232	1.4729727	1.43334267	0.8502
Ag2 (cm^2)	1.339629	2.5956614	2.55785851	2.00160159
Dicoba profil 2L 50x50x5				
A (cm^2)	4.8	4.8	4.8	4.8
r (cm)	1.51	1.51	1.51	1.51
W (kg/m)	3.77	3.77	3.77	3.77
Abruto (cm^2)	9.6	9.6	9.6	9.6
Anetto (cm^2)	8.4094	8.4094	8.4094	8.4094
Aeffektif (cm^2)	7.14799	7.14799	7.14799	7.14799
Kontrol Tegangan:				
T/Aprofil	24.41125	230.15198	223.959792	132.84375
0.6 fy (kg/m^2)	1500	1500	1500	1500
T/Aprofil < 0,6 Fy	Aman	Aman	Aman	Aman
T/Aeffektif	32.78516	309.10214	300.785815	178.4137918
0,5.Fu (kg/m^2)	1850	1850	1850	1850
T/Aeffektif < 0.5Fu	**Aman**	**Aman**	**Aman**	**Aman**

Tabel 4.4 Perencanaan dimensi batang tekan kuda-kuda -1

Batang Tekan	Btg Atas	Btg Bawah	Btg Vertikal	Btg Diagonal
Gaya tekan maksimal(P) (kg)	2261.316	592.847	1568.04	289.124
Panjang Batang Maks (cm)	169.7	144.2	80	144.2
Fy (kg/m^2)	2500	2500	2500	2500
Fu (kg/m^2)	3700	3700	3700	3700
E (Mpa)	2100000	2100000	2100000	2100000
K (Sendi - Sendi)	1	1	1	1
r min (cm)	0.8485	0.721	0.4	0.721
Dicoba Profil 2L 50x50x5				
A (cm^2)	4.8	4.8	4.8	4.8
R (cm)	1.51	1.51	1.51	1.51
W (kg/m)	3.77	3.77	3.77	3.77
Ix=Iy (cm^4)	11	11	11	11
ix=iy (cm)	1.51	1.51	1.51	1.51
e (cm)	1.4	1.4	1.4	1.4
tp (cm)	1	1	1	1
x (cm)	1.9	1.9	1.9	1.9
Ix gabungan (cm^4)	22	22	22	22
Iy gabungan (cm^4)	40.24	40.24	40.24	40.24
ix gabungan (cm)	1.513825	1.5138252	1.51382518	1.513825177
iy gabungan (cm)	2.047356	2.047356	2.04735602	2.047356019
Dipakai r (cm)	1.51	1.51	1.51	1.51
Syarat :				
K.L / r	112.3841	95.496689	52.9801325	95.49668874
Cc	128	128	128	128
	K.L/r <Cc	K.L/r <Cc	K.L/r < Cc	K.L/r > Cc
Fs (kg/m^2)	1.911312	1.8945328	1.81301811	-
Fa (kg/m^2)	803.8421	952.33419	1260.79874	1185.757347
Kontrol kapasitas				
P (kg)	7716.884	9142.4082	12103.6679	11383.27053
P > P terjadi	**Aman**	**Aman**	**Aman**	**Aman**

4.1.4 Perencanaan Pelat Kuda-kuda -1



Gambar 4.8 pelat kuda-kuda -1

P =1784,78 kg ; f'c = 25 Mpa = 250 kg/cm^2

$$\text{A perlu} = \frac{P}{0{,}33.f'c} = \frac{1784{,}78}{0{,}33.250} = 21{,}634 \text{ cm}^2$$

Diambil ukuran pelat : 15 x 20 = 300 cm^2 > A perlu

$$q = \frac{P}{BxL} = \frac{1784.78 \times 1}{15 \times 20} = 5{,}949 \text{ kg/cm}$$

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

M = ½ . q . x^2 = ½ . 8,622.4,5^2 = 87,298 kg.cm

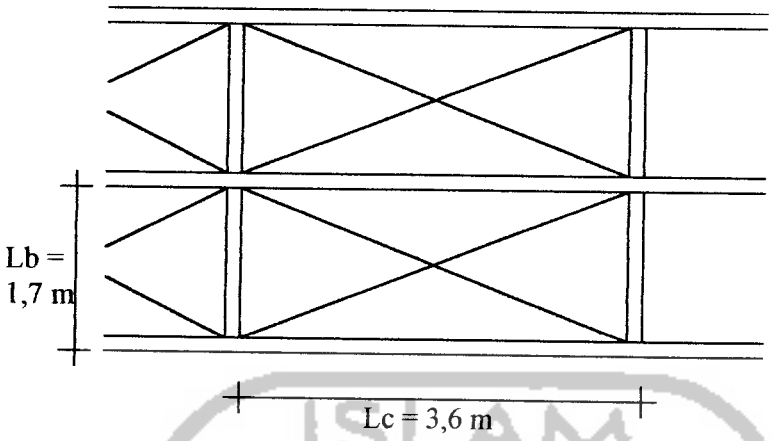
Syarat : $0{,}6 \text{ Fy} = \frac{M}{1/6.1.tp^2}$

$$tp = \sqrt{\frac{10M}{Fy}} = \sqrt{\frac{10.87{,}298}{2500}} = 0{,}591 \text{ cm} \approx 1 \text{ cm}$$

Sehingga dipakai pelat dengan tebal 1 cm

Pelat kuda-kuda berukuran : **15 x 20 x 1**

4.1.5 Perencanaan dukungan arah lateral



Gambar 4.9 dukungan arah lateral

Diketahui :

Lb = jarak antar gording = 1,7 m

Lc = jarak antar kuda-kuda = 3,6 m

$$L = \sqrt{Lb^2 + Lc^2} = \sqrt{(1,7)^2 + (3,6)^2} = 3,981\,\text{m}$$

Syarat : L/r ≤ 300 sehingga :

$$r\,\text{min} \geq \frac{L}{300} = \frac{3,981m}{300} = \frac{398,1cm}{300} = 1,327\ \text{cm}$$

Keterangan :

1. L ≤ 3 m → dipakai baja tulangan ∅12 mm
2. L ≥ 5 m → dipakai baja tulangan ∅19 mm
3. 3 m < L = 3,981 m ≤ 5 m → dipakai baja tulangan ∅16 mm

⇒ Sehingga dipakai baja tulangan ∅16 mm > r min = 1,327 cm **.....Ok**

4.1.6. Perencanaan Sambungan

Dalam perencanaan sambungan pada tiap joint menggunakan baut ϕ ½" (1,27 cm), dan pelat baja BJ 37 (Fy = 2500 kg/cm^2 , Fu = 3700 kg/cm^2) dengan tebal 0,8 cm. Baut yang digunakan adalah A325x (baut non full drat), dengan kekuatan ultimit (Fu) = 8250 kg/cm^2, Fv = 2050 kg/cm^2

Sehingga didapat kekuatan 1 baut untuk menahan gaya adalah :

P_{tumpu} = Tebal pelat . ϕ baut . 1,2 Fu pelat . n

= 0,8 . 1,27 . 1,2 . 3700 . 1 = 4511,04 kg

P_{geser} = Abaut . 0,33 . Fu . 2n

= ¼ . π . 1,27^2 . 0,33 . 8250 . 2 . 1 = 5431,14611 kg

dipakai P yang kecil yaitu P = 4511,04 kg

Jarak penggunaan baut 1/2"

- Jarak baut ke tepi (min 1,2 D),

  diambil = 1,5 D = 1,5 x 1,27= 1,905 ~ 3 cm

- Jarak antar baut (2D s/d 7D)

  diambil = 3D = 3 x 1,27 = 3,81 ~ 5 cm

Perhitungan jumlah baut untuk masing-masing joint adalah sebagai berikut :

Rangka Kuda-kuda I

1. Joint 1

   - Batang Atas 1 (tarik)

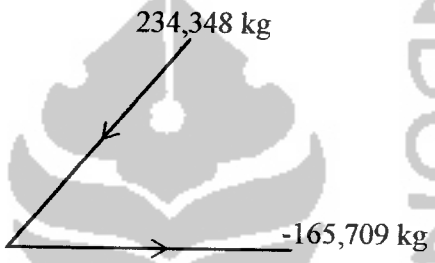
P = 234,348 kg

$n = \frac{234,348}{4511,04} = 0.051$, diambil jumlah minimal baut = 3 buah

- Batang Bawah 1 (tekan)

P = 165,709 kg

$n = \frac{165,709}{4511,04} = 0,037$, diambil jumlah minimal baut 3 buah



2. Joint 2

- Batang Atas 1 (tarik)

P = 234,348 kg

$n = \frac{234,348}{4511,04} = 0.051$, diambil jumlah minimal baut 3 buah

- Batang Atas 2 (tekan)

P = 1569,2 kg

$n = \frac{1569,2}{4511,04} = 0,348$; diambil jumlah minimal baut 3 buah

- Batang Vertikal 1 (desak)

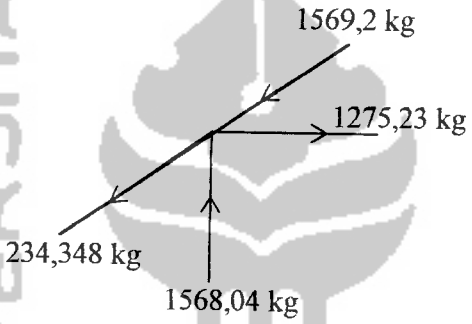
P = 1568,04 kg

$$n = \frac{1568,04}{4511,04} = 0.348$$; diambil jumlah minimal baut 3 buah

- Batang Diagonal 1 (tarik)

P = 1275,23 kg

$$n = \frac{1275,23}{4511,04} = 0,283$$, diambil jumlah minimal baut 3 buah



Untuk sambungan pada joint berikutnya, dengan perhitungan yang sama didapat jumlah baut yang sama pula yaitu 3 buah, karena gaya-gaya batang yang terjadi kurang dari kekuatan 1 baut untuk menahan gaya (= 4511,04 kg). Perhitungan baut meliputi setengah bentang rangka kuda-kuda untuk mewakili perhitungan satu bentang.

Tabel 4.5 Jumlah baut terpakai

Joint	Elemen/Batang	Jumlah Baut
1	1	3 Buah
	16	3 Buah
2	1	3 Buah
	2	3 Buah
	17	3 Buah
	18	3 Buah
3	2	3 Buah
	3	3 Buah
	19	3 Buah
	20	3 Buah
4	3	3 Buah
	4	3 Buah
	21	3 Buah
	22	3 Buah
5	4	3 Buah
	5	3 Buah
	23	3 Buah
13	12	3 Buah
	13	3 Buah
	22	3 Buah
	23	3 Buah
	23	3 Buah
14	13	3 Buah
	14	3 Buah
	20	3 Buah
	21	3 Buah
15	14	3 Buah
	15	3 Buah
	18	3 Buah
	19	3 Buah
16	15	3 Buah
	16	3 Buah
	17	3 Buah

Kontrol berat kuda-kuda :

Tabel 4.6 Profil terpakai dan berat profil terpakai

Batang	Profil (mm)	Berat Profil (kg/m)	Panjang (m)	Berat (kg)
Batang Atas	2L 50x50x5	2 x 3,77 = 7,54	12,445	93,835
Batang Bawah	2L 50x50x5	2 x 3,77 = 7,54	10,253	77,308
Batang Vertikal	2L 50x50x5	2 x 3,77 = 7,54	9,200	69,368
Batang Diagonal	2L 50x50x5	2 x 3,77 = 7,54	7,814	58,918
				299,429

- Berat total kuda-kuda = 299,429 kg
- Berat baut, plat sambung Ø baut =(20%.berat total kuda-kuda) = 59,886 kg

  Jumlah (Σ) = B. total kuda-kuda + 20%. B total kuda-kuda

  = 299,429 kg + 59,886 kg = 359,315 kg
- Panjang rangka kuda-kuda = L = 8,8 m

  $\frac{\Sigma}{L}$ < Berat taksiran kuda-kuda

  $\frac{359,315}{8,8} < 55,2$ kg

  40,831 kg/m < 55,2 kg/m.....**Ok**

Perencanaan profil untuk jenis kuda-kuda yang lain di tabelkan.

4.2. PERENCANAAN PELAT

4.2.1. Perencanaan Pelat Lantai

Pada perhitungan ini akan diberikan contoh perhitungan perencanaan pelat lantai type 1.

4.2.1.1. Pembebanan Pelat Lantai

- Beban mati pelat lantai (qD) :

  - berat sendiri pelat (perkiraan) : 0,12 x 24 = 2,88 kN/m^2
  - pasir (tebal 5 cm) : 0,05 x 16 = 0,80 kN/m^2
  - spesi (tebal 3 cm) : 0,03 x 21 = 0,63 kN/m^2
  - keramik : 0,01 x 20 = 0,20 kN/m^2
  - eternit + plafond = 0,18 kN/m^2 +

  beban mati total (qD) = 4,69 kN/m^2

- Beban hidup pelat lantai :

  - fungsi bangunan sebagai ruang laboratorium ⟶ qL = 2,5 kN/m^2

  (PPIUG, 1983 tabel 3.1, halaman 17)

- Kombinasi Pembebanan (SK SNI T-15-1991-03, Pasal 3.2.2)

  - qU = 1,2.qD + 1,6.qL = 1,2.4,69 + 1,6. 2,5 = 9,628 kNm

- Digunakan tulangan pokok ∅ 10 mm
- Penutup beton digunakan : Pb = 20 mm
- Digunakan h = 120 mm
- Mutu beton (fc') = 22,5 Mpa
- Mutu baja (fy) = 240 Mpa

- Tinggi manfaat tulangan pelat :

  - Arah lapangan – x : dx = h – Pb - ½$\varnothing_{tul.x}$

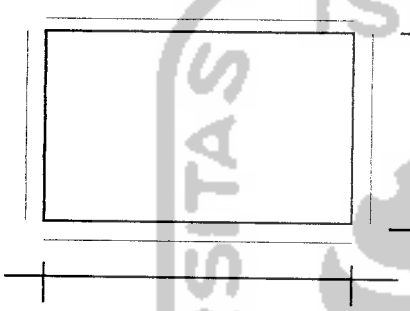
    = 120 – 20 - ½.10 = 95 mm

  - Arah lapangan – y : dy = h – Pb - $\varnothing_{tul.x}$ - ½$\varnothing_{tul.y}$

    = 120 – 20 – 10 - ½.10 = 85 mm

  - Arah tumpuan – x dan y : 95 mm

4.2.1.2. Perhitungan Tulangan Pokok Pelat Lantai Tipe I



$\frac{ly}{lx} = \frac{7,20}{3,60} = 2$

Dari tabel 13.3.2, halaman 203 PBBI NI-2 (pelat dianggap jepit elastis)

Didapat : Clx = -Ctx = 62

Cly = -Cty = 35

- Momen-momen yang bekerja pada pelat :

  Mu = 0,001. qU. lx^2 . C

  Mulx = -Mutx = 0,001 . 9,628 . $3,6^2$. 62 = 7,736 KNm

  Muly = -Muty = 0,001 . 9,628 . $3,6^2$. 35 = 4,367 KNm

a. Perencanaan Tulangan Mulx dan Mutx

Mulx = - Mutx = 7,736 kNm

$Mu/\phi = 7,736/0,8 = 9,67$ kNm

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{9{,}67.10^6}{1000.95^2} = 1{,}072 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}00583$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{240}.\left(\frac{600}{600+240}\right) = 0{,}048$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}048 = 0{,}0363$$

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

$$= \frac{1}{12{,}549}.\left(1-\sqrt{1-\frac{2.12{,}549.1{,}071}{240}}\right) = 0{,}0046 < \rho_{maks} = 0{,}0363$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33.\rho_{aktual} = 1{,}33.\ 0{,}0046 = 0{,}00611 > \rho_{min} = 0{,}00583$

sehingga dipakai : $\rho_{pakai} = 0{,}00583$

$As_{perlu} = \rho_{pakai}.\ b.\ d = 0{,}00583.1000.95 = 554{,}167 \text{ mm}^2$

Digunakan tulangan bagi ∅10 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 10^2 = 78{,}5 \text{ mm}^2$

Jarak antar tulangan : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{78{,}5.1000}{554{,}167}$

$$\le 141{,}783 \text{ mm}$$

$$s \leq 2h \leq 2.120 \leq 240$$

$$s \leq 250$$

Dipakai tulangan pokok : P10 – 140 mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{78,5.1000}{140} = 561,224 \text{ mm}^2 > As_{perlu} = 554,167 \text{ mm}^2$$

Kontrol Kapasitas Momen (Mn) :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{561,224.240}{0,85.22,5.1000} = 7,4043 \text{ mm}$$

$$Mn = As_{ada}.fy.\ (d - {}^{a}/_{2}) = 561,224.240\ (95 - {}^{7,4043}/_{2})$$

$$= 12,322 \text{ KNm} \geq {}^{Mu}/_{\phi} = 9,67 \text{ KNm} \text{ .....OK!}$$

b. Perencanaan Tulangan Muly

Muly = - Muty = 4,367 KNm

$${}^{Mu}/_{\phi} = {}^{4,367}/_{0,8} = 5,459 \text{ KNm}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{5,459.10^6}{1000.85^2} = 0,756 \text{ Mpa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{240}.\left(\frac{600}{600+240}\right) = 0,0484$$

$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0484 = 0{,}0363$

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

$$= \frac{1}{12{,}549}\left(1-\sqrt{1-\frac{2.12{,}549.0{,}756}{240}}\right) = 0{,}00321 < \rho_{maks} = 0{,}0363$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33.\rho_{aktual} = 1{,}33.\ 0{,}00321 = 0{,}00427 < \rho_{min} = 0{,}00583$

sehingga dipakai : $\rho_{pakai} = 0{,}00427$

$As_{perlu} = \rho_{pakai}.\ b.\ d = 0{,}00427.1000.85 = 363{,}233\ mm^2$

Digunakan tulangan bagi ∅ 8 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50{,}265\ mm^2$

Jarak antar tulangan pokok : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{50{,}265.1000}{363{,}233} \le 138{,}439$ mm

$s \le 2h \le 2.120 \le 240$

$s \le 250$

<u>Dipakai tulangan pokok : P8 – 130 mm</u>

$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{50{,}265.1000}{130} = 386{,}813\ mm^2 > As_{ada} = 363{,}233\ mm^2$

Kontrol Kapasitas Momen (Mn) :

$$a = \frac{As_{ada}.fy}{0{,}85.f.c.b} = \frac{386{,}813.240}{0{,}85.22{,}5.1000} = 4{,}854\ mm$$

$Mn = As_{ada}.fy.\ (d - a/2) = 386{,}813.240\ (85 - 4{,}854/2)$

$= 7{,}66\ KNm \ge Mu/\phi = 5{,}459\ KNm$ **.....OK!**

c. Perencanaan Tulangan Muty

Muly = - Muty = 4,367 KNm

$$Mu/\phi = 4{,}367/0{,}8 = 5{,}459 \text{ KNm}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{5{,}459.10^6}{1000.95^2} = 0{,}605 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}00583$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{240}\left(\frac{600}{600+240}\right) = 0{,}0484$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0484 = 0{,}0363$$

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

$$= \frac{1}{12{,}549}\left(1-\sqrt{1-\frac{2.12{,}549.0{,}605}{240}}\right) = 0{,}00256 < \rho_{maks} = 0{,}0363$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33.\rho_{aktual} = 1{,}33.\ 0{,}00256 = 0{,}0034 < \rho_{min} = 0{,}00583$

sehingga dipakai : $\rho_{pakai} = 0{,}0034$

$As_{perlu} = \rho_{pakai}.\ b.\ d = 0{,}0034.1000.95 = 323{,}648 \text{ mm}^2$

Digunakan tulangan bagi ∅8 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50{,}265\ mm^2$

Jarak antar tulangan pokok : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{50{,}265.1000}{323{,}648} \le 155{,}372$ mm

$s \le 2h \le 2.120 \le 240$

$s \le 250$

Dipakai tulangan pokok : P8 – 130 mm

$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{50{,}265.1000}{130} = 386{,}813\ mm^2 > As_{ada} = 323{,}648\ mm^2$

Kontrol Kapasitas Momen (Mn) :

$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{386{,}813.240}{0{,}85.22{,}5.1000} = 4{,}854$ mm

$Mn = As_{ada}.fy.\ (d - a/2) = 386{,}813.240\ (95 - 4{,}854/2)$

$= 8{,}594$ KNm $\ge Mu/\phi = 5{,}459$ KNm **.....OK!**

4.2.1.3. Perhitungan Tulangan Bagi Pelat Lantai

$As_{bagi} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 120 = 240\ mm^2$

Digunakan tulangan bagi ∅ 8 mm, sehingga luas tampang 1 tulangan polos :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50{,}24\ mm^2$

Jarak antar tulangan pokok : $s \le \frac{A_{\theta 1}.b}{As_{bagi}} \le \frac{50{,}24.1000}{240} \le 209{,}3333$ mm

Dipakai tulangan bagi : P8 – 200 mm

Tabel 4.7 Perencanaan pelat lantai tipe -1

	Mulx	Mutx	Muly	Muty
fc' (Mpa)	22.5	22.5	22.5	22.5
fy (Mpa)	240	240	240	240
ly (m)	7.2	7.2	7.2	7.2
lx (m)	3.6	3.6	3.6	3.6
ly/lx	2	2	2	2
C	62	62	35	35
qu (kNm)	9.628	9.628	9.628	9.628
Mu (KNm)	7.736	7.736	4.367	4.367
Mu/ϕ (KNm)	9.670	9.670	5.459	5.459
d (mm)	95	95	85	95
m	12.549	12.549	12.549	12.549
Rn (MPa)	1.072	1.072	0.756	0.605
ρ_{min}	0.00583	0.00583	0.00583	0.00583
ρb	0.04838	0.04838	0.04838	0.04838
ρ_{maks}	0.03629	0.03629	0.03629	0.03629
ρ_{aktual}	0.00460	0.00460	0.00321	0.00256
$1.33.\rho_{aktual}$	0.00611	0.00611	0.00427	0.00341
ρ_{pakai}	0.00583	0.00583	0.00427	0.00341
As ada (mm2)	554.167	554.167	363.233	323.648
dtul.pokok (mm)	10	10	8	8
A1d.pokok (mm2)	78.571	78.571	50.286	50.286
s (mm)	141.783	141.783	138.439	155.372
s pakai (mm)	140	140	130	130
As aktual (mm2)	561.224	561.224	386.813	386.813
a (mm)	7.043	7.043	4.854	4.854
Mn (KNm)	12.322	12.322	7.666	8.594
Tul. Pokok	P10-140	P10-140	P8-130	P8-130
Kontrol	AMAN	AMAN	AMAN	AMAN
As bagi (mm2)		240		240
dtul.bagi (mm)		8		8
A1d.bagi (mm2)		50.286		50.286
x (mm)		209.524		209.524
xpakai (mm)		200		200
Tul. Bagi		P8-200		P8-200

4.2.2 Perencanaan Pelat Atap

Pada perhitungan ini akan diberikan contoh perhitungan perencanaan pelat atap type 1.

4.2.2.1. Pembebanan Pelat Atap

- Beban mati pelat atap :

  1. berat sendiri pelat (perkiraan) : 0,10 x 24 = 2,40 KN/m^2
  2. lapisan kedap air/aspal (tebal 3 cm) : 0,03 x 24 = 0,72 KN/m^2 +

  beban mati total (qD) = 3,12 KN/m^2

- Beban hidup pelat :

  - fungsi bangunan sebagai ruang laboratorium ⟶ qL = 1 kN/m^2

  (PPIUG, 1983 tabel 3.1, halaman 17)

- Kombinasi Pembebanan (SK SNI T-15-1991-03, Pasal 3.2.2)

  - qU = 1,2.qD + 1,6.qL = 1,2. 3,12 + 1,6. 1,0 = 5,344 KNm

- Digunakan tulangan pokok ∅ 10 mm
- Mutu beton (fc') = 22,5 Mpa
- Mutu baja (fy) = 240 Mpa
- Penutup beton digunakan : Pb = 20 mm
- Digunakan h = 100 mm
- Tinggi manfaat tulangan pelat :

  - Arah lapangan – x : $dx = h - Pb - \frac{1}{2}\varnothing_{tul.x}$

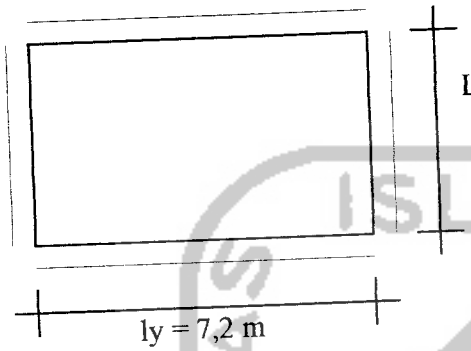
    $= 100 - 20 - \frac{1}{2}.10 = 75$ mm

  - Arah lapangan – y : $dy = h - Pb - \varnothing_{tul.x} - \frac{1}{2}\varnothing_{tul.y}$

= 100 – 20 – 10 - ½.10 = 65 mm

- Arah tumpuan – x dan y = 75 mm

4.2.2.2. Perhitungan Tulangan Pokok Pelat Atap Tipe I



$\frac{ly}{lx} = \frac{7,2}{3,6} = 2,0$

Dari tabel 13.3.2, halaman 203 PBBI NI-2 (pelat dianggap jepit elastis)

Didapat : Clx = -Ctx = 62

Cly = -Cty = 35

- Momen-momen yang bekerja pada pelat :

Mu = 0,001. qU. lx^2 . C

Mulx = -Mutx = 0,001 . 5,344 . $3,6^2$. 62 = 4,294 KNm

Muly = -Muty = 0,001 . 5,344 . $3,6^2$. 35 = 2,424 KNm

a. Perencanaan Tulangan Arah Mulx dan Mutx

Mulx = - Mutx = 4,294 KNm

$Mu/\phi = 4,294/0,8 = 5,3675$ KNm

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{5{,}3675.10^6}{1000.75^2} = 0{,}954 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}00583$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{240}.\left(\frac{600}{600+240}\right) = 0{,}048$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}048 = 0{,}036$$

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

$$= \frac{1}{12{,}549}.\left(1-\sqrt{1-\frac{2.12{,}549.0{,}954}{240}}\right) = 0{,}0041 < \rho_{maks} = 0{,}036$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33.\rho_{aktual} = 1{,}33.\ 0{,}0041 = 0{,}00545 < \rho_{min} = 0{,}00583$

sehingga dipakai : $\rho_{pakai} = 0{,}00545$

$As_{perlu} = \rho_{pakai}.\ b.\ d = 0{,}00545.1000.75 = 408{,}975 \text{ mm}^2$

Digunakan tulangan polos ∅8 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.8^2 = 50{,}265 \text{ mm}^2$

Jarak antar tulangan : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{50{,}265.1000}{408{,}975} \le 122{,}9$ mm

$s \le 2h \le 2.100 \le 200$

$s \le 250$

<u>Dipakai tulangan pokok : P8 – 120 mm</u>

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{50,265.1000}{120} = 418,875 \text{ mm}^2 > As_{ada} = 408,975 \text{ mm}^2$$

Kontrol Kapasitas Momen (Mn) :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{418,875.240}{0,85.22,5.1000} = 5,256 \text{ mm}$$

$$Mn = As_{ada}.fy.\ (d - a/2) = 418,875.240\ (75 - 5,265/2)$$

$$= 7,276 \text{ KNm} \geq Mu/\phi = 5,3675 \text{ KNm} \text{ .....OK!}$$

b. Perencanaan Tulangan Arah Muly

Muly = - Muty = 2,424 KNm

$$Mu/\phi = 2,424/0,8 = 3,03 \text{ KNm}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{3,03.10^6}{1000.65^2} = 0,717 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{240}\left(\frac{600}{600+240}\right) = 0,048$$

$$\rho_{maks} = 0,75.\ \rho b = 0,75.\ 0,048 = 0,036$$

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

$$= \frac{1}{12,549}\left(1-\sqrt{1-\frac{2.12,549.0,717}{240}}\right) = 0,003 < \rho_{maks} = 0,036$$

$$< \rho_{min} = 0,00583$$

$1,33.\rho_{aktual} = 1,33.\ 0,003 = 0,0041 < \rho_{min} = 0,00583$

sehingga dipakai : $\rho_{pakai} = 0,0041$

$As_{perlu} = \rho_{pakai}.\ b.\ d$

$= 0,0041.1000.65 = 263,36\ mm^2$

Digunakan tulangan polos ∅8 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50,265\ mm^2$

Jarak antar tulangan pokok : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{50,265.1000}{263,36} \le 190,86$ mm

$s \le 2h \le 2.100 \le 200$

$s \le 250$

Dipakai tulangan pokok : P8 – 120 mm

$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{50,265.1000}{120} = 418,875\ mm^2 > As_{ada} = 263,36\ mm^2$

Kontrol Kapasitas Momen (Mn) :

$a = \frac{As_{ada}.fy}{0,85.f.c.b} = \frac{418,875.240}{0,85.22,5.1000} = 5,256$ mm

$Mn = As_{ada}.fy.\ (d - a/2) = 418,875.240\ (75 - 5,265/2)$

$= 7,276$ KNm $\ge Mu/\phi = 3,03$ KNm **.....OK!**

c. Perencanaan Tulangan Arah Muty

Muly = - Muty = 2,424 KNm

$Mu/\phi = 2{,}424/0{,}8 = 3{,}03$ KNm

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}549$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mn}{b.d^2} = \frac{3{,}03.10^6}{1000.75^2} = 0{,}54 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}00583$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{240}.\left(\frac{600}{600+240}\right) = 0{,}048$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}048 = 0{,}036$$

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

$$= \frac{1}{12{,}549}.\left(1-\sqrt{1-\frac{2.12{,}549.0{,}54}{240}}\right) = 0{,}0023 < \rho_{maks} = 0{,}036$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33.\rho_{aktual} = 1{,}33.\ 0{,}0023 = 0{,}00303 < \rho_{min} = 0{,}00583$

sehingga dipakai : $\rho_{pakai} = 0{,}00303$

$As_{perlu} = \rho_{pakai}.\ b.\ d = 0{,}00303.1000.75 = 227{,}13\ mm^2$

Digunakan tulangan polos ∅8 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50{,}265\ mm^2$

Jarak antar tulangan : $s \le \frac{A_{\theta 1}.b}{As_{perlu}} \le \frac{50{,}265.1000}{227{,}13} \le 221{,}3$ mm

$s \le 2h \le 2.100 \le 200$

$s \le 250$

Dipakai tulangan pokok : P8 – 120 mm

$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{50{,}265.1000}{120} = 418{,}875\ mm^2 > As_{ada} = 227{,}13\ mm^2$

Kontrol Kapasitas Momen (Mn) :

$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{418{,}875.240}{0{,}85.22{,}5.1000} = 5{,}256$ mm

$Mn = As_{ada}.fy.\ (d - \frac{a}{2}) = 418{,}875.240\ (75 - \frac{5{,}265}{2})$

$= 7{,}276$ KNm $\ge \frac{Mu}{\phi} = 3{,}03$ KNm **.....OK!**

4.2.2.3. Perencanaan Tulangan Bagi Pelat Atap

$As_{bagi} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 100 = 200\ mm^2$

Digunakan tulangan polos ∅6 mm, sehingga luas tampang 1 tulangan bagi :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 8^2 = 50{,}24\ mm^2$

Jarak antar tulangan pokok : $s \le \frac{A_{\theta 1}.b}{As_{bagi}} \le \frac{50{,}24.1000}{200} \le 251{,}2$ mm

Dipakai tulangan bagi : P8 – 200 mm

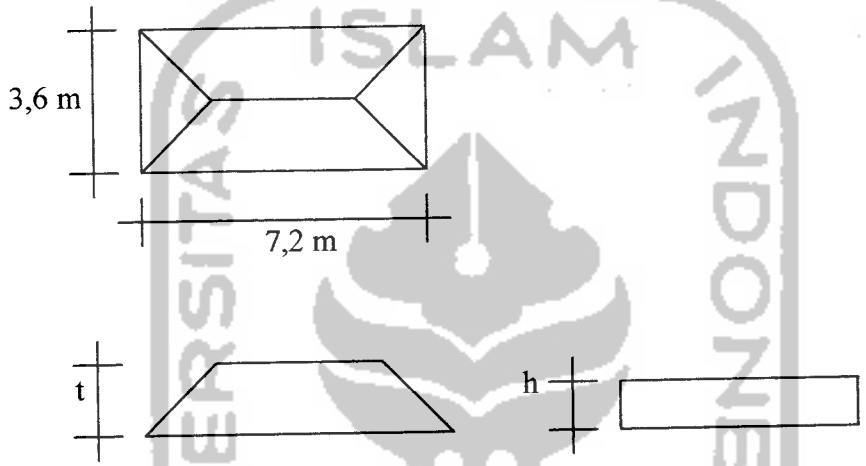
Tabel 4.8 Perencanaan pelat atap

	Mulx	Mutx	Muly	Muty
fc' (Mpa)	22.5	22.5	22.5	22.5
fy (Mpa)	240	240	240	240
ly (m)	7.2	7.2	7.2	7.2
lx (m)	3.6	3.6	3.6	3.6
ly/lx	2.0	2.0	2.0	2.0
C	62	62	35	35
qu (kNm)	5.344	5.344	5.344	5.344
Mu (KNm)	4.294	4.294	2.424	2.424
Mu/ϕ (KNm)	5.368	5.368	3.030	3.030
d (mm)	75	75	65	75
m	12.549	12.549	12.549	12.549
Rn (MPa)	0.954	0.954	0.717	0.539
ρ_{min}	0.00583	0.00583	0.00583	0.00583
ρb	0.04838	0.04838	0.04838	0.04838
ρ_{maks}	0.03629	0.03629	0.03629	0.03629
ρ_{aktual}	0.00408	0.00408	0.00305	0.00228
$1.33.\rho_{aktual}$	0.00543	0.00543	0.00405	0.00303
ρ_{pakai}	0.00543	0.00543	0.00405	0.00303
As ada (mm2)	407.020	407.020	263.250	227.250
dtul.pokok (mm)	8	8	8	8
A1d.pokok (mm2)	50.286	50.286	50.286	50.286
s (mm)	123.546	123.546	191.019	221.279
s pakai (mm)	120	120	120	120
As aktual (mm2)	419.048	419.048	419.048	419.048
a (mm)	5.259	5.259	5.259	5.259
Mn (KNm)	7.278	7.278	6.273	7.278
Tul. Pokok	P8-120	P8-120	P8-120	P8-120
Kontrol	AMAN	AMAN	AMAN	AMAN
As bagi (mm2)		200		200
dtul.bagi (mm)		8		8
A1d.bagi (mm2)		50.286		50.286
x (mm)		251.429		251.429
xpakai (mm)		200		200
Tul. Bagi		P8-200		P8-200

4.3 PERENCANAAN BALOK

Denah rencana balok anak pada lampiran. Hasil perhitungan perencanaan balok anak Gedung Laboratorium Terpadu Universitas Islam Indonesia Jogjakarta sebagai berikut :

4.3.1 Perhitungan Balok Anak B2'



$$t = \frac{1}{2}b \qquad h = t - \left(\frac{4}{3}.\frac{t^3}{L^2}\right)$$

$$t = \frac{1}{2}.3{,}6 = 1{,}8 \text{ m} \qquad h = 1{,}8 - \frac{4}{3}.\frac{1{,}8^3}{7{,}2^2} = 1{,}65 \text{ m}$$

Gambar 4.10 Tipe Pembebanan

4.3.1.1Data

a) berat jenis beton = 24 KN/m^3

b) q_D pelat = 4,69 KN/m^2

c) qL pelat = 2,5 KN/m^2

d) perkiraan ukuran balok

L = 7,2 m = 720 cm

$h = \sim \frac{1}{12} . 720 = 60 \text{ cm} = 0{,}6 \text{ m}$

sehingga asumsi ukuran balok = 0,3 x 0,6 m

e) tinggi tembok (lantai 1) = 3,85 m

4.3.1.2 Perhitungan Tulangan Lentur

a) Pembebanan

beban pelat = $h.qD.n = 1{,}65.4{,}69.2 = 15{,}477$ KN/m

berat balok = $b_{blk}. (h_{blk} - t_{pelat}).bj$

= $0{,}3.(0{,}6-0{,}12).24 = 3{,}456$ KN/m

beban tembok = $berat_{tembok} (t_{tembok} - h_{blk})$

= $2{,}5.(3{,}85-0{,}6) = 8{,}125$ KN/m

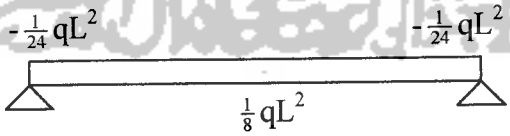
- q_D balok anak = $15{,}477 + 3{,}456 + 8{,}125 = 27{,}058$ KN/m

- qL balok anak = $h.qL.n = 1{,}65.2{,}5{,}2 = 8{,}25$ KN/m

$\therefore$ qu balok anak = $1{,}2.qD + 1{,}6.qL$

= $1{,}2.\ 27{,}058 + 1{,}6.\ 8{,}25 = 45{,}67$ KN/m

b) Mengitung momen



Gambar 4.11 Koefisien Momen (Sumber : PBI 1971 sesuai pasal 13.2)

$Mu_1 = \frac{1}{24} . q_u . l^2 = \frac{1}{24} . 45{,}67 . 7{,}2^2 = 98{,}647$ KNm

$Mu_2 = \frac{1}{8} . q_u . l^2 = \frac{1}{8} . 45{,}67 . 7{,}2^2 = 295{,}942$ KNm

c) penulangan balok

data :

f'c = 22,5 Mpa

fy ulir = 400 Mpa

∅ tul pokok = 22 mm

∅ tulangan sengkang = 10 mm

untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0,85$

f'c > 30 Mpa ⇒ $\beta_1 = 0,85-0,008\ (f'c-30) \geq 0,65$

perhitungan

$$\rho_b = \frac{0,85.f'c}{fy}\beta\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{400}0,85\left(\frac{600}{600+400}\right) = 0,0244$$

$$\rho_{maks} = 0,75.\ \rho_b = 0,75.\ 0,0244 = 0,0183$$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$$

diambil $\rho = 0,5\ \rho_{maks} = 0,00915$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22,5} = 20,915$$

$$Rn = \rho.fy.(1-0,5.\ \rho.m) = 0,00915.400.(1-\frac{1}{2}.0,00915.20,915) = 3,322$$

1) $\frac{Mu_2}{\phi} = \frac{295,942}{0,8} = 369,93$ KN/m

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{369,93.10^6}{3,322} = 111357615,9\ mm^2$$

diambil b = 0,5.d

$0,6.d.d^2 = 111357615,9\ mm^2$

$d_{perlu} = 570,41$ mm

$b = 0,6\ d = 0,6.570,41 = 342,25$ ambil b = 350 m

ambil h = 650 mm

d_{pakai} = h-pb-Øsengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 650 - 40 - 10 - \frac{1}{2}.22 = 589 \text{ mm}$$

$d_{pakai} > d_{perlu}$ maka dipakai tulangan sebelah

Karena ukuran balok yang dipakai adalah 0,35 x 0,65 m maka momen :

berat balok = $b_{blk}.(h_{blk} - t_{pelat}).bj.1,2 = 0,35.(0,65-0,12).24 = 4,452$ KN/m

q_D balok anak = 15,477 + 4,452 + 8,125 = 28,054 KN/m

Jadi qu balok anak = 1,2.28,054 + 1,6. 8,25 = 46,865 KN/m

Sehingga :

$$Mu_1 = \frac{1}{24}.q_u.l^2 = \frac{1}{24}.46,865.7,2^2 = 101,23 \text{ KNm}$$

$$Mu_2 = \frac{1}{8}.q_u.l^2 = \frac{1}{8}.46,865.7,2^2 = 303,685 \text{ KNm}$$

Untuk perhitungan menggunakan tulangan sebelah, maka penulangan lapangan dengan Mu = 303,685 KNm :

$$\frac{Mu_2}{\phi} = \frac{303,685}{0,8} = 379,61 \text{ KN/m}$$

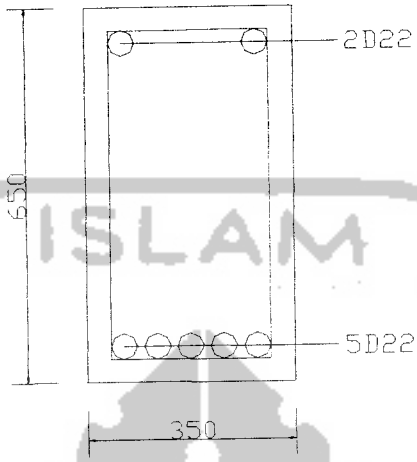
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{379,61.10^6}{350.589^2} = 3,126$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{3,126}{3,322}.0,00915 = 0,00861$$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00861.350.589 = 1774{,}952\ mm^2$

$A\phi_{22} = 380{,}133\ mm^2$

dipakai $5\phi_{22}$ $As_{ada} = 1900{,}66\ mm^2 > As_{perlu} = 1774{,}952\ mm^2$



Gambar 4.12 Penampang melintang balok anak

$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$$

$$= \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35\ mm$$

Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f.c.b} = \frac{1900{,}66.400}{0{,}85.22{,}5.350} = 113{,}58\ mm$$

$$Mn = As_{ada}.fy.\ (d - {}^{a}/_{2}) = 1900{,}66.400\ (589 - {}^{113{,}58}/_{2})$$

$$= 404{,}62\ KNm \geq {}^{Mu}/_{\phi} = 379{,}61\ KNm\text{.....}\mathbf{OK!}$$

2) Penulangan tumpuan untuk $Mu_1 = 101{,}23$ KNm

$$\frac{Mu}{\phi} = \frac{101{,}23}{0{,}8} = 126{,}5375\ KNm$$

Mx (KNm)	265.7201
My (KNm)	349.0082
t kolom (mm)	0.6
d (mm)	619.00
m (m)	1.4435
Bx	7
By	2.8
x (m)	1.219
y (m)	1.219
f'c (MPa)	22.5
qu max (KN/m2)	173.3035102
qu min (KN/m2)	79.21244388
qu1 (KN/m2)	124.7962051
qu2 (KN/m2)	127.719749
qu terjadi (KN/m2)	126.257977
Vu (KN)	802.2481102
Vu/ϕ (KN)	**1337.080184**
βc	1.0
bo (mm)	4876
Vc1 (KN)	85900.7302
Vc2 (KN)	57267.1534
Vc pakai(KN)	**57267.1534**
Kontrol	**AMAN**

Kuat tumpuan pondasi	
luas pondasi/A2 (m2)	19.6000
luas Kolom/A3 (m2)	0.3600
(A2/A3)^0,5	7.3786
jika lebih besar dari 2, dipakai nilai 2	
ϕPn (KN)	9639.0
Kuat tumpuan kolom	
ϕPn (KN)	4819.5
Kontrol ϕPn kolom <= ϕPN pondasi	
AMAN	
Tul Lentur sisi Panjang arah X	

qux (KN/m2)	173.3035
L (m)	7.00
h kolom (m)	0.60
l1 (m)	3.20
Mu1 (KNm)	887.3140
tebal pelat/h (mm)	700
Pb (mm)	70
d (mm)	619.00
f'c (MPa)	22.5
fy (MPa)	400
β1	0.85
m	20.9150
Rn (MPa)	2.3158
ρ_b	0.02438
ρ_{min}	0.00350
ρ_{maks}	0.01829
ρ	0.00619
1,33.ρ	0.00823
ρpakai	0.00613
As pertu (mm2)	3794.4700
dtul.pokok (mm)	22
A1d.pokok (mm2)	379.9400
jrk tul. pokok/s (mm)	100.1299
jrk tul. pakai/s (mm)	90
tul pokok pakai	**P22 - 90**
As aktual (mm2)	4221.5556
a (mm)	88.2940
Mn (kNm)	970.7096
Kontrol	**AMAN**
dtul.susut (mm)	12
A1d.susut (mm2)	113.0400
As susut (mm2)	1238.0000
jrk tul. susut/s (mm)	91.3086
jrk tul. pakai/s (mm)	90
tul pokok pakai	**P12 - 90**

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{126{,}5375.10^6}{350.589^2} = 1{,}0421$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{1{,}0421}{3{,}322}.0{,}00915 = 0{,}00287 < \rho_{min} = 0{,}0035$$

$1{,}33\ \rho_{baru} = 1{,}33.0{,}00287 = 0{,}00382 > \rho_{min} = 0{,}0035$

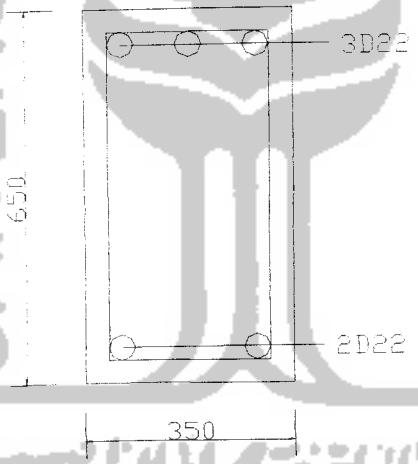
sehingga $\rho_{pakai} = 1{,}33\ \rho_{baru} = 0{,}00382$

$As_{perlu} = \rho_{pakai}.b.d = 0{,}00382.350.589 = 787{,}493\ mm^2$

$As_{min} = \rho_{min}.b.d = 0{,}0035.350.589 = 721{,}525\ mm^2$

$A\phi_{22} = 380{,}133\ mm^2$

dipakai $3\phi 22$ $As_{ada} = 1140{,}4\ mm^2 > As_{perlu} = 787{,}493\ mm^2$



Gambar 4.13 Penampang melintang balok anak

$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$$

$$= \frac{350 - 2.40 - 2.10 - 3.22}{(3-1)} = 92mm$$

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0{,}85.f.c.b} = \frac{1140{,}4.400}{0{,}85.22{,}5.350} = 68{,}15$$

$$Mn = As_{ada}.fy.\ (d - a/2) = 1140{,}4.400\ (589 - 68{,}15/2)$$

$$= 253{,}14\ KNm \geq Mu/\phi = 126{,}5375\ KNm\text{.....}\mathbf{OK!}$$

4.3.2 Perhitungan penulangan geser balok anak

a. Perhitungan tulangan geser balok B2'

diketahui : qu = 46,865 KN/m b = 300 mm fy = 240 Mpa

L = 7,2 m h = 650 mm ql = 8,25 KN/m

f'c = 22,5 Mpa d = 589 mm

Gaya geser pada tumpuan :

$$VU_{tumpuan} = \frac{1}{2}.1{,}15.qu.L = \frac{1}{2}.1{,}15.46{,}865.7{,}2 = 194{,}02\ KN$$

$$\frac{Vu_{tumpuan}}{\phi} = \frac{194{,}02}{0{,}6} = 323{,}37\ KN$$

Gaya geser pada setengah bentang :

$$VU_{tengah} = \frac{1}{8}.ql.L = \frac{1}{8}.(1{,}6.8{,}25).7{,}2 = 11{,}88\ KN$$

$$\frac{VU_{tengah}}{\phi} = \frac{11{,}88}{0{,}6} = 19{,}8\ KN$$

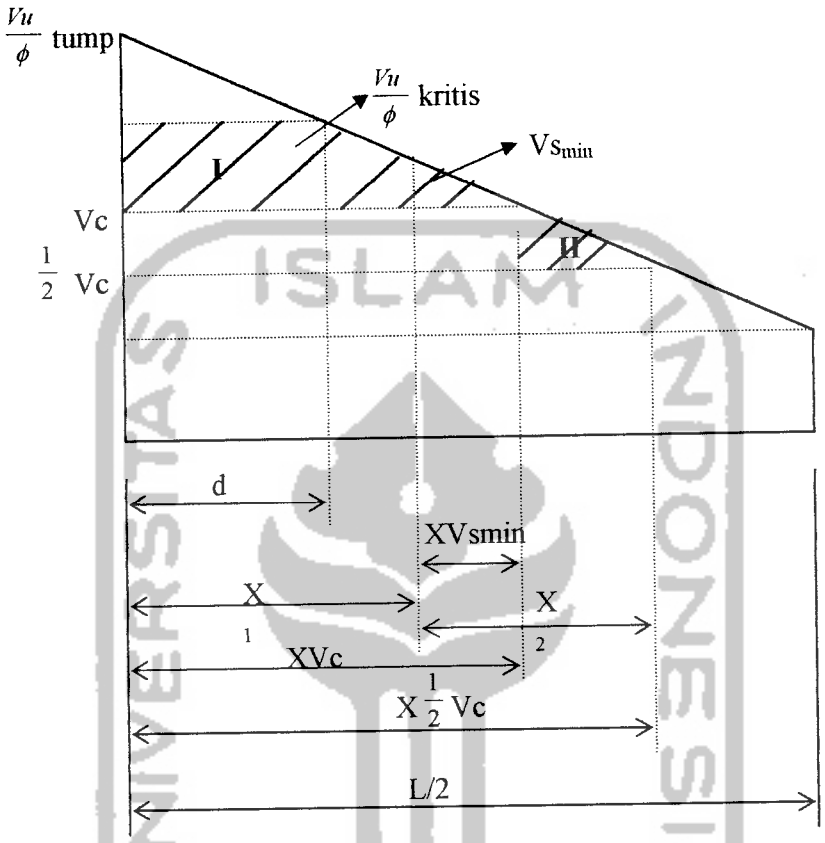
Gaya geser beton :

$$Vc = \frac{1}{6}\sqrt{f'c}.b.d = \frac{1}{6}\sqrt{22{,}5}.350.589 = 162{,}976\ KN$$

$$\frac{1}{2}Vc = 81{,}488\ KN$$

$3Vc = 488{,}928$ KN

$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3}.350.589 = 68{,}717$ KN



Gambar 4.14 Gaya geser pada penampang kritis

$\frac{Vu}{\phi}$ kritis untuk perencaan diambil sejauh d dari tumpuan

$$\frac{N}{(323{,}37-19{,}8)} = \frac{1/2L-d}{1/2L} \Rightarrow N = \frac{3{,}6-0{,}589}{3{,}6}.303{,}398 = 253{,}76$$

$\frac{Vu}{\phi}$ kritis $= 253{,}76 + 19{,}8 = 273{,}56$ KN

Jarak Vs_{min} (XVs_{min}) $= \frac{68{,}717}{303{,}398} x 3{,}6 = 0{,}82$ m

$(Vc + Vs_{min}) < \frac{Vu}{\phi}$ kritis $\leq 3Vc$

231,693 KN < 273,56 KN ≤ 488,928 KN

Daerah I

digunakan sengkang ∅10mm

$Av = 2.\ 1/4\ \pi\ D^2 = 2.\ 1/4\ \pi\ 10^2\ mm^2 = 157{,}08\ mm^2$

$S_1 \leq \frac{Av.fy.d}{Vs} = \frac{157{,}08.240.589}{(273{,}56 - 162{,}976).10^3} = 200{,}8\ mm \approx 200\ mm$

$S_2 \leq \frac{d}{2} = \frac{589}{2} = 294{,}5\ mm$

$S_3 \leq 600\ mm$

Dipakai P10-200

Daerah II

digunakan sengakang ∅8 mm, dengan $Av = 100{,}53\ mm^2$

$S_1 \leq \frac{Av.fy.d}{Vs\,min} = \frac{100{,}53.240.589}{68{,}717.10^3} = 206{,}8\ mm \approx 200\ mm$

$S_2 \leq \frac{d}{2} = \frac{589}{2} = 294{,}5\ mm$

$S_3 \leq 600\ mm$

Dipakai P8-200

Tabel 4.9 Perencanaan lentur balok anak tipe –B1'

	Tumpuan	Lapangan
asumsi balok (m)	0.3 x 0.6	0.3 x 0.6
L (m)	7.2	7.2
qu (kN/m')	15.477	15.477
Mu (kNm)	33.430	100.291
f'c (Mpa)	22.5	22.5
fy (Mpa)	400	400
β1	0.85	0.85
ρb	0.0244	0.0244
ρmaks	0.0183	0.0183
ρmin	0.0035	0.0035
ρpakai	0.0091	0.0091
m	20.915	20.915
Rn (Mpa)	3.308	3.308
Mu/∅ (kNm)	-	125.364
b.d2 (mm3)	-	37898355.5
dperlu (mm)	-	398.251
b (mm)	-	238.950
bpakai (mm)	350	350
h (mm)	650	650
dpakai (mm)	589	589
pakai	tul.sebelah	tul.sebelah
pakai balok (m)	0.35 x 0.65	0.35 x 0.65
qu (kN/m')	46.7148	46.7148
Mu (kNm)	100.904	302.712
Mu/∅ (kNm)	126.130	378.390
Rnbaru (Mpa)	1.039	3.116
ρbaru	0.00287	0.00861
1.33ρ	0.00382	0.01146
ρpakai	0.00382	0.00861
As perlu (mm2)	787.311	1775.888
As min (mm2)	721.525	721.525
∅tul.pokok (mm)	**22**	**22**
A1∅ (mm2)	380.286	380.286
tul.terpakai (n buah)	**3**	**5**
As ada	1140.857	1901.429
jarak (mm)	92	35
a (mm)	68.174	113.624
Mn (kNm)	253.230	404.767
kontrol	**Aman**	**Aman**

Tabel 4.10 Perencanaan geser balok anak tipe –B1'

Vu tump (kN)	193.399
Vu tump/∅ (kN)	322.332
Vu teng (kN)	11.88
Vu teng/∅ (kN)	19.8
Vc (kN)	162.976
3Vc (kN)	488.928
Vsmin (kN)	68.717
Vu/∅kritis (kN)	272.835
Daerah I	
Vs (kN)	109.859
s (mm)	202.122
s (mm)	294.500
s (mm)	600.000
spakai (mm)	200
Perencanaan	**P10-200**
Daerah II	
s (mm)	206.805
s (mm)	294.500
s (mm)	600.000
spakai (mm)	200
Perencanaan	**P8-200**

4.4 ANALISIS STRUKTUR PORTAL

1. Beban mati yang digunakan

Beban mati seperti yang tercantum pada tabel berikut :

No	Jenis Material	Beban
1	Beton bertulang	24 kN/m^3
2	Tegel per	24 kN/m^3
3	Spesi	21 kN/m^3
4	Plafond	0,18 kN/m^2
5	Tembok	2,5 kN/m^2

Perhitungan pembebanan pelat lantai untuk beban mati per m2

- Pelat beton = 0,12 . 24 = 2,88 kN/m^2
- Keramik = 0,02 . 24 = 0,48 kN/m^2
- Spesi = 0,04 . 21 = 0,84 kN/m^2
- Plafond = 0,18 kN/m^2

4,40 kN/m^2

Perhitungan pembebanan pelat atap untuk beban mati per m^2

- Pelat beton = 0,12 . 24 = 2,88 kN/m^2
- Spesi = 0,04 . 21 = 0,84 kN/m^2
- Plafond = 0,18 kN/m^2

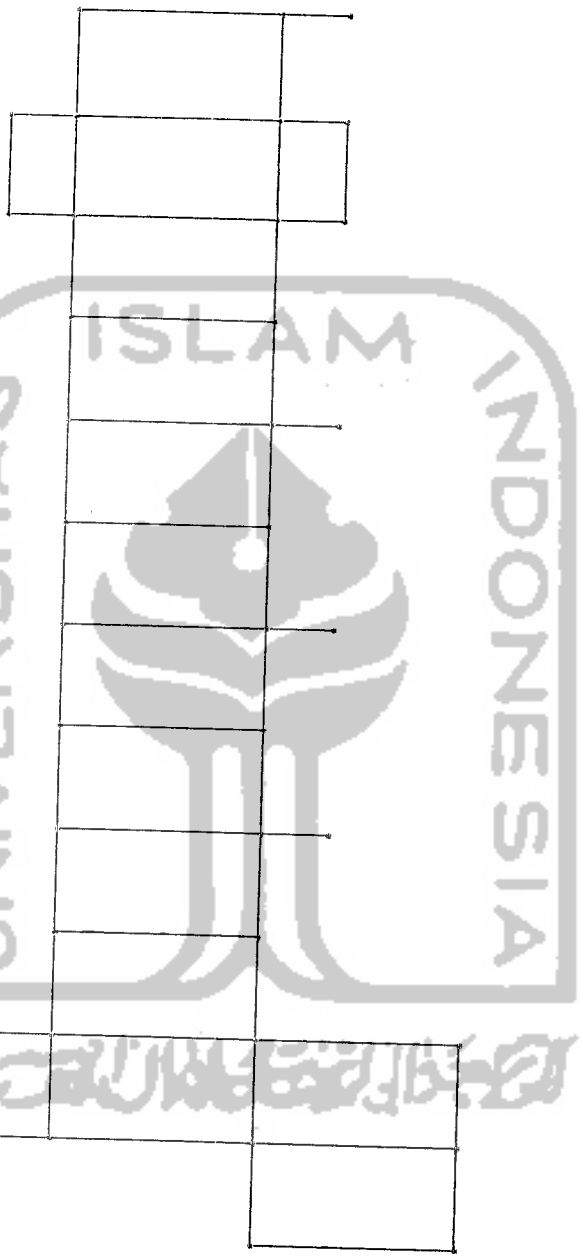
3,90 kN/m^2

2. Beban hidup yang digunakan

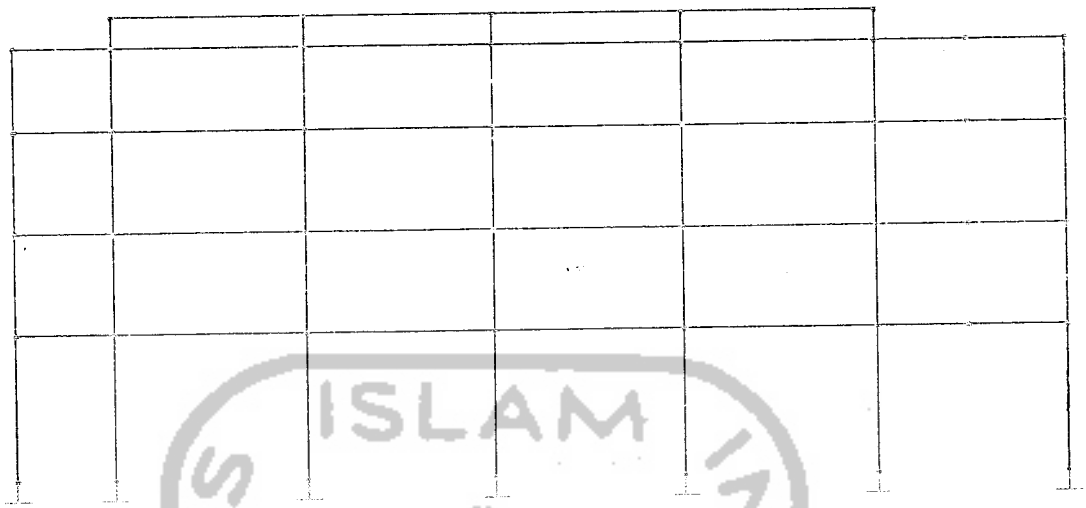
Beban hidup pelat lantai = 2,5 KN/m^2

Beban hidup pelat selasar = 3,0 KN/m^2

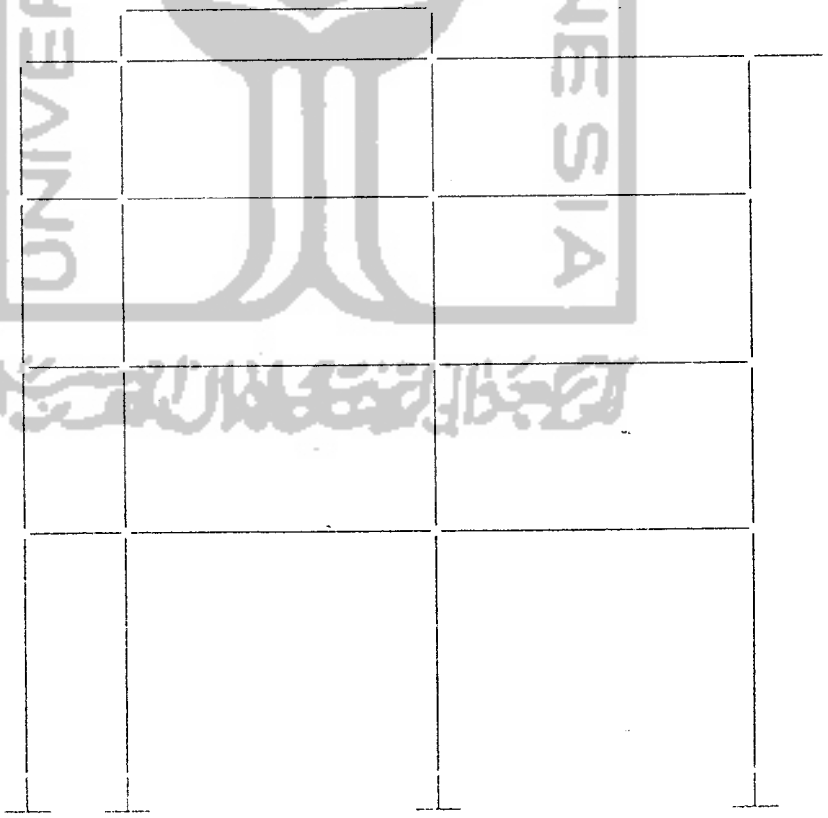
Beban hidup pekerja atap = 1,0 KN/m^2



GAMBAR DENAH STRUKTUR LANTAI 1, 2, DAN 3
TAMPAK ATAS



Gambar denah struktur tampak samping



Gambar denah strukur tampak depan

4.4.1 Perhitungan Beban Akibat Gravitasi

4.4.1.1 Portal as A^o

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,14^2/3,6^2))$ = 0,866

- Pelat lantai = 0,866. 1,14 . 4,4 = 8,69 kN/m'
- Dinding = (3,85-0,7) . 2,5 = 7, 87 kN/m' +

qd_1 = 16,56 kN/m'

b. Beban merata atap pelat

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,14^2/6^2))$ = 0,967

- Pelat atap = 0,866 . 1,14 . 3,9 = 3,85 kN/m'
- Mangkokan =1 . 3,9 = 3,9 kN/m' +

qd_2 = 7,75 kN/m'

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

-Ruang laboratorium = 0,866 . 1,14 . 2,5 = ql_1 = 2,47 kN/m'

b. Beban merata atap pelat

- Pekerja = 0,866 . 1,14 . 1 = 0,99 kN/m'
- Pekerja = 1 . 1 = 1 kN/m' +

ql_2 = 1,99 kN/m'

4.4.1.2 Portal as A

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang 1'-2

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1{,}14^2/3{,}6^2)) = 0{,}866$

- Pelat lantai = 0,866 . 1,14 . 4,4 . 2 = 8,68 kN/m'
- Dinding = 3,15 . 2,5 = 7, 87 kN/m' +

qd_1 = 16,55 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

- Pelat tritisan = .1 . 3,9 = 3,9 kN/m'
- Pelat lantai = 2 . 2/3 . 1.8 . 4,4 = 10,56 kN/m'
- Dinding = 3,15 .2,5 = 7,87 kN/m' +

qd_2 = 22,33 kN/m'

b. Beban merata atap pelat

- Bentang 1'-2

- Pelat atap = 0,866 . 1,14 . 3,9 = 3,85 kN/m'
- Pelat atap = 2/3 . 1,8 . 3,9 = 4,68 kN/m' +

qd_3 = 8,53 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1{,}14^2/7{,}2^2)) = 0{,}967$

- Pelat atap = 0,967 . 1,14 . 3,9 qd_3 = 4,3 kN/m'

- Bentang 6-7

- Pelat atap = 2 . 2/3 . 1.8 . 3,9 = 9,36 kN/m'

= 0,967 . 1,14 . 3,9 = 4,3 kN/m' +

qd_4 = 13,66 kN/m'

d. Beban terpusat lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 0,917 . 1,8 . 7,2 . 4,4 = 52,29 kN
- Balok anak = 0,35 . 0,6 . 3,6 . 24 = 18,14 kN
- Dinding = 3,15 . 3,6 . 2,5 = 28,35 kN +

P_1 = 98,78 kN

e. Beban terpusat atap pelat

- Pelat atap = 0,917 . 1,8 . 7,2 . 3,9 = 46,35 kN
- Balok anak = 0,35 . 0,6 . 3,6 . 24 = 18,14 kN +

P_2 = 64,49 kN

e. Beban terpusat atap genteng

- kuda-kuda 1 P_3 = 17,85 kN
- kuda-kuda 2 P_4 = 1,10 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

- Bentang 1'-2

-Ruang laboratorium = 0,866 . 1,14 . 2,5 . 2 = 5,94 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

-Ruang laboratorium = 2 . 2/3 . 1,8 . 2,5 = 6,00 kN/m'

- Pekerja = 1 . 1 = 1,00 kN/m' +

ql_2 = 7,00 kN/m

b. Beban merata atap pelat

- Bentang 1'-2

-Pekerja = 0,866 . 1,14 . 1 = 0,99 kN/m'

-Pekerja = 2/3 . 1,8 . 1 = 1,2 kN/m' +

ql_4 = 2,19 kN/m

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6

- Pekerja = 0,967 . 1,14 . 1 = 1,10 kN/m'

- Bentang 6-7

- Pekerja = 0,967 . 1,14 . 1 = 1,10 kN/m'

-Pekerja = 2/3 . 3,6 . 1 . 2 = 4,8 kN/m' +

ql_5 = 5,9 kN/m'

c. Beban titik lantai 1, 2, dan 3

- Pelat lantai = 0,917 . 1,8 . 7,2 . 2,5 = 29,71 kN

d. Beban titik atap pelat

- Pelat atap = 0,917 . 1,8 . 7,2 . 1 = 11,88 kN

4.4.1.3 Portal as B

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang 1-1'= Bentang 1'-2

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1{,}14^2/3{,}6^2)) = 0{,}866$

- Pelat lantai = 0,866 . 1,14 . 4,4 = 4,34 kN/m'

- Pelat lantai = 2/3 . 1,8 . 4,4 = 5,28 kN/m'

- Dinding = 3,15 . 2,5 = 7, 87 kN/m' +

qd_1 = 17,49 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,2^2/7,2^2))$ = 0,963

- Pelat selasar = 0,963 . 1,2 . 4,4 = 5,08 kN/m'

- Pelat lantai = 2 . 2/3 . 1.8 . 4,4 = 10,56 kN/m'

- Dinding = 3,15 .2,5 = 7,87 kN/m' +

qd_2 = 23,51 kN/m'

b. Beban merata atap

- Bentang 1'-2

- Pelat atap = 0,866 . 1,14 . 3,9 = 3,85 kN/m'

- Pelat atap = 2/3 . 1,8 . 3,9 = 4,68 kN/m' +

qd_4 = 8,53 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6

- Pelat atap = 0,917 . 1,8 . 3,9 qd_5 = 6,44 kN/m'

- Bentang 6-7

- Pelat atap = 2 . 2/3 . 1.8 . 3,9 = 9,36 kN/m'

= 0,917 . 1,8 . 3,9 = 6,44 kN/m' +

qd_6 = 15,80 kN/m'

d. Beban terpusat lantai 1, 2, dan 3

- Bentang 1'-2

- Pelat atap = 2 . 2/3 . 1,8 . 1,8 . 4,4 = 19,01 kN

- Balok anak = 0,35 . 0,6 . 1,05 . 24 = 5,29 kN

P_1 = 32,57 kN

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

- Pelat atap =0,917 . 1,8 . 7,2 . 4,4 = 52,29 kN
- Balok anak = 0,35 . 0,6 . 3,6 . 24 = 18,14 kN
- Dinding = 3,15 . 3,6 . 2,5 = 28,35 kN +

P_2 = 98,78 kN

- Bentang 4-5

- Balok anak = 0,35 . 0,6 . 2,4 . 24 = 12,10 kN

e. Beban terpusat atap pelat

- Balok anak = 0,35 . 0,6 . 5,4 . 24 = P_4 = 27,22 kN
- Balok anak = 0,35 . 0,6 . 2,4 . 24 = P_5 = 12,10 kN
- Pelat = 0,917 . 1,8 . 7,2 . 3,9 = 46,35 kN
- Balok anak = 0,35 . 0,6 . 3,6 . 24 = 18,14 kN +

P_6 = 64,49 kN

e. Beban terpusat atap genteng

- kuda-kuda 1 $P_5 = P_6$ = 17,85 kN
- kuda-kuda 2 P_7 = 1,10 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

- Bentang 1'-2

-Ruang laboratorium = 0,866 . 1,14 . 2,5 = 2,56 kN/m'
-Ruang laboratorium = 2/3 . 1,8 . 2,5 = 3 kN/m' +

ql_1 = 5,56 kN/m

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

-Ruang laboratorium = 2 . 2/3 . 1,8 . 2,5 = 6,00 kN/m'

- Selasar = 0,963 . 1,2 . 3 = 3,47 kN/m' +

ql_2 = 9,47 kN/m

b. Beban merata atap pelat

- Bentang 1'-2

-Pekerja = 0,866 . 1,14 . 1 = 0,99 kN/m'

-Pekerja = 2/3 . 1,8 . 1 = 1,20 kN/m' +

ql_3 = 2,19 kN/m

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6

- Pekerja = 0,917 . 1,8 . 1 = 1,65 kN/m'

- Bentang 2-3 = Bentang 3-4 = Bentang 4-5 = Bentang 5-6 = Bentang 6-7

- Pekerja = 0,917 . 1,8 . 1 = 1,65 kN/m'

-Pekerja = 2/3 . 3,6 . 1 . 2 = 4,80 kN/m' +

ql_4 = 6,45 kN/m'

d. Beban terpusat lantai 1, 2, dan 3

- Pelat atap = 2 . 2/3 . 1,8 . 1,8 . 2,5 = 10,80 kN

- Pelat atap =0,917 . 1,8 . 7,2 . 2,5 = 29,71 kN

e. Beban terpusat atap pelat

- Pelat = 0,917 . 1,8 . 7,2 . 1 = 11,88 kN

4.4.1.4 Portal as C

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang 1-1'= Bentang 1'-2

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,14^2/3,6^2))$ = 0,866

- Pelat lantai = 0,866 . 1,14 . 4,4 = 4,34 kN/m'
- Pelat lantai = 2/3 . 1,8 . 4,4 = 5,28 kN/m'
- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_1 = 17,49 kN/m'

b. Beban merata atap

- Pelat atap = 0,866 . 1,14 . 3,9 = 3,85 kN/m'
- Pelat atap = 2/3 . 1,8 . 3,9 = 4,68 kN/m' +

qd_4 = 8,53 kN/m'

c. Beban terpusat atap

- Balok anak = 0,35 . 0,6 . 5,4 . 24 = P_4 = 27,22 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

-Ruang laboratorium = 0,866 . 1,14 . 2,5 = 2,56 kN/m'
-Ruang laboratorium = 2/3 . 1,8 . 2,5 = 3 kN/m' +

ql_1 = 5,56 kN/m

b. Beban merata atap

-Pekerja = 0,866 . 1,14 . 1 = 0,99 kN/m'
-Pekerja = 2/3 . 1,8 . 1 = 1,20 kN/m' +

ql_3 = 2,19 kN/m

4.4.1.5 Portal as 1 (Bentang B-C)

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 0,917 . 1,8 . 4,4 = 7,26 kN/m'
- Pelat tritisan = 1,2 . 4,4 = 5,28 kN/m'
- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_1 = 20,41 kN/m'

b. Beban merata atap

- Pelat atap = 0,917 . 1,8 . 3,9 = 6,44 kN/m'
- Pelat atap = 1,8 . 3,9 = 7,02 kN/m' +

qd_1 = 13,46 kN/m'

c. Beban terpusat atap

- Balok anak = 0,35 . 0,6 . 1,8 . 24 = P_1 = 9,07 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

-Ruang laboratorium = 0,917 . 1,8 . 2,5 = 4,13 kN/m'
- Pekerja = 1,2 . 1 = 1,2 kN/m' +

ql_1 = 5,33 kN/m

b. Beban merata atap

-Pekerja = 0,917 . 1,8 . 1 = 1,65 kN/m'
-Pekerja = 1,8 . 1 = 1,80 kN/m' +

ql_2 = 3,45 kN/m

4.4.1.6 Portal as 1'

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang A^0-A

- Pelat lantai = 2/3 . 1,14 . 4,4 = 3,34 kN/m'

- Dinding = 3,15 . 2,5 = 7, 87 kN/m' +

qd_1 = 11,21 kN/m'

- Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 0,917 . 1,8 . 4,4 = 7,26 kN/m'

- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_2 = 15,13 kN/m'

- Bentang B-C

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 2 . 0,917 . 1,8 . 4,4 = 14,52 kN/m'

- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_3 = 22,39 kN/m'

b. Beban merata atap

- Bentang A^0-A

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,14^2/2,3^2))$ = 0,672

- Pelat atap = 0,672 . 1,14 . 3,9 = 2,99 kN/m'

- Pelat atap = 2/3 . 1,14 . 3,9 = 2,96 kN/m' +

qd_4 = 5,95 kN/m'

- Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 0,917 . 1,8 . 3,9 = qd_5 = 6,44 kN/m'

- Pelat lantai = 2 . 0,917 . 1,8 . 3,9 = qd_6 = 14,52 kN/m'

c. Beban terpusat lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/3,6^2))$ = 0,667

- Pelat lantai = 0,667 . 1,8 . 3,6 . 4,4 = 18,02 kN

- Balok anak = 0,35 . 0,6 . 1,8 . 24 = 9,07 kN

- Dinding = 3,1 . 2,5 . = 13,95 kN/m' +

P1 = 41,04 kN

c. Beban terpusat atap

- Pelat lantai = 2 . 0,667 . 1,8 . 3,6 . 3,9 = 33,71 kN

- Balok anak = 0,35 . 0,6 . 1,8 . 24 = 9,07 kN +

P2 = 42,78 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

- Bentang A^0-A

-R. laboratorium = 2/3 . 1,14 . 2,5 qd_1 = 1,90 kN/m'

- Bentang A-B

- R. laboratorium = 0,917 . 1,8 . 2,5 qd_2 = 4,13 kN/m'

- Bentang B-C

- R. laboratorium = 2 . 0,917 . 1,8 . 2,5 qd_3 = 8,26 kN/m

b. Beban merata atap

- Bentang A^0-A

- Pelat atap = 2/3 . 1,14 . 1 = 0,76 kN/m'

- Pelat atap = 0,672 . 1,14 . 1 = 0,77 kN/m' +

qd_4 = 1,53 kN/m'

- Bentang A-B

- Pelat lantai = 0,917 . 1,8 . 1 = qd_5 = 1,65 kN/m'

- Bentang B-C

- Pelat lantai = 2 . 0,917 . 1,8 . 1 = qd_6 = 3,30kN/m'

c. Beban terpusat lantai 1, 2, dan 3

- Pelat lantai = 0,667 . 1,8 . 3,6 . 2,5 = 10,81 kN

c. Beban terpusat atap

- Pelat lantai = 2 . 0,667 . 1,8 . 3,6 . 1 = 8,64 kN

4.4.1.7 Portal as 2

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang A^0-A

- Pelat lantai = 2 . 2/3 . 1,14 . 4,4 = 3,34 kN/m'

- Dinding = 3,15 . 2,5 = 7, 87 kN/m' +

qd_1 = 14.55 kN/m'

- Bentang A-B = Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2)) = 0,917$

- Pelat lantai = 2 . 0,917 . 1,8 . 4,4 = 14,52 kN/m'

- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_3 = 22,39 kN/m'

b. Beban merata atap pelat

- Bentang A^0-A

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,14^2/2,3^2)) = 0,672$

- Pelat atap = 0,672 . 1,14 . 3,9 = 2,99 kN/m'
- Pelat atap = 2/3 . 1,14 . 3,9 = 2,96 kN/m' +

qd_4 = 5,95 kN/m'

- Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2)) = 0,917$

- Pelat lantai = 0,917 . 1,8 . 3,9 = qd_5 = 22,76 kN/m'

- Bentang B-C

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2)) = 0,917$

- Pelat lantai = 2 . 0,917 . 1,8 . 3,9 = qd_5 = 45,52 kN/m'

c. Beban terpusat lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,2^2/7,2^2)) = 0,963$

- Pelat lantai = 0,5 . 0,963 . 1,2 . 7,2 . 4,4 = 18,30 kN
- Balok anak = 0,35 . 0,6 .3,6 . 24 = 18,14 kN
- Dinding = 3,1 . 2,5 . 3,6 = 28,33 kN/m' +

P1 = 64,77 kN

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/3,6^2)) = 0,667$

- Pelat lantai = 0,667 . 1,8 . 3,6 . 4,4 = 18,02 kN
- Balok anak = 0,35 . 0,6 . 1,8 . 24 = 9,07 kN
- Dinding = 3,1 . 2,5 . = 13,95 kN/m' +

P2 = 41,04 kN

c. Beban terpusat atap pelat

- Pelat atap = 0,5 . 0,963 . 1,2 . 7,2 . 4,4 = 18,30 kN

- Balok anak = 0,35 . 0,6 .3,6 . 24 = 18,14 kN +

P1 = 36,44 kN

- Pelat atap = 0,5 . 2/3 . 3,6 . 7,2.. 3,9 = 33,70 kN

- Balok anak = 0,35 . 0,6 . 3,6 . 24 = 18,14 kN +

P3 = 51,84 kN

- Pelat lantai = 2 . 0,667 . 1,8 . 3,6 . 3,9 = 33,71 kN

- Balok anak = 0,35 . 0,6 . 1,8 . 24 = 9,07 kN +

P4 = 42,78 kN

d. Beban terpusat atap pelat

- kuda-kuda 3 = 13,84 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

- Bentang A^0-A

-R. laboratorium = 2 . 2/3 . 1,14 . 2,5 = 3,8 kN/m'

- Bentang A-B = Bentang B-C

- R. laboratorium = 2 . 0,917 . 1,8 . 2,5 = 8,26 kN/m'

b. Beban merata atap pelat

- Bentang A^0-A

- Pelat atap = 2/3 . 1,14 . 1 = 0,76 kN/m'

- Pelat atap = 0,672 . 1,14 . 1 = 0,77 kN/m' +

qd_4 = 1,53 kN/m'

- Bentang A-B

- Pelat lantai = 0,917 . 1,8 . 1 = qd_5 = 1,65 kN/m'

- Bentang B-C

- Pelat lantai = 2 . 0,917 . 1,8 . 1 = qd_5 = 3,30 kN/m'

c. Beban terpusat lantai 1, 2, dan 3

-selasar = 0,5 . 0,963 . 1,2 . 7,2 . 3 = 12,48 kN

- Pelat lantai = 0,667 . 1,8 . 3,6 . 2,5 = 10,81 kN

d. Beban terpusat atap pelat

- Pelat atap = 0,5 . 0,963 . 1,2 . 7,2 . 1 = 4,16 kN

- Pelat atap =0,5 . 2/3 . 3,6 . 7,2 . 1 = 8,64 kN

- Pelat lantai = 2 . 0,667 . 1,8 . 3,6 . 1 = 8,64 kN

4.4.1.8 Portal as 3

A. Beban Mati

a. Beban merata lantai 1, 2, dan 3

- Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 2 . 0,917 . 1,8 . 4,4 = 14,52 kN/m'

- Dinding = 3,1 . 2,5 = 7, 87 kN/m' +

qd_3 = 22,39 kN/m'

- Bentang B-B'

- Pelat lantai = 2 .2/3 . 1,2 . 4,4 = 7,04 kN/m'

b. Beban merata atap pelat

- Bentang A-B

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,8^2/7,2^2))$ = 0,917

- Pelat lantai = 2 . 0,917 . 1,8 . 3,9 = qd_5 = 45,52 kN/m'

- Bentang A^0-A

- Pelat atap = 2 .2/3 . 1,3 . 3,9 = 6,76 kN/m'

- Bentang B-B''

- Pelat atap = 2 .2/3 . 1,8 . 3,9 = 9,36 kN/m'

c. Beban terpusat lantai 1, 2, dan 3

$\gamma = (1 - (4/3 . C^2/L^2)) = (1 - (4/3 . 1,2^2/7,2^2))$ = 0,963

- Pelat lantai = 0,963 . 1,2 . 7,2 . 4,4 = 32,60 kN

- Balok anak = 0,35 . 0,6 .7,2 . 24 = 32,28kN

- Dinding = 3,1 . 2,5 . 7,2 = 52,66 kN/m' +

P1 = 129,54 kN

c. Beban terpusat atap

- Pelat atap = 0,963 . 1,2 . 7,2 . 4,4 = 32,6 kN

- Balok anak = 0,35 . 0,6 .7,2 . 24 = 32,28 kN +

P1 = 72,88 kN

- Pelat lantai = 2/3 . 3,6 . 7,2.. 3,9 = 67,40 kN

- Balok anak = 0,35 . 0,6 . 7,2 . 24 = 32,28 kN +

P3 = 103,68 kN

B. Beban Hidup

a. Beban merata lantai 1, 2, dan 3

- Bentang A-B

- R. laboratorium = 2 . 0,917 . 1,8 . 2,5 = 8,26 kN/m'

- Bentang B-B''

- Pelat lantai = 2 .2/3 . 1,2 . 3 = 4,8 kN/m'

b. Beban merata atap pelat

- Bentang A-B

- Pelat lantai = 2 . 0,917 . 1,8 . 1 = qd_5 = 3,30 kN/m'

- Bentang A^0-A

- Pelat atap = 2 .2/3 . 1,3 . 1 = 1,73 kN/m'

- Bentang B-B''

- Pelat atap = 2 .2/3 . 1,8 . 1 = 2,40 kN/m'

c. Beban terpusat lantai 1, 2, dan 3

- Selasar = 0,963 . 1,2 . 7,2 . 3 = 24,96 kN

c. Beban terpusat atap pelat

- Pelat atap = 0,963 . 1,2 . 7,2 . 1 = 8,32 kN

- Pelat atap = 2/3 . 3,6 . 7,2. 1 = 17,28 kN

4.4.2 Perhitungan Gaya Geser Dasar Horizontal Total Akibat Gempa.

A. Lantai 1, 2, dan 3

a. Beban mati

- Pelat = 7,2 . 39,6 . 4,4 = 1254,53 kN
- Pelat = 2,4 . 36 . 4,4 = 380,16 kN
- Pelat = 2,275 . 3,6 . 4,4 = 36,04 kN
- Pelat = 7,2 . 7,2 . 4,4 = 228,10 kN
- Balok = 0,35 . 0,70 . 177,48 . 24 = 1043,58 kN
- Balok anak = 0,35 . 0,60 . 36 . 24 = 181,44 kN

- kolom = 0,7 . 0,7 . 3,85 . 24 . 24 = 1086,62 kN
- Dinding = 3,1 . 213,48. 2,5 = 1654,47 kN +

Wd = 5864,94 kN

b Beban hidup

- Pelat = 7,2 . 39,6 . 2,5 = 712,8 kN
- Pelat = 2,4 . 36 . 3 = 87,12 kN
- Pelat = 2,275 . 3,6 . 1 = 8,19 kN
- Pelat = 7,2 . 7,2 . 2,5 = 129,60 kN +

= 937,71 kN

Wt_1 = 5864,94 + 937,71 = 6802,65 kN

B. Atap Pelat

a. Beban mati

- Pelat =3,6 . 3,6 . 3,9 . 5 = 252,72 kN
- Pelat =2,4 . 3,6 . 3,9 . 3 = 101,09 kN
- Pelat =3,6 . 7,2 . 3,9 . 5 = 505,44 kN
- Pelat = 2,4 . 3,6 . 3,9 . 9 = 303,26 kN
- Balok ring = 0,35 . 0,60 . 140,8 . 24 = 709,63 kN +

= 1872,14

b. Beban hidup

- Pelat =3,6 . 3,6 . 1 . 5 = 64,80 kN
- Pelat =2,4 . 3,6 . 1 . 3 = 25,92 kN
- Pelat =3,6 . 7,2 . 1 . 5 = 129,60 kN

- Pelat = 2,4 . 3,6 . 1 . 9 = 77,76 kN +

= 298,08 kN

$\mathbf{Wt_1}$ = 1872,14 + 298,08 = 2170,22 kN

C. Atap Genteng

a. Beban mati

- kuda-kuda 1 = 14 . 17,5 = 245 kN
- kuda-kuda 2 = 1 . 1,10 = 1,10 kN
- kuda-kuda 2 = 1 . 13,84 = 13,84 kN +

Wd = 259,94 kN

b. Beban hidup

- Pekerja = 7,2 . 28,8 . 1 = 207,36 kN

Wl = 207,36 kN

$\mathbf{Wt_3}$ = 259,94 + 207,36 = 467,3 kN

- **Wtotal =** $4Wt_1 + Wt_2 + Wt_3 = 4 . 6802,65 + 2170,22 + 467,3 = 29848,12$ kN

D. Waktu getar bangunan (T)

$T = 0,06 . H^{2/3} = 0,06 . 17,62^{2/3} = 0,406$ dt

E. Koefisien gempa dasar

T = 0,406 dt ; Zona 3 dan jenis tanah lunak diperoleh C = 0,07

F. Faktor keutamaan I dan faktor jenis struktur K

I = 1,0 ; K = 1,0

G. Gaya geser horizontal akibat gempa

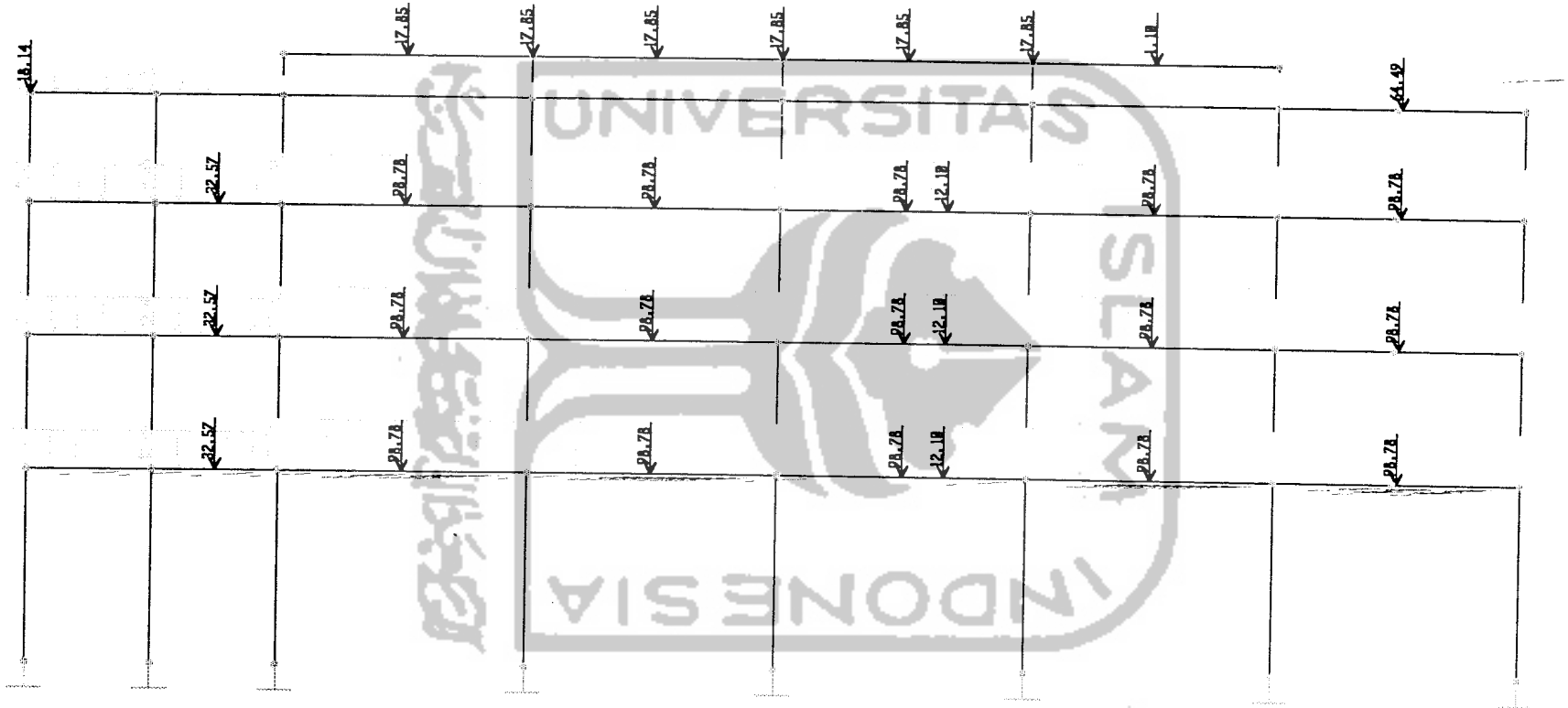
V = C . I . K . Wt = 0,07 . 1 . 1 . 29848,12 = 2089,37 kN

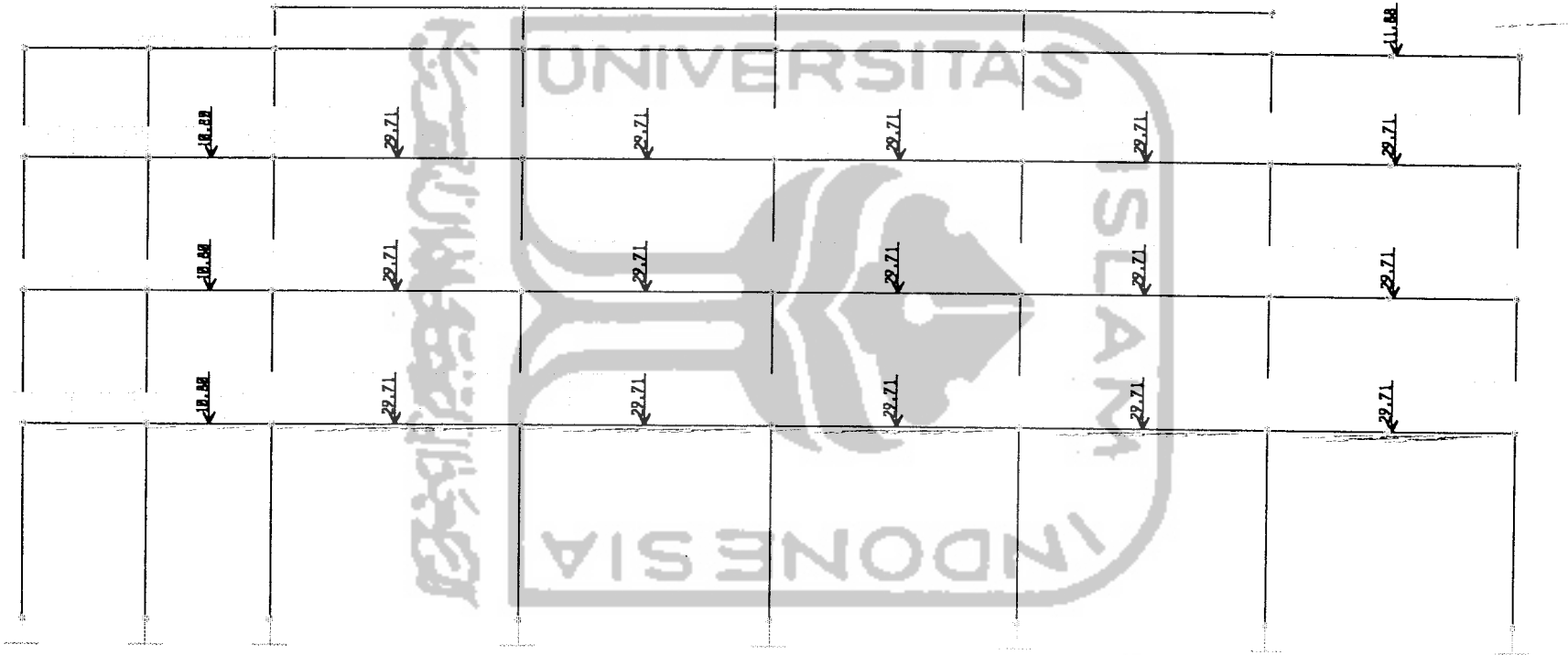
H. Distribusi gaya horizontal total akibat gempa ke sepanjang tinggi gedung.

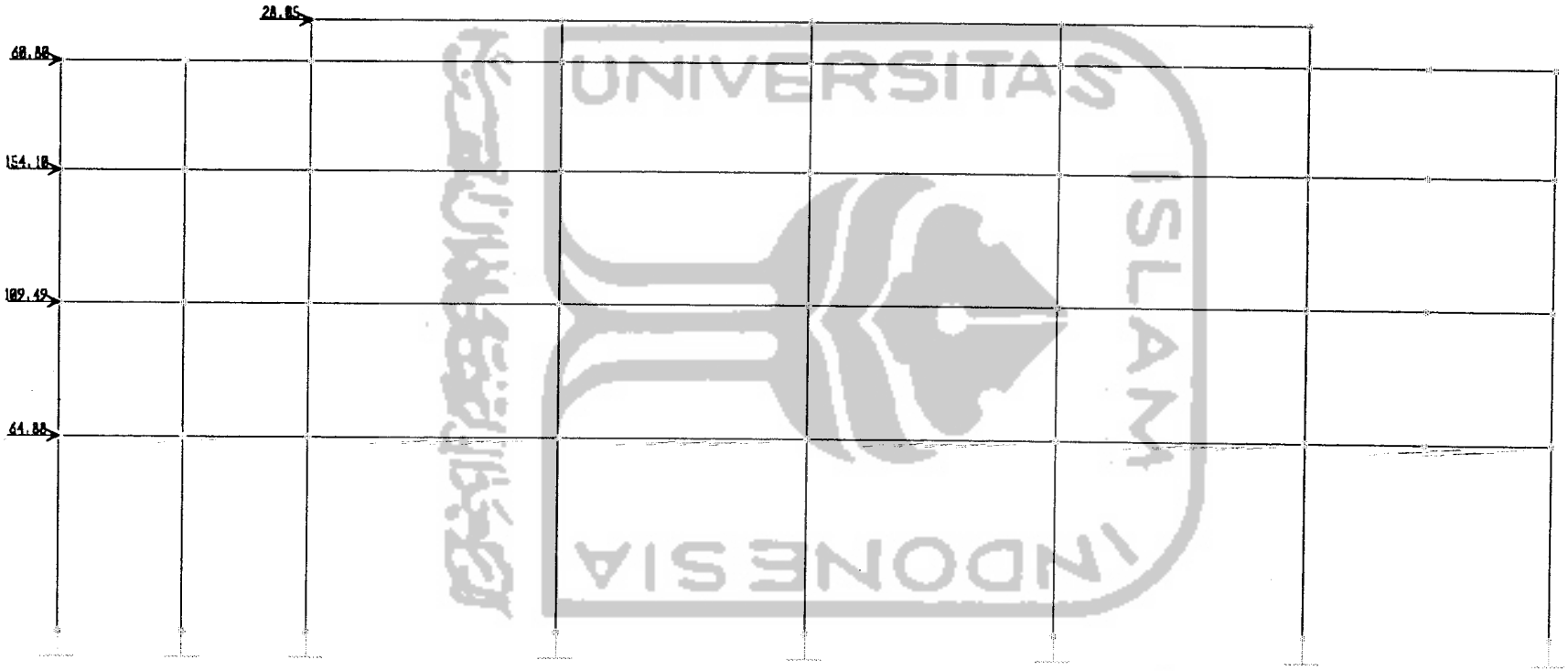
tingkat	hi (m)	Wi (kN)	V (kN)	Wi.hi (kN.m)	Fi (kN)
atas	17.62	467.3	2089.370	8233.826	69.17549
balok 4	16.45	2170.22	2089.370	35700.119	299.9302
3	13.3	6802.65	2089.370	90475.245	760.1168
2	9.45	6802.65	2089.370	64285.0425	540.083
1	5.6	6802.65	2089.370	38094.84	320.0492
basemen	1.75	6802.65	2089.370	11904.6375	100.0154
		29848.12		248693.71	2089.37

Tabel 4.11 Distribusi Gaya Geser Dasar Horisontal Total ke Arah X dan Y

Fi (kN)	Fix (kN)	Fiy(kN)
69.18	13,84	34,59
299.93	33,33	74,98
760.12	84,46	190,03
540.08	60,01	135,02
320.05	35,56	80,01
100.02	11,11	25,00


18.14
17.85
17.85
17.85
17.85
17.85
17.85
1.18
44.42
22.57
28.78
28.78
28.78
12.18
28.78
28.78
22.57
28.78
28.78
28.78
12.18
28.78
28.78
22.57
28.78
28.78
28.78
12.18
28.78
28.78


29.71
29.71
29.71
29.71
29.71
29.71
29.71
29.71
29.71
29.71
29.71
29.71


28.85
68.88
154.18
189.49
64.88

4.5 PERENCANAAN BALOK

4.5.1 Perencanaan Tulangan Lentur Balok

A. Momen Rencana Balok

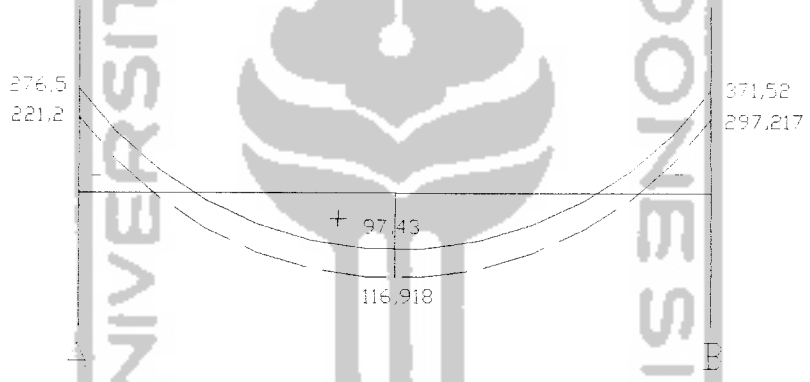
Momen rencana balok diambil yang terbesar setelah dikombinasikan sebagai berikut:

$1{,}2\ M_D + 1{,}6\ M_L$

$0{,}9\ M_D \pm M_E$

$1{,}05\ (M_D + 0{,}6\ M_L \pm M_E)$

Berikut diberikan contoh perhitungan balok tipe – B2



Gambar 4.15 Momen pada portal As-4 bentang A-B

B. Tulangan Tumpuan A

Dipakai dimensi rencana 350/650

fc' = 22,5 Mpa fy = 400 Mpa

Mu = 276,5 kNm

Mu akibat distribusi momen 20% = 276,5 – (0,2.276,5) = 221,2 kNm

$$\frac{Mu}{\phi} = \frac{221{,}2}{0{,}8} = 276{,}5 \text{ kNm}$$

$$\rho_b = \frac{0{,}85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = 0{,}0244$$

$\rho_{maks} = 0{,}75\ \rho b = 0{,}0183$

rasio tulangan rencana = 0,5. $\rho_{maks} = 0{,}00914$

$\rho_{min} = 1{,}4/fy = 0{,}0035$

$$m = \frac{fy}{0{,}85.fc'} = 20{,}915$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0{,}00914.400.\left(1-\frac{1}{2}.0{,}00914.20{,}915\right) = 3{,}31\ \text{Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{276{,}2.10^6}{3{,}31.350}} = 488{,}6977\ \text{mm}$$

$d_{pakai} = h - Pb - \phi sengkang - z = 650\text{-}40\text{-}10\text{-}\frac{1}{2}.22 = 589$ mm

ambil $d_{pakai} = 589$ mm > d_{perlu} → dipakai tul sebelah

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{276{,}5.10^6}{350.589^2} = 2{,}277$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}.0{,}00914 = 0{,}00629 > \rho_{min} = 0{,}0035$$

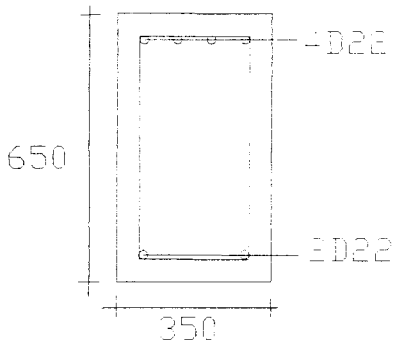
$< \rho_{maks} = 0{,}0183$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00629.350.589 = 1297{,}706\ mm^2$

Dipakai 4D22 dengan $As_{ada} = 1520{,}5\ mm^2$

$$s = \left| = \frac{350 - 2.40 - 2.10 - 4.22}{(4-1)} = 54 \right.$$

mm



Gambar 4.16 tulangan pokok balok tumpuan

Kontrol

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{1520,5.400}{0,85.22,5.350} = 90,899 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 1520,5.400.(589 - {}^{90,899}/_{2}) = 330.727 \text{ kNm} > {}^{Mu}/_{\phi} = 276,5 \text{ kNm}$$

C. Tulangan Lapangan

Dipakai dimensi rencana 350/650

fc' = 22,5 Mpa fy = 400 Mpa

Mu = 97,43 kNm

Mu akibat distribusi momen 20% = 97,43 + 97,43.0,2 = 116,918 kNm

$$\frac{Mu}{\phi} = \frac{116,918}{0,8} = 146,15 \text{ kNm}$$

$$\rho_b = \frac{0,85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = 0,0244$$

$\rho_{maks} = 0,75\ \rho b = 0,0183$

rasio tulangan rencana = 0,5. ρ_{maks} = 0,00914

$\rho_{min} = 1,4/fy = 0,0035$

$$m = \frac{fy}{0{,}85.fc'} = 20{,}915$$

$$\text{Rn} = \rho.\text{fy}.\left(1 - \frac{1}{2}.\rho.m\right) = 0{,}00914.400.\left(1 - \frac{1}{2}.0{,}00914.20{,}915\right) = 3{,}31\ \text{Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{146{,}15.10^6}{3{,}31.350}} = 355{,}293\ \text{mm}$$

$\text{d}_{\text{pakai}} = 550\ \text{mm} > \text{d}_{\text{perlu}} \rightarrow$ dipakai tul sebelah

$$\text{Rn}_{\text{baru}} = \frac{Mu/\phi}{b.d^2} = \frac{146{,}15.10^6}{350.589^2} = 1{,}204\ \text{Mpa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn} = \frac{1{,}204}{3{,}31}0{,}00914 = 0{,}00333 < \rho_{\text{min}} = 0{,}0035$$

$$< \rho_{\text{maks}} = 0{,}0183$$

$1{,}33\ .\ 0{,}00333 = 0{,}00443$

sehingga $\rho_{\text{perlu}} = \rho_{\text{min}} = 0{,}0035$

$\text{As}_{\text{perlu}} = \rho_{\text{perlu}}.\text{b.d} = 0{,}0035.350.589 = 721{,}525\ \text{mm}^2$

Dipakai 2D22 dengan $\text{As}_{\text{ada}} = 760{,}6\ \text{mm}^2$

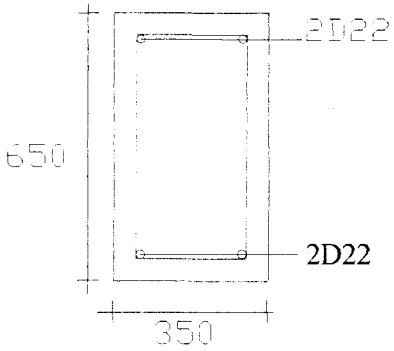
Kontrol

$$\text{a} = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{760{,}6.400}{0{,}85.22{,}5.350} = 45{,}45\ \text{mm}$$

$$\text{Mn} = \text{As}_{\text{ada}}.\text{fy}.(\text{d} - a/2) = 760{,}6.400.(589 - 45{,}45/2)$$

$$= 172{,}277\ \text{kNm} > Mu/\phi = 146{,}15\ \text{kNm}$$

$$\text{s} = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 2.22}{(2-1)} = 206\ \text{mm}$$



Gambar 4.17 tulangan pokok balok lapangan

D. Tulangan Tumpuan B

Dipakai dimensi rencana 350/650

fc' = 22,5 Mpa fy = 400 Mpa

Mu = 371,52 kNm

Mu akibat distribusi momen 20% = 371,52 – (0,2.371,52) = 297,217 kNm

$$\frac{Mu}{\phi} = \frac{297,217}{0,8} = 371,52 \text{ kNm}$$

$$\rho_b = \frac{0,85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = 0,0244$$

$\rho_{maks} = 0,75\ \rho b = 0,0183$

rasio tulangan rencana = 0,5. ρ_{maks} = 0,00914

$\rho_{min} = 1,4/fy = 0,0035$

$$m = \frac{fy}{0,85.fc'} = 20,915$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0,00914.400.\left(1-\frac{1}{2}.0,00914.20,915\right) = 3,31 \text{ Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{371,52.10^6}{3,31.350}} = 566,476 \text{ mm}$$

d_{pakai} = h - Pb - ϕsengkang – z = 650-40-10-(22 = 589 mm

l d_{pakai} = 589 mm > d_{perlu} → dipakai tul sebelah

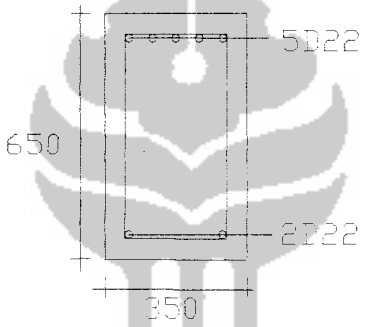
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{371{,}52.10^6}{350.589^2} = 3{,}06$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn} = \frac{3{,}06}{3{,}31}.0{,}00914 = 0{,}00846 > \rho_{min} = 0{,}0035$$

$< \rho_{maks} = 0{,}0183$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00846.350.589 = 1743{,}65\ mm^2$

Dipakai 5D22 dengan $As_{ada} = 1901{,}43\ mm^2$



Gambar 4.18 tulangan pokok balok tumpuan

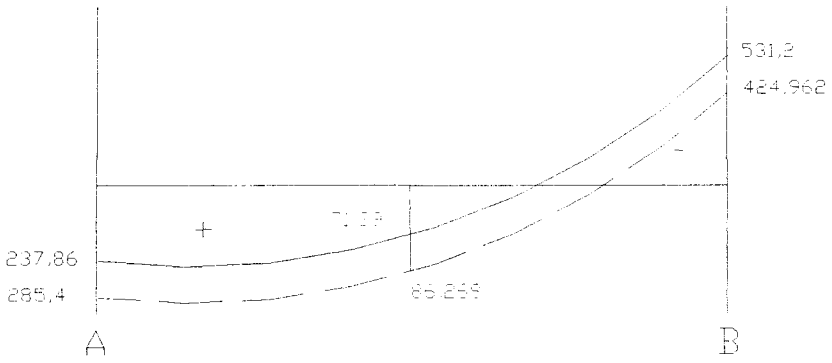
$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35\ mm$$

Kontrol

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{1901{,}43.400}{0{,}85.22{,}5.350} = 113{,}624\ mm$$

$$Mn = As_{ada}.fy.(d - a/2) = 1901{,}43.400\ (589 - 113{,}624/2)$$

$$= 404{,}767\ kNm > Mu/\phi = 371{,}52\ kNm$$



Gambar 4.19 momen pada portal As-4 bentang A-B

E. Tulangan Tumpuan daerah A

Dipakai dimensi rencana 350/650

fc' = 22,5 Mpa fy = 400 Mpa

Mu = 237,86 kNm

Mu akibat distribusi momen 20% = 237,86 + (0,2. 237,86) = 285,43 kNm

$$\frac{Mu}{\phi} = \frac{285,43}{0,8} = 356,79 \text{ kNm}$$

$$\rho_b = \frac{0,85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = 0,0244$$

$\rho_{maks} = 0,75\ \rho b = 0,0183$

rasio tulangan rencana = 0,5. ρ_{maks} = 0,00914

$\rho_{min} = 1,4/fy = 0,0035$

$$m = \frac{fy}{0,85.fc'} = 20,915$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0,00914.400.\left(1-\frac{1}{2}.0,00914.20,915\right) = 3,31 \text{ Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{356,79.10^6}{3,31.350}} = 555,135 \text{mm}$$

$d_{pakai} = 589$ mm $> d_{perlu}$ $\rightarrow$ dipakai tul sebelah

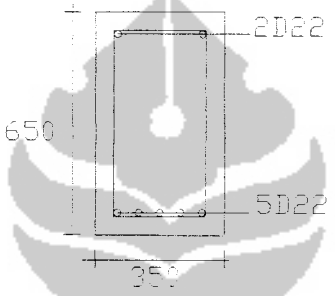
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{356,79.10^6}{350.589^2} = 2,9385 \text{ Mpa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn} = \frac{2,9385}{3,31}.0,00914 = 0,00812 > \rho_{min} = 0,0035$$

$$< \rho_{maks} = 0,0183$$

$As_{perlu} = \rho_{baru}.b.d = 0,00812.350.589 = 1674,53$ mm^2

Dipakai 5D22 dengan $As_{ada} = 1901,429$ mm^2



Gambar 4.20 tulangan balok tumpuan

$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35 \text{ mm}$$

Kontrol

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{1901,429.400}{0,85.22,5.350} = 113,624 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - a/2) = 1901,429.400.(589 - 113,624/2)$$

$$= 404,767 \text{ kNm} > Mu/\phi = 356,79 \text{ kNm}$$

F. Tulangan Lapangan

Dipakai dimensi rencana 350/650

fc' = 25 Mpa fy = 400 Mpa

Mu = 71,89 kNm

Mu akibat distribusi momen = 71,89 + (20%.71,89) = 86,268 kNm

$$\frac{Mu}{\phi} = \frac{86,268}{0,8} = 107,84 \text{ kNm}$$

$$\rho_b = \frac{0,85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = 0,0244$$

$\rho_{maks} = 0,75\ \rho b = 0,0183$

rasio tulangan rencana = 0,5. ρ_{maks} = 0,00914

$\rho_{min} = 1,4/fy = 0,0035$

$$m = \frac{fy}{0,85.fc'} = 20,915$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0,00914.400.\left(1-\frac{1}{2}.0,00914.20,915\right) = 3,31 \text{ Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{107,84.10^6}{3,31.350}} = 305,1907 \text{ mm}$$

d_{pakai} = 589 mm > d_{perlu} → dipakai tul sebelah

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{107,84.10^6}{350.589^2} = 0,888 \text{ Mpa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}.0,00914 = 0,00246 < \rho_{min} = 0,0035$$

$$< \rho_{mak} = 0,0183$$

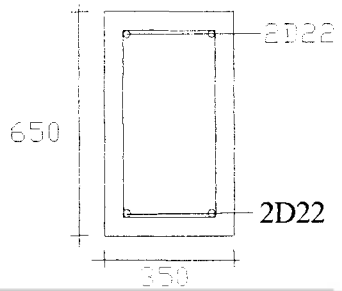
1.33 . 0,00246 = 0,00327

sehingga $\rho_{perlu} = 1,33.\rho_{baru} = 0,00327$

$As_{perlu} = \rho_{perlu}.b.d = 0,00327.350.589 = 673,116 \text{ mm}^2$

$As_{min} = \rho_{min}.b.d = 0{,}0035.350.589 = 721{,}525\ mm^2$

Dipakai 2D22 dengan $As_{ada} = 760{,}57\ mm^2$



Gambar 4.21 tulangan balok lapangan

$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 2.22}{(2-1)} = 206\ mm$$

Kontrol

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{760{,}57.400}{0{,}85.22{,}5.350} = 45{,}45\ mm$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2}) = 760{,}57.400.(589 - {}^{45{,}45}/_{2})$$

$$= 172{,}277\ kNm > {}^{Mu}/_{\phi} = 107{,}84\ kNm$$

G. Tulangan Tumpuan daerah B

Dipakai dimensi rencana 350/650

fc' = 22,5 Mpa fy = 400 Mpa

Mu = 531,2 kNm

Mu akibat distribusi momen 20% = 531,2 - (0,2. 531,2) = 424,962 kNm

$$\frac{Mu}{\phi} = \frac{424{,}962}{0{,}8} = 531{,}2\ kNm$$

$$\rho_b = \frac{0{,}85.fc'}{fy}\beta_1\left(\frac{600}{600 + fy}\right) = 0{,}0244$$

$\rho_{maks} = 0{,}75\ \rho b = 0{,}0183$

rasio tulangan rencana = $0{,}5.\ \rho_{maks} = 0{,}00914$

$\rho_{min} = 1{,}4/fy = 0{,}0035$

$$m = \frac{fy}{0{,}85.fc'} = 20{,}915$$

$$Rn = \rho.fy.\left(1 - \frac{1}{2}.\rho.m\right) = 0{,}00914.400.\left(1 - \frac{1}{2}.0{,}00914.20{,}915\right) = 3{,}31\ \text{Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{531{,}2.10^6}{3{,}31.350}} = 677{,}3613\ \text{mm}$$

$d_{pakai} = 589$ mm $< d_{perlu}$ → dipakai tul rangkap

$d' = Pb + \phi_{sengkang} + 0{,}5.\ \phi_{tul.pokok} = 40+10+11 = 61$ mm

$\rho_1 = \rho - \rho' = \rho$ tulangan sebelah $= 0{,}00914$

$As_1 = \rho_1.b.d = 0{,}00914.350.589 = 1885{,}0646\ mm^2$

$$a = \frac{As1.fy}{0{,}85.fc'.b} = \frac{1885{,}0646.400}{0{,}85.22{,}5.350} = 112{,}6463\ \text{mm}$$

$Mn_1 = As_1.fy.(d-a/2) = 1885{,}0646.400.(589-(112{,}6463/2)) = 401{,}652$ kNm

$Mn_2 = 531{,}2 - 401{,}652 = 129{,}5506$ kNm

1. untuk tulangan desak

$$fs' = 600\left\{1 - \frac{0{,}85.f'c.\beta_1}{(\rho-\rho').fy}\frac{d'}{d}\right\} = 600\left\{1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}00914.400}\frac{61}{589}\right\}$$

$= 323{,}8257$ Mpa

fs' < fy dipakai fs' = 323,8257 Mpa

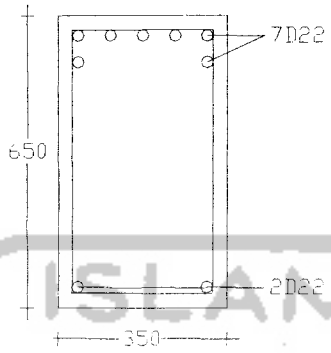
$$As' = As_2 = \frac{Mn_2}{fs'.(d-d')}. = \frac{129{,}5506.10^6}{323{,}8257.(589-61)} = 757{,}6945\ mm^2$$

Dipakai 2D22 $As_{ada} = 760{,}57\ mm^2$

2. untuk tulangan tarik

$As = As_1 + As' = 1885{,}0646 + 757{,}6945 = 2645{,}636\ mm^2$

Dipakai 7D22 $As_{ada} = 2662\ mm^2$



Gambar 4.22 tulangan pokok balok tumpuan

$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35\ mm$$

Kontrol kapasitas momen

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{2662}{350.589} = 0{,}01291$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{757{,}6945}{350.589} = 0{,}00369$$

$$\rho_1 = \rho - \rho' = 0{,}01291 - 0{,}00369 = 0{,}0009$$

$$fs' = 600\left\{1 - \frac{0{,}85.f'c.\beta_1\ d'}{(\rho - \rho').fy\ d}\right\} = 600\left\{1 - \frac{0{,}85.22{,}5.0{,}85\ \ 61}{0{,}0009.400\ \ 589}\right\} = 358{,}53\ Mpa$$

$fs' < fy$ dipakai $fs' = fs' = 326{,}2025$ Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0{,}85.fc'.b}$$

$$= \frac{(2662.400) - (757{,}6945.326{,}2025)}{0{,}85.22{,}5.350} = 122{,}009\ mm^2$$

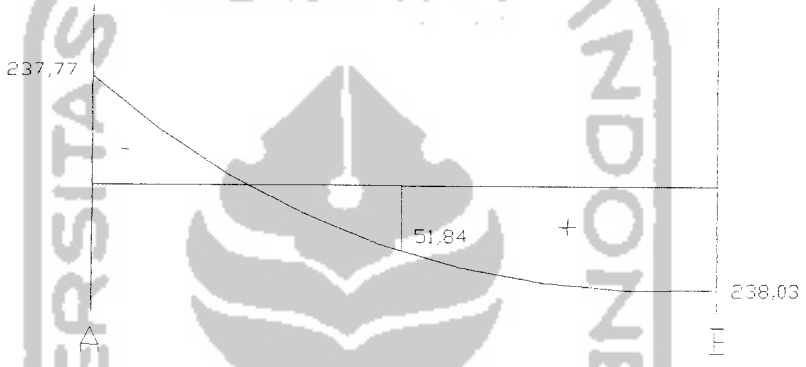
$Mn_1 = ((2662.400) - (757,2025.\ 326,2025)).(589 - {}^{122,009}/_{2})$

$= 431,214$ kNm

$Mn_2 = (757,2025.\ 326,2025).(589 - 61)$

$= 130,997$ kNm

$Mn_1 + Mn_2 = 562,211$ kNm $> {}^{Mu}/_{\phi} = 531,2$ kNm



Gambar 4.23 momen portal As-4 bentang A-B

Dengan perhitungan yang sama dengan perhitungan diatas diperoleh:

Tumpuan A dipakai 4D22 dengan $As_{ada} = 1521,143$ mm^2

Lapangan dipakai 2D22 dengan $As_{ada} = 760,57$ mm

Tumpuan B dipakai 4D22 dengan $As_{ada} = 1521,143$ mm^2

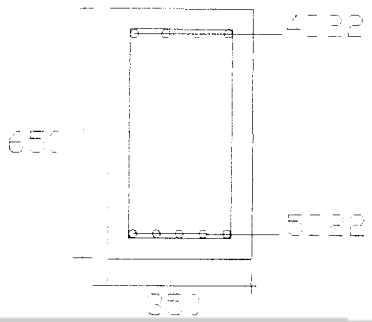
H. Perencanaan Balok

Sehingga dari ke-3 bentuk momoen diatas dapat disimpulkan bahwa untuk balok tipe-B2 dipakai :

1. daerah tumpuan A dipakai tulangan rangkap

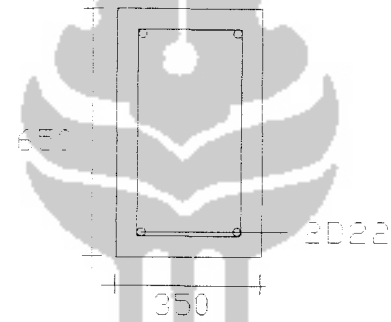
   - daerah desak dipakai 4D22 dengan $As_{ada} = 1521,1429$ mm^2

   - daerah tarik dipakai 5D22 dengan $As_{ada} = 1901,4286$ mm^2



$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35 \text{ mm}$$

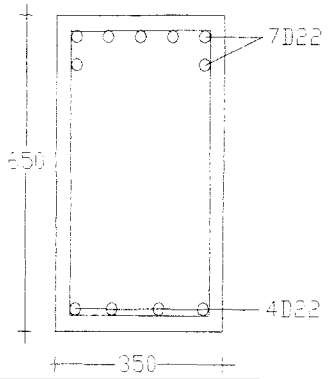
2. daerah lapangan (tul.sebelah) dipakai 2D22 dengan As_{ada}= 760,57 mm^2



$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 2.22}{(2-1)} = 206 \text{ mm}$$

3. daerah tumpuan B dipakai tulangan rangkap

- daerah desak dipakai 4D22 dengan As_{ada} = 1521,143 mm^2
- daerah tarik dipakai 7D22 dengan As_{ada} = 2662 mm^2



$$s = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul}{(n-1)} = \frac{350 - 2.40 - 2.10 - 5.22}{(5-1)} = 35 \text{ mm}$$

Kontrol kapasitas momen

1. daerah tumpuan A

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{1901,4286}{350.589} = 0,00922$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{760,57}{350.589} = 0,00369$$

$$\rho_1 = \rho - \rho' = 0,00922 - 0,00369 = 0,0055$$

$$fs' = 600\left\{1 - \frac{0,85.f'c.\beta_1}{(\rho - \rho').fy}\frac{d'}{d}\right\} = 600\left\{1 - \frac{0,85.22,5.0,85}{0,0055.400}\frac{61}{589}\right\}$$

$$= 143,67 \text{ Mpa}$$

fs'< fy dipakai fs'= fs' = 143,67 Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0,85.fc'.b}$$

$$= \frac{(1901,4286.400) - (760,57.143,67)}{0,85.22,5.350} = 97,2996 \text{ mm}^2$$

$Mn_1 = ((1901,4286.400) - (760,57.143,59)).(589 - {}^{97,2996}/_{2})$

$= 351,953$ kNm

$Mn_2 = (757,2025.\ 326,2025).(589 - 61)$

$= 57,673$ kNm

$Mnak = Mn_1 + Mn_2 = 409,626$ kNm $> {}^{Mu}/_{\phi} = 356,79$ kNm

2. daerah tumpuan B

dengan cara yang sama didapat :

$\rho_1 = \rho - \rho' = 0,01291 - 0,00738 = 0,00553$

fs' = 143,671 Mpa < fy 400 Mpa sehingga fs' = fs' = 143,671 Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0,85.fc'.b} = \frac{(2662.400) - (760,57.143,671)}{0,85.22,5.350}$$

$= 142,75\ mm^2$

$Mn_1 = (As_{ada}.fy - As'_{ada}.fs').(d - {}^{a}/_{2})$

$= (2662.400 - 1521,143.\ 143,671).(589 - {}^{142,75}/_{2}) = 438,044$ kNm

$Mn_2 = (as'_{ada}.fs').(d - d')$

$= (1521,143.\ 143,671).(589 - 61) = 115,341$ kNm

$Mnak = Mn_1 + Mn_2$

$= 553,435$ kNm $> {}^{Mu}/_{\phi} = 531,2$ kNm

4.5.2 Perencanaan Tulangan Geser Balok

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

$V_D = 131,11$ kN ; $V_L = 35,973$ kN ; $V_E = 66,0842$ kN

$$Vu,b = 0,7\ \phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln}\right] + 1,05.Vg$$

$$Vu,b = 0,7.1,25\left[\frac{409,626 + 553,435}{6,5}\right] + 1,05.(131,11 + 35,973) = 303,1152\ \text{kN}$$

Dengan syarat tidak lebih besar dari :

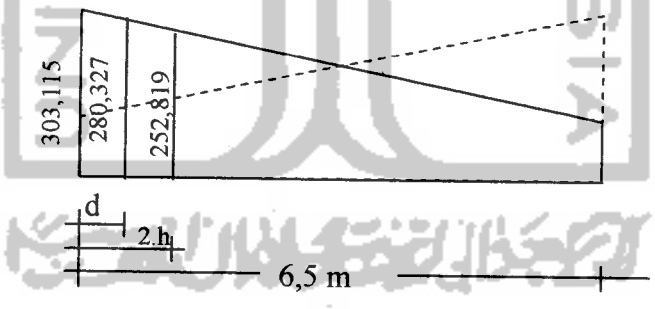
Vu,b = 1,05 (131,11 + 35,973 + 4/1 . 66,0842) = 452,9908 kN

Vu,b pakai =

$$\left[1,05Vg - 0,7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] + \frac{Ln - d}{Ln}\left[Vu,b - 0,7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right]$$

$$\left[1,05.175,437 - 0,7.1,25\frac{409,626 + 553,435}{6,5}\right] + \frac{6,5 - 0,589}{6,5}\left[303,12 - 0,7.1,25\frac{409,626 + 553,435}{6,5}\right]$$

= 280,.327 kN



Gambar 4.24 Gaya geser pada penampang kritis dan daerah sendi plastis

dalam daerah sendi plastis

Vu,b = 280,327 KN

Vc = 0

$$Vs = \frac{Vu,b}{\phi} = \frac{280,327}{0,6} = 467,211\ \text{kN}$$

$$S = \frac{Av.fy.d}{Vs} = \frac{(2.0{,}25.\pi.10^2).240.589}{467{,}211.10^3} = 47{,}545\ \text{mm}$$

Syarat spasi

d/4 = 147,25 mm

dipakai $2P_{10}$ - 90

Diluar sendi plastis

Diambil jarak sejauh 2h = 1300 mm dengan Vu,b = 252,819 kN

$Vc = 1/6.\sqrt{fc'}.b.d = 162{,}9759\ \text{kN}$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{252{,}819}{0{,}6} - 162{,}9759 = 258{,}387\text{kN}$$

$$S = \frac{Av.fy.d}{Vs} = \frac{(2.1/4.\pi.10^2).240.589}{258{,}387.10^3} = 85{,}971\ \text{mm}$$

Syarat spasi

$S \leq d/2 = 294{,}5$ mm

S ≤ 600 mm

Dipakai P_{10} - 85

4.5.3 Perencanaan Tulangan Torsi

Tu = 10,04 kNm

$\sum x^2.y = 350^2.650 = 79{,}625.10^6\ \text{mm}^2$

$$\text{Ø}\left(\tfrac{1}{9}.\sqrt{fc'}.\sum x^2.y\right) = 0{,}6.\left(\tfrac{1}{9}.\sqrt{22{,}5}.79{,}625.10^6\right)$$

$$= 25{,}18.10^6\ \text{Nmm} = 25{,}18\ \text{kNm}$$

Kontrol

$$\text{Tu} = 10{,}04\ \text{kNm} < \text{Ø}\left(\tfrac{1}{9}.\sqrt{fc'}.\sum x^2.y\right) = 25{,}18\ \text{kNm}$$

⟶ Tulangan torsi diabaikan

Tabel 4.12 Balok Induk Tipe-B1

	Tump. Kr-1 (-)	Tump. Kr-2 (+)	Tump. Kr-3 (-)
Mu (KNm)	276.50	237.86	237.77
Mu redistribusi 20%	221.20	285.43	-
Mu/ϕ (KNm)	276.50	356.79	297.22
f'c (MPa)	22.5	22.5	22.5
fy (MPa)	400	400	400
β1	0.85	0.85	0.85
m	20.915	20.915	20.915
ρ_b	0.0244	0.0244	0.0244
ρ_{min}	0.0035	0.0035	0.0035
ρ_{maks}	0.0183	0.0183	0.0183
ρpakai	0.00914	0.00914	0.00914
Rn (MPa)	3.31	3.31	3.31
b.d2 perlu (mm3)	83588892.0153	107861292.3405	89850438.5064
b (mm)	350	350	350
dperlu (mm)	488.6977	555.1352	506.6710
h (mm)	650	650	650
dpakai (mm)	589	589	589
Perencanaan	**Tul. Sebelah**	**Tul. Sebelah**	**Tul. Sebelah**
Rn aktual	2.2772	2.9385	2.4478
p aktual	0.00629	0.00812	0.00677
nilai paktual baru	0.00629	0.00812	0.00677
As min (mm2)	721.525	721.525	721.525
As perlu (mm2)	1297.706	1674.531	1394.915
dtul.pokok (mm)	22	22	22
A1d.pokok (mm2)	380.29	380.29	380.29
jumlah tul. perlu	3.4124	4.4034	3.6681
tul. terpasang (n buah)	4	5	4
As aktual (mm2)	1521.1429	1901.4286	1521.1429
s(mm) > 25mm	54	35	53
a (mm)	90.899	113.624	90.899
Mn (kNm)	**330.727**	**404.767**	**330.727**
Kontrol	**AMAN**	**AMAN**	**AMAN**

Kesimpulan

Perencanaan	Tul. Rangkap
Tul.Atas n buah	4
Tul. Bawah n buah	5
ρ1 (ρ - ρ')	0.0050
fs' (MPa)	143.67
fs' pakai (MPa)	143.67

a (mm2)	97.2996
Mn1 (kNm)	351.953
Mn2 (kNm)	57.673
Mn	409.626
Kontrol	**AMAN**
Mkap	512.033

	Lap. -1 (+)	Lap. -2 (+)	Lap. -3 (+)
Mu (KNm)	97.43	71.89	51.84
Mu redistribusi 20 % (KNm)	116.918	86.268	-
Mu/ϕ (KNm)	146.15	107.84	64.80
f'c (MPa)	22.5	22.5	22.5
fy (MPa)	400	400	400
β1	0.85	0.85	0.85
m	20.915	20.915	20.915
ρ_b	0.0244	0.0244	0.0244
ρ_{min}	0.0035	0.0035	0.0035
ρ_{maks}	0.0183	0.0183	0.0183
ρpakai	0.00914	0.00914	0.00914
Rn (MPa)	3.31	3.31	3.31
b.d2 perlu (mm3)	44181600.0987	32599483.7808	19589509.8557
b (mm)	350	350	350
dperlu (mm)	355.2930	305.1907	236.5799
h (mm)	650	650	650
dpakai (mm)	589	589	589
Perencanaan	**Tul. Sebelah**	**Tul. Sebelah**	**Tul. Sebelah**
Rn aktual	1.2036	0.8881	0.5337
p aktual	0.00333	0.00246	0.00148
nilai paktual baru	0.00350	0.00327	0.00200
As min (mm2)	721.525	721.525	721.525
As perlu (mm2)	721.525	673.116	412.300
dtul.pokok (mm)	22	22	22
A1d.pokok (mm2)	380.29	380.29	380.29
jumlah tul. perlu	1.8973	1.8973	1.8973
tul. terpasang (n buah)	2	2	2
As aktual (mm2)	760.5714	760.5714	760.5714
s(mm) > 25mm	206	206	202
a (mm)	45.450	45.450	45.450
Mn (kNm)	**172.277**	**172.277**	**172.277**
Kontrol	**AMAN**	**AMAN**	**AMAN**
Kesimpulan			
Perencanaan	Tulangan Sebelah		
tul. terpasang (n buah)	2		
Mkap	215.346		

	Tump. Kn-1 (-)	Tump. Kn-2 (-)	Tump. Kn-3 (+)
Mu (KNm)	371.52	531.20	238.03
Mu redistribusi 20 % (KNm)	297.217	424.962	-
Mu/ϕ (KNm)	371.52	531.20	297.54
f'c (MPa)	22.5	22.5	22.5
fy (MPa)	400	400	400
β1	0.85	0.85	0.85
m	20.915	20.915	20.915
ρ_b	0.0244	0.0244	0.0244
ρ_{min}	0.0035	0.0035	0.0035
ρ_{maks}	0.0183	0.0183	0.0183
ρpakai	0.00914	0.00914	0.00914
Rn (MPa)	3.31	3.31	3.31
b.d2 perlu (mm3)	112313431.6853	160586427.8865	89949255.1890
b (mm)	350	350	350
dperlu (mm)	566.4764	677.3613	506.9496
h (mm)	650	650	650
dpakai (mm)	589	589	589
Perencanaan	**Tul. Sebelah**	**Tul. Rangkap**	**Tul. Sebelah**
Rn aktual	3.0597	-	2.4505
p aktual	0.00846	-	0.00677
nilai paktual baru	0.00846	-	0.00677
As min (mm2)	721.525	-	721.525
As perlu (mm2)	1743.650	-	1396.449
dtul.pokok (mm)	22	-	22
A1d.pokok (mm2)	380.29	-	380.29
jumlah tul. perlu	4.5851	-	3.6721
tul. terpasang (n buah)	5	-	4
As aktual (mm2)	1901.4286	-	1521.1429
s(mm) > 25mm	34	-	53
a (mm)	113.624	-	90.899
Mn (kNm)	404.767	-	330.727
Kontrol	**AMAN**	-	**AMAN**
d' (mm)	-	61	-
ρ1 (ρ – ρ')	-	0.00914	-
As1 perlu (mm)	-	1885.0646	-
a1 (mm)	-	112.6463	-
Mn1 (KNm)	-	401.6521	-
Mn2(KNm)=Mu/ϕ-Mn1	-	129.5506	-
fs' (MPa)	-	323.8257	-
fs' pakai (MPa)	-	323.8257	-
As' perlu (mm)	-	757.6945	-
dtul.pokok (mm)	-	22	-
A1d.pokok (mm2)	-	380.29	-
tul. desak perlu (n buah)	-	1.992434734	-

tul desak pakai (n buah)	-	2	-
As' ada (mm)	-	760.571	-
As perlu total (mm2)	-	2645.6360	-
tul. tarik perlu (n buah)	-	6.9570	-
tul tarik pakai (n buah)	-	7	-
As ada (mm2)	-	2662	-
Kontrol	-		-
r' ada	-	0.00369	-
r ada	-	0.01291	-
(rada - r'ada)	-	0.0092	-
fs' (MPa)	-	326.2025	-
fs' pakai (MPa)	-	326.2025	-
a (mm2)	-	122.0092931	-
Mn1 (kNm)	-	431.214	-
Mn2 (kNm)	-	130.997	-
Mn	-	562.211	-
s (mm)	-	35	-
Kontrol	-	AMAN	-
Mkap	505.959	702.763	413.409

Kesimpulan

Perencanaan	Tul. Rangkap
Tul. Atas n buah	7
Tul. Bawah n buah	4
ρ1 (ρ - ρ')	0.0055
fs' (MPa)	143.6708
fs' pakai (MPa)	143.6708
a (mm2)	142.7493237
Mn1 (kNm)	438.044
Mn2 (kNm)	115.391
Mn	553.435
Kontrol	**AMAN**
Mkap	691.793

4.6 PERENCANAAN KOLOM

Di bawah ini akan diberikan contoh perhitungan kolom tipe-K4 pada portal As- B4.

4.6.1 Perhitungan Momen dan Gaya Aksial Rencana (Mc)

a. Momen untuk portal arah X.

Data momen :

$M_{Dy\ atas}$ = - 5,688 kNm

$M_{Dy\ bawah}$ = 6,196 kNm

$M_{Ly\ atas}$ = - 0,844 kNm

$M_{Ly\ bawah}$ = 1,102 kNm

$M_{Ey\ atas}$ = -135,913 kNm

$M_{Ey\ bawah}$ = 235,499 kNm

$M_{Eyy\ atas}$ = 116 kNm

$M_{Eyy\ bawah}$ = --201,89 kNm

Daerah Atas

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2. (-5,688) + 1,6. (-0,844) = -8,176 kNm

$1,05\ (M_{Dy} + M_{Ly})$ = 1,05. ((-5,688) + ((-0,844). 0,6))

Mby = -6,504 kNm

$1,05\ M_{Ey}$ = 1,05.(- 135,913) = -142,709 kNm

Msy = -142,709 kNm

Mby + Msy =(-6,504) +(-142,709) = -149,213 kNm

Tetapi tidak perlu lebih besar dari :

$$1,05(M_{Dy} + M_{Ly} + \frac{4}{k} M_{Ey}) = 1,05\ ((-5,688)+(-0,844) + \frac{4}{1}(-135,913))$$

= -577,635 kNm

Daerah Bawah

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2. 6,196 + 1,6. 1,102 = 9,199 kNm

$1,05\ (\ M_{Dy} + M_{Ly}\ )$ = 1,05. (6,196 + (1,102. 0,6)

Mby = 7,2005 kNm

$1,05\ M_{Ey}$ = 1,05. 235,499 = 247,274 kNm

Msy = 247,274 kNm

Mby + Msy = 7,2005 + 247,274 = 254,4745 kNm

Tetapi tidak perlu lebih besar dari :

$1,05\ (\ M_{Dy} + M_{Ly} + \frac{4}{k}\ M_{Ey}\ )$ = 996,76 kNm

M pakai : Atas = -149,213 kNm

Bawah = 254,4745 kNm

b. Momen untuk portal arah Y

Data momen

$M_{Dx\ atas}$ = 92,155 kNm

$M_{Dx\ bawah}$ = 20,541 kNm

$M_{lx\ atas}$ = 16,421 kNm

$M_{Lx\ bawah}$ = 6,178 kNm

$M_{Ex\ atas}$ = -127,74 kNm

$M_{Ex\ bawah}$ = 340,63 kNm

$M_{Exx\ atas}$ = 128,09 kNm

$M_{Exx\ bawah}$ = -341,79 kNm

Daerah atas

$1{,}2\ M_{Dx} + 1{,}6\ M_{Lx}$ = 1,2. 92,155 + 1,6. 16,421 = 136,86 kNm

$1{,}05\ (\ M_{Dx} + M_{Lx})$ = 1,05. (92,155 + (16,421. 0,6))

Mbx = 107,108 kNm

$1{,}05\ M_{Ex}$ = 1,05. 128,09 = 134,489 kNm

Msx = 134,489 kNm

Mbx + Msx = 107,108 + 134,489 = 241,597 kNm

Tetapi tidak perlu lebih besar dari :

$$1{,}05\ (\ M_{Dx} + M_{Lx} + \frac{4}{k}\ M_{Ex}\ ) = 651{,}962 \text{ kNm}$$

Daerah bawah

$1{,}2\ M_{Dx} + 1{,}6\ M_{Lx}$ = 1,2. 20,541 + 1,6. 6,178 = 34,533 kNm

$1{,}05\ (\ M_{Dx} + M_{L,rx}\ )$ = 1,05. (20,541 + 6,178. 0,6)

Mbx = 25,46 kNm

$1{,}05\ M_{Ex}$ = 1,05. (-341,79) = -358,881 kNm

Msx = -358,881 kNm

Mbx + Msx = 25,46 + -358,881 = -333,421 kNm

Tetapi tidak perlu lebih besr dari :

$$1{,}05\ (\ M_{Dx} + M_{Lx} + \frac{4}{k}\ M_{Ex}\ ) = -384{,}3406 \text{ kNm}$$

M pakai : Atas = 241,597 kNm

Bawah = -333,421 kNm

c. Gaya aksial

Data Gaya Aksial

$P_{D\ atas}$ = -2339,3 kN

$P_{D\ bawah}$ = -2403,953 kN

$P_{L\ atas}$ = -559,224 kN

$P_{L\ bawah}$ = -559,224 kN

$P_{E\ atas}$ = -171,748 kN

$P_{E\ bawah}$ = -171,748 kN

Daerah Atas

$1{,}2\ P_D + 1{,}6\ P_L$ = 1,2. (-2339,3) + 1,6. (-559,224) = -3701,918 kN

$1{,}05\ (P_D + P_L)$ = 1,05.((- 2339,3) + (-559,224) . 0,6)

Pb = -2808,576 kN

$1{,}05\ P_E$ = 1,05. -171,748 = -180,335 kN

Ps = -180,335 kN

Pb + Ps = -2808,576 + -180,335 = -2988,911 kN

Tetapi tidak perlu lebih besar dari :

$1{,}05\ (P_D + P_L + \frac{4}{k} . P_E)$ = -3764,789 kN

Daerah Bawah

$1{,}2\ P_D + 1{,}6\ P_L$ = 1,2. (-2403,953) + 1,6. (-559,224) = -3779,501 kN

$1{,}05\ (P_D + P_L)$ = 1,05.((- 2403,953) + (-559,224) . 0,6)

Pb = -2876,462 kN

$1{,}05\ P_E$ = 1,05. -171,748 = -180,335 kN

Ps = -180,335 kN

Pb + Ps = -2876,462 + -180,335 = -3056,796 kN

Tetapi tidak perlu lebih besar dari :

$$1{,}05\ (P_D + P_L + \frac{4}{k}.P_E) = -3832{,}675\ \text{kN}$$

Pu pakai : Atas = -3701,918 kN

Bawah = -3779,501 kN

4.6.2 Kriteria Kolom dan Pembesaran Momen

Menghitung kekakuan kolom

a. Arah X

$$Ec = Eg = 4700.\sqrt{f'c} = 4700.\sqrt{22{,}5} = 22294{,}1\ \text{Mpa}$$

Dicoba dimensi kolom 600 x 600 mm

$$Ic\ (\text{inersia kolom}) = \frac{1}{12}.600.600^3 = 1{,}08.10^{10}\ \text{mm}^4$$

$$\beta_d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.6{,}196}{1{,}2.6{,}196 + 1{,}6.1{,}102} = 0{,}808$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta_d)} = \frac{22294{,}1.1{,}08.10^{10}}{2{,}5(1+0{,}808)} = 5{,}326.10^{13}\ \text{Nmm}^2$$

Menghitung momen inersia balok di kanan kiri kolom. Dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok dikanan kiri ujung atas kolom yaitu :

$$I_{cr} = \frac{I_g}{2} = \frac{1}{2}\left[\frac{1}{12}.350.650^3\right] = 4{,}005.10^9\ \text{mm}^4$$

2. Momen inersia balok dikanan kiri ujung bawah kolom = 0, karena

ujung jepit

Lu (panjang kolom) = 5,25 m

Lg (panjang bersih balok) = 6,6 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lu}\right)}{\sum\left(\frac{E_c.I_{cr}}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{5{,}326.10^{13}}{3200}\right)+\left(\frac{5{,}326.10^{13}}{5250}\right)}{\left(\frac{22294{,}1.4{,}005.10^{9}}{6600}\right)+\left(\frac{22294{,}1.5{,}7167.10^{9}}{1{,}925}\right)} = 0{,}34$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,05

$\frac{k.lu}{r} = \frac{1{,}05.5250}{0{,}3.600} = 30{,}625 > 22$ (termasuk kolom panjang)

Beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLu)^2} = \frac{\pi^2.5{,}326.10^{13}}{(1{,}05.5250)^2} = 17280590{,}97 \text{ N}$$

menghitung faktor pembesaran momen δ_{by}

$$\delta b_y = \frac{Cm}{1-\left(\frac{Pu}{\phi Pc}\right)} \geq 1$$

cm = 1 (portal tanpa pengaku)

$$\delta b_y = \frac{1}{1-\left(\frac{3779501}{0{,}65.17280590{,}97}\right)} = 1{,}00034 > 1$$

menghitung factor pembesaran δ_{sy}

kolom As-A4

$$\psi_{atas} = \frac{\left(\frac{5{,}326.10^{13}}{3200}\right)+\left(\frac{5{,}326.10^{13}}{5250}\right)}{\left(\frac{22294{,}1.4{,}005.10^{9}}{6600}\right)} = 2{,}0$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,28

beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLu)^2} = \frac{\pi^2.5{,}326.10^{13}}{(1{,}28.5250)^2} = 11832334{,}48 \text{ N}$$

$\Sigma Pc = 17280590{,}97 + 11832334{,}48 = 29112925{,}45$ N

$\Sigma Pu = P_{kolom\ As\text{-}A} + P_{kolom\ As\text{-}B}$

$= 2356{,}7272 + 3779{,}501 = 6136{,}2284$ kN

$$\delta_{sy} = \frac{1}{1-\left(\frac{\sum Pu}{\phi.\sum Pc}\right)} \geq 1$$

$$= \frac{1}{1-29112925{,}45} = 1{,}00032 > 1$$

Momen akibat pembesaran momen :

Muy, bawah $= \delta_{by}\, Mby + \delta_{sy}\, Msy$

$= 1{,}00034.\ 134{,}489 + 1{,}00032.\ 358{,}881 = 254{,}56$ kNm

Muy, atas $= \delta_{by}\, Mby + \delta_{sy}\, Msy$

$= 1{,}00034.\ 107{,}108 + 1{,}00032.\ 25{,}4596 = 149{,}26$ kNm

b. Arah Y

$Ec = Eg = 4700.\sqrt{f'c} = 4700.\sqrt{22.5} = 22294{,}1$ Mpa

Ic (inersia kolom) $= \frac{1}{12} . 600^3 . 600 = 1{,}08.10^{10}$ mm^4

$$\beta_d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.92{,}155}{1{,}2.92{,}155 + 1{,}6.16{,}421} = 0{,}808$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta_d)} = \frac{22294{,}1.1{,}08.10^{10}}{2{,}5(1+0{,}808)} = 5{,}327.10^{13} \text{ Nmm}^2$$

Menghitung momen inersia balok di kanan kiri kolom. Dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok dikanan kiri ujung atas kolom yaitu :

$$I_{cr} = \frac{I_g}{2} = \frac{1}{2}\left[\frac{1}{12}.350.650^3\right] = 4{,}005.10^9 \text{ mm}^4$$

2. Momen inersia balok dikanan kiri ujung bawah kolom = 0, karena ujung jepit

Lc (panjang kolom) = 5,25 m

Lg (panjang bersih balok) = 6,6 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{E_c I_{cr}}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{5{,}327.10^{13}}{3200}\right) + \left(\frac{5{,}327.10^{13}}{5250}\right)}{\left(\frac{22294{,}1.4{,}005.10^9}{6600}\right) + \left[\frac{22294{,}1.4{,}005.10^9}{6600}\right]} = 0{,}99$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,15

$$\frac{k.lu}{r} = \frac{1,15.5250}{(0,3.600)} = 33,54 > 22$$ (termasuk kolom panjang)

Beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLc)^2} = \frac{\pi^2.5,327.10^{13}}{(1,15.5250)^2} = 14408320,57 \text{ N}$$

menghitung faktor pembesaran momen δ_{by}

$$\delta b_y = \frac{Cm}{1-\left(\frac{Pu}{\phi Pc}\right)} \geq 1$$

cm = 1 (portal tanpa pengaku)

$$\delta b_x = \frac{1}{1-\left(\frac{3779492}{0,65.14408320,57}\right)} = 1,0004 > 1$$

menghitung faktor pembesaran δ_{sy}

kolom portal As-B1

$$\psi_{atas} = \frac{\left(\frac{5,208.10^{13}}{3200}\right)+\left(\frac{5,208.10^{13}}{5250}\right)}{\left(\frac{22294,1.4,005.10^{9}}{3000}\right)} = 0,44$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,08

Beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLc)^2} = \frac{\pi^2.5,208.10^{13}}{(1,08.5250)^2} = 7965141,052 \text{ N}$$

kolom portal As-B1'

$$\psi_{atas} = \frac{\left(\frac{5{,}208.10^{13}}{3200}\right)+\left(\frac{5{,}208.10^{13}}{5250}\right)}{\left(\frac{22294{,}1.4{,}005.10^{9}}{3150}\right)+\left(\frac{22294{,}1.4{,}005.10^{9}}{3000}\right)} = 0{,}22$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,04

Beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLc)^2} = \frac{\pi^2.5{,}208.10^{13}}{(1{,}04.5250)^2} = 8426571{,}16 \text{ N}$$

kolom portal As-B2

$$\psi_{atas} = \frac{\left(\frac{5{,}493.10^{13}}{3200}\right)+\left(\frac{5{,}493.10^{13}}{5250}\right)}{\left(\frac{22294{,}1.4{,}005.10^{9}}{6600}\right)+\left(\frac{22294{,}1.4{,}005.10^{9}}{6600}\right)} = 0{,}99$$

$\psi_{bawah} = 0$ (ujung jepit)

Dari nomogram portal tanpa pengaku, didapat k = 1,15

Beban tekuk Euler yang terjadi adalah:

$$P_c = \frac{\pi^2 EI}{(kLc)^2} = \frac{\pi^2.5{,}493.10^{13}}{(1{,}15.5250)^2} = 14858607{,}6 \text{ N}$$

kolom portal As-B3

Dengan cara yang sama didapat:

- $\psi_{atas} = 0{,}99$
- $\psi_{bawah} = 0$
- k = 1,14

- Pu = 3666,2913 kN
- EI = $5,33.10^{13}$ Nmm^2
- P_c = 14679620,31 N

kolom portal As-B5

Dengan cara yang sama didapat:

- ψ_{atas} = 0,99
- ψ_{bawah} = 0
- k = 1,14
- Pu = 3664,1789 kN
- EI = $5,33.10^{13}$ Nmm^2
- P_c = 14675768,8 N

kolom portal As-B6

Dengan cara yang sama didapat:

- ψ_{atas} = 1,03
- ψ_{bawah} = 0
- k = 1,15
- Pu = 2346,96 kN
- EI = $5,549.10^{13}$ Nmm^2
- P_c = 15009846,84 N

kolom portal As-B7

Dengan cara yang sama didapat:

- ψ_{atas} = 0,99
- ψ_{bawah} = 0

- k = 1,14
- Pu = 2894,8091 kN
- EI = $5,33.10^{13}$ Nmm^2
- P_c = 14669901,18 N

Σ Pc = 14408320,57 + 796514,052 + 8426571,16 + 14858607,6
+ 14679620,31 + 14675768,8 + 15009846,84 + 14669901,18
= 104693777,5 N

Σ Pu = 3779,492 + 1176,405 + 1734,1422 + 3080,308 + 3666,29
+ 3664,179 + 2346,961 + 2894,809
= 22342,5875 kN

$$\delta s_x = \frac{1}{1-\left(\frac{\sum Pu}{\sum Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{22342587,5}{0,65.104693777,5}\right)} = 1,00033$$

Momen akibat pembesaran momen :

Mux bawah = δ_{bx} Mbx+ δ_{sx} Msx
= 1,0004. 134,4894 + 1,00033. 358,881
= 493,54 kNm

Mux atas = δ_{bx} Mbx+ δ_{sx} Msx
= 1,0004. 107,108 + 1,00033. 25,46
= 132,619 kNm

4.6.3 Analisis Gaya Aksial dan Momen akibat balok

h = 5,6 m

hn = 5,25 m

Rv = 1 (jumlah lantai diatasnya; n = 3)

ωd = 1 (untuk lantai dasar)

k = 1

a. Perhitungan Arah X

$Mkap_{(kiri)}$ = 1,25. 553,435 = 691,794

$Mkap_{(kanan)}$= 1,25. Mnak = 1,25. 606,303 = 757,879 kNm

menghitung gaya aksial rencana :

$$Nu,k_x = \frac{0,7.Rv.(Mkap_{ki} + Mkap_{ka})}{l} + 1,05.\ Ng$$

$$= \frac{0,7.1.(691,794 + 757,879)}{7,2} + 1,05(\ 2403,953 + 559,2235\ )$$

= 3252,276 kN

tidak perlu melebihi :

Nu,k_x = 1,05 (N_D+ N_L+ 4.N_E)

= 1,05 (2403,953 +559,2235 + 4.171,7475)

= 3832,675 kN

dipakai Nu,k_x = 3252,276 kN

menghitung α :

Lantai 1

$M_{E,K\ atas}$ = 272,13 kNm

$M_{E,K\ bawah}$ = -

Lantai 2

$M_{E,K\ atas}$ = 220,4 kNm

$M_{E,K\ bawah}$ = 134,64 kNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{272,13}{272,13+134,64} = 0,67$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = -$$

menghitung momen rancang kolom :

$$Mu,k_x atas = \frac{h}{hn}\ \omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}} Mkap,ki + \frac{l_{ka}}{l'_{ka}} Mkap,ka\right)$$

$$= \frac{5,6}{5,25}.1.0,67.0,7.\left(\frac{2,4}{1,925}.691,794 + \frac{7,2}{6,6}.757,879\right) = 412,993\ \text{kNm}$$

$Mu,k_x baw$ = 617,327 kNm

tidak perlu melebihi :

$Mu,k = 1,05\ (M_D + M_L + 4.M_E) = 1,05\ (6,196 + 1,102 + 4\ .235,499\ )$

= 996,759 kNm

Mu,k_x pakai = 617,327 kNm

b. .Perhitungan Arah Y

$Mkap_{(kiri)} = Mkap_{(kanan)}$ = 1,25. 409,6256 = 512,032 kNm

menghitung gaya aksial rencana :

$$Nu,k_y = \frac{0,7.Rv.(Mkap_{ki} + Mkap_{ka})}{l} + 1,05.\ Ng$$

$$= \frac{0,7.1.(512,032 + 512,032)}{7,2} + 1,05(\ 2403,953 + 559,2235)$$

= 3210,897 kN

tidak perlu melebihi :

$Nu,k_y = 1,05\ (Nd + Nl + 4.Ne)$

= 1,05 (2403,953 + 559,224+ 4.171,748)

= 3832,675 kN

dipakai Nu,k_y = 3210,897 kN

menghitung α : 128.0851

$$\alpha \text{ ka} = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{128,085}{128,085 + 78,1} = 0,621$$

α bawah = -

menghitung momen rancang kolom :

$$Mu,k_y \text{ atas} = \frac{h}{hn} \omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}} Mkap,ki + \frac{l_{ka}}{l'_{ka}} Mkap,ka\right)$$

$$= \frac{5,6}{5,25} 1.\ 0,621.0,7.\left(\frac{7,2}{6,5}.512,032 + \frac{7,2}{6,5}.512,032\right)$$

= 529,092 kNm

Mu,k_y bwh = 417,073 kNm

tidak perlu melebihi :

$Mu,k_y = 1,05\ (M_D + M_L + 4.M_E) = 1,05\ (92,155 + 16,42 + 4.\ 340,63\ )$

= 1544,64 kNm

Mu,k_y pakai = 417,073 kNm

4.6.4 Perencanaan Penulangan Kolom

$$Pn = \frac{Pu}{\theta} = \frac{3779,5012}{0,65} = 5814,617 \text{ kN}$$

$$Mnx = \frac{Mux}{\theta} = \frac{617{,}327}{0{,}65} = 949{,}734 \text{ kNm}$$

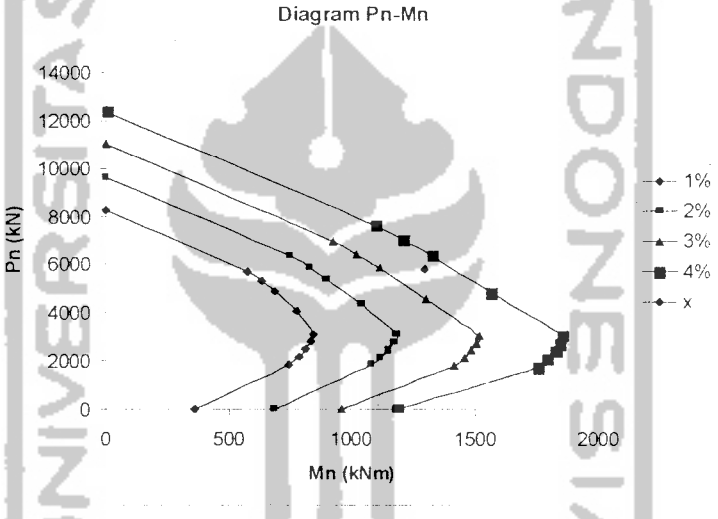
$$Mny = \frac{Muy}{\theta} = \frac{417{,}073}{0{,}65} = 641{,}65 \text{ kNm}$$

Digunakan Mox untuk perencanaan .

$$Mox \text{ perlu} = Mnx + Mny \left(\frac{b}{h}\right)\left(\frac{1-\beta}{\beta}\right)$$

$$= 949{,}734 + 641{,}65 \left(\frac{0{,}6}{0{,}6}\right)\left(\frac{1-0{,}65}{0{,}65}\right)$$

$$= 1295{,}238 \text{ kNm}$$



Gambar 4.25 Grafik Pn-Mn

Ast = 0,036.600.600= 11880 mm^2

As = As` = 0,5.Ast = 5940 mm^2

dipakai 18D22 dengan $As_{ada} = As`_{ada} = 6842{,}39$ mm^2

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600 + fy} = 323{,}4 \text{ mm}$$

$ab = \beta_1 . Xb = 0{,}85 . 323{,}4 = 274{,}89$ mm

$$f's = 600\frac{(Xb - d')}{Xb} = 600\frac{(323,4 - 61)}{323,4} = 486,83 \text{ MPa} > fy = 400 \text{ MPa}$$

Dengan demikian digunakan f's = fy = 400 MPa

Ccb = 0,85.f'c.b.ab = 0,85.22,5.600. 274,89 = 3154362,75 N

Csb = As'(fs'- 0,85.f'c) = 5940 (400 – 0,85.22,5) = 2262397,5 N

Tsb = As.fy = 5940.400 = 2376000 N

Pnb = Ccb + Csb – Tsb = 3154362,75 + 2262397,5 – 2376000

= 3154,363 N

$$Mnb = Ccb\left[\frac{h}{2} - \frac{ab}{2}\right] + Csb\left(\frac{h}{2} - d'\right) + Tsb\left(d - \frac{h}{2}\right)$$

$$= 3154362,75\left[\frac{600}{2} - \frac{274,89}{2}\right] + 2262397,5\left(\frac{600}{2} - 61\right)$$

$$+ 2376000\left(539 - \frac{600}{2}\right)$$

= 1676,127 kNm

$$eb = \frac{Mnb}{Pnb} = \frac{1676,127}{3154,363} = 0,531$$

$$e = \frac{Mox}{Pn} = \frac{1295,238}{5814,617} = 0,223$$

karena e_b > e runtuh desak

Kontrol kekuatan kolom terhadap patah desak

$$Pn = \frac{As'.fy}{\frac{e}{d - d'} + 0,5} + \frac{b.h.f'c}{\frac{3.h.e}{d^2} + 1,18}$$

$$\frac{6842,39.400}{\frac{223}{539 - 61} + 0,5} + \frac{600.600.22,5}{\frac{3.600.223}{539^2} + 1,18}$$

= 5993,326 kN > Pn = 5814,617 kN

Hitung momen tahanan nominal Moxn terhadap sumbu x bila Moy = 0

$$a = \frac{Pn}{0{,}85.f'c.b} = \frac{5993{,}326.10^3}{0{,}85.22{,}5.600} = 522{,}294 \text{ mm}$$

$$X = \frac{522{,}294}{0{,}85} = 614{,}464 \text{ mm}$$

$$fs' = 600.\left(\frac{X - d'}{X}\right) = 600.\left(\frac{614{,}464 - 61}{614{,}464}\right) = 540{,}44 \text{ Mpa}$$

Pakai fs' = fy = 400 Mpa

Cc = 0,85.f'c.b.a = 0,85.22,5.600.522,294 = 5993323,65 N

Cs = As'.fs' = 6842,39.400 = 2736956 N

Ts = As.fy = 6842,39.400 = 2736956N

$$Moxn = Cc.\left(\frac{h}{2} - \frac{a}{2}\right) + Cs.\left(\frac{h}{2} - d'\right) + Ts.\left(d - \frac{h}{2}\right)$$

$$= 5993323{,}65.\left(\frac{600}{2} - \frac{522{,}294}{2}\right) + 2736956.\left(\frac{600}{2} - 61\right)$$

$$+ \ 2736956.\left(539 - \frac{600}{2}\right)$$

= 1541,124 kNm > Mox perlu = 1295,238 kNm

Penampang diasumsikan bujur sangkar sehingga :

Moxn = Moyn = 1541,124 kNm

$$\frac{Mnx}{Moxn} = \frac{949{,}734}{1541{,}124} = 0{,}616$$

Dari diagram faktor kontur β untuk kolom segiempat yang mengalami lentur biaksial, dicoba β = 0,5 dengan Mnx / Moxn = 0,616 didapat :

$$\frac{Mny}{Moyn} = 0{,}53$$

Mny = 0,53. 1434,56 = 760,318 kNm > Mny_{perlu}= 641,65 kNm **..Ok**

4.6.5 Perencanaan Penulangan Geser

$$V_{u,k} = \frac{Mu,k_{atas} + Mu,k_{bawah}}{h'_k} = \frac{337{,}442 + 617{,}327}{5{,}25} = 181{,}8608 \text{ kN}$$

tidak perlu melebihi

$$V_{u,k} = 1{,}05\,(V_{D,k} + V_{L,k} + \frac{4}{K} V_{E,k}) = 1{,}05\,(12{,}79 + 1{,}83 + 4.83{,}91)$$

$$= 367{,}773 \text{ kN}$$

$V_{u,k}$ terpakai = 181,8608 kN

Daerah sendi plastis

d = 0,539 m

Vc = 0 (pada daerah sendi plastis,Vc dianggap 0)

$$V_{u,k} \text{ terhit.} = \frac{h'k - d}{h'k} V_{u,k}terpakai = \frac{5{,}25 - 0{,}539}{5{,}25}.181{,}8608 = 163{,}1897 \text{ kN}$$

$$Vs = \frac{V_{u,k}terhitung}{\phi} = \frac{163{,}1897}{0{,}6} = 271{,}9829 \text{ kN}$$

Dipakai sengkang D10 dengan Av = 2.0,25.π.d^2 = 157,143 mm^2

$$\text{Jarak (S)} < \frac{Av.fy.d}{Vs} = \frac{157{,}143.240.539}{271{,}9829.10^3} = 74{,}74 \text{ mm}$$

< d/4 = 134,75 mm

maka dipakai P10 - 70

Daerah luar sendi plastis

$V_{u,k}$ terhitung = 163,1897 kN

$$\frac{Vu,k\ terhitung}{0,6} = \frac{163,1897}{0,6} = 271,9829\ kN$$

Nu,k = 5814,62 kN

$$Vc = \left(1+\frac{Nu,k}{14.Ag}\right)\frac{1}{6}.\sqrt{f'c}.b.d = \left(1+\frac{5814620}{14.600.600}\right)\frac{1}{6}.\sqrt{22,5}.600.539$$

= 551,1092 kN

$$Vs = \frac{Vu,k\ terhitung}{0,6} - Vc = 453,304 - 271,9829 = -279,1263\ kN$$

Karena beton sudah mampu untuk menahan geser maka jarak sengkang diambil jarak sejauh 200 mm.

maka dipakai P10–200

4.6.6 Pertemuan Balok Kolom

4.6.6.1 Pertemuan Balok Kolom Luar

1. Perhitungan gaya-gayaa dalam

Mnak, b = 409,626 kNm

Mkap,b = 1,25. Mnak, b = 1,25.409,626 =512,0325 kNm

$$Vkol = \frac{0,7\left(Lki/Lki'.Mkap,ki + Lka/Lka'.Mkap,ka\right)}{1/2.(hk,a + hk,b)}$$

$$Vkol = \frac{0,7.\left(0+7,2/6,6.512,0325\right)}{1/2.(3,85+5,6)} = 84,026\ kN$$

T = 0,7.Mkap,ka/zka

d = 539 mm = 0,539 m

zka = 0,85.d = 0,85.0,539 = 0,4582 m

T = 0,7.512,0325 / 0,4582 = 782,326 kN

Vj,h = T – Vkol = 782,326 – 84,026 = 698,3 kN

2. *Kontrol tegangan geser horisontal minimal*

$$Vj,h = \frac{Vj,h}{bj.hc} \leq 1{,}5\sqrt{f'c} \quad , \text{ bj} = \text{b} = 600 \text{ mm}$$

$$Vj,h = \frac{698{,}3}{0{,}6.0{,}6} = 1940 \text{ kN/m}^2$$

$$= 1{,}94 \text{ N/m}^2 < 1{,}5.\sqrt{22{,}5} = 7{,}12 \text{ N/m}^2 \ldots \textbf{OK}$$

3. *Penulangan geser horisontal*

Nu = 3779,5012 kN

$$\frac{Nu}{Ag} = \frac{3779{,}5012}{0{,}6.0{,}6} = 10499 \text{ kN/m}^2$$

$$= 10{,}499 \text{ N/mm}^2 > 0{,}1.\text{fc'} = 0{,}474 \text{ Mpa}$$

$$\text{Vc,h} = 2/3.\sqrt{\left\{\left(Nu,k/Ag\right) - 0{,}1.f'c\right\}}.\text{bj.hc}$$

$$\text{Vc,h} = 2/3.\sqrt{\left\{\left(3779501{,}2/600.600\right) - 0{,}1.22{,}5\right\}}.600.600$$

$$= 689290 \text{ N} = 689{,}29 \text{ kN}$$

Vs,h + Vc,h = Vj,h

Vs,h = 698,3 – 689,29 = 9,011 kN

$$\text{Aj,h} = \frac{Vs,h}{fy} = \frac{9011}{400} = 22{,}526 \text{ mm}^2$$

Digunakan sengkang rangkap P10 dengan Av = 314,159 mm^2

$$\text{Jumlah lapis sengkang} = \frac{22{,}536}{314{,}159} = 0{,}082 = 2 \text{ lapis}$$

4. Penulangan geser vertikal

$$Vc,v = \frac{Asc'}{Asc} Vj,h.\left(0,6 + {}^{Nu,k}/_{Ag.f'c}\right)$$

$$Vc,v = 1.698,3.10^3.\left(0,6 + {}^{3779,5012.10^3}/_{600.600.22,5}\right)$$

$$= 33002 \text{ N} = 33,002 \text{ kN}$$

Vj,v = d/hc.Vj,h = 0,539/0,6. 698,3 = 627,306 kN

Vs,v = Vj,v – Vc,v = 627,306 – 33,002 = 594,304 kN

$$Aj,v = \frac{Vs,v}{fy} = \frac{594304}{400} = 1485,761 \text{ mm}^2$$

pakai 4 D22 dengan As = 1521,144 mm^2 > 1485,761 mm^2

4.6.6.2 Pertemuan balok kolom dalam

1. Perhitungan gaya-gaya dalam

Mnak,b kiri = 553,435 kNm

Mnak,b kanan = 606,303 kNm

Mkap,b kiri = 1,25.553,435 = 691,794 kNm

Mkap,b kanan = 1,25.606,303 = 757,879 kNm

$$Vkol = \frac{0,7\left({}^{Lki}/_{Lki'}.Mkap,ki + {}^{Lka}/_{Lka'}.Mkap,ka\right)}{{}^{1}/_{2}.(hk,a + hk,b)}$$

$$Vkol = \frac{0,7\left({}^{7,2}/_{6,6}.691,794 + {}^{7,2}/_{6,6}.757,879\right)}{{}^{1}/_{2}.(3,85 + 5,6)} = 111,949 \text{ kN}$$

Cki = 0,7.Mkap,b ki/zki = 0,7. 691,794 / 0,4582 = 1056,981 kN

Tka = 0,7.Mkap,b ka/zka = 0,7.757,879/ 0,4582 = 1157,951 kN

Vj,h = Cki + Tka – Vkol

$$= 1056{,}981 + 1157{,}951 - 111{,}949 = 2102{,}982 \text{ kN}$$

2. *Kontrol tegangan geser horisontal*

$$Vj,h = \frac{Vj,h}{bj.hc} < 1{,}5\sqrt{f'c} \quad ; \text{bj pakai 600 mm}$$

$$Vj,h = \frac{2102{,}982}{0{,}6.0{,}6} = 5842 \text{ kN/m}^2$$

$$= 5{,}842 \text{ N/mm}^2 < 1{,}5 . \sqrt{22{,}5} = 7{,}115 \text{ N/mm}^2 \ldots \textbf{OK}$$

3. *Penulangan geser horisontal*

Nu,k = 3779,5012 kN

$$\frac{Nu}{Ag} = \frac{3779{,}5012}{0{,}6.0{,}6} = 10499 \text{ kN/m}^2$$

$$= 10{,}499 \text{ N/m}^2 > 0{,}1.\ fc' = 2{,}25 \text{ Mpa}$$

$$Vc,h = 2/3.\sqrt{\left\{\left(Nu,k/Ag\right) - 0{,}1.f'c\right\}}.bj.hc$$

$$Vc,h = 2/3.\sqrt{\left\{\left(3779501{,}2/600.600\right) - 0{,}1.22{,}5\right\}}.600.600$$

$$= 689290 \text{ N} = 689{,}29 \text{ kN}$$

Vs,h + Vc,h = Vj,h

Vs,h = 2102,982 – 689,29 = 665,098 kN

$$Aj,h = \frac{Vs,h}{fy} = \frac{665098}{400} = 1662{,}745 \text{ mm}^2$$

Dipakai sengkang rangkap Av = 314,159 mm^2

Jumlah lapis sengkang = 1662,74 /314,159 = 5,291 = 6 lapis

4. Penulangan vertikal

$$Vc,v = \frac{Asc'}{Asc} Vj,h \left(0,6 + {}^{Nu,k}/_{Ag.f'c} \right)$$

$$Vc,v = 1.2102982 \left(0,6 + {}^{3779,5012}/_{600.600.22,5} \right)$$

$= 99388$ N $= 99,388$ kN

Vj,v = d/hc. Vj,h = 539/600. 2102,982 = 1301,08 kN

Vs,v = Vj,v – Vc,v = 1301,08 – 99,388 = 1201,692 kN

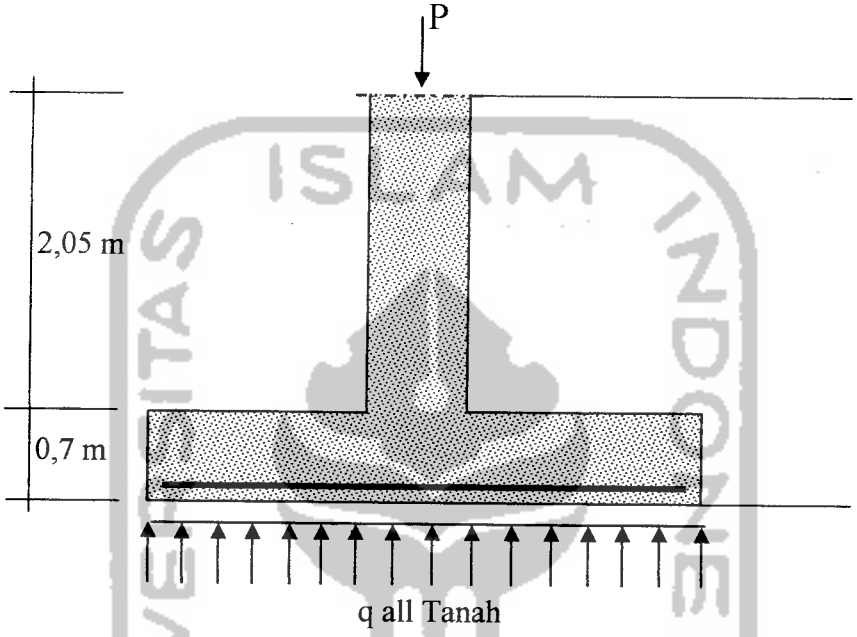
Aj,v = Vs,v / fy =1201,692 /400 = 3004,23 mm^2

pakai 8 D22 dengan As = 3042,288 mm^2 > 3004,23 mm^2

4.7 PERENCANAAN PONDASI

4.7.1 Perencanaan Dimensi Pondasi (P1)

1. Tinjauan Terhadap Beban Tetap



σ tanah = 425 kN/m^2 γbtanah = 15,64 KN/m^3

F'c = 22,5 Mpa γbeton = 24 KN/m^3

Fy = 400 Mpa Asumsi tebal pondasi (h) = 700 mm

P = 1318,345 kN Ukuran kolom : 600/600 mm

Mx = 274,2178 kNm

My = 326,3857 kNm

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

$= 425 - (0,7.24) - (2,05.15,64)$

$= 376,138\ kN/m^2$

Dimensi luas pelat pondasi : (terdapat momen yang bekerja pada arah x dan y)

$$\sigma_{\text{netto tanah}} = \frac{P}{A_{perlu}} + \frac{My}{1/6.Bx^2.By} + \frac{Mx}{1/6.By^2.Bx}$$

dicoba dengan nilai B = 2,7 m

$$A_{\text{perlu}} = \frac{P}{\sigma netto\ \tan ah - \left(\frac{My}{1/6.By^2.Bx}\right) - \left(\frac{Mx}{1/6.Bx^2.By}\right)}$$

$$= \frac{1318{,}345}{376{,}138 - \left(\frac{274{,}2178}{1/6.2{,}7^2.2{,}7}\right) - \left(\frac{326{,}3857}{1/6.2{,}7^2.2{,}7}\right)} = 6{,}32\ \text{m}^2$$

Digunakan penampang bujur sangkar dengan :

$P = L = \sqrt{6{,}32} = 2{,}61$ m $\longrightarrow P_{ada} = L_{ada} = 2{,}7$ m

Luas penampang pelat pondasi : $A_{ada} = P \times L = 2{,}7 \times 2{,}7 = 7{,}29\ \text{m}^2$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$A_{ada} = 7{,}29\ \text{m}^2 > A_{perlu} = 6{,}32\ \text{m}^2$ **.....Ok.**

Tegangan kontak yang terjadi di dasar pondasi :

$$\sigma_{\text{netto tanah}} = \frac{P}{A_{ada}} + \frac{My}{1/6.P^2.L} + \frac{Mx}{1/6.L^2.P}$$

$$\sigma_{\text{netto tanah}} = \frac{1318{,}345}{7{,}29} + \frac{274{,}2178}{1/6.2{,}7^2.2{,}7} + \frac{326{,}3857}{1/6.2{,}7^2.2{,}7}$$

$= 363{,}926\text{NkN/m}^2 < \sigma_{\text{nettotanah}} = 376{,}138\ \text{kN/m}^2$ **.....Aman.**

Jarak pusat tulangan tarik ke serat tekan beton :

$d = h - Pb - \frac{1}{2}.\varnothing_{\text{tul. pokok}} = 700 - 70 - \frac{1}{2}.22 = 619$ mm

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{274,2178}{1318,345} = 0,2586 \text{ m}$$

$$ey = \frac{My}{P} = \frac{326,3857}{1318,345} = 0,208 \text{ m}$$

Kontrol tegangan yang terjadi :

$$\sigma = \frac{P}{(L.(P-2.ex))+(P.(L-2.ey))}$$

$$= \frac{1318,345}{(2,7.(2,7-2.0,2176))+(2,7.(2,7-2.0,208))}$$

$= 108,775 \text{ kNm} < 1,5.\sigma_{netto} = 1,5.\ 376,138 = 564,207 \text{ kNm}$ **.....Aman.**

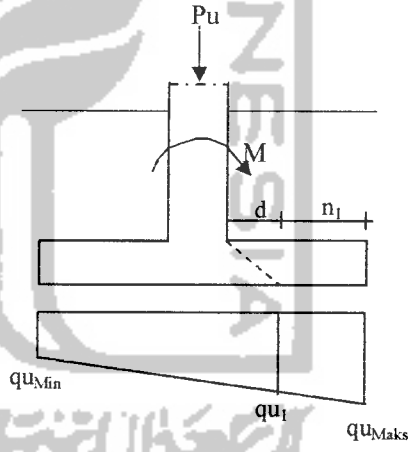
4.5.2 Perencananaan Geser Satu Arah

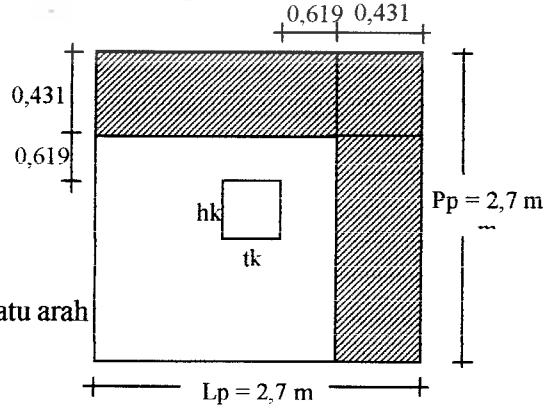
→ Ditinjau pada arah momen terbesar .

Pu = 1642,046 kN

Mux = 369,4413 kNm

Muy = 537,979 kNm





Gambar 4.27 Pondasi dengan geser satu arah

$$n_1 = \frac{Lp - tk - 2.d}{2} = \frac{2,7 - 0,60 - 2.0,619}{2} = 0,431 \text{ m}$$

Arah X

- Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{Mx}{1/6.L^2.P}$$

$$= \frac{1642,046}{7,29} \pm \frac{537,979}{1/6.2,7^2.2,7}$$

$qux_{max} = 389,441 \text{ kN/m}^2$

$qux_{min} = 61,253 \text{ kN/m}^2$

$$qu_m = \frac{(Lp - m).qux_{max} + m.qux_{min}}{Lp}$$

$$= \frac{(2,7 - 0,431).389,441 + 0,431.61,253}{2,7} = 336,883 \text{ kN/m}^2$$

$qux_{terjadi} = qux_{max} + qux_m) = ½. (389,441 + 336,883) = 363,061 \text{ kN/m}^2$

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

$Vu = qu_{xterjadi}. n_1 . L = 363,061. 0,431 . 2,7 = 422,494 \text{ kN}$

$$Vu/\phi = 422,494/0,6 = 704,157 \text{ kN}$$

- Kekuatan beton menahan geser:

$Vc = 1/6. \sqrt{f'c} . L. d = 1/6. \sqrt{22,5}\ 2,7. 0,619 . 10^3 = 1321,279 \text{ kN}$

- Kontrol gaya geser :

$Vc = 1321,279 \text{ kN} \geq Vu/\phi = 704,157 \text{ kN}$ **.....Aman.**

Arah Y

- Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{My}{1/6.L^2.P}$$

$$= \frac{1642,046}{7,29} \pm \frac{369,979}{1/6.2,7^2.2,7}$$

$qux_{max} = 337,864 \text{ kN/m}^2$

$qux_{min} = 112,910 \text{ kN/m}^2$

$$qu_m = \frac{(Lp - m).qux_{max} + m.qux_{min}}{Lp}$$

$$= \frac{(2,7 - 0,431).337,8648 + 0,431.112,629}{2,7} = 301,910 \text{ kN/m}^2$$

$qux_{terjadi} = qux_{max} + qux_m) = ½. (337,864 + 301,910) = 319,886 \text{ kN/m}^2$

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

$Vu = qu_{xterjadi}. n_1 . L = 192,866. 0,431 . 2,7 = 372,252 \text{ kN}$

$Vu/\phi = 372,252/0,6 = 620,420 \text{ kN}$

- Kekuatan beton menahan geser:

$Vc = 1/6. \sqrt{f'c} . L. d = 1/6. \sqrt{22,5}\ 2,7. 0,619 . 10^3 = 1321,279 \text{ kN}$

- Kontrol gaya geser :

$Vc = 1321,279 \text{ kN} \geq Vu/\phi = 620,420 \text{ kN}$ .....**Aman.**

4.5.3 Perencananaan Geser Dua Arah

→ Ditinjau pada arah momen terbesar.

x = hk + d

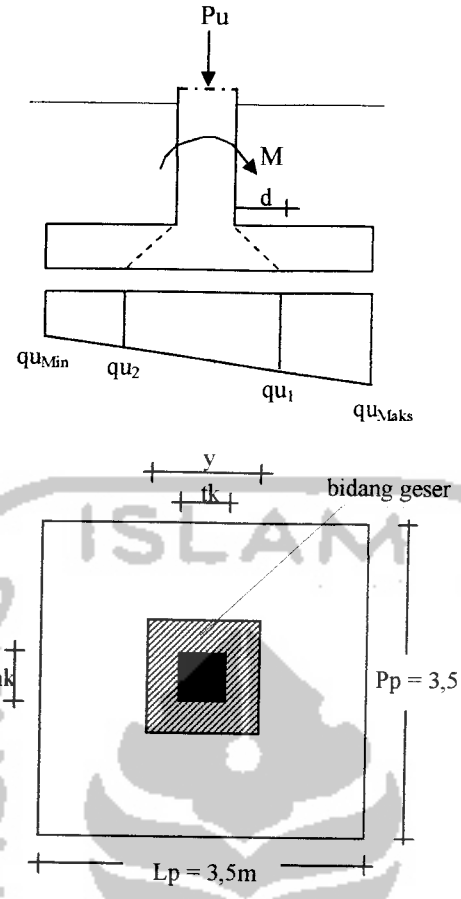
= 600 + 619

= 1219 mm = 1,219 m

y = tk + d

= 600 + 619

= 1219 mm = 1,219 m



Gambar 4.28 Pondasi dengan geser dua arah

• Tegangan kontak yang terjadi :

$$qu = \frac{P}{A_{perlu}} \pm \frac{My}{1/6.Bx^2.By} \pm \frac{Mx}{1/6.By^2.Bx}$$

$$= \frac{1642,046}{7,29} \pm \frac{537,979}{1/6.2,7^2.2,7} \pm \frac{369,979}{1/6.2,7^2.2,7}$$

qu_{max} = 501,857 kN/m^2

qu_{min} = 51,364 kN/m^2

qu_T = ½ (qu_1 + qu_2) = ½ (501,857 + 51,364) = 225,246 kN/m^2

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

Vu = qu_T. ((Pp .Lp) – (x.y))

=225,246 . ((2,7.2,7) – (1,219 .1,219)) = 1307,339 kN

$Vu/\phi = 1307{,}339/0{,}6 = 2178{,}898$ kN

- Kekuatan beton menahan geser :

$$\beta c = \frac{sisipanjang}{sisipendek} = \frac{Pp}{Lp} = \frac{2{,}7}{2{,}7} = 1$$

bo $= 2 . (x + y) = 2 . (1219 + 1219) = 4876$ mm

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

$= (1 + 2/1).(2.\sqrt{22{,}5}).\ 4876.\ 619.\ 10^{-3} = 85900$ kN

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

$= 4.\sqrt{22{,}5}.\ 4876.\ 619.\ 10^{-3} = 57267{,}15$ kN

- Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc_1 dan Vc_2, yaitu $Vc_2 = 57267{,}15$ KN

$Vc_2 = 57267{,}15$ KN $\geq Vu/\phi = 2178{,}898$ kN.....**Aman**.

4.5.1.4 Kuat Tumpuan Pondasi

- Kuat tumpuan Pondasi :

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

Luas pelat pondasi (A_2) $= P . L = 2{,}7 . 2{,}7 = 7{,}29$ m^2

Luas penampang kolom (A_1) $= hk . tk = 0{,}60 . 0{,}60 = 0{,}3600$ m^2

$\sqrt{A_2/A_1} = \sqrt{7{,}29/0{,}3600} = 4{,}5 > 2$ (jika lebih besar dari 2, dipakai nilai 2)

$\phi.Pn = \phi.(0{,}85.\ f'c.\ A_1.2)$

$= 0,7.\ (0,85.\ 22,5.\ 360000.2).10^{-3} = 9639$ kN

- Kuat tumpuan kolom :

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

$= 0,7.\ (0,85.\ 22,5.\ 360000).10^{-3} = 4819,5$ KN

- Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 9639$ kN$>\phi.Pn_{kolom} = 4819,5$ kKN .....Aman.

4.5.5 Perencanaan Tulangan Lentur Telapak Pondasi

Momen yang terjadi :

$$l = \frac{Lp - tk}{2} = \frac{2,7 - 0,60}{2} = 1,05 \text{ m}$$

$qu_{maks} = 501,8567$ kN/m^2

$Mu = 0,5.\ qu_{maks.}\ l^2 = 0,5\ .\ 501,857\ .\ 1,05 = 276,648$ kNm

$$Mu/\phi = \frac{276,648}{0,8} = 345,811 \text{ kNm}$$

- Digunakan tulangan bagi $\varnothing_{19}$ mm, sehingga luas tampang 1 tulangan pokok :

  $A_{1\varnothing} = \frac{1}{4}.\ \pi.\ D^2 = \frac{1}{4}.\ \pi.\ 19^2 = 283,385$ mm^2

- Tebal pelat pondasi : h = 700 mm, selimut beton (Pb) = 70 mm

  $d = h - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 700 - 70 - 0,5.\ 19 = 620,5$ mm

  $$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22,5} = 20,915$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d} = \frac{345,8115.10^6}{1000.620,5^2} = 0,898 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,00350$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{400}.\left(\frac{600}{600+400}\right) = 0,0244$$

$$\rho_{maks} = 0,75.\ \rho b = 0,75.\ 0,0244 = 0,0183$$

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

$$= \frac{1}{20,915}.\left(1-\sqrt{1-\frac{2.20,915.0,898}{400}}\right) = 0,0023 < \rho_{maks} = 0,0183$$

$$< \rho_{min} = 0,00350$$

$0,002 < 1,33\rho_{ada} = 0,00306 > \rho_{min}$

.....sehingga dipakai : $\rho_{min} = 0,00306$

$As_{perlu} = \rho_{perlu}$. b . d $= 0,00306$. 1000 . 620,5 $= 1898,73\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283,385.1000}{1898,73} = 149,25\ mm$$

$s \leq 2.h = 2$ .700 $= 1400$ mm

$s \leq 250$ mm

→ Dipakai Tulangan Pokok : D_{19} – 140 mm

$$As_{aktual} = \frac{A_{1\theta}.1000}{s} = \frac{379,94.1000}{140} = 2024,1786\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{aktual}.fy}{0{,}85.f.c.b} = \frac{2024{,}1785.400}{0{,}85.22{,}5.1000} = 42{,}3358 \text{ mm}$$

$$Mn = As_{aktual}.fy.\ (d - a/2)$$

$$= 2024{,}1785\ .\ 400\ (620{,}5 - 42{,}3358/2)$$

$$= 485{,}2621 \text{ kNm} \geq Mu/\phi = 345{,}8107 \text{ kNm} \text{ .....Aman.}$$

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 700 = 1400 \text{ mm}^2$

• Digunakan tulangan bagi ∅12 mm, sehingga luas tampang 1 tulangan susut :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 12^2 = 113{,}04 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113{,}04.1000}{1400} = 80{,}7428 \text{ mm} \approx 80 \text{ mm}$$

→ **Dipakai Tulangan Susut : P_{12} – 80 mm**

Perencanaan Pondasi Bujur sangkar	
σ tanah (KN/m2)	425
f'c (MPa)	22.5
fy (MPa)	400
γb beton (KN/m3)	24
γb tanah (KN/m3)	15.64
P (KN)	1318.345
Mx (KNm)	274.2178
My (KNm)	326.3857
h kolom (mm)	2.05
t kolom (mm)	0.6
tebal pelat (h) (mm)	0.7
σ netto tanah (KN/m2)	**376.138**
Dicoba nilai B (m)	2.7
A perlu (m2)	6.828854026
B perlu	2.613207613
B ada	2.7
A ada (m2)	7.29
σ kontak	**363.9258497**
Kontrol tegangan σnetto tanah ≥ σ kontak	
AMAN	
Pb (mm)	70
θ tul.pokok (mm)	22
d (mm)	619.00

Tinjauan Beban Sementara	
P (KN)	1318.345
Mx (KNm)	326.3857
My (KNm)	274.2178
ex (m)	0.247572297
ey (m)	0.208001547
B ada	2.7
σ netto tanah (KN/m2)	376.138
1,5.σnetto tanah(KN/m2)	**564.207**
σ kontak (KN/m2)	**108.7752263**
Kontrol 1,5 σnetto tanah ≥ σ kontak	
AMAN	

Perencanaan Geser 1 Arah	
P (KN)	1642.046
Mx (KNm)	537.979
My (KNm)	369.4413
t kolom (mm)	0.6
d (mm)	619.00
m (m)	0.431
ex (m)	0.247572297
ey (m)	0.208001547
L (m)	2.7
f'c (MPa)	22.5
Perencanaan Geser Arah X	
qux max (KN/m2)	389.2393538
qux min (KN/m2)	61.25337601
qux m (KN/m2)	336.8830736
qux terjadi (KN/m2)	363.0612137
Vu (KN)	422.4943344
Vu/ϕ (KN)	**704.1572239**
Vc (KN)	**1321.278663**
kontrol Vc > Vu/ϕ	
AMAN	
Perencanaan Geser 1 Arah (y)	
qux max (KN/m2)	337.8637403
qux min (KN/m2)	112.6289895
qux m (KN/m2)	301.9096004
qux terjadi (KN/m2)	319.8866704
Vu (KN)	372.2521183
Vu/ϕ (KN)	**620.4201972**
Vc (KN)	**1321.278663**
kontrol Vc > Vu/ϕ	
AMAN	

Perencanaan Geser 2 Arah	
P (KN)	1642.046
Mx (KNm)	537.979
My (KNm)	369.4413
t kolom (mm)	0.6
d (mm)	619.00
m (m)	0.431
L (m)	2.7
x (m)	1.219

y (m)	1.219
f'c (MPa)	22.5
qu max (KN/m2)	501.8567292
qu min (KN/m2)	-51.36399939
qu1 (KN/m2)	413.5463092
qu2 (KN/m2)	36.94642061
qu terjadi (KN/m2)	225.2463649
Vu (KN)	1307.338686
Vu/ϕ (KN)	**2178.897811**
βc	1.0
bo (mm)	4876
Vc1 (KN)	85900.7302
Vc2 (KN)	57267.1534
Vc pakai(KN)	**57267.1534**
Kontrol Vu/Ø ≤ Vc pakai	
AMAN	

Kuat tumpuan pondasi	
luas pondasi/A2 (m2)	7.2900
luas Kolom/A3 (m2)	0.3600
(A2/A3)^0,5	4.5000
jika lebih besar dari 2, dipakai nilai 2	
ϕPn (KN)	**9639.0**
Kuat tumpuan kolom	
ϕPn (KN)	**4819.5**
Kontrol ϕPn kolom ≤ ϕPN pondasi	
AMAN	

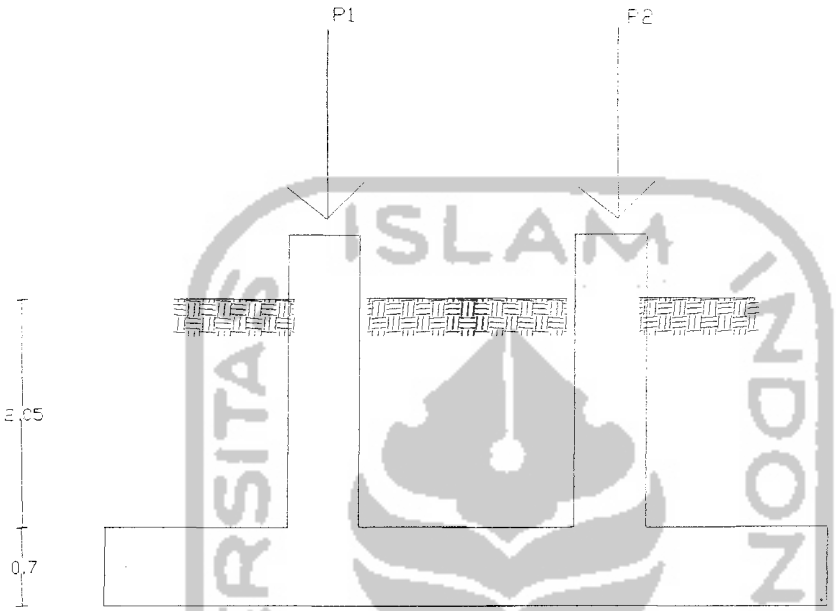
Tul Lentur sisi Panjang arah X	
qux (KN/m2)	501.8567
L (m)	2.70
h kolom (m)	0.60

l1 (m)	1.05
Mu1 (KNm)	276.6485
MuØ (KNm)	**345.8107**
tebal pelat/h (mm)	700
Pb (mm)	70
d (mm)	620.50
f'c (MPa)	22.5
fy (MPa)	400
β1	0.85
m	20.9150
Rn (MPa)	0.8982
ρ_b	0.02438
ρ_{min}	0.00350
ρ_{maks}	0.01829
ρ	0.00230
1,33.ρ	0.00306
ρpakai	0.00306
As perlu (mm2)	1898.7300
dtul.pokok (mm)	19
A1d.pokok (mm2)	283.3850
jrk tul. pokok/s (mm)	149.2498
jrk tul. pakai/s (mm)	140
tul pokok pakai	**P19 - 140**
As aktual (mm2)	2024.1786
a (mm)	42.3358
Mn (kNm)	**485.2621**
Kontrol Mn ≥ Mu/Ø	
AMAN	
dtul.susut (mm)	12
A1d.susut (mm2)	113.0400
As susut (mm2)	1400.0000
jrk tul. susut/s (mm)	80.7429
jrk tul. pakai/s (mm)	80
tul pokok pakai	**P12 - 80**

4.5.2 Pondasi Gabungan

4.5.2.1 Perencanaan Dimensi Pondasi

1. Tinjauan Terhadap Beban Tetap



Gambar 4.29 Penampang pondasi

σ tanah = 425 kN/m^2 γbtanah = 15,64 KN/m^3

F'c = 22,5 Mpa γbeton = 24 KN/m^3

Fy = 400 Mpa Asumsi tebal pondasi (h) = 700 mm

P_1 = 1041,975 kN Ukuran kolom : 600/600 mm

Mx_1 = 181,9573 kNm

My_1 = 320,2238 kNm

P_2 = 2094,646 kN

Mx_2 = 254,9642 kNm

My_2 = 331,1364 kNm

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

$= 425 - (0,7.24) - (2,05.15,64)$

$= 376,138\ kN/m^2$

$R = P_1 + P_2 = 1041,975 + 2094,646 = 3136,621\ kN$

$R\ .\ r_1 = P_2\ .\ r$

$r_1 = 2094,646\ .\ 2,275 / 3136,621 = 1,51925\ m$

$r_2 = 2,275 - 1,51925 = 0,75575\ m$

$ex = 2,275 - 0,75575 = 0,38175\ m$

$ey = 0$

$Mx_{tot} = 181,9573 + 254,9642 = 436,9215\ kNm$

$My_{tot} = 320,2238 + 331,1364 = 651,360\ kNm$

Dimensi luas pelat pondasi : (terdapat momen yang bekerja pada arah x dan y)

$$\sigma_{netto\ tanah} = \frac{R}{Bx.By} + \frac{My}{1/6.Bx^2.By} + \frac{Mx}{1/6.By^2.Bx}$$

dicoba dengan nilai Bx = 7 m dan By = 2,8 m

$$\sigma_{nettotanah} = \frac{3136,621}{6.2,8} + \frac{651,360}{1/6.7^2.2,8} + \frac{1634,335}{1/6.2,8^2.7}$$

$= 353,211\ kN/m^2 \geq 376,138\ kN/m^2$ ..... **oke !**

Jarak pusat tulangan tarik ke serat tekan beton :

$d = h - Pb - \frac{1}{2}.\varnothing_{tul.\ pokok} = 700 - 70 - \frac{1}{2}.22 = 619\ mm$

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{1634,335}{3136,621} = 0,521\ m$$

$$ey = \frac{My}{P} = \frac{651{,}360}{3136{,}621} = 0{,}2077 \text{ m}$$

Kontrol tegangan yang terjadi :

$$\sigma = \frac{P}{(L.(P-2.ex)) + (P.(L-2.ey))}$$

$$= \frac{3136{,}621}{(2{,}8.(7-2.0{,}521)) + (7.(2{,}8-2.0{,}2077))}$$

$= 102{,}029 \text{ kNm} < 1{,}5.\sigma_{netto} = 1{,}5.\ 376{,}138 = 564{,}207 \text{ kNm}$ **.....Aman.**

4.5.2 Perencananaan Geser Satu Arah

→ Ditinjau pada arah momen terbesar .

Pu = 2421,114 kN

Mux = 265,7201 kNm

Muy = 349,0082 kNm

Arah X

• Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{Mx}{1/6.L^2.P}$$

$$= \frac{22421{,}114}{7.2{,}8} \pm \frac{265{,}720}{1/6.2{,}8^2.7}$$

$qux_{max} = 135{,}147 \text{ kN/m}^2$

$qux_{min} = 111{,}906 \text{ kN/m}^2$

$$n_1 = \frac{(Lp-r)-tk-2.d}{2} = \frac{7-2{,}275-0{,}60-2.0{,}619}{2} = 1{,}4435 \text{ m}$$

$$qu_m = \frac{(Lp-m).qux_{max} + m.qux_{min}}{Lp}$$

$$= \frac{(7-1{,}4435).135{,}147+1{,}4435.111{,}906}{7} = 130{,}354 \text{ kN/m}^2$$

$qux_{terjadi}$ $= qux_{max} + qux_m) = \frac{1}{2}. (135{,}147 + 130{,}354) = 132{,}750$ kN/m^2

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

$Vu = qu_{xterjadi}. n_1 . L = 132{,}750. 1{,}4435 . 7 = 1341{,}376$ kN

$\frac{Vu}{\phi} = \frac{1341{,}3761}{0{,}6} = 2235{,}626$ kN

- Kekuatan beton menahan geser:

$Vc = 1/6. \sqrt{f'c} . L. d = 1/6. \sqrt{22{,}5}\ 7. 0{,}619 . 10^3 = 3425{,}537$ kN

- Kontrol gaya geser :

$Vc = 3425{,}537$ kN $\geq \frac{Vu}{\phi} = 2235{,}626$ kN **.....Aman.**

Arah Y

- Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{My}{1/6.L^2.P}$$

$$= \frac{2421{,}114}{7.2{,}8} \pm \frac{349{,}0082}{1/6.7^2 2{,}8}$$

$qux_{max} = 138{,}789$ kN/m^2

$qux_{min} = 108{,}263$ kN/m^2

$$n_1 = \frac{Lp - tk - 2.d}{2} = \frac{7 - 0{,}60 - 2.0{,}619}{2} = 0{,}481 \text{ m}$$

$$qu_m = \frac{(Lp-m).qux_{max} + m.qux_{min}}{Lp}$$

$$= \frac{(2{,}8-0{,}481).138{,}789+0{,}481.108{,}263}{2{,}8} = 133{,}545 \text{ kN/m}^2$$

$qux_{terjadi}$ $= qux_{max} + qux_m) = \frac{1}{2}. (138{,}7898 + 133{,}545) = 136{,}167$ kN/m^2

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

Vu = $qu_{xterjadi}$. n_1 . L = 136,167. 0,481 . 2,8 = 183,390 kN

Vu/ϕ = $183,390/0,6$ = 305,650 kN

• Kekuatan beton menahan geser:

Vc = 1/6. $\sqrt{f'c}$. L. d = 1/6. $\sqrt{22,5}$ 2,8. 0,619 . 10^3 = 5256,551 kN

• Kontrol gaya geser :

Vc = 5256,551 kN ≥ Vu/ϕ = 305,650 kN .....**Aman.**

4.5.3 Perencananaan Geser Dua Arah

→ Ditinjau pada arah momen terbesar.

x = hk + d = 600 + 619

= 1219 mm = 1,219 m

y = tk + d = 600 + 619

= 1219 mm = 1,219 m

• Tegangan kontak yang terjadi :

$$qu = \frac{P}{A_{perlu}} \pm \frac{My}{1/6.Bx^2.By} \pm \frac{Mx}{1/6.By^2.Bx}$$

$$= \frac{2421,114}{7.2,8} \pm \frac{265,720}{1/6.2,8^2.7} \pm \frac{349,008}{1/6.7^2.2,8}$$

qu_{max} = 167,84 kN/m^2

qu_{min} = 79,212 kN/m^2

qu_T = ½ (qu_1 + qu_2) = ½ (167,84 + 79,212) = 123,526 kN/m^2

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

$Vu = qu_T . ((Pp . Lp) - (x.y))$

$= 123,526 . ((7.2,8) - (1,219 . 1,219)) = 2237,559$ kN

$Vu/\phi = 2237,599/0,6 = 3729,265$ kN

• Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{Pp}{Lp} = \frac{7}{2,8} = 2,5$

$bo = 2 . (x + y) = 2 . (1219 + 1219) = 4876$ mm

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

$= (1 + 2/2,5).(2.\sqrt{22,5}).\ 4876.\ 619.\ 10^{-3} = 51540,438$ kN

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

$= 4.\sqrt{22,5}\ .\ 4876.\ 619.\ 10^{-3} = 57267,15$ kN

• Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc_1 dan Vc_2, yaitu $Vc_2 = 57267,15$ KN

$Vc_2 = 51540,438$ KN $\geq Vu/\phi = 3729,265$ kN.....**Aman.**

4.5.4 Kuat Tumpuan Pondasi

• Kuat tumpuan Pondasi :

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

Luas pelat pondasi (A_2) = P . L = 7 . 2,8 = 19,6 m^2

Luas penampang kolom (A_1) = hk . tk = 0,60 . 0,60 = 0,3600 m^2

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{19,6}{0,3600}} = 7,3786 > 2$$ (jika lebih besar dari 2, dipakai nilai 2)

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

$= 0,7.\ (0,85.\ 22,5.\ 360000.2).10^{-3} = 9639$ kN

- Kuat tumpuan kolom :

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

$= 0,7.\ (0,85.\ 22,5.\ 360000).10^{-3} = 4819,5$ KN

- Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 9639$ kN$>\phi.Pn_{kolom} = 4819,5$ kKN .....Aman.

4.5.5 Perencanaan Tulangan Telapak Pondasi

Momen yang terjadi :

$$l = \frac{Lp - tk}{2} = \frac{7 - 0,60}{2} = 3,2 \text{ m}$$

$qu_{maks} = 167,84$ kN/m^2

$Mu = 0,5.\ qu_{maks}.\ l^2 = 0,5\ .\ 167,84\ .\ 3,2^2 = 859,341$ kNm

$$Mu/\phi = \frac{859,341}{0,8} = 1074,176 \text{ kNm}$$

- Digunakan tulangan bagi $\varnothing_{22}$ mm, sehingga luas tampang 1 tulangan pokok :

  $A_{1\varnothing} = \frac{1}{4}.\ \pi.\ D^2 = \frac{1}{4}.\ \pi.\ 25^2 = 490,625$ mm^2

- Tebal pelat pondasi : h = 700 mm, selimut beton (Pb) = 70 mm

  $d = h - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 700 - 70 - 0,5.\ 25 = 617,5$ mm

  $$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22,5} = 20,915$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d} = \frac{1074{,}176.10^6}{1000.619^2} = 0{,}7253 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}00350$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0244$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0244 = 0{,}0183$$

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

$$= \frac{1}{20{,}915}.\left(1-\sqrt{1-\frac{2.20{,}915.0{,}7253}{400}}\right) = 0{,}00766 < \rho_{maks} = 0{,}0183$$

$$< \rho_{min} = 0{,}00350$$

.....sehingga dipakai : $\rho_{aktual} = 0{,}00766$

$$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00766.\ 1000\ .\ 617{,}5 = 4730{,}05 \text{ mm}^2$$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{490{,}625.1000}{4730{,}05} = 103{,}725 \text{ mm}$$

$$s \le 2.h = 2\ .700 = 1400 \text{ mm}$$

$$s \le 250 \text{ mm}$$

→ Dipakai Tulangan Pokok : D_{25} – 100 mm

$$As_{aktual} = \frac{A_{1\theta}.1000}{s} = \frac{490{,}625.1000}{100} = 4906{,}25 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{aktual}.fy}{0{,}85.f'c.b} = \frac{4906{,}25.400}{0{,}85.22{,}5.1000} = 102{,}614 \text{ mm}$$

$$Mn = As_{aktual}.fy.\ (d - a/2)$$

$$= 4906{,}25\ .\ 400\ (617{,}5 - 102{,}614/2)$$

$$= 1111{,}1564 \text{ kNm} \geq Mu/\phi = 1074{,}176 \text{ kNm} \text{ .....Aman.}$$

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 700 = 1400 \text{ mm}^2$

• Digunakan tulangan bagi ∅12 mm, sehingga luas tampang 1 tulangan susut :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 12^2 = 113{,}04 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113{,}04.1000}{1400} = 80{,}7428 \text{ mm} \approx 80 \text{ mm}$$

→ **Dipakai Tulangan Susut : P_{12} – 80 mm**

Perencanaan Pondasi	
σ tanah (KN/m2)	425
f'c (MPa)	22.5
fy (MPa)	400
γb beton (KN/m3)	24
γb tanah (KN/m3)	15.64
P1 (KN)	1041.975
MX1 (KNm)	181.9573
MY1 (KNm)	320.2238
P2 (KN)	2094.646
MX2 (KNm)	254.9642
MY2 (KNm)	331.1364
R (KN)	3136.621
r	2.275
r1	1.519252613
r2	0.755747387
ex	0.381752613
ey	0
Mxtotal	436.9215
Mytotal	651.3602
h kolom (mm)	2.05
t kolom (mm)	0.6
tebal pelat (h) (mm)	0.7
σ netto tanah (KN/m2)	**376.138**
Dicoba nilai Bx (m)	7
Dicoba nilai By (m)	2.8
σ kontak	**367.1977379**
Kontrol tegangan	**AMAN**
Pb (mm)	70
θ tul.pokok (mm)	22
d (mm)	619.00

Tinjauan Beban Sementara	
R (KN)	3136.621
Mx (KNm)	436.9215
My (KNm)	651.3602
ex (m)	0.381752613
ey (m)	0
Mxtotal (KNm)	1634.334763
Mytotal (KNm)	651.3602
Bx	7

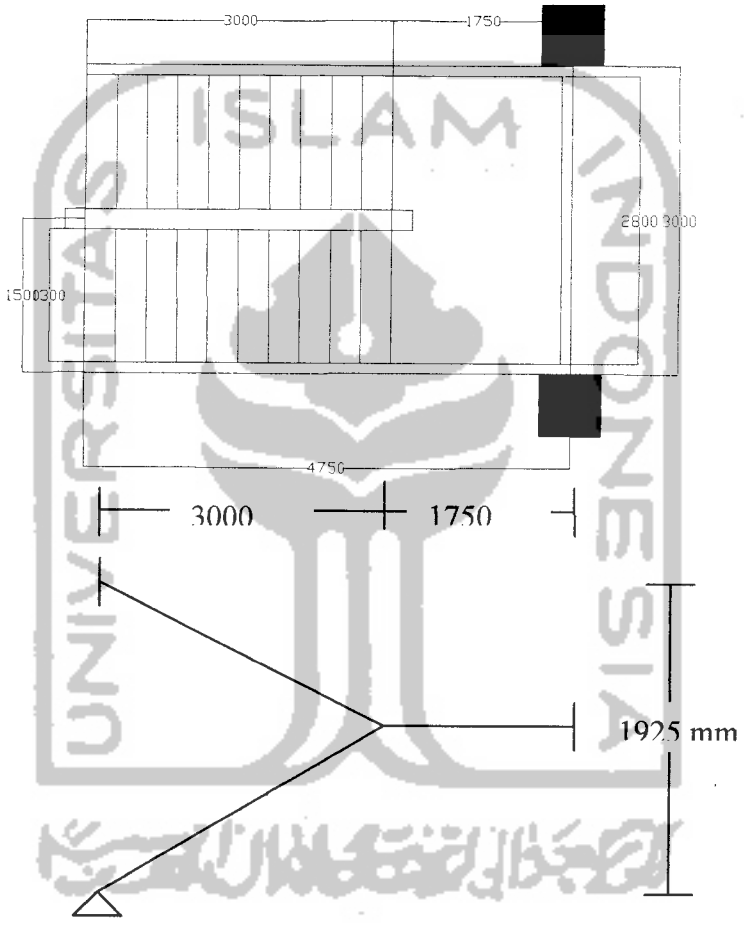
By	2.8
σ netto tanah (KN/m2)	376.138
1,5.σnetto tanah(KN/m2)	**564.207**
σ kontak (KN/m2)	**92.64741**
Kontrol	**AMAN**

Perencanaan Geser 1 Arah	
P	2421.114
Mx	265.7201
My	349.0082
t kolom (mm)	0.6
d (mm)	619.00
m (m)	1.4435
ex (m)	0.521049487
ey (m)	0.207663023
Bxeff	7
Byeff	2.8
f'c (MPa)	22.5
Perencanaan Geser Arah X	
qux max (KN/m2)	135.146637
qux min (KN/m2)	111.905812
qux m (KN/m2)	130.3540469
qux terjadi (KN/m2)	132.750342
Vu (KN)	1341.37583
Vu/ϕ (KN)	**2235.626384**
Vc (KN)	**3425.537275**
Vc > Vu/ϕ	**AMAN**
Perencanaan Geser 1 Arah (y)	
qux max (KN/m2)	161.6830977
qux min (KN/m2)	85.36935131
qux m (KN/m2)	157.233425
qux terjadi (KN/m2)	159.4582613
Vu (KN)	644.4984006
Vu/ϕ (KN)	**1074.164001**
Vc (KN)	**5256.55111**
Vc > Vu/ϕ	**AMAN**

Perencanaan Geser 2 Arah	
P (KN)	2421.114

4.8 PERENCANAAN TANGGA

Perencanaan tangga meliputi perencanaan optrede dan antrede, pembebanan tangga dan bordes, penulangan pelat tangga dan bordes, dan penulangan balok bordes.



Gambar 4.30 Detail tangga

4.8.1 Spesifikasi Struktur

1. Tinggi antar lantai (h) = 3,85 m = 385 cm
2. Sudut kemiringan tangga = tg α = 192,5/300 = 0,642

$$\alpha = 32{,}7^0$$

3. Tinggi optrade rencana diambil 19 cm

   Jumlah optrade = 385/18 = 21,3 dipakai 22 buah

   Tinggi optrade pakai = 385/22 = 17,5 cm

4. Jumlah antrade = 22 – 2 = 20 buah

   Diambil panjang antrade = 30 cm

5. Panjang tangga (Lo)

   Lo = (Panjang antrade x jml antrade/2) + LB

   = (30 x 20/2) + 175 = 475 cm

   Lebar bersih tangga = 150 cm

6. Tebal pelat diambil 15 cm

7. Beban sandaran tangga :

   Tinggi sandaran = 1m

   Tebal sandaran = 0,12 m

   Berat sandaran total = (0,12. 1. 24.2)/1,5 = 3,29 kN/m^2

4.6.2 Pembebanan

4.6.2.1 Pembebanan bordes

- Beban mati

- Berat sendiri pelat	= 0,15.24	= 3,60 kN/m^2
- Berat spesi	= 3. 0,24	= 0,72 kN/m^2
- Berat keramik	= 1.0,20	= 0,20 kN/m^2
- Sandaran		= 3,29 kN/m^2 +
	Q_D	= 7,81 kN/m^2

- Beban hidup

Q_L = 300 Kg/m^2 = 3 kN/m^2

Qu = 1,2.Q_D + 1,6.Q_L = 1,2.7,81 + 1,6.3 = 14,17 KN/m^2

qu = 14,17.1,75 = 24,80 kN/m

4.6.2.2 Pembebanan Tangga

- Beban mati

- Berat sendiri tangga = $\left(\frac{0,15}{\cos 32,7^0} + \frac{0,175}{2}\right)$.24 = 6,38 kN/m^2

- Spesi = 3. 0,24 = 0,72 kN/m^2

- Lantai keramik = 1.0,20 = 0,20 kN/m^2

- Sandaran = 3,29 kN/m^2 +

Q_D = 10,59 kN/m^2

- Beban hidup

Q_L = 300 Kg/m^2 = 3 kN/m^2

Qu = 1,2 Q_D + 1,6 Q_L = 1,2.10,59 + 1,6.3 = 17,51kN/m^2

qu = 17,51.1,75 = 30,64 kN/m

4.6.3 Penulangan Tangga

4.6.3.1 Perhitungan pelat bordes

Mu maks = 26,18 kNm

$Mu/\phi = \frac{26,18}{0,8}$ = 32,725 kNm

Digunakan tulangan ∅13 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing}$ = ¼. π. D^2 = ¼. π.13^2 = 132,73 mm^2

tebal pelat tangga = 150 mm, selimut beton (pb) = 20 mm

$d = h - Pb - 0{,}5.\ \varnothing_{tul.\ pokok} = 150 - 20 - 0{,}5.\ 13 = 123{,}5$ mm

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.22{,}5} = 20{,}915$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d} = \frac{32{,}725.10^6}{1000.123{,}5^2} = 2{,}146 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}0035$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0244$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0244 = 0{,}0183$$

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

$$= \frac{1}{20{,}915}.\left(1-\sqrt{1-\frac{2.20{,}915.2{,}146}{400}}\right) = 0{,}00571 < \rho_{maks} = 0{,}0183$$

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

sehingga dipakai : $\rho_{perlu} = 0{,}00571$

$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00571\ .\ 1000\ .123{,}5 = 705{,}185\ m^2$

Jarak antar tulangan : $s \le \frac{A_{\emptyset 1}.b}{As_{perlu}} = \frac{132{,}73.1000}{705{,}185} = 188{,}22$ mm

$s \le 2.h = 2.150 = 300$ mm

$s \le 250$ mm

→ **Dipakai Tulangan Pokok : D13 – 180 mm**

$$As_{aktual} = \frac{A_{1\theta}.1000}{s} = \frac{132,73.1000}{180} = 737,39 \text{ mm}^2$$

Kontrol Kapasitas Lentur Pelat tangga :

$$a = \frac{As_{aktual}.fy}{0,85.f.c.b} = \frac{737,39.400}{0,85.22,5.1000} = 15,423 \text{ mm}$$

$$Mn = As_{aktual}.fy.\ (d - a/2)$$

$$= 737,39 .400\ (123,5 - 15,423/2)$$

$$= 34,15 \text{ kNm} \geq Mu/\phi = 32,725 \text{ kNm} \ldots\ldots\ldots\ldots\ldots\ldots\textbf{Ok.}$$

Tulangan bagi pelat bordes

As bagi = 0,002. b. h = 0,002. 1000. 150 = 300 mm^2

Digunakan ∅ 8 dengan A_{1tul} = 50,265 mm^2

Jarak tulangan = s = $\frac{A_{1tul}.1000}{Asbagi} = \frac{50,265.1000}{300} = 167,55$ mm

Dipakai **P8-150**

4.6.3.2 Perhitungan pelat tangga

Mu maks = 33,69 kNm

$Mu/\phi = \frac{33,69}{0,8} = 42,11$ kNm

Digunakan tulangan bagi ∅16 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 13^2 = 132,73 \text{ mm}^2$

tebal pelat tangga = 150/cos 32,7 = 180 mm, selimut beton (pb) = 20 mm

$d = h - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 180 - 20 - 0,5.\ 13 = 153,5$ mm

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.22{,}5} = 20{,}915$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d} = \frac{42{,}11.10^6}{1000.153{,}5^2} = 1{,}787 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}0035$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}0244$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0244 = 0{,}0183$$

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

$$= \frac{1}{20{,}915}\left(1-\sqrt{1-\frac{2.20{,}915.1{,}787}{400}}\right) = 0{,}0047 \quad < \rho_{maks} = 0{,}0183$$

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

maka ρperlu = 0,0047

$$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}0047\ .\ 1000\ .\ 153{,}5 = 721{,}45 \text{ mm}^2$$

Jarak antar tulangan : $$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{132{,}73.1000}{721{,}45} = 183{,}98 \text{ mm}$$

$$s \le 2.h = 2.180 = 360 \text{ mm}$$

$$s \le 250 \text{ mm}$$

→ Dipakai Tulangan Pokok : D13 – 180 mm

$$As_{aktual} = \frac{A_{1\theta}.1000}{s} = \frac{132{,}73.1000}{180} = 737{,}9 \text{ mm}^2$$

Kontrol Kapasitas Lentur Pelat tangga :

$$a = \frac{As_{aktual} \cdot fy}{0{,}85 . f.c.b} = \frac{737{,}9.400}{0{,}85.22{,}5.1000} = 15{,}43 \text{ mm}$$

$$Mn = As_{aktual}.fy.\ (d - a/2)$$

$$= 737{,}9\ .400\ (152 - 15{,}43/2)$$

$$= 42{,}587 \text{ kNm} \geq Mu/\phi = 42{,}113 \text{ kNm} \ldots\ldots\ldots\ldots\ldots\ldots\textbf{Ok.}$$

Tulangan bagi pelat tangga

As bagi = 0,002. b. h = 0,002. 1000. 180 = 360 mm^2

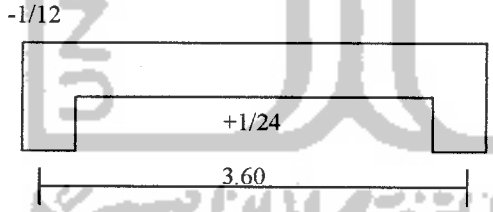
Digunakan ∅ 8 dengan A_{1tul} = 50,265 mm^2

Jarak tulangan = s = $\frac{A_{1tul}.1000}{Asbagi} = \frac{50{,}265.1000}{360} = 139{,}625$ mm

Dipakai **P8-130**

4.6.3.3 Perhitungan Tulangan Lentur Balok Bordes

Dicoba balok ukuran 30/50



Pembebanan

- berat akibat pelat bordes = = 33,21 kN/m
- berat sendiri balok bordes = 1,2.0,3.0,5.24 = 4,32 kN/m +

Q_D = 37,55 kN/m

Momen :

$M^+ = 1/24.qu.L^2 = 1/24\ .\ 37{,}55\ .\ 3{,}6^2 = 20{,}28$ kNm

$M^{-} = 1/12.qu.L^2 = 1/12 \cdot 37,55 \cdot 3,6^2 = 40,55$ kNm

1. Tulangan Lapangan

Mu = 20,28 kN.m

$$Mn = \frac{Mu}{\varphi} = \frac{20,28}{0,8} = 25,35 \text{ kN.m}$$

d = 500 – 40 – 8 - 0,5.13 = 445,5 mm

$$\rho b = \frac{0,85.f'c.\beta}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{400}.\left(\frac{600}{600+400}\right) = 0,0244$$

ρ max = 0,75. ρb = 0,75 . 0,0244 = 0,0183

rasio tulangan rencana = ρ = 0,5.ρ max = 0,5 . 0,0183 = 0,00915

$$\rho \text{ min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22,5} = 20,915$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0,00915.400.\left(1-\frac{1}{2}.0,00915.20,915\right) = 3,31 \text{ Mpa}$$

$$b.d_{perlu}^{\ 2} = \frac{Mn}{Rn}$$

$$d_{perlu} = \sqrt{\frac{Mn}{Rn.b}} = \sqrt{\frac{25,35.10^6}{3,31.300}} = 159,78 \text{ mm}$$

d_{ada} = 445,5 mm

$d_{ada} > d_{perlu}$ maka dipakai tulangan sebelah

$$Rn_{baru} = \frac{Mu/\phi.}{b.d^2} = \frac{25,35.10^6}{300.445,5^2} = 0,426 \text{ MPa}$$

$$\rho_{baru} = \frac{0,426}{3,31} \times 0,00915 = 0,00118$$

$1,33\rho_{baru} = 0,00157$

$1,33\rho_{baru} < 0,002 < \rho_{min}$

$\rho_{pakai} = 0,002$

$As_{perlu} = \rho_{baru} \,.\, b \,.\, d = 0,002.300 .445,5 = 279,3 \text{ mm}^2$

digunakan $\phi_{13} \Rightarrow As = 132,732 \text{ mm}^2$

dipakai $3\phi_{13}$ As tul $= 398,196 \text{ mm}^2 > As_{perlu} = 279,3 \text{ mm}^2$

Tulangan Tumpuan

Mu $= 40,55$ kN.m

$$Mn = \frac{Mu}{\varphi} = \frac{40,55}{0,8} = 50,69 \text{ kN.m}$$

d $= 500 - 40 - 8 - 0,5.13 = 445,5$ mm

$$\rho b = \frac{0,85.f'c.\beta}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{400}.\left(\frac{600}{600+400}\right) = 0,0244$$

ρ max $= 0,75.\ \rho b = 0,75 \,.\, 0,0244 = 0,0183$

rasio tulangan rencana $= \rho = 0,5.\rho$ max $= 0,5 \,.\, 0,0183 = 0,00915$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22,5} = 20,915$$

$$Rn = \rho.fy.\left(1 - \frac{1}{2}.\rho.m\right) = 0,00915.400.\left(1 - \frac{1}{2}.0,00915.20,915\right) = 3,31 \text{ Mpa}$$

$$b.d_{perlu}{}^2 = \frac{Mn}{Rn}$$

$$d = \sqrt{\frac{Mn}{Rn.b}} = \sqrt{\frac{50{,}69.10^6}{3{,}31.300}} = 225{,}94 \text{ mm}$$

$d_{pakai} = 445{,}5$ mm

$d_{ada} > d_{perlu}$ maka dipakai tulangan sebelah

$$Rn_{baru} = \frac{Mu/\phi.}{b.d^2} = \frac{50{,}69.10^6}{300.445{,}5^2} = 0{,}851 \text{ MPa}$$

$$\rho_{baru} = \frac{0{,}851}{3{,}31} \times 0{,}00915 = 0{,}00235$$

$1{,}33\rho_{baru} = 0{,}00313$

$0{,}002 < 1{,}33\rho_{baru} < \rho_{min}$

$\rho_{pakai} = 1{,}33\rho_{baru} = 0{,}00313$

$As_{perlu} = \rho_{baru} . b . d = 0{,}00313.300 .445{,}5 = 418{,}32 \text{ mm}^2$

digunakan $\phi_{13} \Rightarrow As = 132{,}732 \text{ mm}^2$

dipakai $4\phi_{13}$ As tul $= 530{,}93 \text{ mm}^2 > As_{perlu} = 418{,}32 \text{ mm}^2$

2. Perencanaan Geser dan Torsi

- kontrol torsi

$Tu_1 = 26{,}18$ kNm

$$Tu_2 = \left(\frac{1}{9}.\sqrt{f'c}.\left(\Sigma x^2.y.\frac{1}{3}\right)\right) = \left(\frac{1}{9}.\sqrt{22{,}5}.\left(300^2.500.\frac{1}{3}\right)\right) = 7{,}906 \text{ kNm}$$

Digunakal Tu yang terkecil Tu = 7,906

$$Ct = \frac{bw.d}{\Sigma x^2 y} = \frac{300.445{,}5}{300^2 500} = 0{,}00297 \text{ mm}^{-1}$$

$$Tc = \frac{\left(\frac{1}{15}.\sqrt{f'c}.\Sigma x^2 y\right)}{\sqrt{1+\left(\frac{0{,}4.Vu}{Ct.Tu}\right)^2}}$$

$$= \frac{\left(1/15.\sqrt{22,5}.300^2 500\right)}{\sqrt{1+\left(\frac{0,4.64,314.10^3}{0,00297.26,18.10^6}\right)^2}} = 13,51 \text{ kNm}$$

Kekuatan baja menahan torsi

$$Ts = Tu/\phi - Tc$$

Karena $Tu/\phi = 7,906/0,6 = 13,177 < Tc = 13,51$

Maka tidak diperlikan tulangan torsi

3. Perencanaan Geser

Tulangan geser balok

Vu = 0,5.qu.L = 0,5.37,55.3,6 = 67,59 kN

bt = b - 2Pb - 2Ø sengkang ; Ø sengkang =10 mm

= 300 – 2.40 – 2.10

= 200 mm

(bt + d) = 200 + 445,5 = 645,5 mm

$$Vu_{pakai} = Vu\ (bt + d) = \left(\frac{3,6 - 0,6455}{3,6}\right) . 64,314 = 52,782 \text{ kN}$$

Gaya geser yang mampu ditahan oleh beton :

- $Vc = 1/6 . \sqrt{fc'} . b.d = 1/6. \sqrt{22,5} .300.445,5 = 105,66$ kN

- $\emptyset Vc = 0,6.105,66 = 63,396$ kN

- $½.\emptyset Vc = ½. 63,396 = 31,70$ kN

$Vu > 1/2 \emptyset Vc \longrightarrow$ perlu tulangan geser

$Vs\ min = 1/3. b.d = 1/3.300.445,5.10^{-3} = 44,55$ kN

Jika $½.\emptyset Vc < Vu \leq \emptyset Vc$

Gaya geser yang harus ditahan oleh sengkang

$$s \leq \frac{Av.fy.d}{Vs} = \frac{2.50,265.240.445,5}{44,55.10^3} = 241,272 \text{ mm}$$

$\leq ½.d = 160$ mm

≤ 600 mm

Dipakai **P8-160**

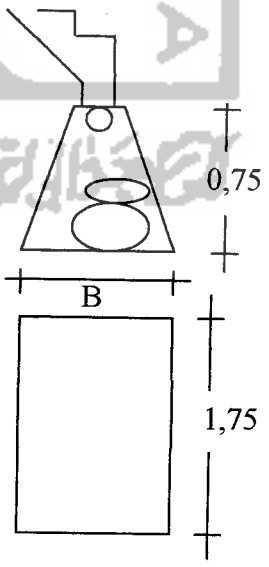
4.8.4 Perencanaan Pondasi Tangga

σ tanah = 120 kN/m^2

γ batu = 22 KN/m^3

Balok diatas pondasi 20/40

tinjauan untuk lebar tangga 1,75 m



Gambar 4.31 Pondasi tangga

Pembebanan:

- akibat tekanan tangga = 124,26 kN

- berat balok diatas pondasi = 1,2 (0,2 . 0,4 . 1,75 . 24) = 3,36 kN

Pu = 124,26 + 3,36 = 127,62 kN

Tegangan ijin tanah pakai:

$\sigma = \sigma$ tanah – σpondasi

$= 120 - 1.22 = 98$ KN/m^2

Diketahui pada kondisi kritis = $\sigma = \frac{Pu}{A}$

$A = \frac{Pu}{\sigma} = \frac{127,62}{98} = 1,302$ m^2

$B = \frac{A}{L} = \frac{1,302}{1,75} = 0,74$ m $\longrightarrow$ 100 cm

Kontrol tegangan tanah:

$\sigma = \frac{Pu}{A} = \frac{127,62}{1.1,75} =$ kN/m$^2 < \sigma$ tanah = 98 **...OK..**