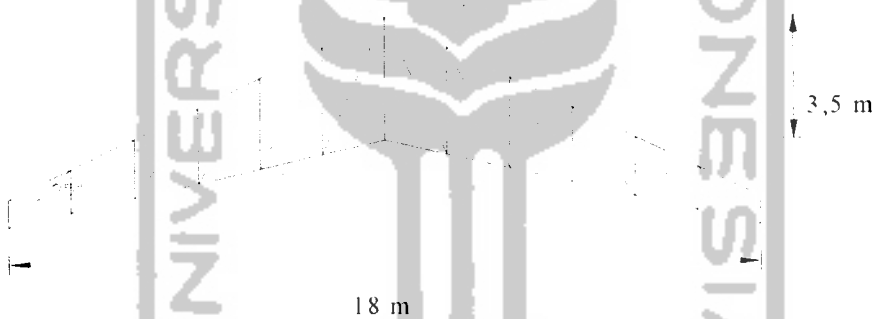
BAB IV

PERENCANAAN

4.1 Perencanaan Atap

Pada perencanaan ulang Gedung Fakultas Hukum Universitas Gadjah Mada (UGM) Yogyakarta ini digunakan rangka atap (kuda-kuda) yang direncanakan dengan menggunakan profil baja.



Gambar 4.1 Rencana rangka kuda-kuda 1

4.1.1 Perencanaan gording pada kuda-kuda 1

Data-data pada perencanaan gording meliputi :

1. Jarak antar kuda-kuda = 3,0 m
2. Jarak antar gording = 1,730 m
3. Dipakai baja

   Mutu baja profil (fy) = 2400 kg/cm^2

   Kuat tarik (Fu) = 3700 kg/cm^2

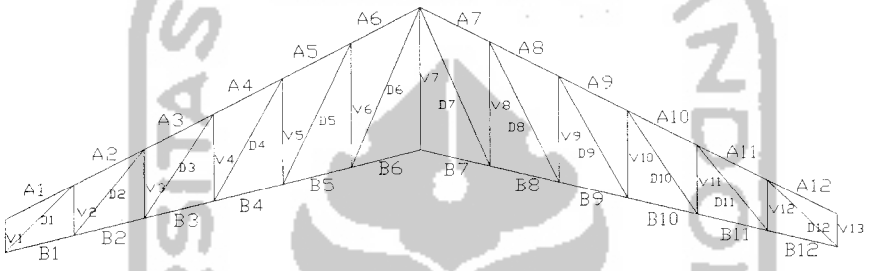
4. Mutu baut non full drat dari AISC A_{325x}

   Fu = 8250 kg/cm^2 dan fy = 2050 kg/cm^2

5. Direncanakan terhadap bangunan di darat

6. Panjang batang

   Berikut ini contoh perencanaan kuda-kuda 1 dengan bentang 18 m.



Gambar 4.2 .Rencana rangka batang

Tabel 4.1. Panjang rangka batang kuda-kuda

Batang	Panjang (m)	Batang	Panjang (m)
A1 – A12	1,730	V1 = V13	0,800
B1 – B12	1,555	V2 = V12	1,250
D1 = D12	2,237	V3 = V11	1,700
D2 = D12	2,589	V4 = V10	2,150
D3 = D10	2,967	V5 = V9	2,600
D4 = D9	3,363	V6 = V8	3,050
D5 = D8	3,771	V7	3,500
D6 = D7	4,188		

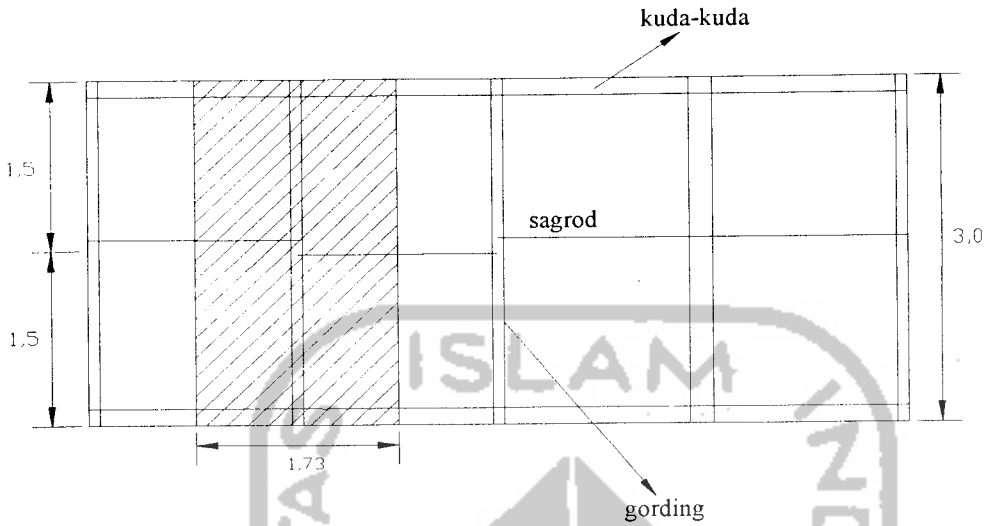
7. Pembebanan gording

   Dipakai profil 100 x 50 x 20 x 2,3 (Light Lip Channel)

   A = 5,172 cm^2 w = 4,06 kg/m

   Ix = 80,7 cm^4 Sx = 16,1 cm^3

   Iy = 19 cm^4 Sy = 6,06 cm^3



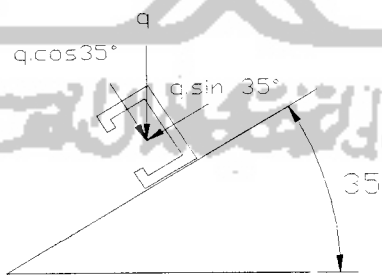
Gambar 4.3. Pembebanan gording

a. Beban tetap

Beban penutup atap (genteng) = 50 x 1,730 = 86,500 kg/m

Beban gording (diambil dari profil) = 4,060 kg/m +

q = 90,560 kg/m



Gambar 4.4. Arah pembebanan gording

$q\perp = q \cos \alpha = 90{,}560 . \cos 35° = 74{,}182$ kg/m

$q // = q \sin \alpha = 90{,}560 . \sin 35° = 51{,}943$ kg/m

b. Beban hidup

Beban hidup yang dipakai adalah beban pekerja (orang)

P = 100 kg

Maka P⊥ = 100 . cos 35° = 81,915 kg

P// = 100 . sin 35° = 57,358 kg

c. Beban angin

Pada daerah daratan w (dalam PPIUG 1983) = 25 kg/m^2

1) Angin tekan (wt) $\alpha < 65°$, diketahui sudut α = 35°

C_1 = 0,02 α - 0,4 = 0,02 . 35 – 0,4 = 0,3

wt = C_1 . w . jarak gording = 0,3 . 25 . 1,730 = 12,975 kg/m

2) Angin hisap (wh), C_2 = -0,4

wh = C_2 . w . jarak gording = -0,4 . 25 . 1,730 = -17,300 kg/m

8. Perhitungan momen

a. Akibat beban tetap

q ⊥ = Mmaks = 1/8 . q ⊥ . b^2 + ¼ . P⊥ . b

= 1/8 . 74,182.3^2 + ¼ . 81,915 . 3

= 144,891 kgm = 14489,1 kgcm

q // = Mmaks = 1/32 . q // . b^2 + ¼ . P// . b

= 1/32. 51,943.3^2 + ¼ . 57,358 . 3

= 36,118 kgm = 3611,8 kgcm

b. Akibat beban angin

Mmaks = 1/8 . w. b^2 = 1/8 . 12,975 . 3^2 = 14,597 kgm

= 1459,7 kgcm

9. Dimensi gording

a. Kontrol tegangan

$$fbx = \frac{M_{\perp}.\max}{Sx} = \frac{(14489{,}1 + 1459{,}7)}{16{,}1} = 990{,}609 \text{ kg/cm}^2$$

$$fby = \frac{M_{//}.\max}{Sy} = \frac{3611{,}8}{6{,}06} = 596{,}007 \text{ kg/cm}^2$$

$$\frac{fbx}{0{,}66\,fy} + \frac{fby}{0{,}75\,fy} \leq 1{,}0$$

$$\frac{990{,}609}{0{,}66.2400} + \frac{596{,}007}{0{,}75.2400} = 0{,}956 \leq 1{,}0 \Rightarrow \text{Ok!}$$

b. Kontrol lendutan

$$\delta \perp = \frac{5}{384}\frac{q_{\perp}.L^4}{EIx} + \frac{1}{48}\frac{P_{\perp}.L^3}{EIx} \leq \frac{L}{360}$$

$$= \frac{5}{384}.\frac{0{,}74584.300^4}{2{,}1.10^6.80{,}7} + \frac{1}{48}.\frac{0{,}81915.300^3}{2{,}1.10^6.80{,}7}$$

$$= 0{,}467 \text{ cm} \leq \frac{300}{360} = 0{,}833 \text{ cm} \Rightarrow \text{Ok!}$$

$$\delta // = \frac{5}{384}\frac{q_{//}.(L/(a+1))^4}{EIy} + \frac{1}{48}\frac{P_{//}.(L/(a+1))^3}{EIy} \leq \frac{L}{360}$$

$$= \frac{5}{384}.\frac{0{,}51943.(300/1+1))^4}{2{,}1.10^6.19} + \frac{1}{48}.\frac{0{,}57358.(300/1+1))^3}{2{,}1.10^6.19}$$

$$= 0{,}087 \text{ cm} \leq \frac{300}{360} = 0{,}833 \text{ cm} \Rightarrow \text{Ok!}$$

$$\delta = \sqrt{\delta \perp^2 + \delta //^2} = \sqrt{0{,}467^2 + 0{,}087^2}$$

$$= 0{,}475 \text{ cm} \leq \frac{L}{360} = 0{,}833 \text{ cm} \Rightarrow \text{Ok!}$$

Jadi profil Light Lip Channel 100 x 50 x 20 x 2,3 dapat digunakan.

4.1.2 Perencanaan sagrod dan tierod

1. Beban sagrod

   berat penutup atap x (½ L / cos α) = 50 .(½ .18 / cos 35°)

   = 549,349 kg/m

   jumlah gording satu sisi miring x berat gording = 7 . 4,06

   = 28,420 kg/m

   P = 549,349 + 28,420 = 577,769 kg/m

   P// = P.sin α . Ss + beban hidup = 577,769 .sin 35°.1,5 + 100

   = 597,092 kg

2. Dimensi sagrod

   Asagrod $= \frac{P_{//}}{0.33 \cdot Fu} = \frac{1}{4} \cdot \pi \cdot D^2 sagrod$

   $= \frac{597{,}092}{0.33 \cdot 3700} = \frac{1}{4} \cdot \pi \cdot D^2$

   Dsagrod $= \sqrt{\frac{(597{,}092 \cdot 4)}{(0{,}33 \cdot 3700 \cdot \pi)}}$ = 0,789 cm ≈ 8 mm

   Dsagrod = 8 + 3 = 11 mm , pakai ∅ 12 mm

3. Dimensi tierod

   Beban tierod; T = P//.cos 35° = 597,092. cos 35° = 489,109 kg

   Atierod $= \frac{T}{0{,}33Fu} = \frac{1}{4} \cdot \pi \cdot D^2 tierod$

   Dtierod $= \sqrt{\frac{(489{,}109 \cdot 4)}{(0{,}33 \cdot 3700 \cdot \pi)}}$ = 0,714 cm ≈ 8 mm

   Dtierod = 8 + 3 = 11 mm , pakai ∅ 12 mm

4.1.3 Pembebanan kuda-kuda

Pembebanan kuda kuda meliputi :

Gambar 4.5. Pembebanan kuda – kuda

1. Beban tetap

   berat gording = 4,06 kg/m

   berat eternit + penggantung = 18 kg/m^2

   berat penutup atap (genteng) = 50 kg/m^2

Tabel 4.2 Profil terpakai dan berat profil terpakai kuda-kuda 1 :

Batang	Profil (mm)	Berat Profil (kg/m)	Panjang (m)	Berat (kg)
Batang Atas	2L 60x60x6	2 x 5,42 = 10,84	20,760	225,038
Batang Bawah	2L 60x60x6	2 x 5,42 = 10,84	18,660	202,274
Batang Vertikal	2L 60x60x6	2 x 5,42 = 10,84	26,600	288,994
Batang Diagonal	2L 60x60x6	2 x 5,42 = 10,84	38,230	414,413
		Wtotal	=	1130,719

Berat kuda-kuda yang digunakan :

Berat total kuda-kuda = 1130,719 kg

Berat baut dan plat sambung = (20 % x berat total kuda-kuda)

= 226,144 kg

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

= 1130,719 + 226,144 = 1356,863 kg

Panjang rangka kuda-kuda, L = 18 m

$$\frac{\Sigma}{L} = \frac{1356{,}863}{18} = 75{,}381 \text{ kg/m}$$

Beban masing-masing joint :

a. P1 = P13

Berat gording = 4,06 x 3,0 = 12,180 kg

Berat penutup atap = 50 x 3,0 x ½ x 1,730 = 129,750 kg +

= 141,930 kg

b. P2 = P3 = P4 = P5 = P6 = P8 = P9 = P10 = P11 = P12

Berat gording = 4,06 x 3,0 = 12,180 kg

Berat penutup atap = 50 x 3,0 x 1,730 = 259,500 kg +

= 271,680 kg

c. P7

Berat gording = 2 x 4,06 x 3,0 = 24,360 kg

Berat penutup atap = 50 x 3,0 x 1,730 = 259,500 kg +

= 283,860 kg

d. P'1 = P'13

Berat eternit + plafond = 18 x 3,0 x ½ x 1,555 = 41,985 kg

Berat profil kuda-kuda = 75,381 x ½ x 1,555 = 58,609 kg +

= 100,594 kg

e. P'2= P'3=P'4= P'5= P'6= P'7= P'8= P'9= P'10= P'11= P'12

Berat eternit + plafond = 18 x 3,0 x 1,555 = 83,970 kg

Berat profil kuda-kuda = 75,381 x 1,555 = 117,218 kg +

= 201,188 kg

2. Beban hidup

Beban hidup dipakai P = 100 kg (beban pekerja/beban orang)

3. Beban angin

Muatan angin di darat (PPUIUG 1983) untuk $\alpha < 65° = 25$ kg/m^2

a. Koefisien angin :

Angin tekan (wt)

C1 = 0,02 .α - 0,4 = 0,02 .35 - 0,4 = 0,3

Angin hisap (wh)

C2 = -0,4

b. Beban yang bekerja

wt = C1 x w = 0,3 x 25 = 7,5 kg/m^2

wh = C2 x w = -0,4 x 25 = -10 kg/m^2

Angin kiri :

wt_1 = 7,5 x ½ x 1,730 x 3,0 = 19,463 kg

$wt_2 = wt_3 = wt_4 = wt_5 = wt_6$ = 7,5x1,730x3,0 = 38,925 kg

wt_7 = 7,5 x ½ x 1.730 x 3,0 = 19,463 kg

wh_1 = -10 x ½ x 1,730 x 3,0 = -25,950 kg

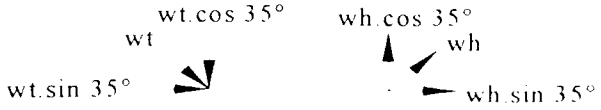
$wh_2 = wh_3 = wh_4 = wh_5 = wh_6$ = -10x1,730x3,0 = -51,900 kg

wh_7 = -10 x ½ x 1,730 x 3,0 = -25,950 kg

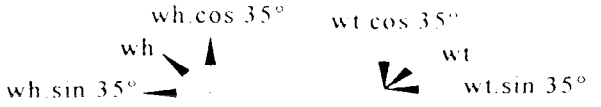
Angin kanan :

Besar angin kanan sama dengan besar angin kiri.

Keterangan :



Angin kiri



Angin kanan

4.1.4 Perencanaan dimensi batang

Contoh perhitungan dimensi batang kuda – kuda 1:

1. Batang tarik

   Gaya tarik maksimal (P) = 1811,616 kg (batang D6)

   Panjang batang (L) = 4,188 m = 418,8 cm

   fy = 2400 kg/cm^2 , Fu = 3700 kg/cm^2

   E = 2,1x10^6 kg/cm^2

   K = 1 (sendi-sendi)

   a. Syarat batang tarik

   $$\frac{L}{r} \leq 240 \text{ s/d } 300 \qquad r\,\min = \frac{KL}{240} = \frac{418{,}8}{240} = 1{,}745 \text{ cm}$$

   b. Luas tampang perlu :

   $$Ag1 = \frac{T}{0{,}6\,fy} = \frac{1811{,}616}{0{,}6.2400} = 1{,}258 \text{ cm}^2$$

   $$Ag2 = \frac{T}{0{,}5Fu.\mu} + \left(\frac{1''}{8} + \phi baut\right).tp.n$$

   $$= \frac{1811{,}616}{0{,}5.3700.0{,}75} + \left(\frac{1''}{8} + \frac{1''}{2}\right).1.2 = 4{,}481 \text{ cm}^2$$

dicoba profil 2L 60x60x6 :

$A = 6{,}91\ cm^2$ $r = 1{,}82\ cm$ $W = 5{,}42\ kg/m$

Abruto = 2 x 6,91 = 13,82 cm^2

$$\text{Alubang} = \left(\frac{1''}{8} + \phi baut\right).tp.n = \left(\frac{1''}{8} + \frac{1''}{2}\right).1.2 = 3{,}175\ cm^2$$

Anetto = Abruto Alubang 13,82 3,175

$= 10{,}645\ cm^2$

Aeffektif = 0,75.Anetto = 0,75. 10,645 = 7,984 cm^2

c. Kontrol tegangan :

$$\frac{T}{Aprofil} \le 0{,}6.fy \rightarrow \frac{1811{,}616}{13{,}82} \le 0{,}6.\ 2400$$

$131{,}087\ kg/cm^2 \le 1440\ kg/cm^2$ .....Ok!

$$\frac{T}{Aeffektif} \le 0{,}5.Fu \rightarrow \frac{1811{,}616}{7{,}984} \le 0{,}5.3700$$

$226{,}906\ kg/cm^2 \le 1850\ kg/cm^2$ .....Ok!

Jadi profil 2L 60x60x6 dapat digunakan.

2. Batang tekan

Gaya tekan maksimal (P) = - 3907,326 kg (batang A4)

Panjang batang (L) = 1,730 m = 173 cm

fy = 2400 kg/cm^2, Fu = 3700 kg/cm^2

E = $2{,}1 \times 10^6\ kg/cm^2$

K = 1 (sendi-sendi)

a. Syarat batang tekan

$\frac{L}{r} \le 200$ $r\ min = \frac{KL}{200} = \frac{173}{200} = 0{,}865\ cm$

dicoba profil 2L 60x60X6

A = 6,91 cm^2 Atotal = 13,82 cm^2 W = 5,42 kg/m

r = 1,82 cm ≥ rmin = 0,865 cm ➔ dipakai r = 1,82 cm

Ix = Iy = 22,8 cm^4 ix = iy = 1,82 cm e = 1,69

x = e + (0,5.tp) = 1,69 + (0,5.1) = 2,19 cm

Ix gab = 2 . Ix = 2 .22,8 = 45,6 cm^4

Iy gab = Ix gab + 2A.x^2 = 45,6 + 2. 6,91. $2,19^2$ = 111,882 cm^4

$$\text{ix gab} = \sqrt{\frac{Ixgab}{2A}} = \sqrt{\frac{45,6}{2.6,91}} = 1,816 \text{ cm}$$

$$\text{iy gab} = \sqrt{\frac{Iygab}{2A}} = \sqrt{\frac{111,882}{2.6,91}} = 2,845 \text{ cm}$$

dipakai r = 1,816 cm

b. Syarat :

$$\frac{KL}{r} \le Cc = \frac{6440}{\sqrt{fy}}$$

$$\frac{KL}{r} = \frac{1.173}{1,816} = 95,240 \le Cc = \frac{6440}{\sqrt{2400}} = 131,456$$

maka digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3.(KL/r)}{8.Cc} - \frac{1.(KL/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3.(95,240)}{8.131,456} - \frac{1.(95,240)^3}{8.131,456^3}$$

$$= 1,891$$

$$Fa = \frac{fy}{Fs}\left(1 - 0,5\left(\frac{KL}{Cc}\frac{r}{}\right)^2\right) = \frac{2400}{1,891}\left(1 - 0,5\left(\frac{95,240}{131,456}\right)^2\right)$$

$$= 936,168 \text{ kg/cm}^2$$

c. Kontrol Kapasitas

P Fa.Atotal > Pterjadi

936,168. 13,82 > 3907,326

12937,838 kg > 3634,906 kg ..... Ok!

Jadi profil 2L 60x60x6 dapat digunakan.

4.1.5 Perencanaan pelat kuda-kuda

P = 3579,266 kg (diambil dari reaksi maksimum yang terjadi)

fc' = 25 Mpa

= 250 kg/cm^2

$$\text{Aperlu} = \frac{P}{0{,}33.f'c} = \frac{3579{,}266}{0{,}33.250} = 43{,}385 \text{ cm}^2$$

Diambil ukuran pelat = 15 x 20 = 300 cm^2 > Aperlu

$$Q = \frac{P}{BxL} = \frac{3579{,}266}{15x20} = 11{,}931 \text{ cm}^2$$

$$x = \frac{20-(6+1+6)}{2} = 3{,}5 \text{ cm}$$

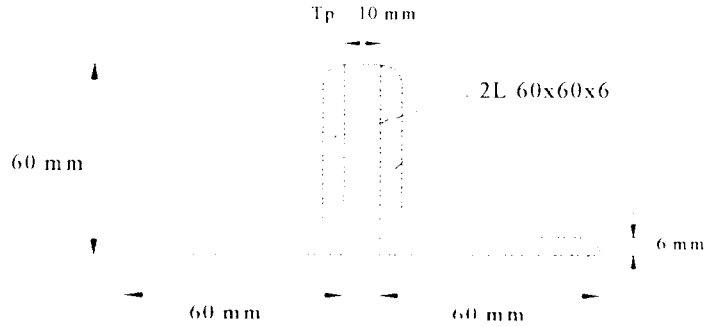
$M = {}^1/_2 . q . x^2 = {}^1/_2 . 11{,}931 . 3{,}5^2 = 73{,}077$ kg.cm

Syarat :

$$0{,}6.fy = \frac{M}{\frac{1}{6}.1.tp^2} \qquad tp = \sqrt{\frac{M}{0{,}6.1/6.fy}} = \sqrt{\frac{73{,}077}{0{,}6.1/6.2400}} = 0{,}552 \text{ cm}$$

digunakan pelat dengan tebal (tp) = 1 cm

Dimensi pelat yang digunakan 15 x 20 x 1 cm.



Gambar 4.6. Penampang profil terpakai kuda-kuda 1

4.1.6 Perencanaan dukungan arah lateral

Diketahui :

Lb = jarak antar gording = 1,730 m

Lc = jarak antar kuda-kuda = 3 m

$L = \sqrt{Lb^2 + Lc^2} = \sqrt{1{,}730^2 + 3^2}$

$= 3{,}463$ m

Syarat : $L/r \leq 300$ sehingga :

$$r\ min \geq \frac{L}{300} = \frac{346{,}3}{300} = 1{,}154\ cm$$

Keterangan :

1. $L \leq 3$ m → dipakai baja tulangan Ø12 mm
2. $L \geq 5$ m → dipakai baja tulangan Ø19 mm
3. $3\ m < L = 3{,}463\ m \leq 5$ m → dipakai baja tulangan Ø16 mm

Sehingga digunakan baja tulangan Ø16 mm

4.1.7 Perencanaan sambungan

Perhitungan sambungan dilakukan pada setengah bentang pada tiap joint, diambil tebal pelat sambungan (tp) = 1 cm, fy = 2400 kg/cm^2 , Fu = 3700 kg/cm^2

Mutu baut non full drat dari AISC A325, fy = 2050 kg/cm^2 dan Fu = 8250 kg/cm^2

Tegangan tumpu yang terjadi pada pelat :

Fa tumpu = 1,2 . Fu pelat = 1,2 . 3700 = 4440 kg/cm^2

Diameter baut yang dipakai ½ "= 1,27 cm

Sehingga didapat kekuatan 1 baut untuk menahan gaya adalah :

P tumpu = tp . Ø baut . Fa tumpu = 1 . 1,27 . 4440

= 5638,800 kg

P geser = ¼. π . D^2 . 0,22 . Fu baut . 2 = ¼ . 3,14 .(1,27)^2 .0,22.8250 .2

= 4598,370 kg

dipakai P yang kecil = 4598,370 kg maka, Jumlah baut = $\frac{P}{4598,370} buah$

Jarak penggunaan baut ½ " :

- Jarak baut ke tepi minimal 1,2.D

diambil 2D = 2.1,27 = 2,540 cm ~ pakai 3 cm

- Jarak antar baut 2D s/d 7D

diambil 4D = 4.1,27 = 5,080 cm ~ pakai 5 cm

Hitungan jumlah baut dilakukan untuk setengah bentang karena bentang simetris.

1. Joint 2

a. Batang (1) = - 201,018 kg

$$n = \frac{201,018}{4598,370} = 0,044 \sim \text{dipakai baut 2 buah}$$

b. Batang (7) = 42,449 kg

$$n \quad \frac{42,449}{4598,370} = 0,009 \sim \text{dipakai baut 2 buah}$$

2. Joint 1

a. Batang (1) = - 201,018 kg

$$n = \frac{201,018}{4598,370} = 0,044 \sim \text{dipakai baut 2 buah}$$

b. Batang (2) = - 3852,759 kg

$$n = \frac{3852,759}{4598,370} = 0,838 \sim \text{dipakai baut 2 buah}$$

c. Batang (30) = - 1460,913 kg

$$n = \frac{1460,913}{4598,370} = 0,318 \sim \text{dipakai baut 2 buah}$$

Untuk sambungan pada joint berikutnya, dengan perhitungan yang sama didapat jumlah baut yang sama pula yaitu 2 buah, karena gaya-gaya batang yang terjadi kurang dari kekuatan 1 baut untuk menahan gaya.

Tabel 4.3. Jumlah baut terpakai pada setengah bentang kuda-kuda 1

Join	Batang	Gaya batang (kg)	Jumlah baut
2	1	- 201,018	2
	7	42,449	2
7	7	42,449	2
	2	- 3852,759	2
	42	1061,657	2
	8	- 3023,377	2
8	8	- 3023,377	2
	5	- 1273,680	2
	66	290,064	2
	9	- 3872,258	2
9	9	- 3872,258	2
	6	- 69,286	2
	46	- 441,732	2
	10	- 3907,326	2
10	10	- 3907,326	2
	31	881,949	2
	48	- 887,904	2
	11	- 3449,484	2
11	11	- 3449,484	2
	32	1398,961	2
	50	- 1278,585	2
	12	- 2797,704	2
3	12	- 2797,704	2
	33	1811,616	2
	52	- 958,591	2
	34	1811,616	2
	13	- 2797,704	2

Join	Batang	Gaya batang (kg)	Jumlah baut
1	1	- 201,018	2
	2	- 3852,759	2
	30	- 1460,913	2
	30	- 1460,913	2
26	42	1061,657	2
	5	- 1273,680	2
	29	- 681,112	2
25	29	- 681,112	2
	66	290,064	2
	6	- 69,286	2
	28	- 646,127	2
24	28	- 646,127	2
	46	- 441,732	2
	31	881,949	2
	27	- 1057,331	2
23	27	- 1057,331	2
	48	- 887,904	2
	32	1398,961	2
	26	- 1639,128	2
22	26	- 1639,128	2
	50	- 1278,585	2
	33	1811,616	2
	25	- 2314,637	2
6	25	- 2314,637	2
	52	958,591	2
	24	- 2314,637	2

4.2 Perencanaan Pelat Lantai dan Pelat Talang

4.2.1 Mutu bahan

1. Kuat tekan beton yang disyaratkan, f'c = 25 Mpa.
2. Modulus elastisitas beton

   $Ec = 4700.\sqrt{f'c} = 4700.\sqrt{25} = 23500$ Mpa.

   $= 2{,}396.10^7$ KN/m^2.
3. Baja tulangan dengan fy :
   a. Baja tulangan Ø ≤ 12 mm digunakan tulangan polos dengan tegangan leleh (fy) = 240 Mpa.
   b. Baja tulangan Ø ≥ 12 mm digunakan tulangan ulir (*deform*) dengan fy = 400 Mpa.

4.2.2 Pembebanan pelat lantai

1. Beban mati pelat lantai (qD).

a. berat pelat	=	0,12 x 24	=	2,88 KN/m^2.
b. berat pasir (5 cm)	=	0,05 x 16	=	0.80 KN/m^2.
c. berat spesi (3 cm)	=	0,03 x 22	=	0,66 KN/m^2.
d. berat keramik	=	0,01 x 20	=	0,20 KN/m^2. +
qD total	=		=	4,54 KN/m^2.

2. Beban hidup pelat lantai (qL).

   Gedung ini berfungsi sebagai ruang kuliah dan kantor, sehingga beban hidup (qL) diambil sebesar 250 Kg/cm^2 atau 2,5 KN/m^2. (PPIUG, 1983 tabel 3.1 halaman 17).

3. Kombinasi pembebanan (SK SNI T-15-1991-03, pasal 3.2.2).

   qU = 1,2.qD + 1,6.qL = 1,2.4,54 + 1,6.2,5 = 9,448 KNm.

4. Tinggi manfaat pelat.

   a. Pakai tulangan pokok Ø 10 mm.

   b. Penutup beton (Pb) = 20 mm.

   c. Tinggi Manfaat tulangan pelat :

      1) Arah lapangan x : dx = h – Pb – ½.Øtul.x

         = 120 – 20 – ½.10 = 95 mm.

      2) Arah lapangan y : dy = h – Pb – Øtul.y – ½.Øtul.y

         = 120 – 20 – 10 – ½.10 = 85 mm.

      3) Arah tumpuan x dan y = 95 mm.

4.2.3 Perencanaan pelat lantai

1. Perencanaan pelat lantai tipe I (ukuran 4,5m x 3,0 m).

   a. Menentukan tebal pelat minimum (h).

      Pada SK SNI T-15-1991-03 pasal 3.2.5 butir 3.3 memberikan pendekatan empiris mengenai batasan defleksi dilakukan dengan tebal pelat minimum sebagai berikut :

$$\beta = \frac{4,5}{3,0} = 1,5$$

      Perkiraan tebal pelat (h) :

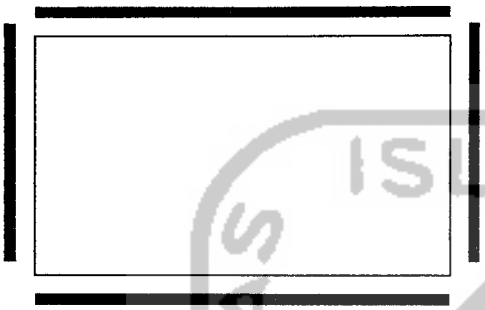
$$h_{min} = \frac{Ln.(0,8 + fy/1500)}{36 + 9.\beta}$$

$$= \frac{3000.(0,8 + 240/1500)}{36 + 9.1,5} = 58,1818 \text{ mm.}$$

$h \geq h_{min}$, digunakan h = 12 cm = 120 mm.

b. Distribusi Momen.

Pelat dianggap terjepit elastis pada keempat sisinya.



Dihitung sebagai pelat dua (2) arah.

$$\frac{Ly}{Lx} = \frac{4,5}{3,0} = 1,5 \text{ m.}$$

Dari tabel 13.3.1 dan 13.3.2 PBBI 1971 didapat faktor :

Ly / Lx	1,5
Mlx = -Mtx	56
Mly	37
-Mty	37

$Mu = 0,001 \cdot qU \cdot Lx^2 \cdot x$

$Mlx = -Mtx = 0,001 \cdot 9,448 \cdot 3^2 \cdot 56 = 4,7618$ KNm.

$Mly = -Mty = 0,001 \cdot 9,448 \cdot 3^2 \cdot 37 = 3,1462$ KNm.

c. Perencanaan tulangan arah Mlx dan Mtx.

Mu = 4,7618 KNm.

$$Mn = \frac{Mu}{\Phi}$$

$$= \frac{4,7618}{0,8} = 5,9523 \text{ KNm.}$$

h = 120 mm.

d = h – Pb – ½.Øtul.x = 120 – 20 – ½.10 = 95 mm.

$$\rho b = \frac{0,85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right)$$

$$= \frac{0,85.25}{240}.0,85.\left(\frac{600}{600+240}\right) = 0,054.$$

ρ max = 0,75 . ρb = 0,75 . 0,054 = 0,0405.

$$\rho \text{ min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{5,9523.10^6}{1000.95^2} = 0.6595 \text{ Mpa.}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11,2941}.\left[1-\sqrt{1-\frac{2.11,2941.0,6595}{240}}\right]$$

= 0,0028 ≤ ρ min = 0,0058.

≤ ρ max = 0,0405

1,33 . ρ ada = 1,33 . 0,0028 = 0,0037.

ρ ada < ρ min.

} ρ pakai = 1,33 . ρ ada = 0,0037.

1,33 . ρ ada < ρ min.

As pakai = ρ pakai . b . d = 0,0037 . 1000 . 95 = 351,50 mm^2.

Pakai tulangan P.10 → A1.Ø = ¼ . π . D^2 = ¼ . π.10^2

= 78,54 mm^2.

Jarak tulangan (S) = $\frac{A1.\phi.1000}{Aspakai} = \frac{78,54.1000}{351,5}$

= 223,4424 mm < 2.h = 2 .120 = 240 mm

< 250 mm.

Jadi pakai jarak (S) = 220 mm → **P 10 - 220 mm.**

As ada = $\frac{78,54.1000}{220}$ = 357 mm^2.

- Kontrol Mn

a = $\frac{Asada.fy}{0,85.f'c.b} = \frac{357.240}{0,85.25.1000}$ = 4,032 mm.

Mn = As ada . fy . (d – a/2).

= 357 . 240 . (95 – 4,032/2).

= 7,9669 KNm > 1,33 . $\frac{Mu}{\phi}$ = 1,33 . 5,9523 = 7,9166 KNm.

d. Perencanaan tulangan arah Mly.

Mu = 3,1462 KNm.

Mn = $\frac{Mu}{\Phi} = \frac{3,1462}{0,8}$ = 3,9328 KNm.

h = 120 mm.

d = h – Pb – ½.Øtul.y – Øtul.x = 120 – 20 – ½.10 - 10 = 85 mm.

ρb = $\frac{0,85.f'c}{fy}.\beta1.\left(\frac{600}{600+fy}\right) = \frac{0,85.25}{240}.0,85.\left(\frac{600}{600+240}\right)$

= 0,054.

ρ max = 0,75 . ρb = 0,75 . 0,043 = 0,0405.

$$\rho \text{ min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{3,9328.10^6}{1000.85^2} = 0.5443 \text{ Mpa.}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11,2941}.\left[1-\sqrt{1-\frac{2.11,2941.0,5443}{240}}\right]$$

$$= 0,0023 \leq \rho \text{ min} = 0,0058.$$

$$\leq \rho \text{ max} = 0,0405$$

1,33 . ρ ada = 1,33 . 0,0023 = 0,0031.

ρ ada $< \rho$ min.

$\}$ ρ pakai = 1,33 . ρ ada = 0,0031.

1,33 . ρ ada $< \rho$ min.

As pakai = ρ pakai . b . d = 0,0031 . 1000 . 85 = 263,5 mm^2.

Pakai tulangan P.10 $\rightarrow$ A1.Ø = ¼ . π . D^2 = ¼ . $\pi.10^2$

= 78,54 mm^2.

$$\text{Jarak tulangan (S)} = \frac{A1.\phi.1000}{Aspakai} = \frac{78,54.1000}{263,5}$$

= 298,0645 mm > 2 . h = 2 .120 = 240 mm

> 250 mm.

Jadi pakai jarak (S) = 240 mm → **P 10 - 240 mm.**

$$\text{As ada} = \frac{78{,}54.1000}{240} = 327{,}25 \text{ mm}^2.$$

- Kontrol Mn

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{327{,}250.240}{0{,}85.25.1000} = 3{,}696 \text{ mm}.$$

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

$$= 327{,}25 \cdot 240 \cdot (85 - 3{,}696/2).$$

$$= 6{,}5308 \text{ KNm} > 1{,}33.\frac{Mu}{\phi} = 1{,}33.3{,}9328 = 5{,}2306 \text{ KNm}$$

e. Perencanaan tulangan arah Mty.

$$Mu = 3{,}1462 \text{ KNm}.$$

$$Mn = \frac{Mu}{\Phi} = \frac{3{,}1462}{0{,}8} = 3{,}9328 \text{ KNm}.$$

$$h = 120 \text{ mm}.$$

$$d = h - Pb - \tfrac{1}{2}.\text{Øtul.y} = 120 - 20 - \tfrac{1}{2}.10 = 95 \text{ mm}.$$

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta 1.\left(\frac{600}{600 + fy}\right)$$

$$= \frac{0{,}85.25}{240}.0{,}85.\left(\frac{600}{600 + 240}\right) = 0{,}054.$$

$$\rho \max = 0{,}75 \cdot \rho b = 0{,}75 \cdot 0{,}043 = 0{,}0405.$$

$$\rho \min = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{3{,}9328.10^6}{1000.95^2} = 0{,}4358 \text{ Mpa.}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.25} = 11{,}2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11{,}2941}.\left[1-\sqrt{1-\frac{2.11{,}2941.0{,}4358}{240}}\right]$$

$$= 0{,}0018 \leq \rho \text{ min} = 0{,}0058.$$

$$\leq \rho \text{ max} = 0{,}0405$$

$1{,}33 \,.\, \rho$ ada $= 1{,}33 \,.\, 0{,}0018 = 0{,}0024.$

ρ ada $< \rho$ min.

$\}$ ρ pakai $= 1{,}33 \,.\, \rho$ ada $= 0{,}0024.$

$1{,}33 \,.\, \rho$ ada $< \rho$ min.

As pakai $= \rho$ pakai $.\ b\ .\ d = 0{,}0024\ .\ 1000\ .\ 95 = 228 \text{ mm}^2.$

Pakai tulangan P.10 $\rightarrow$ A1.Ø $= \frac{1}{4} \,.\, \pi \,.\, D^2 = \frac{1}{4} \,.\, \pi \,.\, 10^2$

$= 78{,}54 \text{ mm}^2.$

$$\text{Jarak tulangan (S)} = \frac{A1.\phi.1000}{Aspakai} = \frac{78{,}54.1000}{228}$$

$= 344{,}4737 \text{ mm} > 2 \,.\, h = 2 \,.120 = 240 \text{ mm}$

$> 250 \text{ mm.}$

Jadi pakai jarak (S) = 240 mm $\rightarrow$ **P 10 - 240 mm.**

$$\text{As ada} = \frac{78{,}54.1000}{240} = 327{,}25 \text{ mm}^2.$$

▪ Kontrol Mn

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{327{,}250.240}{0{,}85.25.1000} = 3{,}696 \text{ mm.}$$

Mn = As ada . fy . (d – a/2).

= 327,25 . 240 . (95 – 3,696/2).

$$= 7{,}3162 \text{ KNm} > 1{,}33 . \frac{Mu}{\phi} = 1{,}33 . 3{,}9328 = 5{,}2306 \text{ KNm.}$$

f. Perencanaan tulangan bagi pelat lantai.

As bagi = 0,002 . b . h

$= 0{,}002 . 1000 . 120 = 240 \text{ mm}^2$.

Dipakai P 8 → A1.Ø = 50,27 mm^2.

$$\text{Jarak tulangan} = \frac{A1.\phi.1000}{Asbagi} = \frac{50{,}27.1000}{240} = 209{,}4583 \text{ mm.}$$

Jadi dipakai tulangan bagi : **P 8 – 200 mm**.

$$As = \frac{50{,}27.1000}{200} = 251{,}35 \text{ mm}^2 > 240 \text{ mm}^2.$$

4.2.4 Pembebanan pelat talang

1. Beban mati pelat talang (qD).

   a. berat pelat = 0,10 x 24 = 2,40 KN/m^2.

   b. lapisan kedap air (2 cm) = 0,02 x 22 = 0,44 KN/m^2. +

   qD total = 2,84 KN/m^2.

2. Beban hidup pelat talang (qL).

   Pada pelat talang beban hidup berupa pekerja atau air hujan (qL) sebesar 100 kg/cm^2 atau 1 KN/m^2 (PPIUG, 1983 tabel 3.1, halaman 17).

3. Kombinasi pembebanan (SK SNI T-15-1991-03, pasal 3.2.2).

   qU = 1,2 . qD + 1,6 . qL = 1,2 . 2,84 + 1,6 . 1,0 = 5,008 KNm.

4. Tinggi manfaat pelat.

   a. pakai tulangan pokok Ø 8 mm.

   b. penutup beton (Pb) = 20 mm.

   c. Tinggi Manfaat tulangan pelat :

      1) Arah lapangan x : dx = h – Pb – ½.Øtul.x

         = 100 – 20 – ½.8 = 76 mm.

      2) Arah lapangan y : dy = h – Pb – Øtul.y – ½.Øtul.y

         = 100 – 20 – 8 – ½.8 = 68 mm.

      3) Arah tumpuan x dan y = 76 mm.

4.2.5 Perencanaan pelat talang

1. Perencanaan pelat talang tipe I (ukuran 3,0m x 1,8m).

   a. Menentukan tebal pelat minimum (h).

      Pada SK SNI T-15-1991-03 pasal 3.2.5 butir 3.3 memberikan pendekatan empiris mengenai batasan defleksi dilakukan dengan tebal pelat minimum sebagai berikut :

      $$\beta = \frac{3,0}{1,8} = 1,667$$

      Perkiraan tebal pelat (h) :

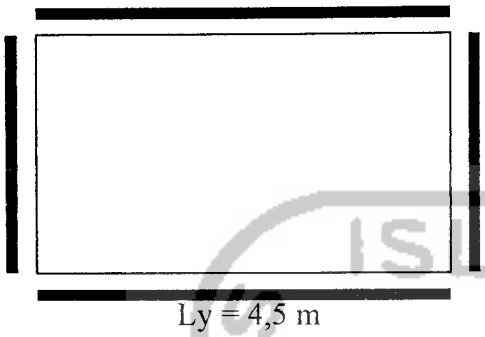
      $$h_{min} = \frac{Ln.(0,8 + fy/1500)}{36 + 9.\beta}$$

      $$= \frac{1800.(0,8 + 240/1500)}{36 + 9.1,6667} = 33,882 \text{ mm.}$$

$h \geq h_{min}$, digunakan h = 10 cm = 100 mm.

b. Distribusi momen.

Pelat dianggap terjepit elastis pada keempat sisinya.



Dihitung sebagai pelat dua (2) arah.

$$\frac{Ly}{Lx} = \frac{3,0}{1,8} = 1,6667 \text{ m.}$$

Dari tabel 13.3.1 dan 13.3.2 PBBI 1971 didapat faktor :

Ly / Lx	1,6	1.6667	1.7
Mlx = -Mtx	58	58,667	59
Mly	36	36	36
-Mty	36	36	36

Mu = 0,001 . qU . Lx^2 . x

Mlx = -Mtx = 0,001 . 5,008 . $1,8^2$. 58,6667 = 0,9519 KNm.

Mly = -Mty = 0,001 . 5,008 . $1,8^2$. 36 = 0,5841 KNm.

c. Perencanaan tulangan arah Mlx dan Mtx.

Mu = 0,9519 KNm.

$$Mn = \frac{Mu}{\Phi}$$

$$= \frac{0,9519}{0,8} = 1,1899 \text{ KNm.}$$

h = 100 mm.

d = h – Pb – ½.Øtul.x = 100 – 20 – ½.8 = 76 mm.

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25}{240}.0{,}85.\left(\frac{600}{600+240}\right)$$

$$= 0{,}054.$$

ρ max = 0,75 . ρb = 0,75 . 0,043 = 0,0405.

$$\rho \text{ min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{1{,}1899.10^6}{1000.76^2} = 0{,}206 \text{ Mpa.}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.25} = 11{,}2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11{,}2941}.\left[1-\sqrt{1-\frac{2.11{,}2941.0{,}206}{240}}\right]$$

$= 0{,}0009 \leq \rho$ min $= 0{,}0058.$

$\leq \rho$ max $= 0{,}0323$

1,33 . ρ ada = 1,33 . 0,0009 = 0,0012.

ρ ada $< \rho$ min.

$\}$ ρ pakai = 1,33 . ρ ada = 0,0012.

1,33 . ρ ada $< \rho$ min.

As pakai = ρ pakai . b . d = 0,0012 . 1000 . 76 = 91,2 mm^2.

Pakai tulangan P.8 $\rightarrow$ A1.Ø = ¼ . π . D^2 = ¼ . π . 8^2

$= 50{,}27$ mm^2.

Jarak tulangan (S) = $\frac{Al.\phi.1000}{Aspakai} = \frac{50{,}27.1000}{91{,}2}$

= 551,2061 mm > 2 . h = 2 .100 = 200 mm

> 250

Jadi pakai jarak (S) = 200 mm → **P 8 - 200 mm.**

As ada = $\frac{50{,}27.1000}{200}$ = 251,35 mm^2.

- Kontrol Mn

a = $\frac{Asada.fy}{0{,}85.f'c.b} = \frac{251{,}350.240}{0{,}85.25.1000}$ = 2,8388 mm.

Mn = As ada . fy . (d – a/2).

= 251,35 . 240 . (76 – 2,8388/2).

= 4,499 KNm > 1,33 . $\frac{Mu}{\phi}$ = 1,33 . 1,1899 = 1,5826 KNm.

d. Perencanaan tulangan arah Mly.

Mu = 0,5841 KNm.

Mn = $\frac{Mu}{\Phi} = \frac{0{,}5841}{0{,}8}$ = 0,7301 KNm.

h = 100 mm.

d = h – Pb – ½.Øtul.y – Øtul.x = 100 – 20 – ½.8 - 8 = 68 mm.

$\rho b = \frac{0{,}85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right)$

$= \frac{0{,}85.20}{240}.0{,}85.\left(\frac{600}{600+240}\right)$ = 0,054.

ρ max = 0,75 . ρb = 0,75 . 0,043 = 0,0405.

$$\rho \text{ min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{0,7301.10^6}{1000.68^2} = 0,1579 \text{ Mpa.}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11,2941}.\left[1 - \sqrt{1 - \frac{2.11,2941.0,1579}{240}}\right]$$

$$= 0,0007 \leq \rho \text{ min} = 0,0058.$$

$$\leq \rho \text{ max} = 0,0323$$

$1,33 . \rho \text{ ada} = 1,33 . 0,0007 = 0,0009.$

ρ ada $< \rho$ min.

$1,33 . \rho \text{ ada} < \rho \text{ min.}$ } ρ pakai $= 1,33 . \rho$ ada $= 0,009.$

As pakai $= \rho$ pakai $. b . d = 0,0009 . 1000 . 68 = 61,2 \text{ mm}^2.$

Pakai tulangan P.8 $\rightarrow$ A1.Ø $= ¼ . \pi . D^2 = ¼ . \pi . 8^2$

$= 50,27 \text{ mm}^2.$

$$\text{Jarak tulangan(S)} = \frac{A1.\phi.1000}{Aspakai} = \frac{50,27.1000}{91,2}$$

$= 821,4052 \text{ mm} > 2 . h = 2 .100 = 200 \text{ mm}$

$> 250 \text{ mm.}$

Jadi pakai jarak (S) = 200 mm $\rightarrow$ **P 8 - 200 mm.**

$$As\ ada = \frac{50,27.1000}{200} = 251,35\ mm^2.$$

- Kontrol Mn

$$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{251,35.240}{0,85.25.1000} = 2,8388\ mm.$$

Mn = As ada . fy . (d – a/2).

= 251,35 . 240 . (68 – 2,8388/2).

$$= 4,0164\ KNm > 1,33 . \frac{Mu}{\phi} = 1,33 . 0,7301 = 0,971\ KNm$$

e. Perencanaan tulangan arah Mty.

Mu = 0,5841 KNm.

$$Mn = \frac{Mu}{\Phi} = \frac{0,5841}{0,8} = 0,7301\ KNm.$$

h = 100 mm.

d = h – Pb – ½.Øtul.y = 100 – 20 – ½.8 = 76 mm.

$$\rho b = \frac{0,85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right)$$

$$= \frac{0,85.25}{240}.0,85.\left(\frac{600}{600+240}\right) = 0,054.$$

ρ max = 0,75 . ρb = 0,75 . 0,043 = 0,0405.

$$\rho\ min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058.$$

- Menentukan luas tulangan (As).

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{0,7301.10^6}{1000.76^2} = 0,1264\ Mpa.$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.25} = 11{,}2941.$$

$$\rho \text{ ada} = \frac{1}{m}.\left[1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right].$$

$$= \frac{1}{11{,}2941}.\left[1 - \sqrt{1 - \frac{2.11{,}2941.0{,}1264}{240}}\right]$$

$$= 0{,}0005 \leq \rho \text{ min} = 0{,}0058.$$

$$\leq \rho \text{ max} = 0{,}0323$$

$1{,}33 . \rho \text{ ada} = 1{,}33 . 0{,}0005 = 0{,}0007.$

ρ ada $< \rho$ min.

ρ pakai $= 1{,}33 . \rho$ ada $= 0{,}0007.$

$1{,}33 . \rho$ ada $< \rho$ min.

As pakai $= \rho$ pakai $. b . d = 0{,}0007 . 1000 . 76 = 53{,}2 \text{ mm}^2.$

Pakai tulangan P.8 $\rightarrow$ A1.Ø $= \frac{1}{4} . \pi . D^2 = \frac{1}{4} . \pi . 8^2$

$= 50{,}27 \text{ mm}^2.$

Jarak tulangan (S) $= \frac{A1.\phi.1000}{Aspakai} = \frac{50{,}27.1000}{53{,}2}$

$= 944{,}9248 \text{ mm} > 2 . h = 2 . 100 = 200 \text{ mm}$

$> 250 \text{ mm}.$

Jadi pakai jarak (S) = 200 mm $\rightarrow$ **P 8 - 200 mm.**

As ada $= \frac{50{,}27.1000}{200} = 251{,}35 \text{ mm}^2.$

- Kontrol Mn

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{251{,}35.240}{0{,}85.25.1000} = 2{,}8388 \text{ mm}.$$

4.3. Pembebanan Balok Induk dan Anak

1. Beban pada balok lantai.

   a. Beban mati (qD) :

      1) berat pelat lantai = 0,12 x 24 = 2,88 KN/m^2.

      2) berat pasir (5cm) = 0,05 x 16 = 0,80 KN/m^2.

      3) berat spesi (3cm) = 0,03 x 22 = 0,66 KN/m^2.

      4) berat keramik = 0,01 x 20 = 0,20 KN/m^2. +

      qD total = 4,54 KN/m^2.

      = 454 Kg/m^2.

   b. Beban dinding / partisi (pasangan ½ bata).

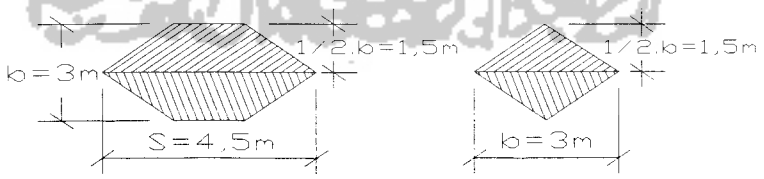
      Q din = Bj bata x Tinggi tembok

      = 2,5.(4,2-0,6) = 9 KN/m = 900 Kg/m.

   c. Beban hidup (qL) = 250 Kg/m^2.

   d. Tipe beban pada balok lantai.

      1) Tipe I



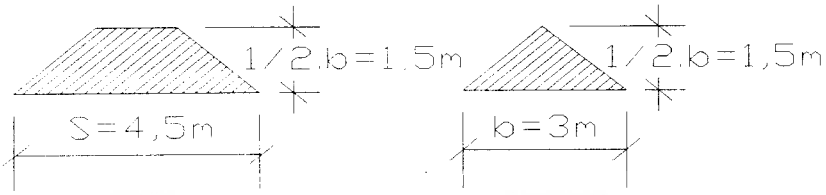
Beban mati (qD) :

q1 (D) = qD . ½ . b . 2 = 454 . ½ . 3 . 2 = 1362 Kg/m.

Beban hidup (qL) :

q1 (L) = qL . ½ . b . 2 = 250 . ½ . 3 . 2 = 750 Kg/m.

2) Tipe II.



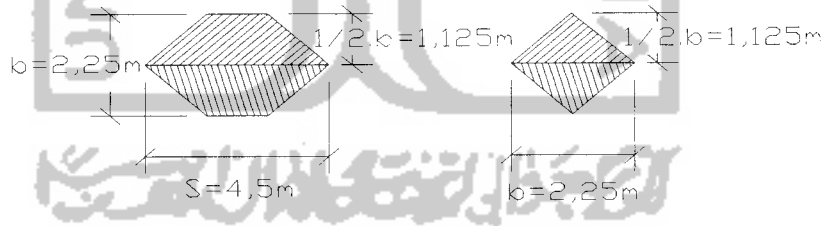
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

3) Tipe III.



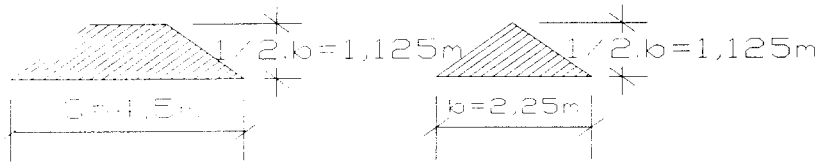
Beban mati (qD) :

q3 (D) = qD . ½ . b . 2 = 454 . ½ . 2,25 . 2 = 1021,5 Kg/m.

Beban hidup (qL) :

q3 (L) = qL . ½ . b . 2 = 250 . ½ . 2,25 2 = 562,5 Kg/m.

4) Tipe IV.



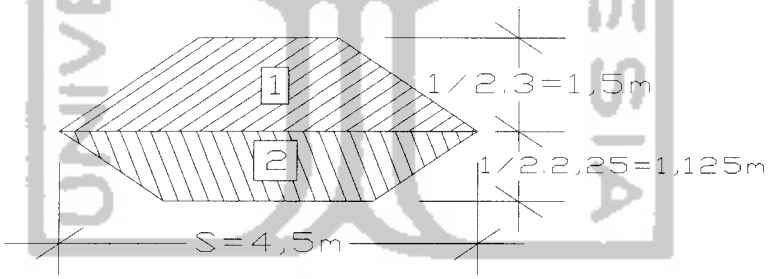
Beban mati (qD) :

q4 (D) = qD . ½ . b = 454 . ½ . 2,25 = 510, 75 Kg/m.

Beban hidup (qL) :

q4 (L) = qL . ½ . b = 250 . ½ . 2,25 = 281,25 Kg/m.

5) Tipe V.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

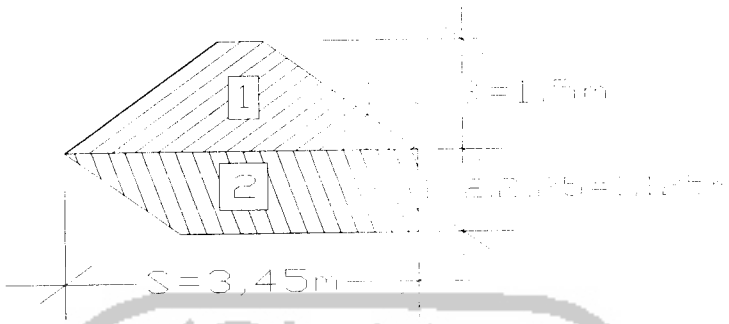
q5 (D2) = qD . ½ . b = 454 . ½ . 2,25 = 510,75 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 2,25 = 281,25 Kg/m.

6) Tipe VI.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

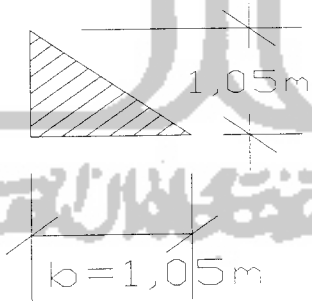
q5 (D2) = qD . ½ . b = 454 . ½ . 2,25 = 510,75 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 2,25 = 281,25 Kg/m.

7) Tipe VII.



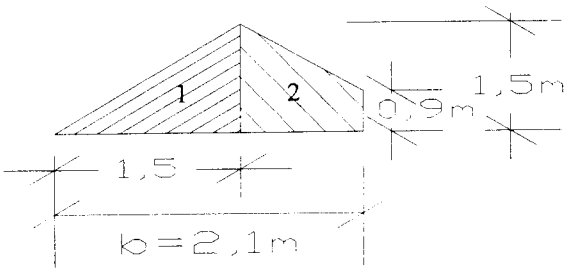
Beban mati (qD) :

q4 (D) = qD . b = 454 . 1,05 = 476,7 Kg/m.

Beban hidup (qL) :

q4 (L) = qL . b = 250 . 1,05 = 262,5 Kg/m.

8) Tipe VIII.



Beban mati (qD) :

q5 (D1) = qD . 1,5 = 454 . 1,5 = 681 Kg/m.

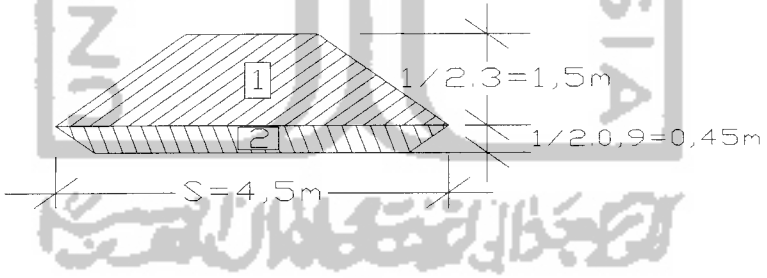
q5 (D2) = qD . 0,9 = 454 . 0,9 = 408,6 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . 1,5 = 250 . 1,5 = 375 Kg/m.

q5 (L2) = qL . 0,9 = 250 . 0,9 = 225 Kg/m.

9) Tipe IX.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

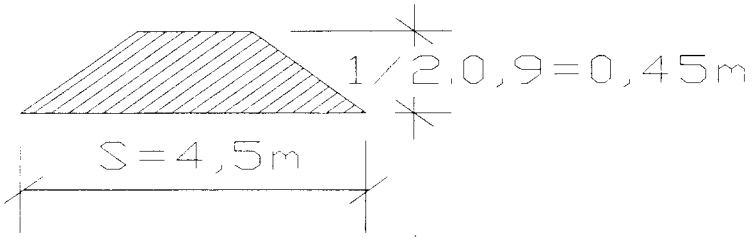
q5 (D2) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

10) Tipe X.



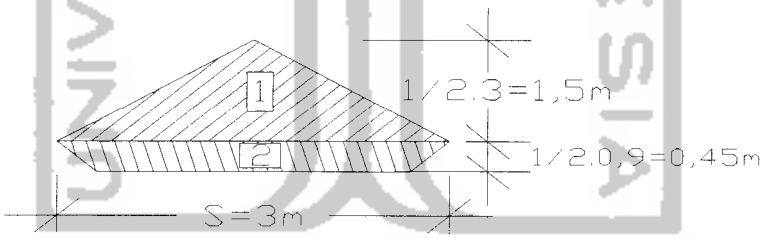
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

11) Tipe XI.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

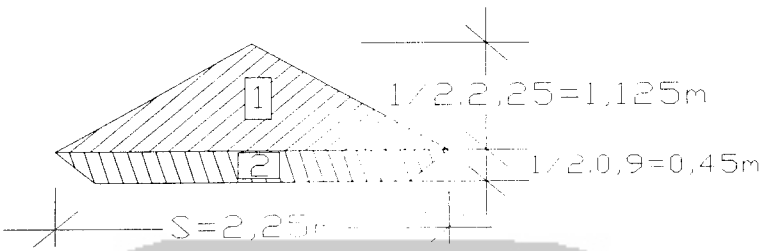
q5 (D2) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

12) Tipe XII.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 2,25 = 510,75 Kg/m.

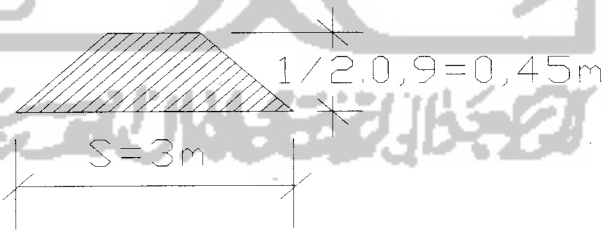
q5 (D2) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 2,25 = 281,25 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

13) Tipe XIII.



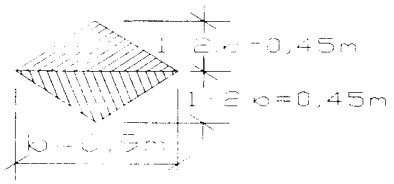
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

14) Tipe XIV.



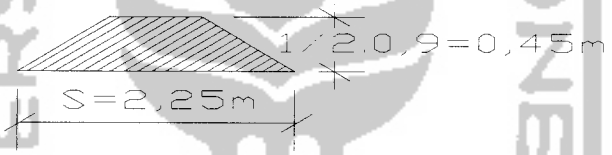
Beban mati (qD) :

q1 (D) = qD . ½ . b . 2 = 454 . ½ . 0,9 . 2 = 408,6 Kg/m.

Beban hidup (qL) :

q1 (L) = qL . ½ . b . 2 = 250 . ½ . 0,9 . 2 = 225 Kg/m.

15) Tipe XV.



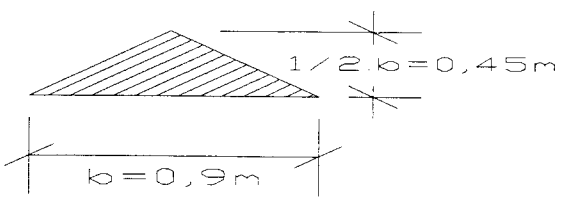
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

16) Tipe XVI.



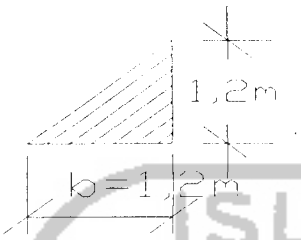
Beban mati (qD) :

q4 (D) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m.

Beban hidup (qL) :

q4 (L) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

17) Tipe XVII.



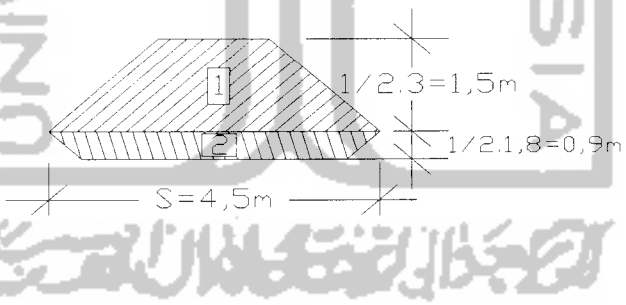
Beban mati (qD) :

q4 (D) = qD . b = 454 . 1,2 = 544,8 Kg/m.

Beban hidup (qL) :

q4 (L) = qL . b = 250 . 1,2 = 300 Kg/m.

18) Tipe XVIII.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

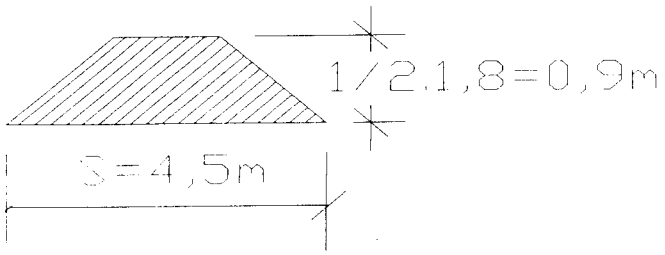
q5 (D2) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

19) Tipe XIX.



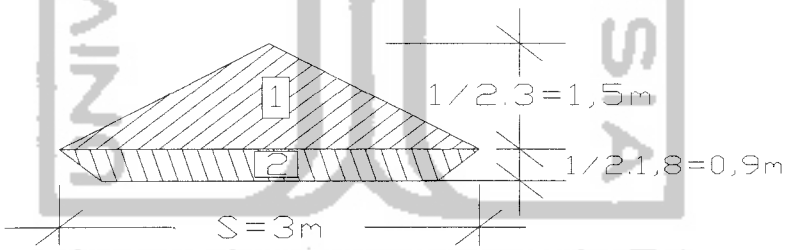
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

20) Tipe XX.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 3 = 681 Kg/m.

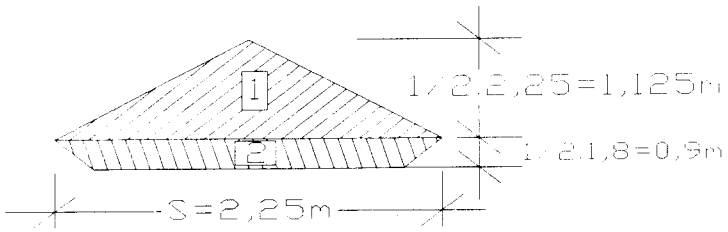
q5 (D2) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 3 = 375 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

21) Tipe XXI.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 2,25 = 510,75 Kg/m.

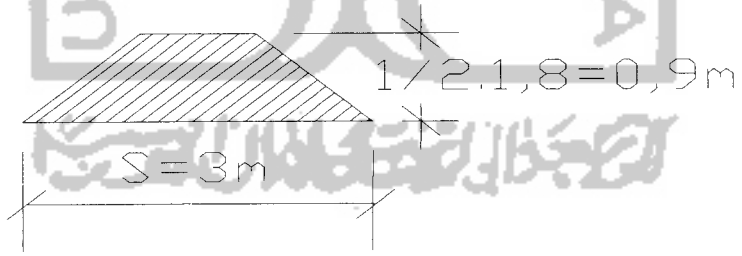
q5 (D2) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 2,25 = 281,25 Kg/m.

q5 (L2) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

22) Tipe XXII.



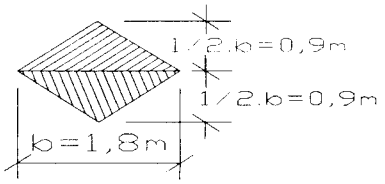
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

23) Tipe XXIII.



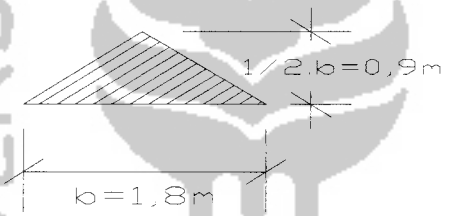
Beban mati (qD) :

q1 (D) = qD . ½ . b . 2 = 454 . ½ . 1,8 . 2 = 817,25 Kg/m.

Beban hidup (qL) :

q1 (L) = qL . ½ . b . 2 = 250 . ½ . 1,8 . 2 = 450 Kg/m.

24) Tipe XXIV.



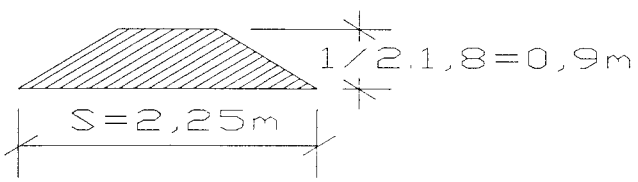
Beban mati (qD) :

q4 (D) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m.

Beban hidup (qL) :

q4 (L) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

25) Tipe XXV.



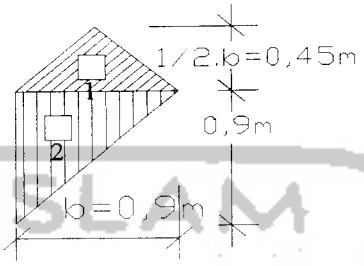
Beban mati (qD) :

q2 (D) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m.

Beban hidup (qL) :

q2 (L) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

26) Tipe XXVI.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 0,9 = 204,3 Kg/m.

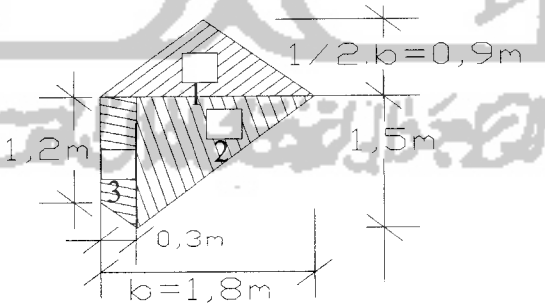
q5 (D2) = qD . b = 454 . 0,9 = 408,6 Kg/m

Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 0,9 = 112,5 Kg/m.

q5 (L2) = qL . b = 250 . 0,9 = 225 Kg/m.

27) Tipe XXVII.



Beban mati (qD) :

q5 (D1) = qD . ½ . b = 454 . ½ . 1,8 = 408,6 Kg/m.

q5 (D2) = qD . 1,5 = 454 . 1,5 = 681 Kg/m

q5 (D3) = qD . 1,2 = 454 . 1,2 = 544,8 Kg/m.

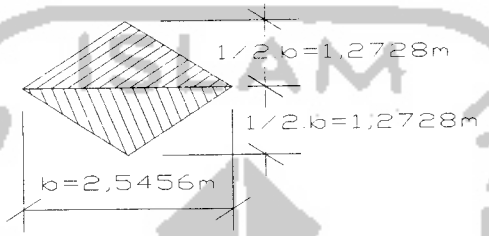
Beban hidup (qL) :

q5 (L1) = qL . ½ . b = 250 . ½ . 1,8 = 225 Kg/m.

q5 (L2) = qL . 1,5 = 250 . 1,5 = 375 Kg/m.

q5 (L3) = qL . 1,2 = 250 . 1,2 = 300 Kg/m.

28) Tipe XXVIII.



Beban mati (qD) :

q1 (D) = qD . ½ . b . 2 = 454 . ½ . 2,5456 . 2 = 1155,7024 Kg/m.

Beban hidup (qL) :

q1 (L) = qL . ½ . b . 2 = 250 . ½ . 2,5456 . 2 = 636,4 Kg/m.

4.4 Perencanaan Beban Gempa

4.4.1 Berat bangunan total

1. Berat balok ring dan pelat talang

- Luas plafond : 918 m^2
- Luas pelat talang : 268,56 m^2
- Luas atap : 1058,76 m^2
- Tebal pelat : 0,1 m
- b balok 1 : 0,4 m
- h balok 1 : 0,6 m
- b balok 2 : 0,2 m
- h balok 2 : 0,4 m
- b kolom : 0,6 m
- h kolom : 0,6 m
- Panjang balok induk : 190,8 m
- Panjang balok anak : 208,79 m
- Panjang balok konsol : 88,8 m
- Panjang dinding bawah : 285,7m
- Tinggi ½ kolom bawah : 2,1 m
- Jumlah kolom bawah (0,6x0,6 m) : 24 buah
- Bj beton : 2400 kg/m^3
- Bj dinding ½ bata : 250 kg/m^2
- Bj plofond : 18 kg/m^2

➢ **Beban mati**

– Genteng	= 50 x 1058,76	= 52938 kg
– Kuda-kuda		= 17476,3 kg
– Gording	= 752,64 x 4,06	= 3055,72 kg
– Balok induk	= 190,8 x 2400 x 0,4 x 0,6	= 109900,8 kg
– Balok anak	= 208,78 x 2400 x 0,2 x 0,4	= 40086,2 kg
– Balok konsol	= 88,8 x 2400 x 0,15 x 0,3	= 9590,4 kg
– Kolom	= 24 x 2,1 x 0,6 x 0,6 x 2400	= 43545,6 kg
– Dinding	= 285,7 x 2,1 x 250	= 149992,5 kg
– Plafond	= 918 x 18	= 16524 kg
– Pelat talang	= 268,56 x 0,1 x 2400	= 64454,4 kg +
		Wm = 507563,923 kg

➢ **Beban hidup**

– qh atap = 100 kg/m^2

– Koefisien reduksi = 0,3

Wh = 0,3 x 100 x 1058,76 = 31762,8 kg

➢ **Beban total (W1)**

W1 = Wm +Wh = 31762,8 + 507563,923

= 539326,723 kg

2. Berat lantai 3

- Luas pelat : 1024,2 m^2
- Tebal pelat : 0,12 m
- b balok 1 : 0,4 m

- h balok 1 : 0,6 m
- b balok 2 : 0,2 m
- h balok 2 : 0,4 m
- b kolom : 0,6 m
- h kolom : 0,6 m
- Panjang balok induk : 377,4 m
- Panjang balok anak : 273,45 m
- Panjang dinding atas : 285,7 m
- Panjang dinding bawah : 356,6 m
- Tinggi ½ kolom atas : 2,1 m
- Tinggi ½ kolom bawah : 2,1 m
- Jumlah kolom atas (0,6x0,6 m) : 32 buah
- Jumlah kolom bawah (0,6x0,6 m) : 32 buah
- Bj beton : 2400 kg/m^3
- Bj dinding ½ bata : 250 kg/m^2
- Bj keramik : 20 kg/m^2
- Bj spesi : 22 kg/m^2
- Tebal spesi : 0,03 m

➢ **Beban mati**

- Pelat = 1024,2 x 0,12 x 2400 = 294969,6 kg
- Balok induk = 377,4 x 2400 x 0,48 x 0,4 = 173905,9 kg
- Balok anak = 273,45 x 2400 x 0,28 x 0,2 = 36751,68 kg
- Kolom atas = 32 x 2,1 x 0,6 x 0,6 x 2400 = 58060,8 kg

- Kolom bawah = 32 x 2,1 x 0,6 x 0,6 x 2400 = 58060,8 kg
- Dinding atas = 285,7 x 2,1 x 250 = 149992,5 kg
- Dinding bawah = 356,6 x 2,1 x 250 = 187215 kg
- Spesi = 1024,2 x 22 x 0,03 = 675,972 kg
- Keramik = 1024,2 x 20 = 20484 kg

\+

Wm = 980116,272 kg

➢ **Beban hidup**

- qh lantai = 250 kg/m^2
- Koefisien reduksi = 0,3

Wh = 0,3 x 250 x 1024,2 = 76815 kg

➢ **Beban total (W2)**

W2 = Wm +Wh = 76815 + 980116,272

= 1056931,272 kg

3. Berat lantai 2

- Luas pelat : 956,7 m^2
- Tebal pelat : 0,12 m
- b balok 1 : 0,4 m
- h balok 1 : 0,6 m
- b balok 2 : 0,2 m
- h balok 2 : 0,4 m
- b kolom : 0,6 m
- h kolom : 0,6 m
- Panjang balok induk : 361,2 m

- Panjang balok anak : 264,45 m
- Panjang dinding atas : 356,6 m
- Panjang dinding bawah : 343,1 m
- Tinggi ½ kolom atas : 2,1 m
- Tinggi ½ kolom bawah : 2,1 m
- Jumlah kolom atas (0,6x0,6 m) : 32 buah
- Jumlah kolom bawah (0,6x0,6 m) : 32 buah
- Bj beton : 2400 kg/m^3
- Bj dinding ½ bata : 250 kg/m^2
- Bj keramik : 20 kg/m^2
- Bj spesi : 22 kg/m^2
- Tebal spesi : 0,03 m

➢ **Beban mati**

- Pelat = 956,7 x 0,12 x 2400 = 275529,6 kg
- Balok induk = 361,2 x 2400 x 0,48 x 0,4 = 166440,96kg
- Balok anak = 264,45 x 2400 x 0,28 x 0,2 = 35542,08 kg
- Kolom atas = 32 x 2,1 x 0,6 x 0,6 x 2400 = 58060,8 kg
- Kolom bawah = 32 x 2,1 x 0,6 x 0,6 x 2400 = 58060,8 kg
- Dinding bawah = 343,1 x 2,1 x 250 = 180127,5 kg
- Dinding atas = 356,6 x 2,1 x 250 = 187215 kg
- Spesi = 956,7 x 22 x 0,03 = 631,422 kg
- Keramik = 956,7 x 20 = 19134 kg

\+

Wm = 980742,162 kg

➢ **Beban hidup**

- qh lantai = 250 kg/m^2
- Koefisien reduksi = 0,3

Wh = 0,3 x 250 x 956,7 = 71752,5 kg

➢ **Beban total (W3)**

W3 = Wm +Wh = 980742,162 + 71752,5

= 1052494,662 kg

4. Berat total seluruhnya (Wt)

Wt = W1 + W2 + W3

= 513916,483 + 1056931,272 + 1052494,662

= 2623342,417 kg

4.4.2 Waktu getar bangunan (T)

Dengan rumus Empiris :

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

H = 2,1 + 4,2 + 4,2 + 4,2 = 14,7 m

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

$= 0,06.14,7^{3/4}$

= 0,45 detik

4.4.3 Koefisien gempa dasar (C)

Untuk Tx = Ty = 0,45 detik zone 3 dan jenis tanah keras, diperoleh C =0,05

Faktor keutamaan I = 1

Faktor jenis struktur K = 1

- **Beban hidup**

  - qh lantai = 250 kg/m^2
  - Koefisien reduksi = 0,3

  Wh = 0,3 x 250 x 956,7 = 71752,5 kg

- **Beban total (W3)**

  W3 = Wm +Wh = 980742,162 + 71752,5

  = 1052494,662 kg

4. Berat total seluruhnya (Wt)

Wt = W1 + W2 + W3

= 539326,723 + 1056931,272 + 1052494,662

= 2623342,417 kg

4.4.2 Waktu getar bangunan (T)

Dengan rumus Empiris :

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

H = 2,1 + 4,2 + 4,2 + 4,2 = 14,7 m

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

$= 0,06.14,7^{3/4}$

= 0,45 detik

4.4.3 Koefisien gempa dasar (C)

Untuk Tx = Ty = 0,45 detik zone 3 dan jenis tanah keras, diperoleh C =0,05

Faktor keutamaan I = 1

Faktor jenis struktur K = 1

4.4.4 Gaya geser horizontal total akibat gempa

Vx = Vy = C.I.K.Wt

= 0,05.1.1.2623115,497

= 131155,775 kg = 131,156 ton

4.4.5 Distribusi gaya geser horizontal total akibat gempa ke sepanjang tinggi gedung

1. Arah x

H/A = 14,7/51 = 0.288 < 3

$$Fi,x = \frac{W_i.H_i}{\Sigma W_i.H_i}.Vx$$

2. Arah y

H/A = 14,7/18 = 0,817 < 3

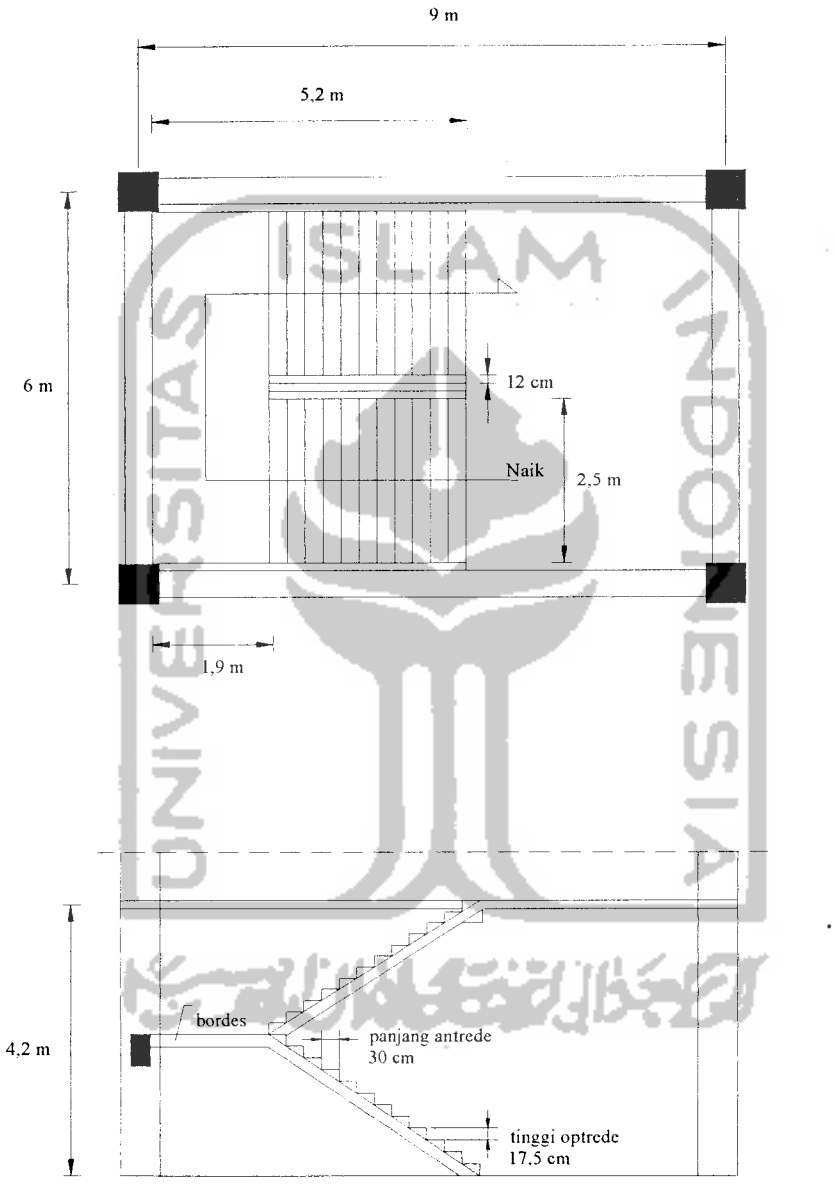
$$Fi,y = \frac{W_i.H_i}{\Sigma W_i.H_i}.Vy$$

Tabel 4.4. Distribusi gaya geser dasar horizontal total akibat gempa

Tingkat	hi (m)	Wi (t)	Wi.hi (tm)	Fix,y total (t)	Untuk tiap portal	
					1/5.Fix(t)	1/10.Fiy(t)
Atap	14,7	539,327	7554,58	39,189	7,835	3,918
3	10,5	1056,931	11097,776	57,57	11,515	5,758
2	6,3	1052,495	6630,719	34,4	6,88	3,44
1 (GF)	2,1	-	-	-	-	-
			25283,075	131,156	26,23	13,116

4.5 Perencanaan Tangga

4.5.1 Perencanaan tangga I



Gambar 4.7 . Denah perencanaan tangga I

1. Data perencanaan tangga I :

a. Tinggi antar lantai = 4,2 m = 420 cm

b. Lebar bordes = 1,9 m = 190 cm

c. Lebar tangga = 2,5 m = 250 cm

d. Beban sandaran tangga

Tinggi sandaran = 1m

Tebal sandaran = 0,12m

e. Tinggi optrede rencana diambil 18 m

Jumlah optrede = 420/18 = 23,333 dipakai 24 buah

Tinggi optrede pakai = 420/24 = 17,5 cm

Jumlah antrede = 24 – 2 = 22 buah

Diambil panjang antrede 30 cm

f. Sudut kemiringan tangga = 17,5/30 = arc tg $\alpha \rightarrow \alpha = 31°$

g. Dimensi tangga

Panjang tangga = (panjang antrede . jml antrede/2) + lebar bordes

= (30 . 22/2) + 190

= 520 cm

Lebar bersih tangga = 250 cm

h. Tebal pelat diambil 15 cm

2. Pembebanan

a. Pembebanan bordes

Beban mati (untuk sepanjang 1 m) :

- berat sendiri pelat = 0,15 . 1 . 24 = 3,60 KN/m
- berat spesi = 0,03 . 1 . 24 = 0,72 KN/m
- berat keramik = 0,01 . 1 . 20 = 0,20 KN/m
- berat sandaran = (0,12 . 24 . 1 . 1)/3 = 0,96 KN/m +

Q_D = 5,48 KN/m

Beban hidup (untuk sepanjang 1 m)

Q_L = 300 . 1 = 3 KN/m

b. Pembebanan tangga

Beban mati (untuk sepanjang 1 m) :

- berat sendiri tangga = $\left\{\frac{0,15}{\cos 31°}+\frac{0,175}{2}\right\}$.1.24 = 6,298 KN/m
- berat spesi = 0,03 . 1 . 24 = 0,720 KN/m
- berat keramik = 0,01 . 1 . 20 = 0,200 KN/m
- berat sandaran = (0,12 . 24 . 1 .1. 2)/2,5 = 2,304 KN/m +

Q_D = 9,522 KN/m

Beban hidup (untuk sepanjang 1 m)

Q_L = 300 . 1 = 3 KN/m

3. Perhitungan penulangan tangga

Tinggi pelat = 15 cm = 150 mm.

$$\rho b = \frac{0,85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0,85.25}{400}.0,85.\left(\frac{600}{600+400}\right) = 0,027$$

ρ maks = 0,75 . ρb = 0,75 . 0,027 = 0,02

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

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$$

d = h – Pb – ½.Øtul = 150 – 20 – ½.13 = 123,5 mm (digunakan tulangan Ø13 mm).

a. Perencanaan pelat bordes

Data dari SAP 2000 seperti tercantum dalam lampiran, didapat momen max. = 20,59 KNm

$$Mn = \frac{Mu}{\phi} = \frac{20{,}59}{0{,}8} = 25{,}738 \text{ KNm}$$

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{25{,}738.10^6}{1000.123{,}5^2} = 1{,}687 \text{ mm}^2$$

$$\rho_{perlu} = \frac{1}{m}.\left[1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right] = \frac{1}{18{,}824}.\left[1 - \sqrt{1 - \frac{2.18{,}824.1{,}687}{400}}\right]$$

$$= 0{,}004$$

$\rho_{perlu} = 0{,}004 > \rho_{min} = 0{,}0035$

sehingga ρ pakai = ρ_{perlu} = 0,004

As perlu = ρ pakai . b . d = 0,004 .1000 . 123,5 = 543,362 mm^2

Pakai tulangan P13 → A1.Ø = ¼ . π . D^2 = ¼ . π.13^2 = 132,73 mm^2

$$\text{Jarak tulangan (x)} \le \frac{A1.\phi.1000}{Asperlu} = \frac{132{,}73.1000}{543{,}362} = 244{,}276 \text{ mm}$$

$$\le 2.h = 2 .150 = 300 \text{ mm}$$

$$\le 250 \text{ mm}$$

Jadi pakai jarak (x) = 130 mm → **D13 - 240 mm.**

$$\text{As ada} = \frac{132{,}73.1000}{240} = 553{,}042 \text{ mm}^2 > \text{As perlu} = 543{,}362 \text{ mm}^2$$

Kontrol Mn :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{553{,}042.400}{0{,}85.25.1000} = 10{,}410 \text{ mm.}$$

Mn = As ada . fy . (d – a/2) = 553,042 .400.(123,5 –10,410/2).

= 26168841,360 Nmm

= 26,169 KNm > $\frac{Mu}{\phi}$ = 25,738 KNm .....Ok !

b. Perencanaan pelat tangga

Data dari SAP 2000 seperti tercantum dalam lampiran, didapat momen max. = 21,41 KNm

$$Mn = \frac{Mu}{\phi} = \frac{21{,}41}{0{,}8} = 26{,}763 \text{ KNm}$$

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{26{,}763.10^6}{1000.123{,}5^2} = 1{,}755 \text{ mm}^2$$

$$\rho_{perlu} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right] = \frac{1}{18{,}824}.\left[1-\sqrt{1-\frac{2.18{,}824.1{,}755}{400}}\right]$$

$$= 0{,}005$$

ρ_{perlu} = 0,005 > ρ_{min} = 0,0035

sehingga ρ pakai = ρ_{perlu} = 0,005

As perlu = ρ pakai . b . d = 0,005 .1000 . 123,5 = 565,630 mm^2

Pakai tulangan P13 → A1.Ø = ¼ . π . D^2 = ¼ . π.13^2 = 132,73 mm^2

$$\text{Jarak tulangan (x)} \leq \frac{A1.\phi.1000}{Asperlu} = \frac{132{,}73.1000}{565{,}630} = 234{,}659 \text{ mm}$$

$$\leq 2.h = 2 .150 = 300 \text{ mm}$$

$$\leq 250 \text{ mm}$$

Jadi pakai jarak (x) = 100 mm → **D13 - 230 mm.**

$$\text{As ada} = \frac{132{,}73.1000}{230} = 577{,}087 \text{ mm}^2 > \text{As perlu} = 565{,}630 \text{ mm}^2$$

Kontrol Mn :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{577{,}087.400}{0{,}85.25.1000} = 10{,}863 \text{ mm.}$$

Mn = As ada . fy . (d – a/2) = 577,087 .400.(123,5 –10,863/2).

= 27254318,58 Nmm

$$= 27{,}254 \text{ KNm} > \frac{Mu}{\phi} = 26{,}763 \text{ KNm} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\text{Ok !}$$

c. Perencanaan tulangan bagi

As bagi = 0,002 .b .h = 0,002 .1000 .150 = 300 mm^2

Pakai tulangan P 8 → A1.Ø = ¼ . π . D^2 = ¼ . π. 8^2 = 50,265 mm^2

$$\text{Jarak antar tulangan (x)} = \frac{A1.\phi.1000}{Asbagi} = \frac{50{,}265.1000}{300}$$

$$= 167{,}55 \text{ mm}^2$$

Dipakai tulangan → **P 8 - 160 mm.**

4. Perencanaan balok bordes

Ukuran balok 40/60

Ø tulangan pokok = 22 mm

Ø tulangan sengkang = 10 mm

Tinggi efektif balok ($d_{diketahui}$) = $h_{diketahui}$ – Pb - $Ø_{sengkang}$ - ½ $Ø_{tul.pokok}$

d = 600 – 40 – 10 – ½ 22 = 539 mm

Pembebanan :

- beban akibat tangga = 54,125 KN/m
- berat sendiri = 1,2.0,40.0,60.24 = 6,912 KN/m +

qu = 61,037 KN/m

Momen tumpuan :

$$Mu = -\frac{1}{16}qu.L^2 = -\frac{1}{16}.61{,}037.5{,}6^2 = -119{,}633 \text{ KNm}$$

Momen lapangan :

$$Mu = \frac{1}{11}qu.L^2 = \frac{1}{11}.61{,}037.5{,}6^2 = 174{,}011 \text{ KNm}$$

a. Perencanaan tulangan lentur balok bordes

Untuk $f'c \leq 30$ MPa $\rightarrow \beta_1 = 0{,}85$

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25}{400}.0{,}85\left(\frac{600}{600+400}\right)$$

$$= 0{,}027$$

$$\rho_{maks} = 0{,}75 \,.\, \rho b = 0{,}75 \,.\, 0{,}027 = 0{,}020$$

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

$$\rho_{pakai} = 0{,}5 \,.\, \rho_{maks} = 0{,}5 \,.\, 0{,}020 = 0{,}010$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$$

$$Rn = \rho.fy\,(1 - \tfrac{1}{2}\rho\, m)$$

$$= 0{,}010 \,.\, 400\,(1 - \tfrac{1}{2}.0{,}010\,.18{,}824) = 3{,}624 \text{ MPa}$$

- Tulangan tumpuan

Mu = 119,633 KNm

$$Mu/\phi = \frac{119,633}{0,8} = 149,541 \text{ KNm}$$

$$b.d^2 = \frac{Mu/\phi}{Rn}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{149,541.10^6}{3,624.400}} = 321,186 \text{ mm} < d = 539 \text{ mm},$$

maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{149,541.10^6}{400.539^2} = 1,287 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1,287}{3,624}.0,010 = 0,0035 = \rho_{min} = 0,0035$$

sehingga ρpakai = ρ_{ada} = 0,0035

As = ρ_{ada}.b.d_{ada} = 0,0035 . 400 .539 = 754,600 mm^2

Dipakai diameter tulangan D22, maka : $A_{1\phi}$ = 380,286 mm^2

$$n = \frac{As}{A_1\phi} = \frac{754,600}{380,286} = 1,98$$

Dipakai tulangan memanjang **2D22**, maka :

As_{ada} = 2. 380,286 = 760,572 mm^2 > As = 754,600 mm^2

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0,85.f'c\ b} = \frac{760,572.400}{0,85.25.400} = 35,792 \text{ mm}$$

Mn = As ada . fy . (d – a/2) = 760,572 .400.(539 –35,792/2).

= 158534844,6 Nmm

= 158,535 KNm > $\frac{Mu}{\phi}$ = 149,541 KNm .....Ok !

- Tulangan lapangan

Mu = 174,011 KNm

$Mu/\phi = \frac{174,011}{0,8} = 217,514$ KNm

$b.d^2 = \frac{Mu/\phi}{Rn}$

$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{217,514.10^6}{3,624.400}} = 387,364$ mm < d = 539 mm,

maka dipakai tulangan sebelah.

$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{174,011.10^6}{400.539^2} = 1,497$ MPa

$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1,497}{3,624}.0,010 = 0,004 > \rho_{min} = 0,0035$

sehingga ρpakai = ρ_{ada} = 0,004

As = $\rho_{ada}.b.d_{ada}$ = 0,004 . 400 .539 = 862,400 mm^2

Dipakai diameter tulangan D22, maka : $A_{1Ø}$ = 380,286 mm^2

$n = \frac{As}{A_1\phi} = \frac{862,400}{380,286} = 2,268$

Dipakai tulangan memanjang **3D22**, maka :

As_{ada} = 3 . 380,286 = 1140,858 mm^2 > As = 862,400 mm^2

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0{,}85.f'c\ b} = \frac{1140{,}858.400}{0{,}85.25.400} = 53{,}687 \text{ mm}$$

Mn = As ada . fy . (d – a/2) = 1140,858 .400.(539 – 53,687/2).

= 233719136,1 Nmm

$$= 233{,}719 \text{ KNm} > \frac{Mu}{\phi} = 217{,}514 \text{ KNm} \ldots\ldots\ldots\ldots\ldots\ldots\text{Ok !}$$

b. Perencanaan tulangan geser balok bordes

Gaya geser dukungan :

$$\text{Vu dukungan} = \frac{1}{2}.qU.L = \frac{1}{2}.61{,}037.5{,}6 = 170{,}904 \text{ KN}$$

$$\text{maka } Vu/\phi = \frac{170{,}904}{0{,}6} = 284{,}839 \text{ KN}$$

Tegangan geser beton (Vc) :

$$Vc = \left(\frac{1}{6}\sqrt{fc'}\right).b.d = \left(\frac{1}{6}\sqrt{25}\right).400.600 = 200000 \text{ N} = 200 \text{ KN}$$

$\frac{Vu}{\phi}$ = 284,839 KN > Vc = 200 KN, maka perlu tulangan geser.

$$Vs = \frac{Vu}{\phi} - Vc = 284{,}839 - 200 = 84{,}839 \text{ KN}$$

Dipakai sengkang P10, maka $Av = 2.¼.\pi.10^2 = 157 \text{ mm}^2$

$$\text{Jarak sengkang } s \le \frac{Av.fy.d}{Vs} = \frac{157.240.539}{84839} = 239{,}389 \text{ mm}$$

$$\le d/2 = 539/2 = 269{,}5 \text{ mm}$$

$$\le 600 \text{ mm}$$

Jadi dipakai sengkang **P10 – 230 mm.**

5. Perhitungan pondasi tangga

a. Data :

- σ tanah = 150 KN/m²
- γ batu = 22 KN/m³
- γ tanah = 18 KN/m³
- balok diatas pondasi 25/50 cm

Tinjauan untuk lebar tangga = 2,5 m

Tinggi pondasi tangga = 1m

b. Pembebanan

Akibat beban tangga = 98,669 KN

Berat balok diatas pondasi = 0,25 .0,5 .2,5 . 24 = 7,500 KN +

P = 106,169 KN

Tegangan ijin tanah pakai :

σ = σ tanah – berat pondasi

= 150 – 1 . 22 = 128 KN/m²

Diketahui pada kondisi kritis → σ = P/A

A = P/σ = 106,169/128 = 0,829 m²

B = A/L = 0,829/2,5 = 0,332 m → diambil lebar (B) = 0,4 m

Kontrol tegangan tanah :

σ = P/A = 106,169 /(0,4 . 2,5)

= 106,169 KN/m² < σ tanah = 128 KN/m² .....Ok !

4.5.2 Perencanaan tangga II

1. Data perencanaan tangga II :

a Tinggi antar lantai = 4,2 m = 420 cm

b Lebar bordes = 1,2 m = 120 cm

c. Lebar tangga = 1,2 m = 120 cm

d. Beban sandaran tangga :

Tinggi sandaran = 1m

Tebal sandaran = 0,12m

e. Tinggi optrede rencana diambil 18 m

Jumlah optrede = 420/18 = 23,333 dipakai 24 buah

Tinggi optrede pakai = 420/24 = 17,5 cm

Jumlah antrede = 24 – 2 = 22 buah

Diambil panjang antrede 30 cm

f. Sudut kemiringan tangga = 17,5/30 = arc tg $\alpha \rightarrow \alpha = 31°$

g. Dimensi tangga

Panjang tangga = (panjang antrede . jml antrede/2) + lebar bordes

= (30 . 22/2) + 120 = 450 cm

Lebar bersih tangga = 120 cm

h. Tebal pelat diambil 15 cm

2. Pembebanan

a Pembebanan bordes

Beban mati (untuk sepanjang 1 m) :

- berat sendiri pelat = 0,15 . 1 . 24 = 3,60 KN/m
- berat spesi = 0,03 . 1 . 24 = 0,72 KN/m
- berat keramik = 0,01 . 1 . 20 = 0,20 KN/m
- berat sandaran = (0,12 . 24 . 1 . 1)/1,5 = 1,92 KN/m +

Q_D = 6,44 KN/m

Beban hidup (untuk sepanjang 1 m)

Q_L = 300 . 1 = 3 KN/m

b Pembebanan tangga

Beban mati (untuk sepanjang 1 m) :

- berat sendiri tangga = $\left\{\frac{0,15}{\cos 31}+\frac{0,175}{2}\right\}.1.24$ = 6,30 KN/m
- berat spesi = 0,03 . 1 . 24 = 0,72 KN/m
- berat keramik = 0,01 . 1 . 20 = 0,20 KN/m
- berat sandaran = (0,12 . 24 . 1 . 2)/1,2 = 4,80 KN/m +

Q_D =12,02 KN/m

Beban hidup (untuk sepanjang 1 m)

Q_L = 300 . 1 = 3 KN/m

3. Perhitungan penulangan tangga

Tinggi pelat tangga = 15 cm = 150 mm.

$$\rho b = \frac{0,85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0,85.25}{400}.0,85.\left(\frac{600}{600+400}\right) = 0,027$$

ρ maks = 0,75 . ρb = 0,75 . 0,027 = 0,02

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

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$$

d = h – Pb – ½.Øtul = 150 – 20 – ½.13 = 123,5 mm.

a. Perencanaan pelat bordes dan pelat tangga (momennya sama)

Data dari SAP 2000 seperti tercantum dalam lampiran, didapat momen max. = 24,19 KNm

$$Mn = \frac{Mu}{\phi} = \frac{24{,}19}{0{,}8} = 30{,}238 \text{ KNm}$$

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{30{,}238.10^6}{1000.123{,}5^2} = 1{,}982 \text{ mm}^2$$

$$\rho_{perlu} = \frac{1}{m}.\left[1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right] = \frac{1}{18{,}824}.\left[1 - \sqrt{1 - \frac{2.18{,}824.1{,}982}{400}}\right]$$

$$= 0{,}005$$

$\rho_{perlu} = 0{,}005 > \rho_{min} = 0{,}0035$

sehingga ρ pakai = ρ_{perlu} = 0,005

As perlu = ρ pakai . b . d = 0,005 .1000 . 123,5 = 643,501 mm²

Pakai tulangan P13 → A1.Ø = ¼ . π . D² = ¼ . π.13² = 132,73 mm²

$$\text{Jarak tulangan (x)} \leq \frac{A1.\phi.1000}{Asperlu} = \frac{132{,}73.1000}{643{,}501} = 206{,}262 \text{ mm}$$

$$\leq 2.h = 2 .150 = 300 \text{ mm}$$

$$\leq 250 \text{ mm}$$

Jadi pakai jarak (x) = 130 mm → **D13 - 200 mm.**

$$\text{As ada} = \frac{132{,}73.1000}{200} = 663{,}650 \text{ mm}^2 > \text{As perlu} = 643{,}501 \text{ mm}^2$$

Kontrol Mn :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{663{,}650.400}{0{,}85.25.1000} = 12{,}492 \text{ mm.}$$

Mn = As ada . fy . (d – a/2) = 663,650 .400.(123,5 –12,492/2).

= 31126246,84 Nmm

$$= 31{,}126 \text{ KNm} > \frac{Mu}{\phi} = 30{,}238 \text{ KNm} \ldots\ldots\ldots\ldots\ldots\ldots\text{Ok !}$$

b. Perencanaan tulangan bagi

As bagi = 0,002 .b .h = 0,002 .1000 .150 = 300 mm^2

Pakai tulangan P 8 → A1.Ø = ¼ . π . D^2 = ¼ . π. 8^2 = 50,265 mm^2

$$\text{Jarak antar tulangan (x)} = \frac{A1.\phi.1000}{Asbagi} = \frac{50{,}265.1000}{300}$$

$= 167{,}55 \text{ mm}^2$

Dipakai tulangan → **P 8 - 160 mm.**

4. Perencanaan balok bordes

Ukuran balok 20/40

Ø tulangan pokok = 16mm

Ø tulangan sengkang = 10 mm

Tinggi efektif balok ($d_{diketahui}$) = $h_{diketahui}$ – Pb - $Ø_{sengkang}$ - ½ $Ø_{tul.pokok}$

d = 400 – 40 – 10 – ½. 16 = 342 mm

Pembebanan :

- beban akibat tangga = 73,475 KN/m
- berat sendiri = 1,2 . 0,20 . 0,40 . 24 = 2,304 KN/m +

qu = 75,779 KN/m

Momen tumpuan :

$$Mu = -\frac{1}{16}qu.L^2 = -\frac{1}{16}.75{,}779.2{,}7^2 = -34{,}527 \text{ KNm}$$

Momen lapangan :

$$Mu = \frac{1}{11}qu.L^2 = \frac{1}{11}.75{,}779.2{,}7^2 = 50{,}221 \text{ KNm}$$

b. Perencanaan tulangan lentur balok bordes

Untuk $f'c \leq 30$ MPa $\rightarrow \beta_1 = 0{,}85$

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25}{400}.0{,}85\left(\frac{600}{600+400}\right)$$

$$= 0{,}027$$

$$\rho_{maks} = 0{,}75 \,.\, \rho b = 0{,}75 \,.\, 0{,}027 = 0{,}020$$

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

$$\rho_{pakai} = 0{,}5 \,.\, \rho_{maks} = 0{,}5 \,.\, 0{,}020 = 0{,}010$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$$

$$Rn = \rho \,.\, fy\,(1 - \tfrac{1}{2}\rho\, m)$$

$$= 0{,}010 \,.\, 400\,(1 - \tfrac{1}{2}.0{,}010\,.18{,}824) = 3{,}624 \text{ MPa}$$

- Tulangan tumpuan

$$Mu = 34{,}527 \text{ KNm}$$

$$Mu/\phi = \frac{34{,}527}{0{,}8} = 43{,}159 \text{ KNm}$$

$$b.d^2 = \frac{Mu/\phi}{Rn}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{43{,}159.10^6}{3{,}624.200}} = 244{,}021 \text{ mm} < d = 342 \text{ mm},$$

maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{\frac{Mu}{\phi}}{b.d_{ada}^2} = \frac{43{,}159.10^6}{200.342^2} = 1{,}845 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1{,}845}{3{,}624}.0{,}010 = 0{,}005 > \rho_{min} = 0{,}0035$$

sehingga ρpakai = ρ_{ada} = 0,005

$As = \rho_{ada}.b.d_{ada} = 0{,}005 \cdot 200 \cdot 342 = 348{,}223 \text{ mm}^2$

Dipakai diameter tulangan D16, maka : $A_{1\emptyset} = 201{,}062 \text{ mm}^2$

$$n = \frac{As}{A_1\phi} = \frac{348{,}223}{201{,}062} = 1{,}732$$

Dipakai tulangan memanjang **2D16**, maka :

$As_{ada} = 2 \cdot 201{,}062 = 402{,}124 \text{ mm}^2 > As = 348{,}223 \text{ mm}^2$

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0{,}85.f'c\ b} = \frac{402{,}124.400}{0{,}85.25.200} = 37{,}847 \text{ mm}$$

Mn = As ada . fy . (d – a/2) = 402,124 .400.(342 – 37,847/2).

= 51966725,79 Nmm

= 51,967 KNm > $\frac{Mu}{\phi}$ = 43,159 KNm .....Ok !

- Tulangan lapangan

Mu = 50,221 KNm

$$Mu/\phi = \frac{50,221}{0,8} = 62,776 \text{ KNm}$$

$$b.d^2 = \frac{Mu/\phi}{Rn}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{62,776.10^6}{3,624.200}} = 294,298 \text{ mm} < d = 342 \text{ mm},$$

maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{62,776.10^6}{200.342^2} = 2,684 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{2,684}{3,624}.0,010 = 0,007 > \rho_{min} = 0,0035$$

sehingga ρpakai = ρ_{ada} = 0,007

As = $\rho_{ada}.b.d_{ada}$ = 0,007 . 200 .342 = 506,5 mm^2

Dipakai diameter tulangan D16, maka : $A_{1Ø}$ = 201,062 mm^2

$$n = \frac{As}{A_1\phi} = \frac{506,5}{201,062} = 2,519$$

Dipakai tulangan memanjang **3D16**, maka :

As_{ada} = 3 . 201,062 = 603,186 mm^2 > As = 506,5 mm^2

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0,85.f'c\ b} = \frac{603,186.400}{0,85.25.200} = 56,770 \text{ mm}$$

Mn = As ada . fy . (d – a/2) = 603,186 .400.(342 – 56,770/2).

= 75667270,96 Nmm

= 75,667 KNm > $\frac{Mu}{\phi}$ = 62,776 KNm .....Ok !

b. Perencanaan tulangan geser balok bordes

Gaya geser dukungan :

Vu dukungan = $\frac{1}{2}$.qU.L = $\frac{1}{2}$.75,779 .2,7 = 102,302 KN

maka $Vu/\phi = \frac{102,302}{0,6}$ = 170,503 KN

Tegangan geser beton (Vc) :

Vc = $\left(\frac{1}{6}\sqrt{fc'}\right)$.b.d = $\left(\frac{1}{6}\sqrt{25}\right)$.200.400 .$10^{-3}$ = 66,667 KN

$\frac{Vu}{\phi}$ = 170,503 KN > Vc = 66,667 KN, maka perlu tulangan geser.

Vs = $\frac{Vu}{\phi}$ – Vc = 170,503 – 66,667 = 103,836 KN

Dipakai sengkang P10, maka Av = 2.¼.π.10^2 = 157 mm^2

Jarak sengkang s ≤ $\frac{Av.fy.d}{Vs} = \frac{157.240.342}{103836}$ = 124,105 mm

≤ $d/2$ = $342/2$ = 171 mm

≤ 600 mm

Jadi dipakai sengkang **P10 – 120 mm.**

5. Perhitungan pondasi tangga

a. Data :

σ tanah = 150 KN/m²

γ batu = 22 KN/m³

γ tanah = 18 KN/m³

balok diatas pondasi 25/50 cm

Tinjauan untuk lebar tangga = 1,2 m

Tinggi pondasi tangga = 1m

b. Pembebanan

Akibat beban tangga = 119,033 KN

Berat balok diatas pondasi = 0,25 .0,5 .1,2 . 24 = 3,600 KN +

P = 122,633 KN

Tegangan ijin tanah pakai :

σ = σ tanah – berat pondasi

= 150 – 1 . 22 = 128 KN/m²

Diketahui pada kondisi kritis → σ = P/A

A = P/σ = 122,633 /128 = 0,958 m²

B = A/L = 0,958/1,2 = 0,798 m → diambil lebar (B) = 0,8 m

Kontrol tegangan tanah :

σ = P/A = 122,633/(0,8 . 1,2)

= 127,743 KN/m² < σ tanah = 128 KN/m² ……Ok !

4.6 Perencanaan Balok

4.6.1 Perencanaan balok penampang persegi menahan lentur tulangan sebelah

1. Data perencanaan balok :

f'c = 25 Mpa

fy = 400 Mpa

penutup beton (Pb) = 40 mm

diameter tulangan pokok = 22 mm

diameter tulangan sengkang = 10 mm

lebar balok (b) = 400 mm

tinggi balok (h) = 600 mm

Ø = 0,8

β = 0,85

d' = 40+10+(22/2) = 61 mm

d = 600-61 = 539 mm

$$\rho_b = \frac{0{,}85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25}{400}.0{,}85.\left(\frac{600}{600+400}\right) = 0{,}027$$

$\rho_{maks} = 0{,}75 \cdot \rho b = 0{,}75 \cdot 0{,}027 = 0{,}020$

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0.004$$

$\rho_{rencana} = 0{,}5 \cdot \rho_{maks} = 0{,}5 \cdot 0{,}020 = 0.010 > \rho_{min}$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$$

$Rn = \rho.fy.(1-½.\rho.m) = 0{,}010.400.(1-½.0{,}010.18{,}824) = 3{,}624$ MPa

Mn = As ada . fy . (d – a/2).

= 251,35 . 240 . (76 – 2,8388/2).

= 4,499 KNm > 1,33 . $\frac{Mu}{\phi}$ = 1,33 . 0,7301 = 0,971 KN

f. Perencanaan tulangan bagi pelat talang.

As bagi = 0,002 . b . h

= 0,002 . 1000 . 100 = 200 mm^2.

Dipakai P 8 → A1.Ø = 50,27 mm^2.

Jarak tulangan = $\frac{A1.\phi.1000}{Aspakai}$ = $\frac{50,27.1000}{200}$ = 251,35 mm.

Jadi dipakai tulangan bagi : **P 8 – 200 mm**.

As = $\frac{50,27.1000}{200}$ = 251,35 mm^2 > 200 mm^2.

2. Perencanaan dimensi balok portal B1a Induk (9 m) lantai 2

   a. Perencanaan tulangan tumpuan

      Data dari SAP 2000 seperti tercantum pada lampiran, di dapat momen maksimum tumpuan (M^-) = 297,071 KNm.

$$Mu/\phi = 297{,}071/0{,}8 = 371{,}339 \text{ KNm}$$

$$b.d^2 = \frac{Mu/\phi}{Rn} \rightarrow d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{371{,}339.10^6}{3{,}624.400}} = 502{,}575 \text{ mm}$$

$d_{diketahui} = 540$ mm $\rightarrow d_{diketahui} > d_{perlu}$ (tul sebelah)

Jika nilai $d_{diketahui} \geq d_{perlu}$, maka digunalan tulangan sebelah

Jika nilai $d_{diketahui} < d_{perlu}$, maka digunalan tulangan rangkap

$$Rn_{baru} = \frac{Mu/\phi}{b.d_{diketahui}^{\,2}} = \frac{371{,}339.10^6}{400.540^2} = 3{,}184 \text{ MPa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}.\rho = \frac{3{,}184}{3{,}624}.0{,}010 = 0{,}009 \geq \rho_{min} = 0{,}004$$

$As = \rho_{baru}.b.d_{diketahui} = 0{,}009.400.540 = 1900{,}938 \text{ mm}^2$

Dipakai Ø tulangan tarik → 22 mm

$A_{1O} = 380{,}286 \text{ mm}^2$

$$n = \frac{As}{A_{1\phi}} = \frac{1900{,}938}{380{,}286} = 4.999 \approx 5$$

$As_{ada} = n.A_{1O} = 5 \,.\, 380{,}286 = 1901{,}429 \text{ mm}^2 > As = 1900{,}938 \text{ mm}^2$

Periksa penempatan tulangan :

$$Jbd = \frac{b - 2.(Pb + \phi_{sengkang}) - (n.\phi_{pokok})}{n-1}$$

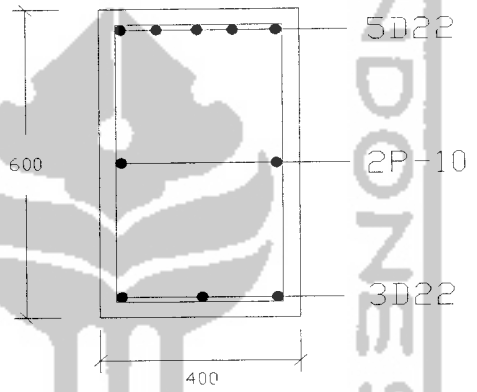
$$= \frac{400 - 2.(40+10) - (5.22)}{5-1} = 47{,}5 \text{ mm} > \text{D } 22 = 22 \text{ mm}$$

Kontrol momen

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1901{,}429.400}{0{,}85.25.400} = 89{,}479$$

$$Mn = As_{ada}.fy.\left(d_{diketahui} - a/2\right) = 1901{,}429.400.\left(540 - 89{,}479/2\right)$$

$$= 379{,}681 \text{ KNm} > Mu/\phi = 371{,}339 \text{ KNm}$$



Gambar 4.8 Detail potongan balok induk pada tumpuan

b. Perencanaan tulangan lapangan

Data dari SAP 2000 seperti tercantum pada lampiran, di dapat momen maksimum lapangan (M^+) = 251,961 KNm.

$$Mu/\phi = 251{,}961/0{,}8 = 314{,}951 \text{ KNm}$$

$$b.d^2 = \frac{Mu/\phi}{Rn} \rightarrow d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{314{,}951.10^6}{3{,}624.400}} = 462{,}847 \text{ mm}$$

$d_{diketahui} = 540$ mm $\rightarrow d_{diketahui} > d_{perlu}$ (tul sebelah)

Jika nilai $d_{diketahui} \geq d_{perlu}$, *maka digunalan tulangan sebelah*

Jika nilai $d_{diketahui} < d_{perlu}$, *maka digunalan tulangan rangkap*

$$Rn_{baru} = \frac{Mu/\phi}{b.d_{diketahui}^{2}} = \frac{314{,}951.10^{6}}{400.540^{2}} = 2{,}700 \text{ MPa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}.\rho = \frac{2{,}700}{3{,}624}.0{,}010 = 0{,}007 \geq \rho_{min} = 0{,}004$$

$$As = \rho_{baru}.b.d_{diketahui} = 0{,}007.400.540 = 1612{,}282 \text{ mm}^2$$

Dipakai Ø tulangan tarik → 22 mm

$$A_{1Ø} = 380{,}286 \text{ mm}^2$$

$$n = \frac{As}{A_{1\phi}} = \frac{1612{,}282}{380{,}286} = 4.240 \approx 5$$

$$As_{ada} = n.A_{1Ø} = 5 \,.\, 380{,}286 = 1901{,}429 \text{ mm}^2 > As = 1612{,}282 \text{ mm}^2$$

Periksa penempatan tulangan :

$$Jbd = \frac{b - 2.(Pb + \phi_{sengkang}) - (n.\phi_{pokok})}{n-1} = \frac{400 - 2.(40+10) - (5.22)}{5-1}$$

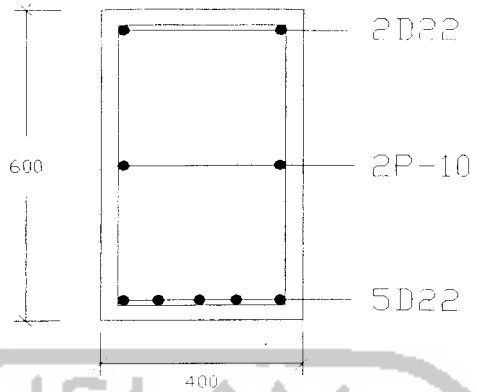
$$= 47{,}5 \text{ mm} > D\ 22 = 22 \text{ mm}$$

Kontrol momen

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1901{,}429.400}{0{,}85.25.400} = 89{,}479$$

$$Mn = As_{ada}.fy.\left(d_{diketahui} - a/2\right) = 1901{,}429.400.\left(540 - 89{,}479/2\right)$$

$$= 379{,}681 \text{ KNm} > Mu/\phi = 314{,}951 \text{ KNm}$$



Gambar 4.9 Detail potongan balok induk pada lapangan

c. Perencanaan gaya geser

Data dari SAP 2000 seperti tercantum pada lampiran, di dapat gaya geser maksimum sebagai berikut :

VD = 107,97 KN

VL = 23,123 KN

VE = 13,807 KN

$$Vu = 0,7.\frac{M_{kap} + M'_{kap}}{l_n} + 1,05.V_g \quad ; Vg = VD + VL$$

$$= 0,7.\phi.\frac{M_{nak} + M'_{nak}}{l_n} + 1,05.(VD + VL) \quad ; \emptyset = 1,25$$

$$= 0,7.1,25.\frac{376,681 + 234,175}{8,6} + 1,05.(107,97 + 23,125)$$

$$= 199,801 \text{ KN}$$

$$Vu' = 1,05.\left(V_{D,b} + V_{L,b} + \frac{4}{k}.V_{E,b}\right)$$

$$= 1,05.\left(107,97 + 23,123 + \frac{4}{1}.13,807\right)$$

$$= 201,325 \text{ KN}$$

Vu,b pakai adalah nilai terkecil antara Vu,b dan Vu,b', sehingga di dapat Vu,b pakai = 199,801 KN.

Daerah sendi plastis (sepanjang 2.h) :

Vc = 0 (beton dianggap tidak menerima geser)

$$Vs = \frac{Vu,b_{pakai}}{\phi} = \frac{199,801}{0,6} = 333,002 \text{ KN}$$

$$Vc = \left(\frac{1}{6}\sqrt{f'c}\right).b.d = \left(\frac{1}{6}\sqrt{25}\right).400.540 = 180 \text{ KN}$$

Vs_{min} = 1/3.b.d = 1/3.400.540 = 72 KN

$(Vc+Vs_{min}) < Vs \leq 3.Vc \rightarrow 252 \text{ KN} < 333,002 \text{ KN} \leq 540 \text{ KN}$

Tersedia tulangan sengkang P-10 → fy = 240 MPa.

$$Av = \tfrac{1}{4}.\pi.D^2 = \tfrac{1}{4}.\pi.10^2.2 = 157,080 \text{ mm}^2.$$

$$S \leq \frac{Av.fy.d}{\left(\frac{Vu}{\phi} - Vc\right)} = \frac{157,080.240.540}{(333,002-0).1000} = 61,134 \text{ mm}$$

$\leq$ d/2 = 540/2 = 270 mm

$\leq$ 600 mm

S pakai → P10-60.

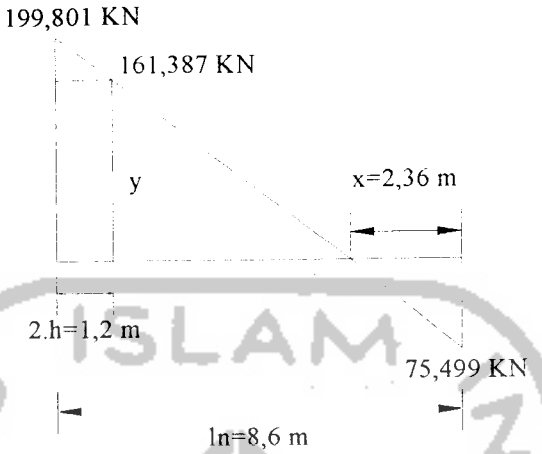
Daerah luar sendi plastis :

$$Vu,b'' = 0,7.\frac{M_{kap} + M'_{kap}}{l_n} - 1,05.V_g \quad ; Vg = VD + VL$$

$$= 0,7.\phi.\frac{M_{nak} + M'_{nak}}{l_n} - 1,05.(VD + VL) \quad ; Ø = 1,25$$

$$= 0,7.1,25.\frac{376,681 + 234,175}{8,6} - 1,05.(107,97 + 23,125)$$

= -75,499 KN



Gambar 4.10 Diagram segitiga tegangan geser

199,801.x = (8,6-x).75,499

199,801.x = 649,291-(75,499.x)

273,3.x = 649,291

x = 2,36 m

$$\frac{199,801}{(8,6-2,36)} = \frac{y}{(8,6-1,2-2,36)}$$

$$\frac{199,801}{6,240} = \frac{y}{5,040}$$

6,240.y = 1006,997

y = 161,387 KN

Vu,b pakai = 161,387 KN

$$Vs = \frac{Vu,b_{pakai}}{\phi} = \frac{161,387}{0,6} = 268,978 \text{ KN}$$

$$Vc = \left(\frac{1}{6}\sqrt{f'c}\right).b.d = \left(\frac{1}{6}\sqrt{25}\right).400.540 = 180 \text{ KN}$$

Vs_{min} = 1/3.b.d = 1/3.400.540 = 72 KN

$(Vc+Vs_{min}) < Vs \leq 3.Vc \rightarrow 252\ KN < 268{,}978\ KN \leq 540\ KN$

Tersedia tulangan sengkang P-10 → fy = 240 MPa.

$Av = \frac{1}{4}.\pi.D^2 = \frac{1}{4}.\pi.10^2.2 = 157{,}080\ mm^2.$

$$S \leq \frac{Av.fy.d}{\left(Vu/\phi - Vc\right)} = \frac{157{,}080.240.540}{(268{,}978-180)1000} = 228{,}885\ mm$$

$\leq d/2 = 540/2 = 270\ mm$

$\leq 600\ mm$

S pakai → P10-225.

4.6.2 Perencanaan tulangan torsi

Perencanaan dimensi balok portal B1a Induk (9 m) lantai 2

Tu = 11,783 KNm

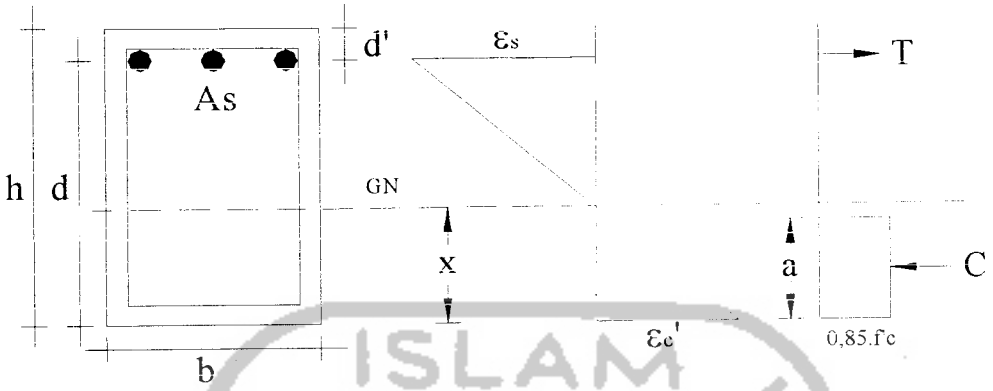
$\Sigma x^2.y = 400^2.600 = 96.10^6\ mm^2.$

$$\phi.\left(\frac{1}{20}.\sqrt{f'c}.\Sigma x^2.y\right) - 0{,}6.\left(\frac{1}{20}.\sqrt{25}.96.10^6\right) - 14{,}4.10^6\ Nmm - 14{,}4\ KNm$$

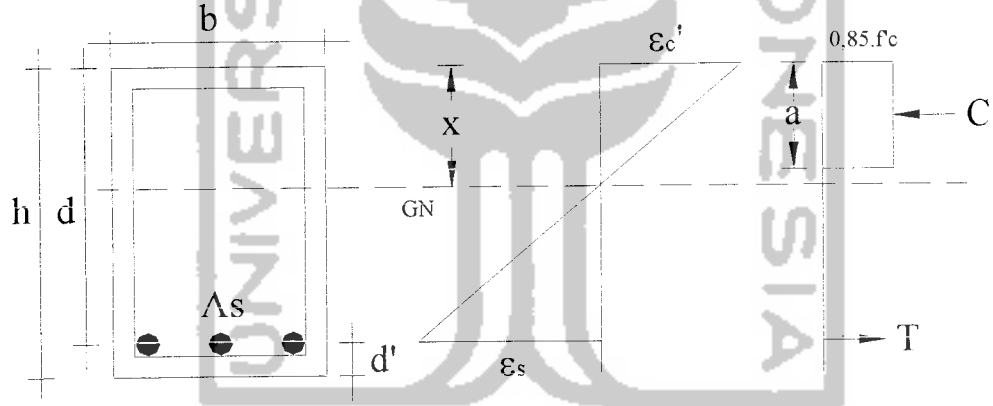
Kontrol :

$$Tu = 11{,}783\ KNm < \phi.\left(\frac{1}{20}.\sqrt{f'c}.\Sigma x^2.y\right) = 14{,}4\ KNm$$

→ Tulangan torsi diabaikan.



Gambar 4.11 Balok persegi tulangan sebelah pada tumpuan



Gambar 4.12 Balok persegi tulangan sebelah pada lapangan

4.7 Perencanaan Kolom

1. Data

   a. Ukuran kolom :

      b = 600 mm

      h = 600 mm

   b. Ukuran balok :

      b = 400 mm

      h = 600 mm

   c. f'c = 25 MPa

   d. Fy ulir = 400 MPa

   e. Fy polos = 240 MPa

   f. Ec = Eg = $4700.\sqrt{f'c}$ = $4700.\sqrt{25}$ = 23500 MPa = 23500000 KN/m^2

   g. Ic (Inersia kolom) = $1/12.b_k.h_k^3 = 1/12.0{,}6.0{,}6^3 = 0{,}011\ m^4$

   h. Ig (Inersia balok) = $1/12.b_b.h_b^3 = 1/12.0{,}4.0{,}6^3 = 0{,}007\ m^4$

   i. Tinggi kolom = 4,2 m

   j. Tinggi (h) pondasi = 2,1 m

   k. Cm = 1

   l. Ø = 0,6

   m. Øo = 1,25

   n. d' = 61 mm

   o. d = h-d' = 600-61 = 539 mm

2. Perencanaan kolom portal As 3 bentang C pada lantai 2

   a. Perhitungan arah X

      Lb_1 (panjang balok) = 6 m

      Lb_1' (bentang bersih balok) = 6-0,6 = 5,4 m

      Lb_2 (panjang balok) = 6 m

      Lb_2' (bentang bersih balok) = 6-0,6 = 5,4 m

      - Data momen diperoleh dari SAP 2000 seperti tercantum pada lampiran :

        MDa = 2,780 KNm MDb = 3,902 KNm

        MLa = 0,752 KNm MLb = 1,182 KNm

        MEa = 0,085 KNm MEb = 0,115 KNm

        Mua = 4,541 KNm Mub = 6,573 KNm

      - Data gaya aksial diperoleh dari SAP 2000 seperti tercantum pada lampiran :

        PDa = 490,028 KNm PDb = 525,651 KNm

        PLa = 137,201 KNm PLb = 137,201 KNm

        PEa = 7,387 KNm PEb = 7,387 KNm

        Pua = 807,556 KNm Pub = 850,303 KNm

      - Data gaya geser diperoleh dari SAP 2000 seperti tercantum pada lampiran :

        VDa = 6,657 KNm VDb = 6,657 KNm

        VLa = 0,896 KNm VLb = 0,896 KNm

        VEa = 56,781 KNm VEb = 56,781 KNm

- Perhitungan momen rencana (Mc)

  e = Mu / Pu = 4,541 / 807,556 = 0,006 m

  e_{min} = (1,5+0,03.h) cm = (1,5+0,03.60) cm = 3,3 cm = 0,033 m

  = 0,033 m > 0,006 m, maka dipakai e = 0,033 m

$$\psi_A = \frac{\Sigma\left(\frac{Ec.Ic}{Lc}\right)}{\Sigma\left(\frac{Eg.Ig}{Lg}\right)} = \frac{\Sigma\left(\frac{23500000.0,011}{4,2}\right)+\left(\frac{23500000.0,011}{4,2}\right)}{\Sigma\left(\frac{23500000.0,007}{5,4}\right)+\left(\frac{23500000.0,007}{5,4}\right)}$$

  = 2,021

$$\psi_B = \frac{\Sigma\left(\frac{Ec.Ic}{Lc}\right)}{\Sigma\left(\frac{Eg.Ig}{Lg}\right)} = \frac{\Sigma\left(\frac{23500000.0,011}{4,2}\right)+\left(\frac{23500000.0,011}{4,2}\right)}{\Sigma\left(\frac{23500000.0,007}{5,4}\right)+\left(\frac{23500000.0,007}{5,4}\right)}$$

  = 2,021

  Lihat nomogram unbranced frames, didapat k = 1,6

$$\frac{k.l}{r} = \frac{1,6.(4,2-0,6)}{(0,3.0,6)} = 32 > 22 \text{ dan} \leq 100$$, maka termasuk kolom panjang, sehingga dipakai konsep pembesaran momen.

$$EI_1 = \frac{\frac{1}{5}.(Ec.Ig)+Es.Ise}{1+\beta.d}$$

$$= \frac{\frac{1}{5}.(23500000.0,007)+(2.10^8.0,6.0,6.0,025.0,249^2)}{\left(1+\frac{588,034}{807,556}\right)}$$

  = 83615,744 KNm^2.

$$EI_2 = \frac{0,4.Ec.Ig}{1+\beta.d} = \frac{0,4.23500000.0,007}{\left(1+\frac{588,034}{807,556}\right)} = 38075,069\ KNm^2.$$

dipakai EI = 83615,744 KNm^2.

$$Pc = \frac{\mu^2.EI}{(k.l)^2} = \frac{\mu^2.83615,744}{(1,6.3,6)^2} = 24873,840 \text{ KN.}$$

$$\delta b = \frac{Cm}{1-\left(\frac{Pu}{\phi.Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{807,556}{0,6.24873,840}\right)} \geq 1$$

$$= 1,057 \geq 1$$

$$\sum Pu = 818,63 + 807,556 + 717,506 + 683,115 + 683,115 + 717,506 + 807,556 + 818,63$$

$$= 6053,614 \text{ KN}$$

$$\sum Pc = 24852,371 + 24893,873 + 28165,672 + 27927,915 + 27927,915 + 28165,672 + 24893,873 + 24852,371$$

$$= 211679,662 \text{ KN}$$

$$\delta s = \frac{Cm}{1-\left(\frac{\Sigma Pu}{\phi.\Sigma Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{6053,614}{0,6.211679,662}\right)} = 1,050 \geq 1$$

$Mu_1 = 1,2.MD + 1,6.ML = 1,2.2,780 + 1,6.0,752 = 4,539$ KNm.

$Mu_2 = ME = 0,085$ KNm

$Mcx = \delta b.Mu_1 + \delta s.Mu_2$

$= 1,057.4,539 + 1,050.0,085 = 4,887$ KNm

b. Perhitungan arah Y

Lb_1 (panjang balok) = 9 m

Lb_1' (bentang bersih balok) = 9-0,6 = 8,4 m

Lb_2 (panjang balok) = 9 m

Lb_2' (bentang bersih balok) = 9-0,6 = 8,4 m

- Data momen diperoleh dari SAP 2000 seperti tercantum pada lampiran :

  MDa = 12,199 KNm MDb = 15,763 KNm

  MLa = 1,649 KNm MLb = 2,112 KNm

  MEa = 99,031 KNm MEb = 139,451 KNm

  Mua = 117,830 KNm Mub = 164,306 KNm

- Data gaya aksial diperoleh dari SAP 2000 seperti tercantum pada lampiran :

  PDa = 490,028 KNm PDb = 525,651 KNm

  PLa = 137,201 KNm PLb = 137,201 KNm

  PEa = 7,387 KNm PEb = 7,387 KNm

  Pua = 807,556 KNm Pub = 850,303 KNm

- Data gaya geser diperoleh dari SAP 2000 seperti tercantum pada lampiran :

  VDa = 1,591 KNm VDb = 1,591 KNm

  VLa = 0,460 KNm VLb = 0,460 KNm

  VEa = 0,048 KNm VEb = 0,048 KNm

- Perhitungan momen rencana (Mc)

  e = Mu / Pu = 117,830 / 807,556 = 0,146 m

  e_{min} = (1,5+0,03.h) cm = (1,5+0,03.60) cm = 3,3 cm = 0,033 m

  = 0,033 m < 0,146 m, maka dipakai e = 0,146 m

$$\psi_A = \frac{\Sigma\left(\frac{Ec.Ic}{Lc}\right)}{\Sigma\left(\frac{Eg.Ig}{Lg}\right)} = \frac{\Sigma\left(\frac{23500000.0,011}{4,2}\right)+\left(\frac{23500000.0,011}{4,2}\right)}{\Sigma\left(\frac{23500000.0,007}{8,4}\right)+\left(\frac{23500000.0,007}{8,4}\right)}$$

$$= 3,143$$

$$\psi_B = \frac{\Sigma\left(\frac{Ec.Ic}{Lc}\right)}{\Sigma\left(\frac{Eg.Ig}{Lg}\right)} = \frac{\Sigma\left(\frac{23500000.0,011}{4,2}\right)+\left(\frac{23500000.0,011}{4,2}\right)}{\Sigma\left(\frac{23500000.0,007}{8,4}\right)+\left(\frac{23500000.0,007}{8,4}\right)}$$

$$= 3,143$$

Lihat nomogram unbranced frames, didapat k = 1,85

$\frac{k.l}{r} = \frac{1,85.(4,2-0,6)}{(0,3.0,6)} = 37 > 22$ dan ≤ 100, maka termasuk kolom panjang, sehingga dipakai konsep pembesaran momen.

$$EI_1 = \frac{\frac{1}{5}.(Ec.Ig)+Es.Ise}{1+\beta.d}$$

$$= \frac{\frac{1}{5}.(23500000.0,007)+\left(2.10^8.0,6.0,6.0,025.0,249^2\right)}{\left(1+\frac{588,034}{807,556}\right)}$$

$$= 83615,744 \text{ KNm}^2.$$

$$EI_2 = \frac{0,4.Ec.Ig}{1+\beta.d} = \frac{0,4.23500000.0,007}{\left(1+\frac{588,034}{807,556}\right)} = 38075,069 \text{ KNm}^2.$$

dipakai EI = 83615,744 KNm^2.

$$Pc = \frac{\mu^2.EI}{(k.l)^2} = \frac{\mu^2.83615,744}{(1,85.3,6)^2} = 18605,414 \text{ KN.}$$

$$\delta b = \frac{Cm}{1-\left(\frac{Pu}{\phi.Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{807,556}{0,6.18605,414}\right)} \geq 1$$

$$= 1,078 \geq 1$$

$$\sum Pu = 807,556 + 923,285 + 919,579$$

$$= 2650,420 \text{ KN}$$

$$\sum Pc = 18620,399 + 10425,896 + 10429,107$$

$$= 39475,402 \text{ KN}$$

$$\delta s = \frac{Cm}{1-\left(\frac{\Sigma Pu}{\phi.\Sigma Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{2650,420}{0,6.39475,402}\right)} \geq 1$$

$$= 1,126 \geq 1$$

$Mu_1 = 1,2.MD + 1,6.ML = 1,2.12,199 + 1,6.1,649 = 17,277$ KNm.

$Mu_2 = ME = 99,031$ KNm

$Mcy = \delta b.Mu_1 + \delta s.Mu_2$

$= 1,078.17,277 + 1,126.99,031 = 130,134$ KNm

3. Analisis gaya aksial dan momen akibat balok

Data momen diperoleh dari SAP 2000 seperti tercantum pada lampiran :

MDa = 12,199 KNm MDb = 15,763 KNm

MLa = 1,649 KNm MLb = 2,112 KNm

MEa = 99,031 KNm MEb = 139,451 KNm

Mua = 117,830 KNm Mub = 164,306 KNm

Data gaya aksial diperoleh dari SAP 2000 seperti tercantum pada lampiran:

PDa = 490,028 KNm PDb = 525,651 KNm

PLa = 137,201 KNm PLb = 137,201 KNm

PEa = 7,387 KNm PEb = 7,387 KNm

Pua = 807,556 KNm Pub = 850,303 KNm

hk = 4,2 m

hk' = 4,2 – h balok

= 4,2 – 0,6 = 3,6 m

Rv = 1 (jumlah lantai diatasnya; n = 1)

Øo = 1,25

$Mnak,bx_{kiri}$ = 443,851 KNm

$Mnak,bx_{kanan}$ = 234,175 KNm

$Mnak,by_{kiri}$ = 587,798 KNm

$Mnak,by_{kanan}$ = 306,789 KNm

$$Nuk_{1x} = 0,7.Rv.\phi o.\left(\frac{Mnak,bx_{kiri}}{lbx_{kiri}} + \frac{Mnak,bx_{kanan}}{lbx_{kanan}}\right) + 1,05.N_{g.k}$$

Ng = PD + PL

$$Nuk_{1x} = 0,7.1.1,25.\left(\frac{443,851}{6} + \frac{234,175}{6}\right) + 1,05.(490,028 + 137,201)$$

$$= 757,469 \text{ KN}$$

$$Nuk_{1y} = 0,7.Rv.\phi o.\left(\frac{Mnak,by_{kiri}}{lby_{kiri}} + \frac{Mnak,by_{kanan}}{lby_{kanan}}\right) + 1,05.N_{g.k}$$

$N_{g,k}$ = PD + PL

$$Nuk_{1y} = 0,7.1.1,25.\left(\frac{587,798}{9} + \frac{306,789}{9}\right) + 1,05.(490,028 + 137,201)$$

$$= 745,564 \text{ KN}$$

$$Nuk_{2x} = Nuk_{2y} = 1,05.\left(P_D + P_L + \left(4/k\right)P_E\right)$$

$$= 1,05.\left(490,028 + 137,201 + \left(4/1\right)7,387\right)$$

$$= 689,616 \text{ KN}$$

dipakai Nuk minimum , yaitu $Nuk_{2x} = Nuk_{2y} = 689,616$ KN

$Mmaks_{atas} = 117,830$ KNm

$Mmaks_{bawah} = 164,306$ KNm

$$\alpha_{k,atas} = \frac{Mmaks_{atas}}{Mmaks_{atas} + Mmak_{bawah}} = \frac{117,830}{117,830 + 164,306} = 0,418$$

$\omega_d = 1,3$

$$Muk_1 = \left(\frac{hk}{hk'}.0,7.\omega_d.\alpha_k.\phi o.\left(\frac{lbx_{ki}}{\ln,bx_{ki}}.Mnak,bx_{ki} + \frac{lbx_{ka}}{\ln,b_{ka}}.Mnak,bx_{ka}\right)\right) +$$
$$\left(\frac{hk}{hk'}.0,7.\omega_d.\alpha_k.\phi o.0,3.\left(\frac{lby_{ki}}{\ln,by_{ki}}.Mnak,by_{ki} + \frac{lby_{ka}}{\ln,by_{ka}}.Mnak,b_{ka}\right)\right)$$

$$= \left(\frac{4,2}{3,6}.0,7.1,3.0,418.1,25.\left(\frac{6}{5,4}.443,851+\frac{6}{5,4}.234,175\right)\right)+$$
$$\left(\frac{4,2}{3,6}.0,7.1,3.0,418.1,25.0,3.\left(\frac{9}{8,4}.587,798+\frac{9}{8,4}.306,789\right)\right)$$

$= 577,413$ KNm

$$\text{Muk}_2 = \left(\frac{hk}{hk'}.0,7.\omega_d.\alpha_k.\phi o.0,3.\left(\frac{lbx_{ki}}{\ln,bx_{ki}}.Mnak,bx_{ki}+\frac{lby_{ka}}{\ln,b_{ka}}.Mnak,bx_{ka}\right)\right)+$$
$$\left(\frac{hk}{hk'}.0,7.\omega_d.\alpha_k.\phi o.\left(\frac{lby_{ki}}{\ln,by_{ki}}.Mnak,by_{ki}+\frac{lby_{ka}}{\ln,by_{ka}}.Mnak,b_{ka}\right)\right)$$

$$= \left(\frac{4,2}{3,6}.0,7.1,3.0,418.1,25.0,3.\left(\frac{6}{5,4}.443,851+\frac{6}{5,4}.234,175\right)\right)+$$
$$\left(\frac{4,2}{3,6}.0,7.1,3.0,418.1,25.\left(\frac{9}{8,4}.587,798+\frac{9}{8,4}.306,789\right)\right)$$

$= 657,064$ KNm

$$\text{Muk}_3 = 1,05.\left(M_{DK}+M_{LK}+\left(4/k\right)M_{EK}\right)$$

$$= 1,05.\left(12,199+1,649+\left(4/1\right)99,031\right)$$

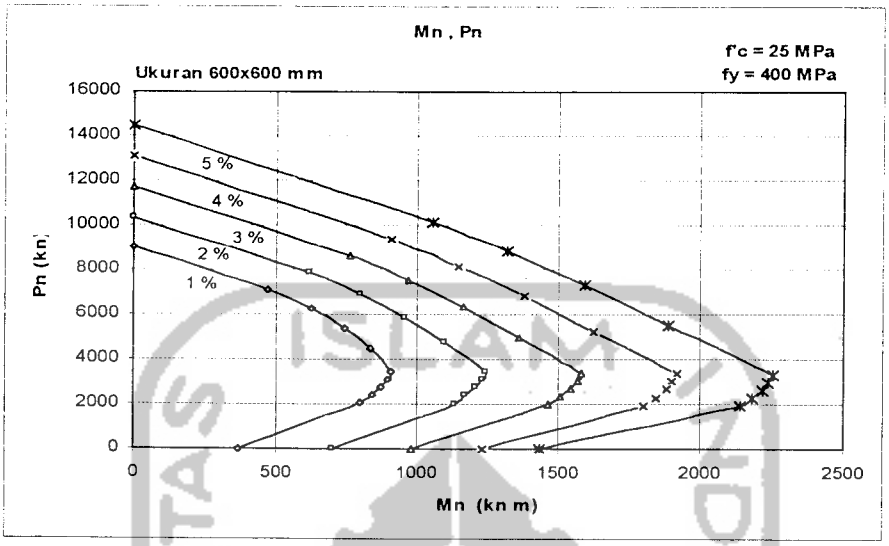
$= 430,471$ KNm

Dipakai Muk minimum, yaitu $\text{Muk}_3 = 430,471$ KNm.

Bandingkan Mcx = 4,887 KNm; Mcy = 130,134 KNm dan Muk pakai = 430,471 KNm, sehingga dipakai yang terbesar, yaitu Muk terpakai = 430,471 KNm.

$$\text{Mn} = Mu/\phi = 430,471/0,6 = 717,452 \text{ KNm}$$

$$\text{Pn} = Nu/\phi = 689,616/0,6 = 1149,360 \text{ KN}$$



Gambar 4.14 Grafik Mn-Pn kolom

Dari grafik Mn – Pn, didapat luas tulangan total (Ast) :

Ast < 1,2 %.b.h = 1,2%.600.600 = 4320 mm^2 → diambil Ast = 1,2%.b.h

As perlu = Ast / 2

= 4320 / 2 = 2160 mm^2

dipakai 6D22 → As ada = 2280,798 mm^2 > As perlu ..... OK !

As = As' = 2280,798 mm^2

Cek eksentrisitas balance (e_b) :

$$Xb = \frac{600.d}{600 + fy} = \frac{600.540}{600 + 400} = 324 \text{ mm}$$

$$fs' = \frac{Xb - d'}{Xb}.600 = \frac{324 - 60}{324}.600 = 488{,}889 \text{ MPa} > fy = 400 \text{ MPa}$$

dipakai fs' = fy = 400 MPa

$Cc = 0{,}85.f'c.b.\beta.Xb = (0{,}85.25.600.0{,}85.324).10^{-3} = 3511{,}350$ KN

$Cs = As'.(fs' - 0{,}85.f'c) = 2280{,}798.(400 - 0{,}85.25).10^{-3} = 863{,}852$ KN

$Ts = As.fy = (2280,798.400).10^{-3} = 912,319$ KN

$Pnb = Cc + Cs - Ts$

$= 3511,35 + 863,852 - 912,319 = 3462,883$ KN

$$Mnb = Cc.\left(h/2 - \left(\frac{\beta.Xb}{2}\right)\right) + Cs.\left(h/2 - d'\right) + Ts.\left(d - h/2\right)$$

$$= (3511,35.\left(600/2 - \left(\frac{0,85.324}{2}\right)\right) + 863,852.\left(600/2 - 60\right) +$$

$$912,319.\left(540 - 600/2\right)).10^{-3}$$

$= 996,173$ KNm

$$eb = \frac{Mnb}{Pnb} = \frac{996,173}{3462,883} = 0,288 \text{ m}$$

$$e = \frac{Mu/\phi}{Nu/\phi} = \frac{717,452}{1149,360} = 0,624 \text{ m}$$

karena e > eb → kolom mengalami patah tarik.

Kontrol tegangan pada daerah tarik :

$$Pn = 0,85.f'c.b.d.\left[(-\rho) + 1 - \frac{e'}{d} + \sqrt{\left(\left(1 - \frac{e'}{d}\right)^2 + 2.\rho.\left((m-1)\left(1 - \frac{d'}{d}\right)\right) + \frac{e'}{d}\right)}\right]$$

$$e = \frac{Mu/\phi}{Nu/\phi} = \frac{717,452}{1149,360} = 0,624 \text{ m} = 624 \text{ mm}$$

$$e' = e + \left(d - \frac{h}{2}\right) = 624 + \left(540 - \frac{600}{2}\right) = 864 \text{ mm}$$

$$\rho = \frac{As}{b.d} = \frac{2280,798}{600.540} = 0,007$$

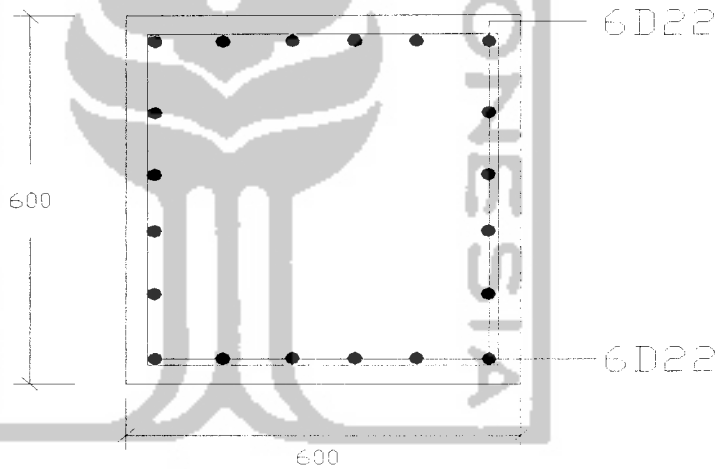
$$Pn = 0{,}85.25.600.540.\left[(-0{,}007)+1-\frac{864}{540}+\sqrt{\left(1-\frac{864}{540}\right)^2+\left(2.0{,}007.(18{,}824-1)\left(1-\frac{60}{540}\right)\right)+\left(\frac{864}{540}\right)}\right]$$

$$= 5990{,}605 \text{ KN} > Pn = {}^{Nu}\!/_{\phi} = {}^{689{,}616}\!/_{0{,}6} = 1149{,}360 \text{ KN}.....\text{OK !}$$

$$Mn = Pn.e$$

$$= 5990{,}605.0{,}624$$

$$= 3738{,}138 \text{ KNm} > {}^{Mu}\!/_{\phi} = {}^{430{,}471}\!/_{0{,}6} = 717{,}452 \text{ KNm}.....\text{OK!}$$



Gambar 4.15 Penampang kolom dengan tulangan

4. Perencanaan tulangan geser kolom

   - Data gaya geser diperoleh dari SAP 2000 seperti tercantum pada lampiran :

     VDa = 6,657 KNm VDb = 6,657 KNm

     VLa = 0,896 KNm VLb = 0,896 KNm

     VEa = 56,781 KNm VEb = 56,781 KNm

- Perhitungan tulangan geser kolom

  Mu,k_{atas} = 430,471 KNm

  Mu,k_{bawah} = 576,910 KNm

  hk' = 3,6 m

  $$Vuk_1 = \frac{Mu,k_{atas} + Mu,k_{bawah}}{hk'}$$

  $$= \frac{430,471 + 576,910}{3,6} = 279,828 \text{ KN}$$

  $$Vuk_2 = 1,05.\left(V_{Dk} + V_{Lk} + {}^{4}\!/_{k}.V_{Ek}\right)$$

  $$= 1,05.\left(6,657 + 0,896 + {}^{4}\!/_{1}.56,781\right) = 246,411 \text{ KN}$$

  Vu pakai adalah nilai terkecil antara Vuk_1 dan Vuk_2, sehingga didapat

  Vuk pakai = 246,411 KN

  $$Vs = \frac{Vuk_{pakai}}{\phi} = \frac{246,411}{0,6} = 410,685 \text{ KN}$$

  Daerah sendi plastis

  Kekutan beton dalam menahan gaya geser dinggap o (Vc=0).

  Dipakai tulangan geser P-10 mm, $Av = 3.1/4.\pi.10^2 = 235,620 \text{ mm}^2$

  $$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{235,620.240.540}{410,685.10^3} = 74,355 \text{ mm}$$

  $$< {}^{d}\!/_{4} = {}^{540}\!/_{4} = 135 \text{ mm}$$

  $$< 8.Ø_{tul.pokok} = 8.22 = 176 \text{ mm}$$

  $$< 100 \text{ mm}$$

  Digunakan sengkang **3P10-70 mm**.

Daerah diluar sendi plastis

$$Vc = \left(1 + \frac{P_{u,k}}{14.Ag}\right).\frac{1}{6}.\sqrt{f'c}.b.d \rightarrow \text{Pu,k} = 807{,}556 \text{ KN}$$

$$= \left(1 + \frac{807{,}556.10^3}{14.600.600}\right).\frac{1}{6}.\sqrt{25}.600.540 = 313{,}262 \text{ KN}$$

$$= 313{,}262 \text{ KN} < \frac{Vuk_{pakai}}{\phi} = 410{,}685 \text{ KN, maka perlu tulangan geser}$$

$$Vs = \frac{Vuk_{pakai}}{\phi} - Vc = 410{,}685 - 313{,}262 = 97{,}423 \text{ KN}$$

Dipakai tulangan geser P-10 mm, maka :

$$Av = 2.1/4.\pi.10^2 = 157{,}080 \text{ mm}^2$$

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157{,}08.240.540}{97{,}423.10^3} = 208{,}961 \text{ mm}$$

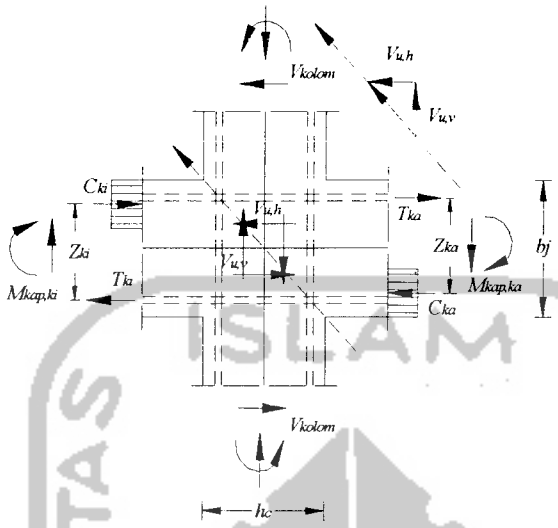
$$< d/4 = 540/4 = 135 \text{ mm}$$

$$< 8.\varnothing_{tul.pokok} = 8.22 = 176 \text{ mm}$$

$$< 100 \text{ mm}$$

Digunakan sengkang **P10-100 mm**.

4.8 Pertemuan Balok Kolom



Gambar 4.16 Join balok-kolom dalam

a. Perhitungan gaya-gaya dalam

1. Arah X

bj = bc = 600 mm

= bb + 0,5.hc = 400 + 0,5.600 = 700 mm

bj pakai = 600 mm

hc = 600 mm

$Mnak,bx_{ki}$ = 234,175 KNm

$Mnak,bx_{ka}$ = 443,851 KNm

$Mnak,by_{ki}$ = 306,789 KNm

$Mnak,by_{ka}$ = 587,798 KNm

$Mkap,bx_{ki}$ = 1,25. $Mnak,bx_{ki}$ = 1,25.234,175 = 292,719 KNm

$Mkap,bx_{ka}$ = 1,25. $Mnak,bx_{ka}$ = 1,25.443,851 = 554,814 KNm

$Mkap,by_{ki}$ = 1,25. $Mnak,by_{ki}$ = 1,25.306,789 = 383,486 KNm

$Mkap,by_{ka}$ = 1,25. $Mnak,by_{ka}$ = 1,25.587,798 = 734,748 KNm

$$V_{kol,x} = \frac{0,7.\left(\Sigma\, {}^{l_x}\!\ln_x .Mkap,b_x + 0,3.\, {}^{l_y}\!\ln_y .Mkap,by\right)}{\frac{1}{2}.(hk,a + hk,b)}$$

$$V_{kol,x} = \left(0,7.\left[\left({}^{6}\!/_{5,4}.292,719 + {}^{6}\!/_{5,4}.554,814\right)\right]\right.$$

$$\left. + 0,3\left[\left({}^{9}\!/_{8.4}.383,486 + {}^{9}\!/_{8,4}.734,748\right)\right]\right) \Big/ \frac{1}{2}.(4,2+0)$$

$$= 433,712 \text{ KN}$$

$z_{ki} = z_{ka} = 0,9.d = 0,9.540 = 486 \text{ mm} = 0,486 \text{ m}$

$C_{ki,x} = T_{ki,x} = 0,7.\ Mkap,bx_{ki} / z_{ki}$

$= 0,7.292,719 / 0,486 = 421,612 \text{ KN}$

$C_{ka,x} = T_{ka,x} = 0,7.\ Mkap,bx_{ka} / z_{ka}$

$= 0,7.554,814 / 0,486 = 799,115 \text{ KN}$

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x} = 421,612+799,115-433,712 = 787,015 \text{ KN}$

Kontrol tegangan geser horizontal :

$$v_{jh,x} = \frac{V_{jh,x}}{bj.hc} \le 1,5.\sqrt{f'c}$$

$$v_{jh,x} = \frac{787,015}{0,6.0,6} = 2186,153 \text{ KN/m}^2$$

$v_{jh,x} = 2,186 \text{ N/mm}^2 \le 1,5.\sqrt{25} = 7,5 \text{ N/mm}^2$ ..... **OK !**

$Nu,k = 807,556 \text{ KN}$

$$\frac{Nu,k}{Ag} = \frac{807,556}{0,6.0,6} = 2243,211 \text{ KN/m}^2$$

$= 2,243 \text{ N/mm}^2 < 0,1.f'c = 2,5 \text{ MPa}$

$V_{ch,x} = 0$

$V_{sh,x} = V_{jh,x} - V_{ch,x} = 787,015 - 0 = 787,015$ KN

2. Arah Y

bj = bc = 600 mm

= bb + 0,5.hc = 400 + 0,5.600 = 700 mm

bj pakai = 600 mm

hc = 600 mm

$Mnak,bx_{ki} = 234,175$ KNm

$Mnak,bx_{ka} = 443,851$ KNm

$Mnak,by_{ki} = 306,789$ KNm

$Mnak,by_{ka} = 587,798$ KNm

$Mkap,bx_{ki} = 1,25.\ Mnak,bx_{ki} = 1,25.234,175 = 292,719$ KNm

$Mkap,bx_{ka} = 1,25.\ Mnak,bx_{ka} = 1,25.443,851 = 554,814$ KNm

$Mkap,by_{ki} = 1,25.\ Mnak,by_{ki} = 1,25.306,789 = 383,486$ KNm

$Mkap,by_{ka} = 1,25.\ Mnak,by_{ka} = 1,25.587,798 = 734,748$ KNm

$$V_{kol,y} = \frac{0,7.\left(0,3.\Sigma\, {}^{l_x}\!/_{\ln_x}.Mkap,b_x + {}^{l_y}\!/_{\ln_y}.Mkap,b_y\right)}{{}^{1}\!/_{2}.(hk,a + hk,b)}$$

$$V_{kol,y} = \left(0,7.\left[0,3.\left({}^{6}\!/_{5,4}.292,719 + {}^{6}\!/_{5,4}.554,814\right)\right]\right.$$

$$\left. + \left[\left({}^{9}\!/_{8.4}.383,486 + {}^{9}\!/_{8,4}.734,748\right)\right]\right) \Big/ \frac{1}{2}.(4,2+0)$$

= 569,628 KN

$z_{ki} = z_{ka} = 0,9.d = 0,9.540 = 486 \text{ mm} = 0,486 \text{ m}$

$C_{ki,y} = T_{ki,y} = 0,7.\ Mkap,by_{ki} / z_{ki}$

$= 0,7.383,486 / 0,486 = 552,346 \text{ KN}$

$C_{ka,y} = T_{ka,y} = 0,7.\ Mkap,by_{ka} / z_{ka}$

$= 0,7.734,748 / 0,486 = 1058,279 \text{ KN}$

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y}$

$= 552,346+1058,279-569,628 = 1040,997 \text{ KN}$

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{bj.hc} \leq 1,5.\sqrt{f'c}$$

$$v_{jh,y} = \frac{1040,997}{0,6.0,6} = 2891,658 \text{ KN/m}^2$$

$v_{jh,y} = 2,892 \text{ N/mm}^2 < 1,5.\sqrt{25} = 7,5 \text{ N/mm}^2$ ..... **OK !**

$Nu,k = 807,556 \text{ KN}$

$$\frac{Nu,k}{Ag} = \frac{807,556}{0,6.0,6} = 2243,211 \text{ KN/m}^2$$

$= 2,243 \text{ N/mm}^2 < 0,1.f'c = 2,5 \text{ MPa}$

$V_{ch,y} = 0$

$V_{sh,y} = V_{jh,y} - V_{ch,y}$

$= 1040,997 - 0 = 1040,997 \text{ KN}$

b. Penulangan geser horizontal

$V_{sh,maks} = V_{sh,y} = 1040,997 \text{ KN}$

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{1040997}{240} = 4337,488 \text{ mm}^2$$

Digunakan sengkang rangkap Ø 10 mm, dengan Av = 314,159 mm^2

Jumlah lapis sengkang = $\frac{4337,488}{314,159}$ = 13,807 = 14 lapis.

c. Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}}.V_{jh,mak}.\left(0,6 + \frac{Pu,k}{Ag.f'c}\right)$$

$$= 1.1040,997.10^3.\left(0,6 + \frac{807,556.10^3}{600.600.25}\right)$$

= 718005,242 N = 718,005 KN

$$V_{jv} = \frac{bj}{hc}.v_{jh,mak} = \frac{0,6}{0,6}.1040,997 = 1040,997 \text{ KN}$$

$V_{sv} = V_{jv} - V_{cv}$ = 1040,997 - 718,005 = 322,992 KN

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{322992}{240} = 1345,800 \text{ mm}^2$$

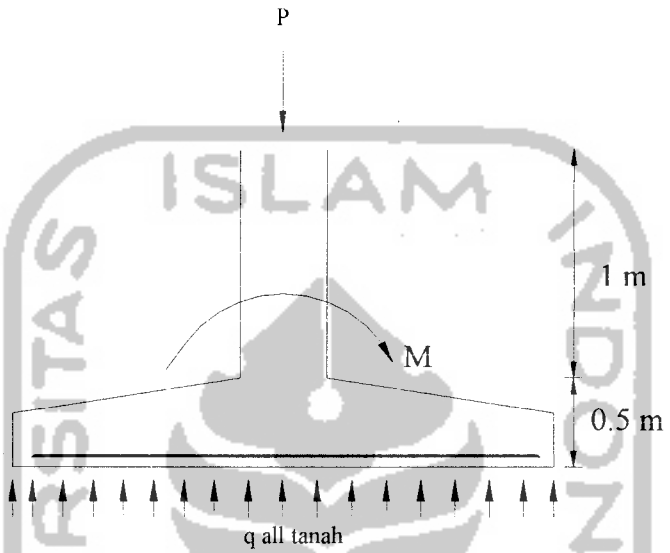
Digunakan sengkang rangkap Ø 10 mm, dengan Av = 314,159 mm^2

Jumlah lapis sengkang = $\frac{1345,8}{314,159}$ = 4,284 = 5 lapis.

4.9 Perencanaan Pondasi

4.9.1 Perencanaan dimensi pondasi (P1)

1. Tinjauan terhadap beban tetap



Gambar 4.17 Rencana pondasi

Data – data :

σ_{tanah} = 150 KN/m² P = 1571,922 KN/m³

f'c = 25 Mpa Mx = 4,748 KNm

fy = 400 Mpa My = 7,183 KNm

γb_{tanah} = 18 KN/m³ Asumsi tebal pondasi (h) = 500 mm

γ_{beton} = 24 KN/m³ Dimensi kolom = 600/600 mm

Direncanakan memakai pondasi telapak

$\sigma_{netto\ tanah} = \sigma_{tanah} - \sum (h \cdot \gamma_{beton}) - \sum (h \cdot \gamma_{tanah})$

$= 150 - (0{,}5 \cdot 24) - (1 \cdot 18)$

$= 120$ KN/m²

Dimensi pelat pondasi (terdapat momen yang bekerja pada arah x dan y) :

$$\sigma_{\text{netto tanah}} = \frac{P}{Aperlu} + \frac{My}{1/6.Bx^2.By} + \frac{Mx}{1/6.By^2.Bx}$$

dicoba dengan nilai B = 3,7 m

$$A_{perlu} = \frac{P}{\sigma.netto\,\text{tan}\,ah - \left(\frac{My}{1/6.Bx^2.By}\right) - \left(\frac{Mx}{1/6.By^2.Bx}\right)}$$

$$= \frac{1571,922}{120 - \left(\frac{4,748}{1/6.3,7^2.3,7}\right) - \left(\frac{7,183}{1/6.3,7^2.3,7}\right)} = 13,255\ m^2$$

Digunakan penampang bujursangkar dengan :

$B = N = \sqrt{13,255} = 3,641\ m \rightarrow B_{ada} = N_{ada} = 3,7\ m$

Luas penampang pelat pondasi :

$A_{ada} = B_{ada} \times N_{ada} = 3,7 \times 3,7 = 13,69\ m^2 > A_{perlu} = 13,255\ m^2$ .....Ok!

Tegangan kontak yang terjadi di dasar pondasi :

$$\sigma_{\text{netto tanah}} = \frac{P}{Aada} + \frac{My}{1/6.N^2.B} + \frac{Mx}{1/6.B^2.N}$$

$$= \frac{1571,922}{13,69} + \frac{7,183}{1/6.3,7^2.3,7} + \frac{4,748}{1/6.3,7^2.3,7}$$

$= 116,236\ KN/m^2 < \sigma_{\text{netto tanah}} = 120\ KN/m^2$ .....Aman !

Perencanaan tebal pondasi telapak (syarat kuat geser) :

Tebal selimut beton (Pb) = 70 mm

Ø tulangan pokok = 22 mm

$d = hp - Pb - \frac{1}{2} . \varnothing_{\text{tul. pokok}} = 500 - 70 - \frac{1}{2} .22 = 419\ m$

2. Tinjauan terhadap beban sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{4{,}748}{1571{,}922} = 0{,}003 \text{ m}$$

$$ey = \frac{My}{P} = \frac{7{,}183}{1571{,}922} = 0{,}005 \text{ m}$$

Kontrol tegangan yang terjadi :

$$\sigma = \frac{P}{(B.(N - 2.ex)) + (N.(B - 2.ey))}$$

$$= \frac{1571{,}922}{(3{,}7.(3{,}7 - 2.0{,}003)) + (3{,}7.(3{,}7 - 2.0{,}005))}$$

$= 57{,}529 \text{ KN/m}^2 < 1{,}5 \cdot \sigma_{netto} = 1{,}5 \cdot 120 = 180 \text{ KN/m}^2$ .....Aman !

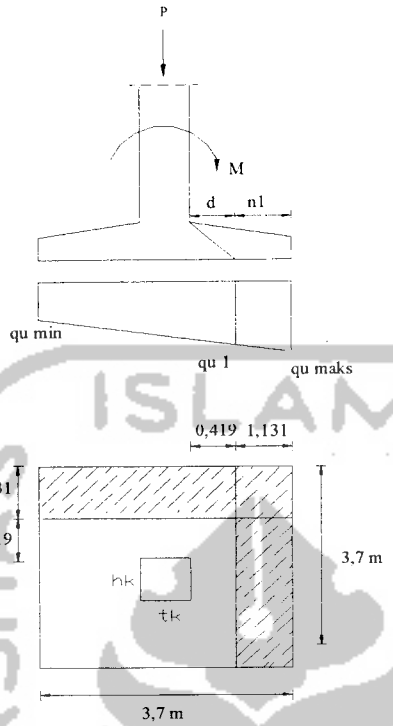
4.9.2 Perencanaan geser satu arah

Ditinjau dari arah momen terbesar :

Pu = 1996,976 KN

Mux = 157,143 KNm

Muy = 22,296 KNm



Gambar 4.18 Pondasi dengan geser satu arah

$$n_1 = \frac{Lp - tk - 2.d}{2} = \frac{3,7 - 0,6 - 2.0,419}{2} = 1,131 \text{ m}$$

Arah X

- Tegangan kontak yang terjadi :

$$qux = \frac{Pu}{Aada} \pm \frac{Mux}{1/6.B^2.N} = \frac{1996,976}{13,69} \pm \frac{157,143}{1/6.3,7^2.3,7}$$

$qux_{max} = 164,485 \text{ KN/m}^2$

$qux_{min} = 127,257 \text{ KN/m}^2$

$$qu_m = \frac{(Lp - n1).qux_{max} + n1.qux_{min}}{Lp}$$

$$= \frac{(3,7 - 1,131).164,485 + 1,131.127,257}{3,7} = 153,105 \text{ KN/m}^2$$

$qux_{terjadi} = ½ .(qux_{max} + qu_m) = ½ .(164,485 + 153,105)$

$= 158,795 \text{ KN/m}^2$

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

$Vu = qux_{terjadi} . n_1 . L = 158,795.1,131.3,7 = 664,510 \text{ KN}$

$Vu/\phi = 664,510/0,6 = 1107,518 \text{ KN}$

- Kekuatan beton menahan geser

$Vc = 1/6 . \sqrt{f'c} . L.d = 1/6 . \sqrt{25} . 3,7.0,419.1000$

$= 1291,917 \text{ KN} \geq Vu/\phi = 1107,518 \text{ KN}$ ..... Aman !

Arah Y

- Tegangan kontak yang terjadi :

$$quy = \frac{Pu}{Aada} \pm \frac{Muy}{1/6.B^2.N} = \frac{1996,976}{13,69} \pm \frac{22,296}{1/6.3,7^2.3,7}$$

$quy_{max} = 148,512 \text{ KN/m}^2$

$quy_{min} = 143,230 \text{ KN/m}^2$

$$qu_m = \frac{(Lp - n1).quy_{max} + n1.quy_{min}}{Lp}$$

$$= \frac{(3,7 - 1,131).148,512 + 1,131.143,230}{3,7} = 146,898 \text{ KN/m}^2$$

$quy_{terjadi} = ½ .(quy_{max} + qu_m) = ½ .(148,512 + 146,898)$

$= 147,705 \text{ KN/m}^2$

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

Vu = $quy_{terjadi}$. n_1. L = 147,705. 1,131 .3,7 = 618,101 KN

Vu/ϕ = $618{,}101/0{,}6$ = 1030,168 KN

- Kekuatan beton menahan geser

Vc = $1/6.\sqrt{f'c}.L.d$ = $1/6.\sqrt{25}.3{,}7.0{,}419.1000$

= 1291,917 KN ≥ Vu/ϕ = 1030,168 KN .....Aman !

4.9.3 Perencanaan geser dua arah

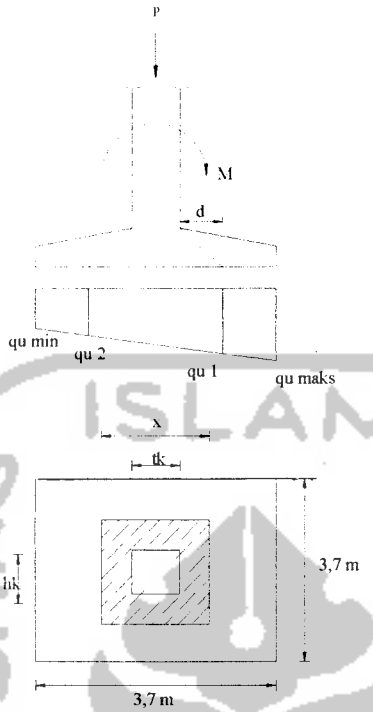
Ditinjau pada arah momen terbesar :

x = hk + d = 600 + 419

= 1019 mm = 1,019 m

y = tk + d = 600 + 419

= 1019 mm = 1,019 m



Gambar 4.19 Pondasi dengan geser dua arah

- Tegangan kontak yang terjadi :

$$qu = \frac{Pu}{Aada} \pm \frac{My}{1/6.Bx^2.By} \pm \frac{Mx}{1/6.By^2.Bx}$$

$$= \frac{1996,976}{13,69} \pm \frac{22,296}{1/6.3,7^2.3,7} \pm \frac{157,143}{1/6.1,8^2.1,8}$$

$qu_{max} = 167,126$ KN/m²

$qu_{min} = 124,616$ KN/m²

$qu_T = ½.(qu_{max} + qu_{min}) = ½ .(167,126 + 124,616) = 145,871$ KN/m²

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

$Vu = qu_T . ((Pp . Lp) - (x,y))$

$= 145,871.((3,7.3,7) - (1,019.1,019)) = 1845,509$ KN

$$Vu/\phi = 1845,509/0,6 = 3075,848 \text{ KN}$$

- Kekuatan beton menahan geser

$$\beta c = \frac{sisipanjangtapak}{sisipendektapak} = \frac{Pp}{Lp} = \frac{3,7}{3,7} \geq 1,0$$

$bo = 2.(x+y) = 2.((h_k+d)+(b_k+d))$

$= 2.(1019 + 1019) = 4076$ mm

$$Vc_1 = \left(1 + \frac{2}{\beta_c}\right)\left(2.\sqrt{f'c}\right)bo.d$$

$$= \left(1 + \frac{2}{1}\right)\left(2.\sqrt{25}\right)4076.419.10^{-3} = 51235,230 \text{ KN}$$

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

$$= 4.\sqrt{25}.4076.419.10^{-3} = 34156,880 \text{ KN}$$

- Kontrol gaya geser

Digunakan nilai terkecil dari Vc_1 dan Vc_2, yaitu Vc = 34156,880 KN

$$Vc = 34156,880 \text{ KN} \geq \frac{Vu}{\phi} = 3075,848 \text{ KN} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{Aman !}$$

4.9.4 Kuat tumpuan pondasi

- Kuat tumpuan pondasi

Luas pelat pondasi (A_2) = B . N = 3,7 . 3,7 = 13,69 m²

Luas penampang kolom (A_1) = hk . tk = 0,6 . 0,6 = 0,36 m²

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{13,69}{0,36}} = 6,167 > 2 \text{ (jika lebih besar dari 2, dipakai nilai 2)}$$

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

$= 0,7.(0,85 .25. 360000. 2). 10^{-3} = 10710$ KN

- Kuat tumpuan kolom

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

$= 0,7.(0,85 .25. 360000). 10^{-3} = 5355$ KN

- Kontrol kuat tumpuan

$\Phi . Pn_{pondasi} = 10710$ KN $> \Phi . Pn_{kolom} = 5355$ KN.....Aman !!

4.9.5 Perencanaan tulangan lentur telapak pondasi

- Momen yang terjadi :

$$l = \frac{Lp - tk}{2} = \frac{3,7 - 0,6}{2} = 1,55 \text{ m}$$

$qu_{maks} = 167,126$ KN/m²

$Mu = 0,5 . qu_{maks} . l^2 = 0,5 . 167,126.1,55^2$

$= 200,760$ KNm

$$\frac{Mu}{\phi} = \frac{200,760}{0,8} = 250,950 \text{ KNm}$$

- Digunakan tulangan Ø 22 mm, maka :

$A1.Ø = ¼ . \pi . D^2 = ¼ . \pi.22^2 = 380,286$ mm²

Dipakai tebal pelat pondasi (h) = 500 mm, selimut beton (Pb) = 70 mm

$d = h - Pb - ½.Øtul = 500 - 70 - ½. 22 = 419$ mm

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.25} = 18,824$$

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

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{250,950.10^6}{1000.419^2} = 1,429 \text{ mm}^2$$

- Rasio tulangan :

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

$$\rho b = \frac{0,85.f'c}{fy}.\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0,85.25}{400}.0,85.\left(\frac{600}{600+400}\right) = 0,027$$

$$\rho_{maks} = 0,75 \;.\; \rho b = 0,75 \;.\; 0,027 = 0,020$$

$$\rho_{perlu} = \frac{1}{m}.\left[1-\sqrt{1-\frac{2.m.Rn}{fy}}\right] = \frac{1}{18,824}.\left[1-\sqrt{1-\frac{2.18,824.1,429}{400}}\right]$$

$$= 0,004 < \rho_{maks} = 0,02$$

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

$$\rho_{terpakai} = \rho_{perlu} = 0,004$$

$$As_{perlu} = \rho_{terpakai} \;.\; b \;.\; d = 0,004.1000 \;.419 = 1676 \text{ mm}^2$$

- Jarak antar tulangan

$$s \leq \frac{Al.\phi.1000}{Asperlu} = \frac{380,286.1000}{1676} = 226,901 \text{ mm}$$

$$s \leq 2 \;.\; h = 2 \;.\; 500 = 1000 \text{ mm}$$

$$s \leq 250 \text{ mm}$$

→ Dipakai tulangan : D22 – 220 mm

$$As_{ada} = \frac{380,286.1000}{220} = 1728,571 \text{ mm}^2 > As_{perlu} = 1676 \text{ mm}^2$$

- Kontrol kapasitas lentur pelat pondasi

$$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{1728,571.400}{0,85.25.1000} = 32,538 \text{ mm.}$$

$Mn = As_{ada} \cdot fy \cdot (d - a/2) = 1728{,}571.400.(419 - 32{,}538/2).\ 10^{-6}.$

$= 278{,}460\ \text{KNm} > \dfrac{Mu}{\phi} = 250{,}950\ \text{KNm}$ .....Ok !

- Perencanaan tulangan bagi

  $As_{susut} = 0{,}002 .b .h = 0{,}002 .1000 .500 = 1000\ mm^2$

  Pakai tulangan P 12 → $A1.Ø = ¼ . \pi . D^2 = ¼ . \pi . 12^2 = 113{,}143\ mm^2$

  Jarak antar tulangan (x) = $\dfrac{A1.\phi.1000}{Asbagi} = \dfrac{113{,}143.1000}{1000}$

  $= 113{,}143\ mm^2$

  → **Dipakai tulangan : P 12 – 110 mm**