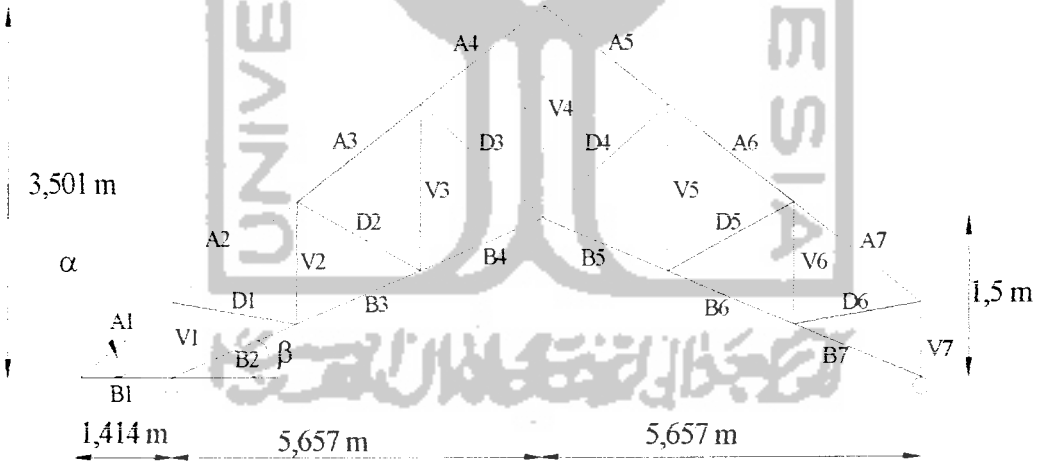
BAB IV

PERENCANAAN KONSTRUKSI

4.1 Perencanaan Atap

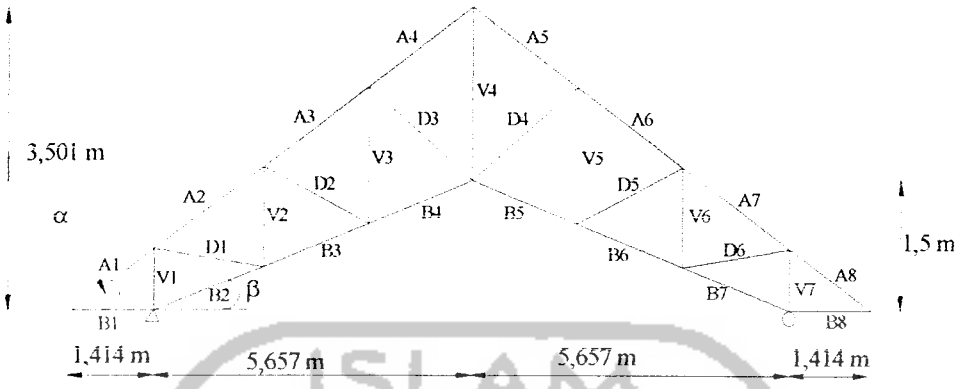
Pada perencanaan ini dipakai 4 macam rangka atap. Rangka atap direncanakan menggunakan profil baja. Di bawah ini gambar-gambar rangka atap yang direncanakan.

1. KK-1 $\alpha = 35°$ $\beta = 20{,}556°$



Gambar 4.1 Rangka kuda kuda KK-1

4. KK-4 $\alpha = 26{,}341°$ $\beta = 14{,}851°$



Gambar 4.4 Rangka kuda-kuda KK-4

4.1.1 Perencanaan Gording

1. Data-data

Jarak antar kuda-kuda maksimum = 4 m

Jenis baja profil A36 dengan :

Tegangan leleh (Fy) = 36 ksi = 2500 kg/cm^2

Kuat tarik (Fu) = 58 ksi = 4000 kg/cm^2

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

Kekuatan tarik (Fu) = 120 ksi = 8250 kg/cm^2

Tegangan Geser (Fv) = 30 ksi = 2070 kg/cm^2

2. Pembebanan gording

A. Beban tetap

- Berat penutup atap = 50 x 2,104 = 105,2 kg/m'
- Beban hidup = 20 x 2,104 = 42,08 kg/m'
- Beban gording = 10 kg/m' +

= 157,28 kg/m'

$q_{\perp} = q \cos \alpha = 157{,}28 \cos 26{,}341° = 140{,}949$ kg/m'

$q_{//} = q \sin \alpha = 157{,}28 \sin 26{,}341° = 69{,}787$ kg/m'

B. Beban angin

Pada daerah daratan w = 25 kg/cm^2 (PPIUG 1983 bab 4 pasal 4.2)

a. angin tekan (wt) untuk $\alpha < 65°$

diketahui $\alpha = 26{,}341°$

$C_1 = 0{,}02\ \alpha - 0{,}4 = 0{,}02 . 26{,}341 - 0{,}4 = 0{,}127$

$W_t = C_1 .$ w . jarak gording $= 0{,}127 . 25 . 2{,}104 = 6{,}671$ kg/m'

b. angin hisap

$C_2 = -0{,}4$

$Wh = C_2 .$ w . jarak gording $= -0{,}4 . 25 . 2{,}104 = -21{,}04$ kg/m'

C. Perhitungan momen

- akibat beban tetap

$Mmaks_{\perp} = 1/8 . q_{\perp} . b^2 = 1/8 . 140{,}949 . 4^2 = 295{,}898$ kgm $= 29589{,}8$ kgcm

$Mmaks_{//} = 1/32 . q_{//} . b^2 = 1/32 . 69{,}787 . 4^2 = 34{,}893$ kgm $= 3489{,}3$ kgcm

- akibat beban angin

$Mmaks_{\perp} = 1/8 . q_{\perp} . b^2 = 1/8 . 6{,}671 . 4^2 = 13{,}342$ kgm $= 1334{,}2$ kgcm

D. Dimensi gording

Dicoba profil 150x50x20x3,2 (Light Lip Channel)

A = 8,607 cm^2 Iy = 28,3 cm^4

W = 6,76 kg/m Sx = 37,4 cm^3

Ix = 280 cm^4 Sy = 8,19 cm^3

- Kontrol tegangan

-Akibat beban tetap dan angin

$$\frac{fbx}{0{,}66.fy}+\frac{fby}{0{,}75.fy}<1{,}25 \qquad \textbf{(3.1.1a)}$$

$$fbx=\frac{M\perp}{Sx}=\frac{29589{,}8+13{,}342}{37{,}4}=826{,}845\ \text{kg/cm}^2$$

$$fby=\frac{M//}{Sy}=\frac{3489{,}3}{8{,}19}=426{,}044\ \text{kg/cm}^2$$

$$\frac{826{,}845}{0{,}66.2500}+\frac{426{,}044}{0{,}75.2500}=0{,}728<1.25$$

- Akibat Beban Tetap

$$\frac{fbx}{0{,}66.fy}+\frac{fby}{0{,}75.fy}<1{,}0 \qquad \textbf{(3.1.1b)}$$

$$\frac{\left(\frac{29589{,}8}{37{,}4}\right)}{0{,}66.2500}+\frac{\left(\frac{3489{,}3}{8{,}19}\right)}{0{,}75.2500}=0{,}706<1{,}0$$

- Kontrol lendutan

$$\delta\perp=\frac{5}{384}.\frac{q_\perp.L^4}{E.I_x}\le\frac{L}{360} \qquad \textbf{(3.1.4)}$$

$$=\frac{5}{384}.\frac{1{,}4762.400^4}{2{,}1.10^6.280}=0{,}836\le\frac{400}{360}=1{,}111$$

$$\delta//=\frac{5}{384}.\frac{q_{//}.\left(\frac{L}{(a+1)}\right)^4}{E.I_y}\le\frac{L}{360} \qquad \textbf{(3.1.5)}$$

$$=\frac{5}{384}.\frac{0{,}69787.[400/(1+1)]^4}{2{,}1.10^6.28{,}3}=0{,}245\le 1{,}111$$

Profil 150x50x20x3,2 bisa digunakan

E. Perencanaan sagrod dan tierod

a. Beban sagrod

- berat penutup atap x (½.L/cosα) = 50 x (½ .14,142/cos 26,341)

  = 394,512 kg/m'

- beban hidupx(½.L/cosα) = 20x(½ .14,142/cos 26,341) =157,805 kg/m'

- jumlah gording satu sisi miring x berat gording = 4 x 6,76 = 27,04 kg/m'

P = 394,512 + 157,805 + 27,04 = 579,35 kg/m'

$P_{//}$ = P . sin α . Ss = 579,35 . sin 26,341 . 2 = 514,129 kg **(3.1.8)**

b. Dimensi sagrod

$$\text{Asagrod} = \frac{P//}{0{,}33Fu} = \frac{514{,}129}{0{,}33.4000} = 0{,}389 \text{ cm}^2 \qquad \textbf{(3.1.9)}$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4.0{,}389}{\pi}} = 0{,}704 \text{ cm} \rightarrow \text{dipakai Ø½ in} \qquad \textbf{(3.1.10)}$$

c. Dimensi tierod

Beban tierod : T = $P_{//}$. cos α = 514,129 . cos 26,341 = 460,746 kg

$$\text{Atierod} = T/0{,}33\ Fu = \frac{460{,}746}{0{,}33.4000} = 0{,}349 \text{ cm}^2 \qquad \textbf{(3.1.14)}$$

$$D = \sqrt{4.A/\pi} = \sqrt{\frac{4.0{,}349}{\pi}} = 0{,}667 \text{ cm} \qquad \textbf{(3.1.15)}$$

Tierod terpakai = D + 0,3 = 0,667 + 0,3 = 0,967 cm → dipakai Ø½ in

4.1.2 Perencanaan Kuda-kuda KK-4(Perhitungan KK-1s/dKK-3 di lamp.II-5)

L = 14,142 m α = 26,341° β = 14,851°

- Pembebanan

Taksiran kuda-kuda : $\left[10 \pm \left(\frac{L-12}{3}\right).5\right]$ jarak kuda-kuda

$$= \left[10 \pm \left(\frac{14{,}142 - 12}{3}\right).5\right].4 = 54{,}28 \text{ kg/m'}$$

a. Beban tetap :

- berat gording	= 6,76 kg/m'
- berat eternit + penggantung	= 18 kg/m'
- berat penutup atap	= 50 kg/m'
- beban hidup	= 20 kg/m'
- taksiran beban kuda-kuda	= 54,28 kg/m'

- Beban masing-masing joint

P1 = P9

Berat gording = 6,76 . 4	= 27,04 kg
Berat penutup atap = 50x4x0,5x1,578	= 157,8 kg
Beban hidup = 20x4x0,5x1,578	= 63,12 kg +
P1 = P9	= 248 kg

P2 = P8

Berat gording = 6,76 . 4	= 27,04 kg
Berat penutup atap = 50x4x(0,5.1,578+0,5.2,104)	= 368,2 kg
Beban hidup = 20x4x(0,5.1,578+0,5.2,104)	= 147,28 kg +
P2 = P8	= 543 kg

P3 = P7

Berat gording = 6,76 . 4	= 27,04 kg
Berat penutup atap = 50x4x(0,5.2,104+0,5.2,104)	= 420,8 kg
Beban hidup = 20x4x(0,5.2,104+0,5.2,104)	= 168,32 kg +

P3 = P7 = 617 kg

P4 = P6

Berat gording = 6,76 . 4 = 27,04 kg

Berat penutup atap = 50x4x(0,5.2,104+0,5.2,104) = 420,8 kg

Beban hidup = 20x4x(0,5.2,104+0,5.2,104) = 168,32 kg +

P3 = P7 = 617 kg

P5

Berat gording = 2x6,76x4 = 54,08 kg

Berat penutup atap = 4x2,104x50 = 420,8 kg

Beban hidup = 20x4x2,104 = 168,32 kg +

P5 = 644 kg

$P1^1$=$P9^1$

Berat eternit + plafont = 18x4x0,5x1,414 = 50,904 kg

Berat taksiran kuda-kuda = 54,28x0,5x1,414 = 38,376 kg +

$P1^1$=$P9^1$ = 90 kg

$P3^1 = P4^1 = P5^1 = P6^1 = P87^1$

Berat eternit + plafont = 18x4x(0,5.2,104+0,5.2,104) = 151,488 kg

Berat taksiran kuda-kuda = 54,28x(0,5.2,104+0,5.2,104) = 114,205 kg +

$P3^1 = P4^1 = P5^1 = P6^1 = P87^1$ = 266 kg

b.Beban angin

Muatan angin di darat = 25 kg/m^2

Koefisien angin menurut Peraturan Pembebanan Indonesia untuk Gedung 1983, untuk $\alpha < 65^o$

Diketahui : $\alpha = 26{,}341°$

Tekan = $C_1 = 0{,}02 \cdot \alpha - 0{,}4 = 0{,}127$ Tarik = $C_2 = -0{,}4$

Beban yang bekerja

$W_t = C_1 \times w = 0{,}127 \cdot 25 = 3{,}170\ kg/m^2$

$W_h = C_2 \times w = -0{,}4.25 = -10\ kg/m^2$

- Sisi kiri

$W_1 = 3{,}17.(0{,}5.1{,}578).4 = 10{,}004\ kg$

$W_2 = 3{,}17.(0{,}5.1{,}578+0{,}5.2{,}104).4 = 23{,}344\ kg$

$W_3 = 3{,}17.(0{,}5.2{,}104+0{,}5.2{,}104).4 = 26{,}679\ kg$

$W_4 = 3{,}17.(0{,}5.2{,}104+0{,}5.2{,}104).4 = 26{,}679\ kg$

$W_5 = 3{,}17.(0{,}5.2{,}104).4 = 13{,}339\ kg$

- Sisi kanan

$W_5^1 = -10.(0{,}5.2{,}104).4 = -42{,}08\ kg$

$W_6 = -10.(0{,}5.2{,}104+0{,}5.2{,}104).4 = -84{,}16\ kg$

$W_7 = -10.(0{,}5.2{,}104+0{,}5.2{,}104).4 = -84{,}16\ kg$

$W_8 = -10.(0{,}5.2{,}104+0{,}5.1{,}578).4 = -73{,}64\ kg$

$W_9 = -10.(0{,}5.1{,}578).4 = -31{,}56\ kg$

4.1.3 Perencanaan Profil

a. Batang bawah

- Batang Tarik

- Gaya tarik (P_{maks}) = 4114,35 kg (**Tabel 4.18** Lampiran II-4)

- Panjang = 1,95083 m = 195,083 cm

- $r_{min} = \dfrac{L}{240} = \dfrac{195{,}083}{240} = 0{,}813$ cm

$$- Ag_1 = \frac{P}{0{,}6.Fy} = \frac{4114{,}35}{0{,}6.2500} = 2{,}7429 \text{ cm}^2 \qquad \textbf{(3.1.18a)}$$

$$- Ag_2 = \frac{P}{0{,}5.Fu.\mu} + \phi_{lbg}.tp.n \quad \mu = 0{,}75 \qquad \textbf{(3.1.18b)}$$

$$= \frac{4114{,}35}{0{,}5.4000.0{,}75} + (1{,}27 + 0{,}3175)0{,}5.2 = 4{,}3304 \text{ cm}^2$$

→ dicoba profil 2L 50x50x5

$A = 9{,}6 \text{ cm}^2$ $W = 3{,}77$ kg/m

$r = 1{,}51 \text{ cm} \geq r_{min} = 0{,}738 \text{ cm}$ → dipakai $r = 1{,}51$ cm

Check kelangsingan :

$$\frac{K.l}{r} = \frac{1.195{,}083}{1{,}51} = 129{,}194 < 240 \ldots \text{Ok} \qquad \textbf{(3.1.21)}$$

$$\text{Anetto} = \text{Aprofil} - (d_{baut} + 1/8").t_p.n \qquad \textbf{(3.1.23)}$$

$$= 9{,}6 - (1{,}27 + 0{,}3175).0{,}5 \,.\, 2$$

$$= 8{,}0125 \text{cm}^2$$

$$\text{Aeff} = \text{Anetto} \times \mu \qquad \textbf{(3.1.24)}$$

$$= 8{,}0125 \times 0{,}75 = 6{,}009 \text{ cm}^2$$

Kontrol tegangan

$$ft = \frac{P}{Aprofil} \qquad \textbf{(3.1.25)}$$

$$= \frac{4114{,}35}{9{,}6} = 428{,}578 < 0{,}6.Fy = 0{,}6.2500 = 1500 \text{ kg/cm}^2$$

$$ft = \frac{P}{Aeff} \qquad \textbf{(3.1.26)}$$

$$= \frac{4114{,}35}{6{,}009} = 684{,}655 < 0{,}5.Fu = 0{,}5.\ 4000 = 2000 \text{ kg/cm}^2$$

Karena batang bawah terdapat juga batang tekan maka dicek juga sebagai batang tekan.

Pmaks = -706,3544 kg (**Tabel 4.18** Lampiran II-4)

Panjang = 195,083 cm

$$\text{Abruto} = \frac{P}{0{,}6.Fy} = \frac{706{,}3544}{0{,}6.2500} = 0{,}47 \text{ cm}^2$$

$$r_{min} = \frac{L}{240} = \frac{195{,}083}{240} = 0{,}813 \text{ cm}$$

$$\text{Aeff}_{perlu} = \frac{P}{0{,}5.Fu} = \frac{706{,}3544}{0{,}5.4000} = 0{,}353 \text{ cm}^2$$

$$\text{Anetto} = \frac{Aeff_{perlu}}{\mu} = \frac{0{,}353}{0{,}75} = 0{,}471 \text{ cm}^2$$

Digunakan profil 2L 50x50x5

A= 2 x 4,8 = 9,6 cm^2

r = 1,51 cm

Check local buckling :

$$\frac{bf}{tw} \leq \frac{76}{\sqrt{fy}} \qquad \textbf{(3.1.27)}$$

$$\frac{50}{5} \leq \frac{76}{\sqrt{36}}$$

10 ≤ 12,667 .... (ok)

Check Kelangsingan :

$$\frac{kl}{r} = \frac{1.195{,}083}{1{,}51} > Cc = \frac{6400}{\sqrt{fy}} \qquad \textbf{(3.1.29)}$$

= 129,194 > 128

maka :

$$Fa = \frac{12}{23}.\frac{\pi^2.E}{(Kl/r)^2} \qquad \textbf{(3.1.33)}$$

$$= \frac{12}{23}.\frac{\pi^2 2{,}1.10^6}{129{,}194}$$

$$= 647{,}8694 \text{ kg/cm}^2$$

Pmax= Fa.A = 647,8694 . 9,6 = 6219,547 kg > 706,3544 kg....(OK)

b. Batang atas

- Batang Tarik (Dihitung dengan rumus 3.1.17 – 3.1.26)

- Gaya tarik (P_{maks}) = 761,8433 kg (**Tabel 4.18** Lampiran II-4)

- Panjang = 1,5777 m = 157,7782 cm

- $r_{min} = \frac{L}{240} = \frac{157{,}7782}{240} = 0{,}657$ cm

- $Ag1 = \frac{P}{0{,}6.Fy} = \frac{761{,}8433}{0{,}6.2500} = 0{,}508 \text{ cm}^2$

- $Ag2 = \frac{P}{0{,}5.Fu.\mu} + \phi_{lb}.tp.n \quad \mu = 0{,}75$

$$= \frac{761{,}8433}{0{,}5.4000.0{,}75} + (1{,}27 + 0{,}3175).0{,}5.2 = 2{,}095 \text{ cm}^2$$

→ dicoba profil 2L 50x50x5

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

r = 1,51 cm ≥ r_{min} = 0,509 cm → dipakai r = 1,51 cm

Check kelangsingan :

$$\frac{K.l}{r} = \frac{1.157{,}7782}{1{,}51} = 104{,}489 < 240 \ldots \text{Ok}$$

Anetto = Aprofil – $(d_{baut} + 1/8").t_p.n$

= 9,6 – (1,27 + 0,3175).0,5.2

= 8,012 cm^2

Aeff = Anetto x μ

= 8,012 x 0,75 = 6,009 cm^2

Kontrol tegangan

$$ft = \frac{P}{Aprofil}$$

$$= \frac{761,8433}{9,6} = 79,359 < 0,6.Fy = 0,6.2500 = 1500 \text{ kg/cm}^2$$

$$ft = \frac{P}{Aeff}$$

$$= \frac{761,8433}{6,009} = 126,784 < 0,5.Fu = 0,5.\ 4000 = 2000 \text{ kg/cm}^2$$

Karena batang atas terdapat juga batang tekan maka dicek juga sebagai batang tekan (Dihitung dengan rumus 3.1.27 – 3.1.34).

P maks = -4437,717 kg (**Tabel 4.18** Lampiran II-4)

Panjang = 210,4156 cm

$$\text{Abruto} = \frac{P}{0,6.Fy} = \frac{4437,717}{0,6.2500} = 2,958 \text{ cm}^2$$

$$r_{min} = \frac{L}{240} = \frac{210,4156}{240} = 0,877 \text{ cm}$$

$$\text{Aeff}_{perlu} = \frac{P}{0,5.Fu} = \frac{4437,717}{0,5.4000} = 2,219 \text{ cm}^2$$

$$\text{Anetto} = \frac{Aeff_{perlu}}{\mu} = \frac{2,219}{0,75} = 2,958 \text{ cm}^2$$

Digunakan profil 2L 50x50x5

A= 2 x 4,8 = 9,6 cm^2

r = 1,51 cm

Check local buckling :

$$\frac{bf}{tw} \leq \frac{76}{\sqrt{fy}}$$

$$\frac{50}{5} \leq \frac{76}{\sqrt{36}}$$

$10 \leq 12{,}667$ .... (ok)

Check Kelangsingan :

$$\frac{kl}{r} = \frac{1.210{,}4156}{1{,}51} \leq Cc = \frac{6400}{\sqrt{fy}}$$

$= 139{,}3481 > 128$

maka :

$$Fa = \frac{12}{23} \cdot \frac{\pi^2 . E}{(Kl/r)^2}$$

$$= \frac{12}{23} \cdot \frac{\pi^2 . 2{,}1.10^6}{139{,}3481^2}$$

$= 556{,}8914$ kg/cm^2

Pmax= Fa.A = 556,8914 . 9,6 = 5346,157 kg > 4437,717 kg....(OK)

c. Batang diagonal

- Batang tarik **(Dihitung dengan rumus 3.1.17 – 3.1.26)**

  - Gaya tarik (Pmaks) = 3977,63 kg (**Tabel 4.18** Lampiran II-4)

  - Panjang = 189,6243 cm

- $r_{min} = \frac{189,6243}{240} = 0,790$

- $Ag1 = \frac{3977,63}{0,6.2500} = 2,652\ cm^2$

- $Ag2 = \frac{3977,63}{0,5.4000.0,75} + (1,27+0,3175).0,5.2 = 4,239\ cm^2$

→ dicoba profil 2L 50x50x5

Check kelangsingan :

$\frac{KI}{r} = \frac{1.189,6243}{1,51} = 125,579 < 240 \ldots Ok$

- Anetto = 9,6 – (1,27+0,3175).0,5.2

  $= 8,012\ cm^2$

- Aeff = $8,012 \times 0,75 = 6,009\ cm^2$

Kontrol tegangan

$ft = \frac{3977,63}{9,6} = 414,336\ kg/cm^2 < 1500\ kg/cm^2$

$ft = \frac{3977,63}{6,009} = 661,945 < 2000\ kg/cm^2$

- Batang tekan **(Dihitung dengan rumus 3.1.27 – 3.1.34)**

  - Gaya tekan (Pmaks) = -497,6931 kg (**Tabel 4.18** Lampiran II-4)

  - Panjang = 216,6781 cm

  - Abruto = $\frac{497,6931}{0,6.2500} = 0,332\ cm^2$

  - $r_{min} = \frac{216,6781}{240} = 0,903$ cm

- $Aeff_{perlu} = \frac{497,6931}{0,5.4000} = 0,249\ cm^2$

- Anetto $= \frac{0,249}{0,75} = 0,332\ cm^2$

→ profil 2L 50x50x5

Check local buckling :

$$\frac{bf}{tw} \leq \frac{76}{\sqrt{Fy}}$$

$10 \leq 12,667 \ldots Ok$

Check kelangsingan

$$\frac{K.l}{r} = \frac{1.216,6781}{1,51} = 143,4954 > 128$$

terjadi tekuk elastis

$$Fa = \frac{12}{23}.\frac{\pi^2.2,1.10^6}{143,4954^2}$$

$= 525,1656\ kg/cm^2$

$Pmax = Fa.A$

$= 525,1656 . 9,6$

$= 5041,59\ kg > 497,6931\ kg \ldots Ok$

D. Batang Vertikal

- Batang Tarik **(Dihitung dengan rumus 3.1.17 – 3.1.26)**

- Gaya tarik (Pmaks) = 2865,341 kg (**Tabel 4.18** Lampiran II-4)

- Panjang = 200,1 cm

- $r_{min} = \frac{L}{240} = \frac{200,1}{240} = 0,834\ cm$

- $Ag1 = \frac{P}{0,6.Fy} = \frac{2865,341}{0,6.2500} = 1,91\ cm^2$

- $Ag2 = \frac{P}{0,5.Fu.\mu} + \phi_{lb}.tp.n$ $\mu = 0,75$

$= \frac{2865,341}{0,5.4000.0,75} + (1,27+0,3175).0,5.2 = 3,497\ cm^2$

→ dicoba profil 2L 50x50x5

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

r = 1,51 cm ≥ r_{min} = 0,833 cm → dipakai r = 1,51 cm

Check kelangsingan :

$\frac{KL}{r} = \frac{1.200,1}{1,51} = 132,516 < 240 \ldots Ok$

Anetto = Aprofil – $(d_{baut} + 1/8").t_p.n$

= 9,6 – (1,27 + 0,3175).0,5.2

= 8,012 cm^2

Aeff = Anetto x μ

= 8,012 x 0,75 = 6,009 cm^2

Kontrol tegangan

$ft = \frac{P}{Aprofil}$

$= \frac{2865,341}{9,6} = 298,473 < 0,6.Fy = 0,6.2500 = 1500\ kg/cm^2$

$ft = \frac{P}{Aeff}$

$= \frac{2865,341}{6,009} = 476,841 < 0,5.Fu = 0,5.\ 4000 = 2000\ kg/cm^2$

Karena batang vertikal terdapat juga batang tekan maka dicek juga sebagai batang tekan (Dihitung dengan rumus 3.1.27 – 3.1.34).

Pmaks = -2920,961 kg (**Tabel 4.18** Lampiran II-4)

Panjang = 0,700 m = 70 cm

$$\text{Abruto} = \frac{P}{0{,}6.Fy} = \frac{2920{,}961}{0{,}6.2500} = 1{,}947 \text{ cm}^2$$

$$r_{min} = \frac{L}{240} = \frac{70}{240} = 0{,}292 \text{ cm}$$

$$\text{Aeff}_{perlu} = \frac{P}{0{,}5.Fu} = \frac{2920{,}961}{0{,}5.4000} = 1{,}46 \text{ cm}^2$$

$$\text{Anetto} = \frac{Aeff_{perlu}}{\mu} = \frac{1{,}46}{0{,}75} = 1{,}947 \text{ cm}^2$$

Digunakan profil 2L 50x50x5

$A = 2 \times 4{,}8 = 9{,}6 \text{ cm}^2$

r = 1,51 cm

Check local buckling :

$$\frac{bf}{tw} \leq \frac{76}{\sqrt{fy}}$$

$$\frac{50}{5} \leq \frac{76}{\sqrt{36}}$$

$10 \leq 12{,}667$ .... (ok)

Check Kelangsingan :

$$\frac{k.l}{r} = \frac{1.70}{1{,}51} \leq Cc = \frac{6400}{\sqrt{fy}}$$

$$= 46{,}358 \leq 128$$

maka :

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

$$= \frac{5}{3} + \frac{3}{8}\frac{46,358}{128} - \frac{1}{8}\frac{46,358^3}{128^3}$$

$$= 1,796$$

$$Fa = \frac{Fy}{Fs}[1 - 0,5\left(\frac{kl/r}{Cc}\right)^2]$$

$$= \frac{2500}{1,796}\left[1 - 0,5\left(\frac{46,358}{128}\right)^2\right]$$

$$= 1300,690$$

Pmax= Fa.A

= 1300,690. 9,6 = 12486,625 kg>2920,961 kg ....(OK)

Tabel 4.1 Jenis dan Berat Profil Terpakai Kuda-Kuda KK-4

Batang	Profil	Berat Profil (kg/m)	Panjang (m)	Berat (kg)
Batang atas	2L 50x50x5	2 x 3,77 = 7,54	15,78	118,981
Batang bawah	2L 50x50x5	2 x 3,77 = 7,54	14,534	109,586
Batang diagonal	2L 50x50x5	2 x 3,77 = 7,54	12,104	91,264
Batang vertikal	2L 50x50x5	2 x 3,77 = 7,54	8,801	66,359
				W_{tot} = 386,190 kg

Kontrol berat kuda-kuda :

- Berat total kuda-kuda = 386,190 kg
- Berat baut dan plat sambung = 20%.berat total kuda-kuda

= 77,238 kg

jumlah ($\sum$) = berat total kuda-kuda + berat baut dan plat sambung

= 386,190 + 77,238

= 463,428 kg

Panjang bentang kuda-kuda = 14,142 m

$$\frac{\sum}{L} < \text{berat taksiran}$$

$$\frac{463,428}{14,142} < 54,28 \text{ kg/m}$$

32,769 kg/m < 54,28 kg/m ..... (OK)

4.1.4 Perencanaan Sambungan

Dalam perencanaan sambungan pada tiap joint menggunakan baut Ø ½" (1,27 cm), dan pelat baja A36 (Fy = 2500 kg/cm^2 , Fu = 4000 kg/cm^2) dengan tebal 0,8 cm. Baut yang digunakan adalah A325x (baut non full drat) dengan kekuatan ultimit (Fu) = 8250 kg/cm^2, Fv = 2070 kg/cm^2.

Sehingga didapat kekuatan 1 baut untuk menahan gaya adalah :

P_{tumpu} = tp . D_{baut} . 1,2 . Fu . n **(3.1.35)**

= 0,8 . 1,27 . 1,2 . 4000 . 1

= 4876,8 kg

P_{geser} = A_{baut} . Fv . m **(3.1.36)**

= ¼ . π . 1,27^2 . 2070 . 2

= 5244,422 kg

dipakai P yang kecil yaitu P = 4876,8 kg

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

Rangka kuda-kuda 4

1. Joint 1

   a. Batang A1 = A8 = 761,8433 kg

   $$n = \frac{761,8433}{4876,8} = 0,156 \sim \text{dipakai 2 baut}$$

   b. Batang B1 = B8 = -682,76 kg

   $$n = \frac{682,76}{4876,8} = 0,14 \sim \text{dipakai 2 baut}$$

2. Joint 2

   a. Batang A1 = A8 = 761,8433 kg

   → dipakai 2 baut

   b. Batang A2 = A7 = -3651,885 kg

   $$n = \frac{3651,885}{4876,8} = 0,749 \sim \text{dipakai 2 baut}$$

   c. Batang V1 = V7 = -2920,961 kg

   $$n = \frac{2920,961}{4876,8} = 0,600 \sim \text{dipakai 2 baut}$$

   d. Batang D1 = D6 = 3977,63 kg

   $$n = \frac{3977,63}{4876,8} = 0,816 \sim \text{dipakai 2 baut}$$

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 (= 4876,8 kg). Perhitungan baut meliputi setengah bentang rangka kuda-kuda untuk mewakili perhitungan satu bentang.

Tabel 4.2 Jumlah Baut pada Kuda-kuda 4

Joint	Elemen/Batang	Jumlah baut
1	A1	2 buah
	B1	2 buah
2	A1	2 buah
	A2	2 buah
	D1	2 buah
	V1	2 buah
3	A2	2 buah
	A3	2 buah
	D2	2 buah
	V2	2 buah
4	A3	2 buah
	A4	2 buah
	D3	2 buah
	V3	2 buah
5	A4	2 buah
	A5	2 buah
	V4	2 buah
10	B1	2 buah
	V1	2 buah
	B2	2 buah
11	B2	2 buah
	B3	2 buah
	D1	2 buah
	V2	2 buah
12	B3	2 buah
	B4	2 buah
	D2	2 buah
	V3	2 buah
13	B4	2 buah
	B5	2 buah
	D3	2 buah
	V4	2 buah
	D4	2 buah

4.2 Perencanaan Pelat

4.2.1 Pembebanan Pelat Atap

Spesifikasi bahan : Mutu beton (f'c) = 28 Mpa

Mutu baja (fy) = 240 Mpa

- Beban mati pelat atap

1. berat pelat (taksiran) : 0,10 x 24 = 2,40 KN/m^2
2. lapisan kedap air/aspal(tebal 3 cm) : 0,03 x 23 = 0,66 KN/m^2 +

beban mati total (qD) = 3,09 KN/m^2

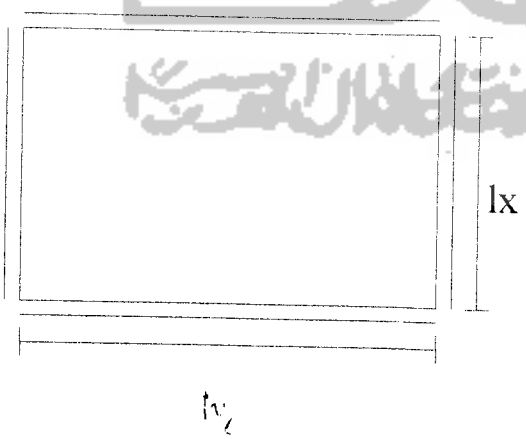
- Beban hidup pelat

Pada pelat atap terdapat beban hidup pekerja atau air hujan (qL) sebesar 100 kg/cm^2 atau 1 KN/m^2 (PPIUG 1983 tabel 3.1).

- Kombinasi pembebanan

qU = 1,2 qD + 1,6 qL = 1,2.3,09 + 1,6. 1,0 = 5,308 KNm

4.2.2 Perencanaan Pelat Atap



$\frac{ly}{lx} = \frac{3,0}{2,0} = 1,5 \rightarrow$ dihitung sebagai pelat 2 arah

Koefisien Momen (Tabel PBI 1971)

Koef. momen pelat (C)	1,5
Mlx = - Mtx	56
Mly	37
- Mty	37

- Momen yang bekerja pada pelat

  $Mu = 0{,}001.\ qU.\ lx^2.\ C$

  $Mulx = - Mutx = 0{,}001.\ 5{,}308.\ 2^2.\ 56 = 1{,}1889$ KNm

  $Muly = 0{,}001.\ 5{,}308.\ 2^2.\ 37 = 0{,}7856$ KNm

  $- Muty = 0{,}001.\ 5{,}308.\ 2^2.\ 37 = 0{,}7856$ KNm

  Tebal pelat dipakai = 100 mm

- Tinggi manfaat (d) pelat atap

  Digunakan tulangan pokok Ø 8 mm

  Penutup beton (Pb) = 20 mm

  - Lapangan arah –x : $d = h - Pb - \frac{1}{2}Ø_{tul.x}$

    $= 100 - 20 - \frac{1}{2}.\ 8 = 76$ mm

  - Lapangan arah – y : $d = h - Pb - Ø_{tul.x} - \frac{1}{2}Ø_{tul.y}$

    $= 100-20-8-\frac{1}{2}.8 = 68$ mm

  - Tumpuan arah –x dan arah –y : d = 76 mm

a. Perencanaan Tulangan Mlx dan Mtx

Mu = 1,1889 KNm

$$Mn = \frac{Mu}{0,8} = \frac{1,1889}{0,8} = 1,4861 KNm$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.28} = 10,0840 \qquad (3.2.13)$$

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

$$Rn = \frac{Mn}{b.d^2} = \frac{1,4861.10^6}{1000.76^2} = 0,2573 \text{ MPa} \qquad (3.2.12)$$

Rasio Tulangan (ρ) :

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

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.28.0,85}{240}\left(\frac{600}{600+240}\right) = 0,0602$$

$$\rho_{maks} = 0,75.\ \rho b = 0,75.\ 0,0602 = 0,0452$$

$$\rho_{perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{10,0840}\left(1-\sqrt{1-\frac{2.10,0840.0,2573}{240}}\right)$$

$$= 0,00116 < \rho_{min} = 0,00583 \qquad (3.2.14)$$

$1,33\ \rho_{perlu} = 1,33.\ 0,00116 = 0,00143 < \rho_{min} = 0,00583$

$\rho_{pakai} = 1,33.\rho_{perlu} = 0,00143$

$As_{perlu} = \rho_{pakai}.\ b.d = 0,00143.\ 1000.\ 76 = 108,68\ mm^2$

$As_{tul.susut} = 0,002\ .\ b\ .\ h = 0,002.1000.100 = 200\ mm^2 > As_{perlu}$ (3.2.15)

$As_{perlu} = As_{tul.susut} = 200\ mm^2$

Digunakan tulangan polos Ø 8 mm, maka :

$A_{10} = ¼\ .\pi.\ D^2 = ¼\ .\ \pi.\ 8^2 = 50,24\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{1\phi}.b}{As_{perlu}} = \frac{50,24.1000}{200} = 251,2 \text{ mm} \quad (3.2.16)$$

$$\leq 2.h = 2 . 100 = 200 \text{ mm} \quad (3.2.17)$$

$$\leq 250 \text{ mm} \quad (3.2.18)$$

Dipakai jarak antar tulangan (s_{pakai}) = 200 mm

DIPAKAI TULANGAN POKOK : P8- 200 mm

Luas tulangan pokok yang digunakan :

$$As_{pakai} = \frac{A_{1\phi}.1000}{s_{pakai}} = \frac{50,24.1000}{200} = 251,2 \text{ mm}^2 > As_{perlu} = 200 \text{ mm}^2 \quad (3.2.20)$$

Tulangan susut

dipakai tulangan polos Ø8 → $A_{1\phi}$ = 50,24 mm^2

$$s \leq \frac{A_{1\phi}.b}{A_{sst}} = \frac{50,24.1000}{200} = 251,2mm$$

$$\leq 5 . h = 5 . 100 = 500 \text{ mm}$$

DIPAKAI TULANGAN SUSUT : P8 - 250

Kontrol Kapasitas Lentur Pelat (arah Mtx dan Mlx) :

$$a = \frac{As_{pakai}.fy}{0,85.f'c.b} = \frac{251,2.240}{0,85.28.1000} = 2,533mm \quad (3.2.21)$$

$$Mn = As_{pakai}. fy. (d-a/2) = 251,2. 240. (76-\frac{2,533}{2}) \quad (3.2.22)$$

$$= 4,5055 \text{ KNm} > 1,4861 \text{ KNm} \ldots\ldots\ldots\ldots\ldots\ldots(OK)$$

b. Perencanaan Tulangan Mly (Dihitung menggunakan rumus 3.2.12 – 3.2.22)

Mu = 0,7856 KNm

$$Mn = \frac{Mu}{0,8} = \frac{0,7856}{0,8} = 0,982 \text{ KNm}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.28} = 10{,}0840$$

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

$$Rn = \frac{Mn}{b.d^2} = \frac{0{,}982.10^6}{1000.68^2} = 0{,}2123 \text{ MPa}$$

Rasio Tulangan (ρ) :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{240}\left(\frac{600}{600+240}\right) = 0{,}0602$$

$$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0602 = 0{,}0452$$

$$\rho_{perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{10{,}0840}\left(1-\sqrt{1-\frac{2.10{,}0840.0{,}2123}{240}}\right)$$

$$= 0{,}00088 < \rho_{min} = 0{,}00583$$

$$1{,}33\ \rho_{perlu} = 1{,}33.\ 0{,}00088 = 0{,}00118 < \rho_{min} = 0{,}00583$$

$$\rho_{pakai} = 1{,}33.\rho_{perlu} = 0{,}00118$$

$$As_{perlu} = \rho_{pakai}.\ b.d = 0{,}00118.\ 1000.\ 68 = 80{,}388 \text{ mm}^2$$

$$As_{tul.susut} = 0{,}002\ .\ b\ .\ h = 0{,}002.1000.100 = 200 \text{ mm}^2 > As_{perlu}$$

$$As_{perlu} = As_{tul.susut} = 200 \text{ mm}^2$$

Digunakan tulangan polos Ø 8 mm, maka :

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

Jarak antar tulangan :

$$s \le \frac{A_{1\phi}.b}{As_{perlu}} = \frac{50{,}24.1000}{200} = 251{,}2 \text{ mm}^2$$

$$\le 300 \text{ mm}$$

$\leq 2.h = 2.100 = 200$ mm

Dipakai jarak antar tulangan (s_{pakai}) = 200 mm

DIPAKAI TULANGAN POKOK : P8- 200 mm

Luas tulangan pokok yang digunakan :

$$As_{pakai} = \frac{A_{1\phi}.1000}{s_{pakai}} = \frac{50,24.1000}{200} = 251,2\,mm^2 > As_{perlu} = 80,388\ mm^2$$

Kontrol Kapasitas Lentur Pelat (arah Mly) :

$$a = \frac{As_{pakai}.fy}{0,85.f'c.b} = \frac{251,2.240}{0,85.28.1000} = 2,533mm$$

$$Mn = As_{pakai}.\ fy.\ (d\text{-}a/2) = 251,2.\ 240.\ (68-\frac{2,533}{2})$$

$$= 4,0232\ KNm > 0,982\ KNm \ldots\ldots\ldots\ldots\ldots (OK)$$

c. Perencanaan Tulangan Mty (Dihitung menggunakan rumus 3.2.12 – 3.2.22)

Mu = 0,7856 KNm

$$Mn = \frac{Mu}{0,8} = \frac{0,7856}{0,8} = 0,982\,KNm$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.28} = 10,0840$$

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

$$Rn = \frac{Mn}{b.d^2} = \frac{0,982.10^6}{1000.76^2} = 0,1700\ MPa$$

Rasio Tulangan (ρ) :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{240}\left(\frac{600}{600+240}\right) = 0{,}0602$$

$\rho_{maks} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0602 = 0{,}0452$

$$\rho_{perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{10{,}0840}\left(1-\sqrt{1-\frac{2.10{,}0840.0{,}1700}{240}}\right)$$

$= 0{,}00071 < \rho_{min} = 0{,}00583$

$1{,}33\ \rho_{perlu} = 1{,}33.\ 0{,}00071 = 0{,}00094 < \rho_{min} = 0{,}00583$

$\rho_{pakai} = 1{,}33.\rho_{perlu} = 0{,}00094$

$As_{perlu} = \rho_{pakai}.\ b.d = 0{,}00094.\ 1000.\ 76 = 71{,}8617\ mm^2$

$As_{tul.susut} = 0{,}002\ .\ b\ .\ h = 0{,}002.1000.100 = 200\ mm^2 > As_{perlu}$

$As_{perlu} = As_{tul.susut} = 200\ mm^2$

Digunakan tulangan polos Ó 8 mm, maka :

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

Jarak antar tulangan :

$$s \le \frac{A_{1\phi}.b}{As_{perlu}} = \frac{50{,}24.1000}{71{,}8617} = 699{,}12\ mm^2$$

Dipakai jarak antar tulangan (s_{pakai}) = 200 mm

DIPAKAI TULANGAN POKOK : P8- 200 mm

Luas tulangan pokok yang digunakan :

$$As_{pakai} = \frac{A_{1\phi}.1000}{s_{pakai}} = \frac{50{,}24.1000}{200} = 251{,}2\,mm^2 > As_{perlu} = 71{,}8617\ mm^2$$

Tulangan susut

dipakai tulangan polos Ó8 → $A_{1Ó} = 50{,}24\ mm^2$

$$s \leq \frac{A_{1\phi}.b}{A_{sst}} = \frac{50{,}24.1000}{200} = 251{,}2mm$$

≤ 5 . h = 5 . 100 = 500 mm

DIPAKAI TULANGAN SUSUT : P8 - 250

Kontrol Kapasitas Lentur Pelat (arah Mty) :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{251{,}2.240}{0{,}85.28.1000} = 2{,}533mm$$

$Mn = As_{pakai}$. fy. (d-a/2) $= 251{,}2.\ 240.\ (76-\frac{2{,}533}{2})$

= 4,505 KNm > 0,982 KNm ..... (OK)

Tabel 4.3 Perencanaan Pelat Atap

	Mlx	Mtx	Mly	Mty
Mu (KNm)	1.1889	1.1889	0.7856	0.7856
Mn (KNm)	1.486125	1.486125	0.982	0.982
h (mm)	100	100	100	100
M	10.084	10.084	10.084	10.084
d (mm)	76	76	68	76
Rn (MPa)	0.25729311	0.257293	0.21237	0.170014
P_{min}	0.00583	0.00583	0.00583	0.00583
Pb	0.0602	0.0602	0.0602	0.0602
P_{maks}	0.00452	0.00452	0.00452	0.00452
P_{perlu}	0.00107791	0.001078	0.000889	0.000711
1.33.ρ_{perlu}	0.00143362	0.001434	0.001182	0.000946
P_{perlu}	0.00143362	0.001434	0.001182	0.000946
As_{perlu} (mm^2)	108.955436	108.9554	80.38646	71.86176
Astul susut(mm^2)	200	200	200	200
		Asperlu < Astul.susut		
A1Ø(mm^2)	50.24	50.24	50.24	50.24
s (mm)	251.2	251.2	251.2	251.2
s_{pakai} (mm)	200	200	200	200
As_{pakai} (mm^2)	251.2	251.2	251.2	251.2
a (mm)	2.53310924	2.533109	2.533109	2.533109
Mn (KNm)	4.50552995	4.50553	4.023226	4.50553
Kontrol	OK	OK	OK	OK
Tul. Pokok	**P8-200**	**P8-200**	**P8-200**	**P8-200**
As_{susut} (mm^2)		200		200
Dtul.susut		8		8
$A_{1Øsusuti}$(mm^2)		50.24		50.24
s_{maks} (mm)		251.2		251.2
s_{pakai} (mm)		250		250
Tul.susut		**P8-250**		**P8-250**

4.2.3 Pembebanan Pelat Lantai

- Beban mati pelat lantai :

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

  beban mati total (qD) = 4,51 kN/m^2

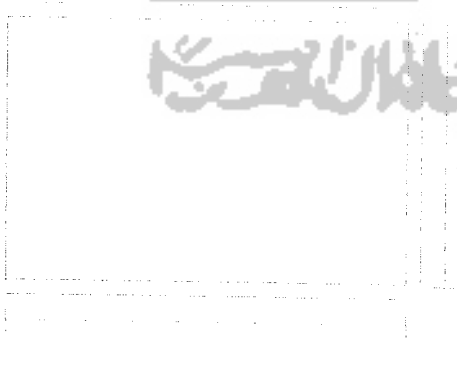
- Beban hidup pelat lantai :

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

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

  qU = 1,2.qD + 1,6.qL = 1,2.4,51 + 1,6.2,5 = 9,412 kNm

4.2.5 Perencanaan Pelat Lantai (Menggunakan Rumus 3.2.1 – 3.2.22)



$$\frac{ly}{lx} = \frac{3,0}{2,0} = 1,5 \rightarrow \text{dihitung sebagai pelat 2 arah}$$

Koefisien Momen (Tabel PBI 1971)

Koef. momen pelat (C)	1,5
Mlx = - Mtx	56
Mly	37
- Mty	37

- Momen yang bekerja pada pelat

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

  Mulx = - Mutx = 0,001. 9,412. 2^2. 56 = 2,1083 KNm

  Muly = 0,001. 9,412. 2^2. 37 = 1,3929 KNm

  - Muty = 0,001. 9,412. 2^2. 37 = 1,3929 KNm

  Tebal pelat dipakai = 120 mm

- Tinggi manfaat (d) pelat lantai

  Digunakan tulangan pokok Ø 10 mm

  Penutup beton (Pb) = 20 mm

  - Lapangan arah –x : d = h – Pb - ½.$Ø_{tul.x}$

    = 120 - 20 - ½. 8 = 96 mm

  - Lapangan arah – y : d = h – Pb - $Ø_{tul.x}$ - ½$Ø_{tul.y}$

    = 120 – 20 – 8 - ½.8 = 88 mm

  - Tumpuan arah –x dan arah –y : d = 95 mm

Perencanaan Tulangan Mlx dan Mtx

Mu = 2,1083 KNm

$$Mn = \frac{Mu}{0,8} = \frac{2,1083}{0,8} = 2,6354\,KNm$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.28} = 10,0840$$

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

$$Rn = \frac{Mn}{b.d^2} = \frac{2,6354.10^6}{1000.96^2} = 0,2859\ MPa$$

Rasio Tulangan (ρ) :

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

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.28.0,85}{240}\left(\frac{600}{600+240}\right) = 0,0602$$

$$\rho_{maks} = 0,75.\ \rho b = 0,75.\ 0,0602 = 0,0452$$

$$\rho_{perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{10,0840}\left(1-\sqrt{1-\frac{2.10,0840.0,2859}{240}}\right)$$

$$= 0,001198 < \rho_{min} = 0,00583$$

$$1,33\ \rho_{perlu} = 1,33.\ 0,001198 = 0,00594 < \rho_{min} = 0,00583$$

$$\rho_{pakai} = 1,33\ \rho_{perlu} = 0,00594$$

$$As_{perlu} = \rho_{pakai}.\ b.d = 0,00594.\ 1000.\ 96 = 153,054\ mm^2$$

$$As_{tul.susut} = 0,002.b.h = 0,002.1000.120 = 240\ mm^2 > As_{perlu}$$

$$As_{perlu} = As_{tul.susut} = 240\ mm^2$$

Digunakan tulangan polos Ø 8 mm, maka :

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

Jarak antar tulangan :

$$s \leq \frac{A_{1\phi}.b}{As_{perlu}} = \frac{50{,}24.1000}{240} = 209{,}333 \text{ mm}$$

≤ 300 mm

$\leq 2.h = 2.120 = 240$ mm

Dipakai jarak antar tulangan (s_{pakai}) = 200 mm

DIPAKAI TULANGAN POKOK : P8- 200 mm

Luas tulangan pokok yang digunakan :

$$As_{pakai} = \frac{A_{1\phi}.1000}{s_{pakai}} = \frac{50{,}24.1000}{240} = 251{,}2\,mm^2 > As_{perlu} = 240\ mm^2$$

Tulangan Susut

Digunakan tulangan polos Ø8mm → A1Ø = 50,24 mm^2

Jarak antar tulangan : $s \leq \frac{A_{1\phi}.1000}{As_{ssi}} = \frac{50{,}24.1000}{240} = 209{,}333$ mm

≤ 500 mm

$\leq 5.h = 5.120 = 600$ mm

DIPAKAI TULANGAN SUSUT : P8-200 mm

Kontrol Kapasitas Lentur Pelat (arah Mtx dan Mlx) :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{251{,}2.240}{0{,}85.28.1000} = 2{,}533mm$$

$$Mn = As_{pakai}.\ fy.\ (d\text{-}a/2) = 251{,}2.\ 240.\ (96 - \frac{2{,}533}{2})$$

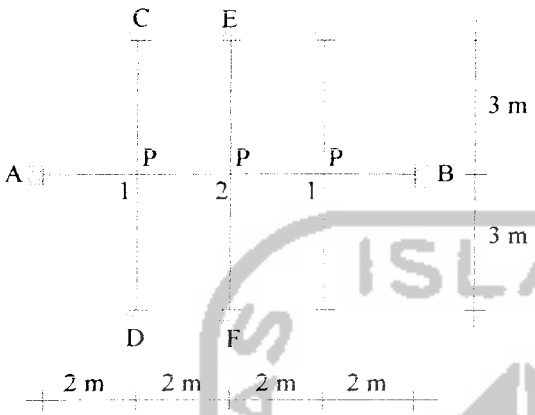
$= 5{,}711$ KNm > 2,6354 KNm .....(OK)

Tabel 4.4 Perencanaan Pelat Lantai Tipe 1

	Mlx	Mtx	Mly	Mty
Mu (KNm)	2,1083	2,1083	1,3929	1,3929
Mn (KNm)	2,635375	2,635375	1,741125	1,741125
h (mm)	120	120	120	120
m	10,084	10,084	10,084	10,084
d (mm)	96	96	88	96
Rn (MPa)	0,285956489	0,2859565	0,224835	0,188924
ρ_{min}	0,00583	0,00583	0,00583	0,00583
pb	0,0602	0,0602	0,0602	0,0602
ρ_{maks}	0,00452	0,00452	0,00452	0,00452
ρ_{perlu}	0,00119873	0,0011987	0,000941	0,00079
1.33.ρ_{perlu}	0,001594312	0,0015943	0,001252	0,001051
ρ_{pakai}	0,001594312	0,0015943	0,001252	0,001051
A_{Sperlu} (mm2)	153,0539097	153,05391	110,1676	100,9098
AStul susut(mm2)	240	240	240	240
		Asperlu < AStul.susut		
Øtul (mm)	8	8	8	8
A1Ø(mm2)	50,24	50,24	50,24	50,24
S_{maks} (mm)	209,3333333	209,33333	209,3333	209,3333
S_{pakai} (mm)	200	200	200	200
A_{Spakai} (mm2)	251,2	251,2	251,2	251,2
a (mm)	2,533109244	2,5331092	2,533109	2,533109
Mn (KNm)	5,711289955	5,71129	5,228986	5,71129
Kontrol	OK	OK	OK	OK
Tul. Pokok	**P8-200**	**P8-200**	**P8-200**	**P8-200**
AStul.susut (mm2)		240		240
Dtul.susut		8		8
A1Øsst(mm2)		50,24		50,24
S_{maks} (mm)		209,33333		209,3333
S_{pakai} (mm)		200		200
Tul.bagi		**P8-200**		**P8-200**

4.3 Perencanaan Balok Anak (Balok Grid)

4.3.1 Balok Grid 1



A-B = balok arah panjang

C-D = E-F = balok arah pendek

EI = sama untuk semua balok

Gambar 4.4 Rencana balok grid 1

4.3.1.1 Pembebanan Balok Grid

Perhitungan beban yang bekerja pada balok silang :

1. Beban mati (PD)

- Beban pelat lantai = (3 x 2).4,51 = 27,06 kN
- Berat balok silang (25/40) = 0,25. (0,4-0,12). 24. 2 = 3,36 kN
- Berat balok bawah (25/40) = 0,25 .(0,4-0,12). 24. 3 = 5,04 kN +

PD = 35,46 kN

2. Beban hidup (PL)

- Beban hidup pelat lantai = (3 x 2). 2,5 = 15 kN

P = 1,2PD + 1,6PL = 1,2. 35,46 + 1,6. 15 = 66,552 kN

Simpul 1

Perhitungan lendutan dengan tabel Makowski :

$$\{ 9(P - x_1) + 11(P - x_2) + 7(P - x_1) \} \frac{L^3}{768EI} = x_1 . \frac{L^3}{48EI}$$

$$\{9(66,552 - x_1) + 11(66,552 - x_2) + 7(66,552 - x_1)\} \frac{8^3}{768EI} = x_1 \cdot \frac{6^3}{48EI}$$

$\{598,968\text{-}9x_1\text{+}732,072\text{-}11x_2\text{+}465,864\text{-}7x_1\}0,667 = 4,5x_1$

$\{1796,904\text{-}16x_1\text{-}11x_2\}0,667 = 4,5x_1$

$1198,5350\text{-}15,172x_1\text{-}7,337x_2 = 0$ ..... (1)

Simpul 2

$$\{11(P\text{-}x_1) + 16(P\text{-}x_2) + 11(P\text{-}x_1)\} \frac{L^3}{768EI} = x_2 \cdot \frac{L^3}{48EI}$$

$$\{11(66,552\text{-}x_1) + 16(66,552\text{-}x_2) + 11(66,552\text{-}x_1)\} \frac{8^3}{768EI} = x_2 \cdot \frac{6^3}{48EI}$$

$\{732,072\text{-}11x_1\text{+}1064,832\text{-}16x_2\text{+}732,072\text{-}11x_1\}0.667 = 4,5.x_2$

$\{2528,976\text{-}22x_1\text{-}16x_2\}0,667 = 4,5x_2$

$1686,8270\text{-}14,674x_1\text{-}15,172x_2 = 0$ ..... (2)

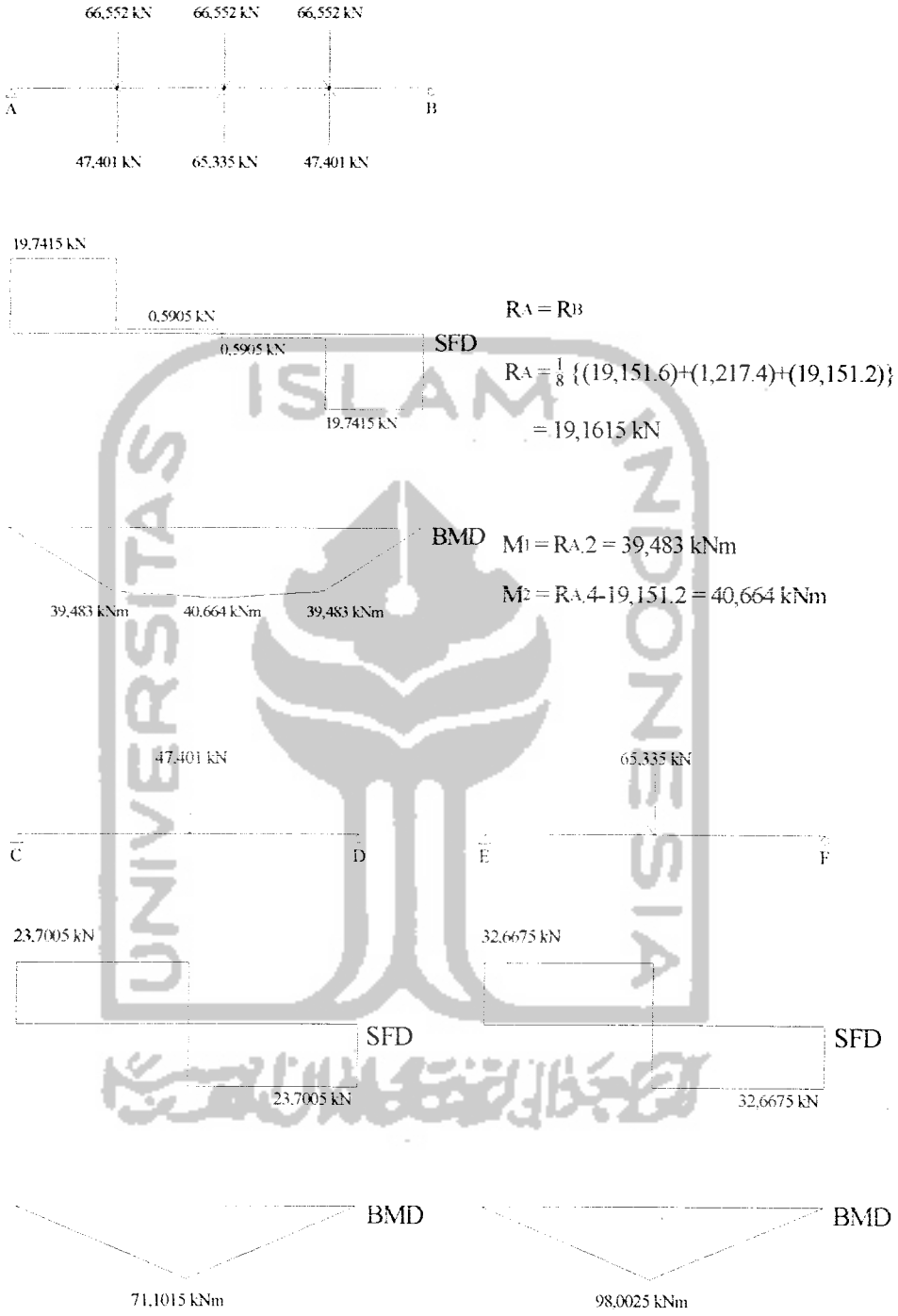
substitusi pers. (1) dan (2) :

$15,172x_1\text{+}7,337x_2 = 1198,5350$

$14,674x_1\text{+}15,172x_2 = 1686,8270$

didapat : $x_1 = 47,401$ kN

$x_2 = 65,335$ kN



Gambar 4.5 Grafik SFD dan BMD Balok Grid 1

$R_C = R_D = 23{,}7005$ kN $\qquad R_E = R_F = 32{,}6675$ kN

$M_1 = R_C \,.\, 3 = 71{,}1015$ kNm $\qquad M_2 = R_E \,.\, 3 = 98{,}0025$ kNm

4.3.1.2 Perhitungan Tulangan Tumpuan Balok Grid 1 Arah Panjang (BG1)

Data : - f'c = 28 MPa

- fy ulir = 400 MPa

- Tul. pokok Ø 16 mm

- Tul. sengkang Ø 8 mm

Perhitungan :

Mmaks pada balok atas = 40,664 kNm (BMD bentang AB hal 105)

Mu = 2/3.40,664 = 27,1093 kNm

$$Mn = \frac{Mu}{\phi} = \frac{27,1093}{0,8} = 33,8866 \text{ kNm}$$

Perencanaan balok dipakai ukuran 25/40

$$\rho_b = \frac{0,85.f'c}{fy}\beta 1\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{240}.0,85\left(\frac{600}{600+400}\right) = 0,0303 \quad \textbf{(3.3.10)}$$

$$\rho_{maks} = 0,75.\ \rho_b = 0,75.\ 0,0303 = 0,0227 \quad \textbf{(3.3.11)}$$

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

$$\rho_{pakai} = 0,5.\ \rho_{maks} = 0,5.\ 0,0227 = 0,0114 \quad \textbf{(3.3.12)}$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,8067 \quad \textbf{(3.3.13)}$$

$$Rn = \rho_{pakai}.fy.(1-0,5.\rho_{pakai}.m) = 0,0114.\ 400\ (1-0,5.0,0114.16,8067) \quad \textbf{(3.3.14)}$$

$$= 4,1232 \text{ MPa}$$

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{33,8866.10^6}{4,1232} = 8231924,286 \text{ mm}^2$$

diambil b = 250 mm, maka :

$$d_{perlu} = \sqrt{\frac{8231924,286}{250}} = 181,4599 \text{ mm}$$

d_{pakai} = h-pb-Ósengkang-0,5.Ótul.pokok

= 400-40-8-0,5.16 = 344 mm > d_{perlu} = 181,4599 mm

Karena $d_{pakai} > d_{perlu}$, maka direncanakan sebagai tulangan sebelah

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{33,8866.10^6}{250.344^2} = 1,1454 \text{ MPa} \qquad \textbf{(3.3.16)}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho_{pakai} = \frac{1,1454}{4,1232}.0,0114 = 0,00317 < \rho_{min} = 0,0035$$

$1,33.\rho_{baru} = 0,00421$

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

$As_{perlu} = \rho_{pakai}.b.d = 0,0035.\ 250.\ 344 = 301 \text{ mm}^2$ **(3.3.18)**

A1Ó16 = 200,96 mm^2

dipakai 2Ó16, maka :

$As_{tul} = 2 \times 200,96 = 401,92 \text{ mm}^2 > As_{perlu} = 340,385 \text{ mm}^2$ **(3.3.20)**

$$s = \frac{b - 2.pb - 2.\phi sengkang - n.\phi tul}{(n-1)} = \frac{250 - 2.40 - 2.8 - 2.16}{2-1}$$

$= 134 \text{ mm} \geq 25 \text{ mm}$

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0,85.f'c.b} = \frac{401,92.400}{0,85.28.250} = 27,01983mm \qquad \textbf{(3.3.21)}$$

$$Mn = As_{pakai}.fy.\ (d - \frac{a}{2}) = 401,92.\ 400\ (344 - \frac{27,0198}{2}) \qquad \textbf{(3.3.22)}$$

= 53,132 KNm > 33,8866 KNm.....(OK)

4.3.1.3 Perhitungan Tulangan Lapangan Arah Panjang

Mmaks pada balok atas = 40,664 kNm (BMD bentang AB hal 105)

Mu = 2/3.40,664 = 27,1093 kNm

$$Mn = \frac{Mu}{\phi} = \frac{27,1093}{0,8} = 33,8866 \text{ kNm}$$

Perencanaan balok dipakai ukuran 25/40

$$\rho_b = \frac{0,85.f'c}{fy}\beta 1\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{240}.0,85\left(\frac{600}{600+400}\right) = 0,0303$$

$\rho_{maks} = 0,75.\ \rho_b = 0,75.\ 0,0303 = 0,0227$

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

$\rho_{pakai} = 0,5.\ \rho_{maks} = 0,5.\ 0,0227 = 0,0114$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,8067$$

$Rn = \rho_{pakai}.\ fy.\ (1-0,5.\ \rho_{pakai}.m) = 0,0114.\ 400\ (1-0,5.0,0114.16,8067)$

$= 4,1232$ MPa

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{33,8866.10^6}{4,1232} = 8231924,286 \text{ mm}^2$$

diambil b = 250 mm, maka :

$$d_{perlu} = \sqrt{\frac{8231924,286}{250}} = 181,4599 \text{ mm}$$

d_{pakai} = h-pb-Øsengkang-Øtul.blk.pendek-0,5.Øtul.lap

= 400-40-8-16-0,5.16 = 328 mm > d_{perlu} = 181,4599 mm

Dipakai tulangan sebelah

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{33{,}8866.10^6}{250.328^2} = 1{,}2599 \text{ MPa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho_{pakai} = \frac{1{,}2599}{4{,}1232}.0{,}0114 = 0{,}00348 > \rho_{min} = 0{,}0035$$

$$< \rho_{maks} = 0{,}0227$$

1,33. $\rho_{baru} = 0{,}00463 > \rho_{min} = 0{,}0035$

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

$As_{perlu} = \rho_{pakai}.b.d = 0\ 0035.\ 250.\ 328 = 287\ mm^2$

A1Ø16 = 200,96 mm^2

Jumlah tulangan terpakai = $\frac{287}{200{,}96} = 1{,}434 \approx 2$

Dipakai tulangan 2Ø16

$As_{pakai} = 2 \times 200{,}96 = 401{,}92\ mm^2 > As_{perlu} = 287\ mm^2$

$$s = \frac{b - 2.pb - 2.\phi sengkang - n.\phi tul}{(n-1)} = \frac{250 - 2.40 - 2.8 - 2.16}{2-1}$$

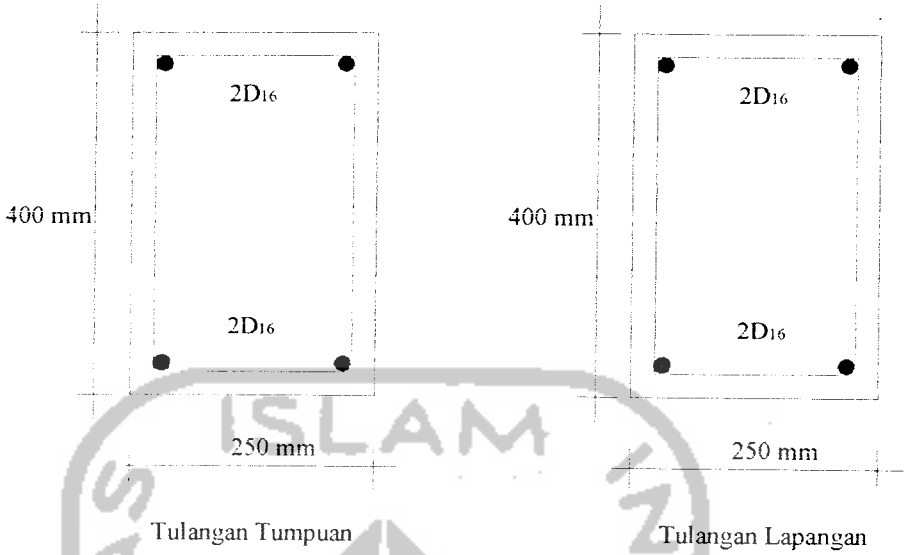
$$= 134 \text{ mm} \geq 25 \text{ mm}$$

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{401{,}92.400}{0{,}85.28.250} = 27{,}0198mm$$

$$Mn = As_{pakai}.fy.\ (d - \frac{a}{2}) = 401{,}92.\ 400\ (328 - \frac{27{,}0198}{2})$$

$$= 50{,}5599 \text{ KNm} > 33{,}8866 \text{ KNm} \ldots\ldots (OK)$$



Gambar 4.6 Gambar Penulangan Balok Grid Arah Panjang

4.3.1.4 Penulangan Geser Balok Grid 1

Dimensi balok : b = 250 mm

h = 400 mm

d = 325 mm

L = 8 m

Mutu bahan : f'c = 28 MPa

fy = 240 MPa

- Gaya geser maksimum pada ujung bentang :

Vu = 19,7415 kN (SFD bentang AB hal 105)

$$\frac{Vu}{\phi} = \frac{19,7415}{0,6} = 32,9025 \text{ kN}$$

- Gaya geser pada penampang kritis

$$\frac{Vu_{kritis}}{\phi} = \frac{4 - 0.325}{4}.32,9025 = 30,2292 \text{ kN}$$

- Kekuatan geser beton

$$Vc = \frac{1}{6}.\sqrt{f'c}.b.d = \frac{1}{6}.\sqrt{28}.250.325 = 71{,}6558 \text{ kN} \quad \textbf{(3.3.38)}$$

$$\frac{1}{2}Vc = \frac{1}{2}.71{,}6558 = 35{,}8279 \text{ kN}$$

$$0{,}5Vc < \frac{Vu_{kritis}}{\phi} < Vc \text{ , diperlukan sengkang minimum} \quad \textbf{(3.3.41)}$$

Dipakai sengkang Ø 8 mm :

Av = 2. (π. 4^2) = 100,48 mm^2

Jarak sengkang : $s \leq \frac{3.Av.fy}{b} = \frac{3.100{,}48.240}{250} = 289{,}3824$ mm **(3.3.42)**

≤ 600 mm

$\leq \frac{d}{2} = \frac{333}{2} = 166{,}5$ mm **(3.3.43)**

Dipakai sengkang Ø8-150

4.3.2 Perhitungan Balok Grid 1 Arah Pendek (BG1')

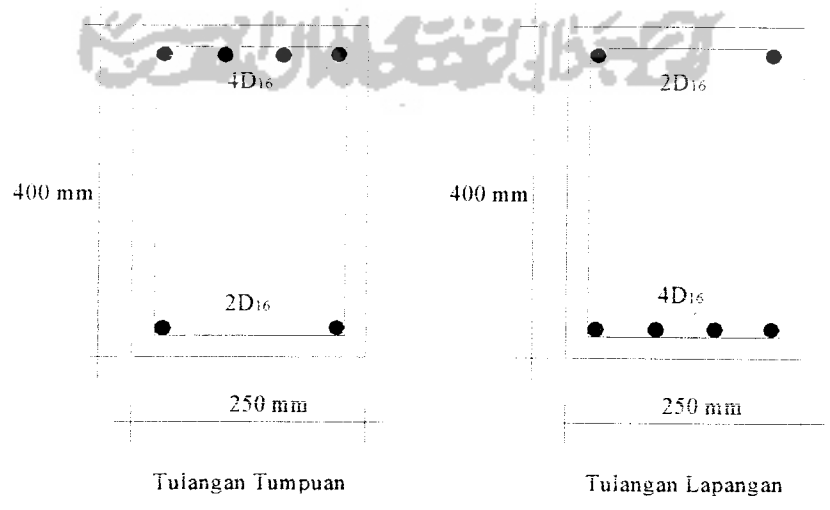
Mmaks = 98,0025 kN (BMD bentang EF hal 105)

Mu = 2/3.98,0025 = 65,3349 kN

Tabel 4.5 Perencanaan Balok Grid 1' (BG 1')

	Tumpuan	Lapangan
Mu (kNm)	65,335	65,335
Mu/Φ (kNm)	81,66875	81,66875
f'c (Mpa)	28	28
fy (Mpa)	400	400
β_1	0,85	0,85
M	16,80672	16,80672
ρ_b	0,030345	0,030345
ρ_{maks}	0,022759	0,022759
ρ_{min}	0,0035	0,0035

ρ_{pakai}	0,011379	0,011379
Rn (Mpa)	4,116489	4,116489
$b.d^2$ perlu	19839419	19839419
b (mm)	250	250
d_{perlu} (mm)	281,7049	281,7049
h (mm)	400	400
d_{pakai} (mm)	344	328
	dpakai > dperlu	
Perencanaan	Tul.sebelah	Tul.sebelah
Rn_{baru} (Mpa)	2,760572	3,036465
ρ_{baru}	0,007631	0,008394
$1,33.\rho_{baru}$	0,010149	0,011164
ρ_{pakai}	0,007631	0,008394
As_{perlu} (mm^2)	656,2796	688,2933
Ótul (mm)	**16**	**16**
$A_{1Ótul}$ (mm^2)	200,96	200,96
n perlu	3,265723	3,425026
jumlah tul. Pakai	**4**	**4**
As_{ada} (mm^2)	803,84	803,84
s > 25 (mm)	40,66667	40,66667
a (mm)	54,03966	54,03966
Mn (kNm)	101,9205	96,77596
Kontrol	Ok !	Ok !



Gambar 4.7 Gambar Penulangan Balok Grid 1 Arah Pendek (BG1')

4.3.2.1 Penulangan Geser Balok Grid 1

Dimensi balok : b = 250 mm

h = 400 mm

d = 323,5 mm

L = 6 m

Mutu bahan : f'c = 28 MPa

fy = 240 MPa

- Gaya geser maksimum pada ujung bentang :

Vu = 32,6675 kN (SFD bentang EF hal 105)

$$\frac{Vu}{\phi} = \frac{32,6675}{0,6} = 54,4458 \text{ kN}$$

- Gaya geser pada penampang kritis

$$\frac{Vu_{kritis}}{\phi} = \frac{3 - 0.3235}{3}.54,4458 = 48,5747 \text{ kN}$$

- Kekuatan geser beton

$$Vc = \frac{1}{6}.\sqrt{f'c}.b.d = \frac{1}{6}.\sqrt{28}.250.323,5 = 71,325 \text{ kN}$$

$$\frac{1}{2}Vc = \frac{1}{2}.71,325 = 35,6625 \text{ kN}$$

$0,5Vc < \frac{Vu_{kritis}}{\phi} < Vc$, diperlukan sengkang minimum

Dipakai sengkang Ø 8 mm :

Av = 2. (π. 4^2) = 100,48 mm^2

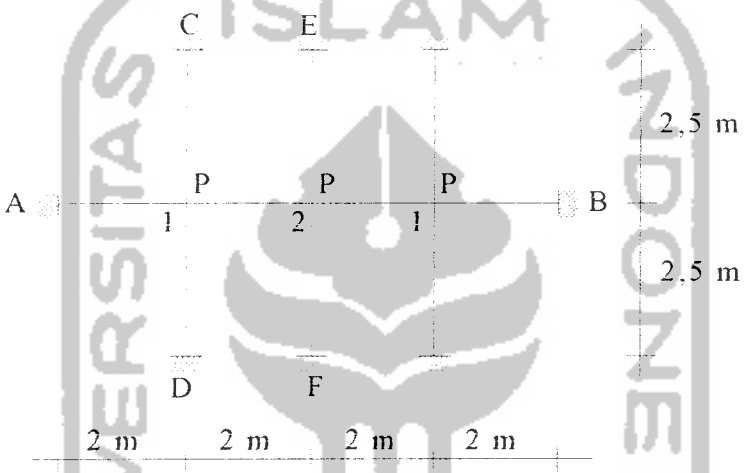
Jarak sengkang : $s \leq \frac{3.Av.fy}{b} = \frac{3.100,48.240}{250} = 289,3824$ mm

$$\leq 600 \text{ mm}$$

$$\leq \frac{d}{2} = \frac{323,5}{2} = 161,75 \text{ mm}$$

Dipakai sengkang Ø8-150

4.3.3 Balok Grid 2



Gambar 4.8 Rencana balok grid 2

4.3.3.1 Pembebanan Balok Grid 2

Perhitungan beban yang bekerja pada balok silang :

1. Beban mati (PD)

- Beban pelat lantai = (2,5 x 2).4,51 = 22,55 kN
- Berat balok silang (25/40) = 0,25. (0,4-0,12). 24. 2 = 3,36 kN
- Berat balok bawah (25/40) = 0,25 .(0,4-0,12). 24. 2,5 = 4,2 kN +

PD = 30,11 kN

2. Beban hidup (PL)

- Beban hidup pelat lantai = (2,5 x 2). 2,5 = 12,5 kN

P = 1,2PD + 1,6PL = 1,2. 30,11 + 1,6. 12,5 = 56,132 kN

Simpul 1

Perhitungan lendutan dengan tabel Makowski :

$$\{ 9(P - x_1) + 11(P - x_2) + 7(P - x_1) \} \frac{L^3}{768EI} = x_1 . \frac{L^3}{48EI}$$

$$\{9(56,132 - x_1) + 11(56,132 - x_2) + 7(56,132 - x_1)\} \frac{8^3}{768EI} = x_1 \frac{5^3}{48EI}$$

$\{505,188-9x_1+617,452-11x_2+392,924-7x_1\}0,667 = 2,604x_1$

$1010,881-10,672x_1-7,337x_2 = 2,604x_1$

$13,276x_1+7,337x_2-1010,881 = 0$ ..... (1)

Simpul 2

$$\{11(P-x_1) + 16(P-x_2) + 11(P-x_1)\} \frac{L^3}{768EI} = x_2 . \frac{L^3}{48EI}$$

$$\{11(56,132-x_1) + 16(56,132-x_2) + 11(56,132-x_1)\} \frac{8^3}{768EI} = x_2 \frac{5^3}{48EI}$$

$\{617,452-11x_1+898,112-16x_2+617,452-11x_1\}0.667 = 2,604.x_2$

$1422,722-14,674x_1-10,672x_2 = 2,604x_2$

$14,674x_1-13,276x_2-1422,722 = 0$ ..... (2)

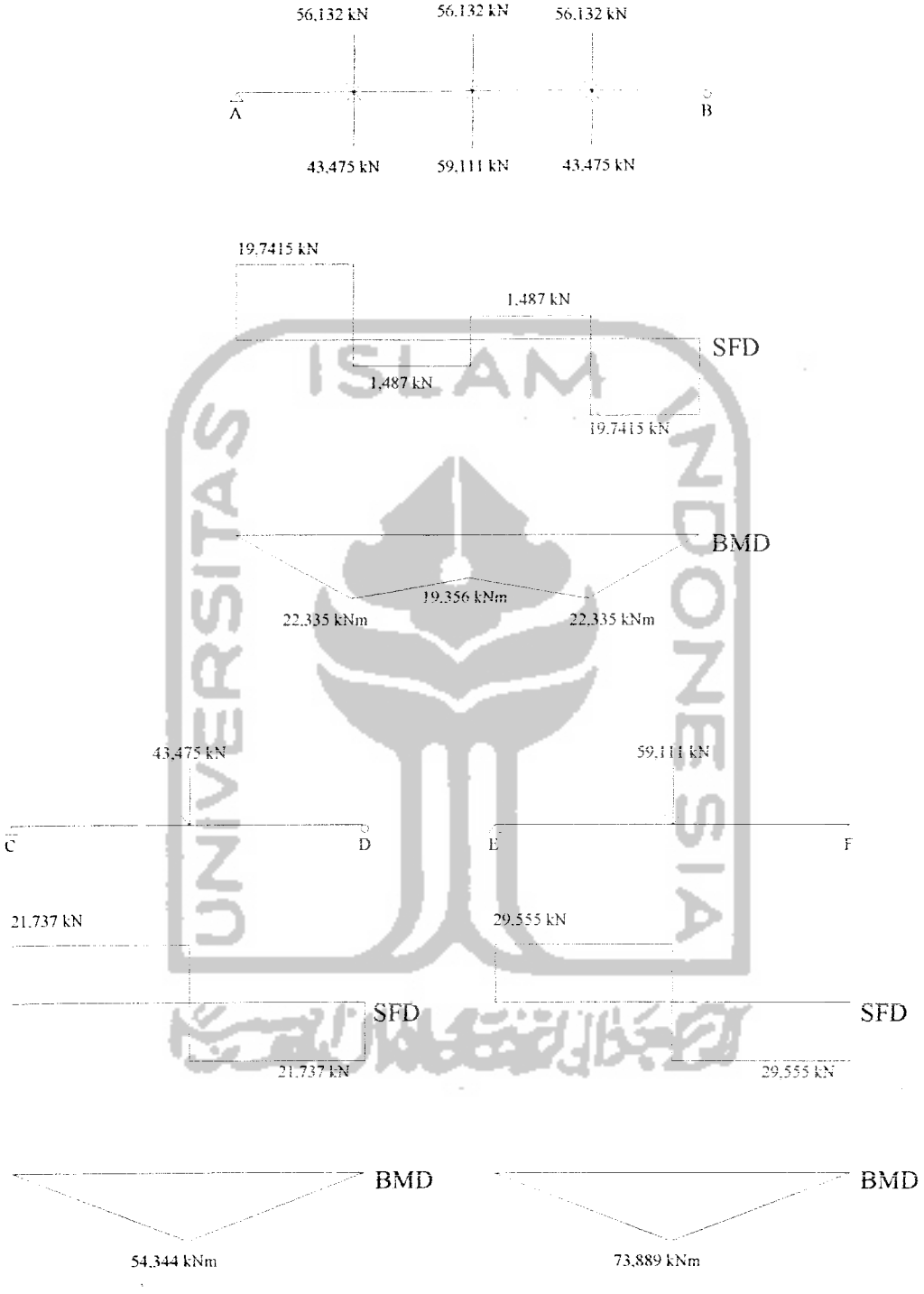
substitusi pers. (1) dan (2) :

$13,276x_1+7,337x_2 = 1010,881$

$14,674x_1+13,276x_2 = 1422,722$

didapat : $x_1 = 43,475$ kN

$x_2 = 59,111$ kN



Gambar 4.9 Grafik SFD dan BMD Balok Grid 2

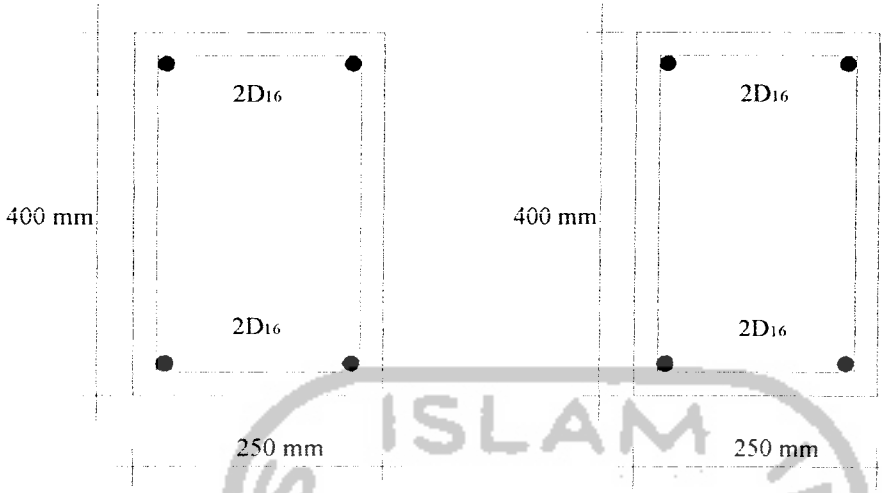
4.3.3.2 Perencanaan Tulangan Balok Grid 2

Mmaks = 22,34 kN (Dari diagram BMD bentang AB hal 116)

$$Mu = \frac{2}{3} . 22,34 = 14,89 \text{ kN}$$

Tabel 4.6 Perencanaan Balok Grid 2 (BG 2)

	Tumpuan	Lapangan
Mu (kNm)	14.89	14.89
Mu/Φ (kNm)	18.6125	18.6125
f'c (Mpa)	28	28
fy (Mpa)	400	400
β_1	0.85	0.85
m	16.80672269	16.80672269
ρb	0.030345	0.030345
ρ_{maks}	0.02275875	0.02275875
ρ_{min}	0.0035	0.0035
ρ_{pakai}	0.011379375	0.011379375
Rn (Mpa)	4.116488906	4.116488906
$b.d^2_{perlu}$	4521450.3	4521450.3
b (mm)	250	250
d_{perlu} (mm)	134.4834607	134.4834607
h (mm)	400	400
d_{pakai} (mm)	344	328
	dpakai > dperlu	
Perencanaan	Tul.sebelah	Tul.sebelah
Rn_{baru} (Mpa)	0.629140752	0.6920174
ρ_{baru}	0.001739159	0.001912971
$1,33.\rho_{baru}$	0.002313081	0.002544252
ρ_{pakai}	0.002313081	0.002544252
As_{perlu} (mm^2)	198.9250012	208.6286598
Ótul (mm)	**16**	**16**
$A_{1Ótul}$ (mm^2)	200.96	200.96
n perlu	0.989873613	1.03816013
jumlah tul. Pakai	**2**	**2**
As_{ada} (mm^2)	401.92	401.92
s > 25 (mm)	154	154
a (mm)	27.01983193	27.01983193
Mn (kNm)	53.13222983	50.55994183
Kontrol	Ok !	Ok !



Gambar 4.10 Tulangan Balok Grid 2 (BG 2)

4.3.3.3 Perencanaan Tulangan Geser Balok Grid 2 (BG 2)

Vu_{maks} = 11,17 kN (SFD bentang AB hal 116)

Tabel 4.7 Penulangan Geser Balok Grid 2

L (m)	8
b (mm)	250
h (mm)	400
d (mm)	328
f'c (Mpa)	28
fy (Mpa)	240
Vu (kN)	11.17
Vu/Φ (kN)	18.61666667
Vu/Φkritis (kN)	17.0901
Vc (kN)	72.3172025
0,5. Vc (kN)	36.15860125
Kondisi	0,5.Vc <(Vu/Φkritis)<Vc
	sengkang minimum
Ösengkang (mm)	**8**
Av (mm^2)	100.48
s (mm)	289.3824
d/2 (mm)	164
Penulangan	**P8-160**

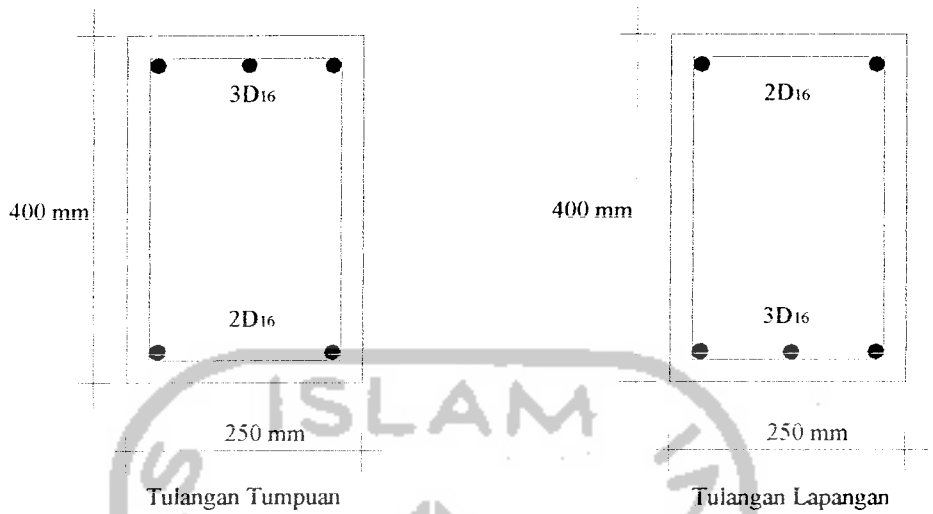
4.3.4 Perencanaan Balok Grid 2' (BG2')

Mmaks = 79,889 kNm (BMD bentang EF hal 116)

Mu = 2/3. 73,889 = 49,259 kNm

Tabel 4.8 Perencanaan Balok Grid 2' (BG 2')

	Tumpuan	Lapangan
Mu (kNm)	49.259	49.259
Mu/Φ (kNm)	61.57375	61.57375
f'c (Mpa)	28	28
fy (Mpa)	400	400
β_1	0.85	0.85
m	16.80672269	16.80672269
ρb	0.030345	0.030345
ρ_{maks}	0.02275875	0.02275875
ρ_{min}	0.0035	0.0035
ρ_{pakai}	0.011379375	0.011379375
Rn (Mpa)	4.116488906	4.116488906
$b.d^2_{perlu}$	14957832.12	14957832.12
b (mm)	250	250
d_{perlu} (mm)	244.6044327	244.6044327
h (mm)	400	400
d_{pakai} (mm)	344	328
	dpakai > dperlu	
Perencanaan	Tul.sebelah	Tul.sebelah
Rn_{baru} (Mpa)	2.081319294	2.289327409
ρ_{baru}	0.005753474	0.006328479
$1,33.\rho_{baru}$	0.007652121	0.008416878
ρ_{pakai}	0.005753474	0.006328479
As_{perlu} (mm^2)	494.7987818	518.9353077
Ótul (mm)	**16**	**16**
$A_{1Ótul}$ (mm^2)	200.96	200.96
n perlu	2.462175467	2.582281587
jumlah tul. Pakai	**3**	**3**
As_{ada} (mm^2)	602.88	602.88
s > 25 (mm)	69	69
a (mm)	40.5297479	40.5297479
Mn (kNm)	78.06937312	74.21094112
Kontrol	Ok !	Ok !



Gambar 4.11 Tulangan Balok Grid 2' (BG 2')

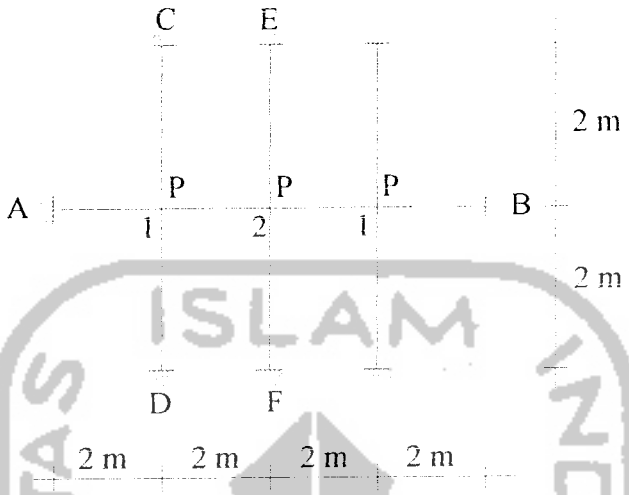
4.3.4.1 Perencanaan Tulangan Geser Balok Grid 2' (BG2')

Vumaks = 29,555 kN (Dari diagram SFD bentang EF hal 116)

Tabel 4.9 Penulangan Geser Balok Grid 2'

L (m)	5
b (mm)	250
h (mm)	400
d (mm)	328
f'c (Mpa)	28
fy (Mpa)	240
Vu (kN)	29.555
Vu/Φ (kN)	49.25833333
Vu/Φkritis (kN)	42.79564
Vc (kN)	72.3172025
0,5. Vc (kN)	36.15860125
Kondisi	0,5.Vc <(Vu/Φkritis)<Vc
	sengkang minimum
Ósengkang (mm)	**8**
Av (mm^2)	100.48
s (mm)	289.3824
d/2 (mm)	164
Penulangan	**P8-160**

4.3.5 Perencanaan Balok Grid 3 (BG 3)



Gambar 4.12 Rencana balok grid 3

4.3.5.1 Pembebanan Balok Grid 3

Perhitungan beban yang bekerja pada balok silang :

1. Beban mati (PD)

- Beban pelat lantai = (2 x 2).4,51 = 18,04 kN
- Berat balok silang (25/40) = 0,25. (0,4-0,12). 24. 2 = 3,36 kN
- Berat balok bawah (25/40) = 0,25 .(0,4-0,12). 24. 2 = 3,36 kN +

PD = 24,76 kN

2. Beban hidup (PL)

- Beban hidup pelat lantai = (2 x 2). 2,5 = 10 kN

P = 1,2PD + 1,6PL = 1,2. 24,76 + 1,6. 10 = 45,712 kN

Simpul 1

Perhitungan lendutan dengan tabel Makowski :

$$\{ 9(P - x_1) + 11(P - x_2) + 7(P - x_1) \} \frac{L^3}{768EI} = x_1 . \frac{L^3}{48EI}$$

$$\{9(45,712 - x_1) + 11(45,712 - x_2) + 7(45,712 - x_1)\} \frac{8^3}{768EI} = x_1 . \frac{4^3}{48EI}$$

$\{411,408-9x_1+502,832-11x_2+319,984-7x_1\}0,667 = 1,333x_1$

$823,227-10,672x_1-7,3373x_2 = 1,333x_1$

$12,005x_1+7,337x_2-823,227 = 0$ ..... (1)

Simpul 2

$$\{11(P-x_1) + 16(P-x_2) + 11(P-x_1)\} \frac{L^3}{768EI} = x_2 . \frac{L^3}{48EI}$$

$$\{11(45,712-x_1) + 16(45,712-x_2) + 11(45,712-x_1)\} \frac{8^3}{768EI} = x_2 . \frac{4^3}{48EI}$$

$\{502,832-11x_1+731,392-16x_2+502,832-11x_1\}0.667 = 1,333.x_2$

$1158,612-14,674x_1-10,672x_2 = 1,333x_2$

$14,674x_1-12,005x_2-1158,612 = 0$ ..... (2)

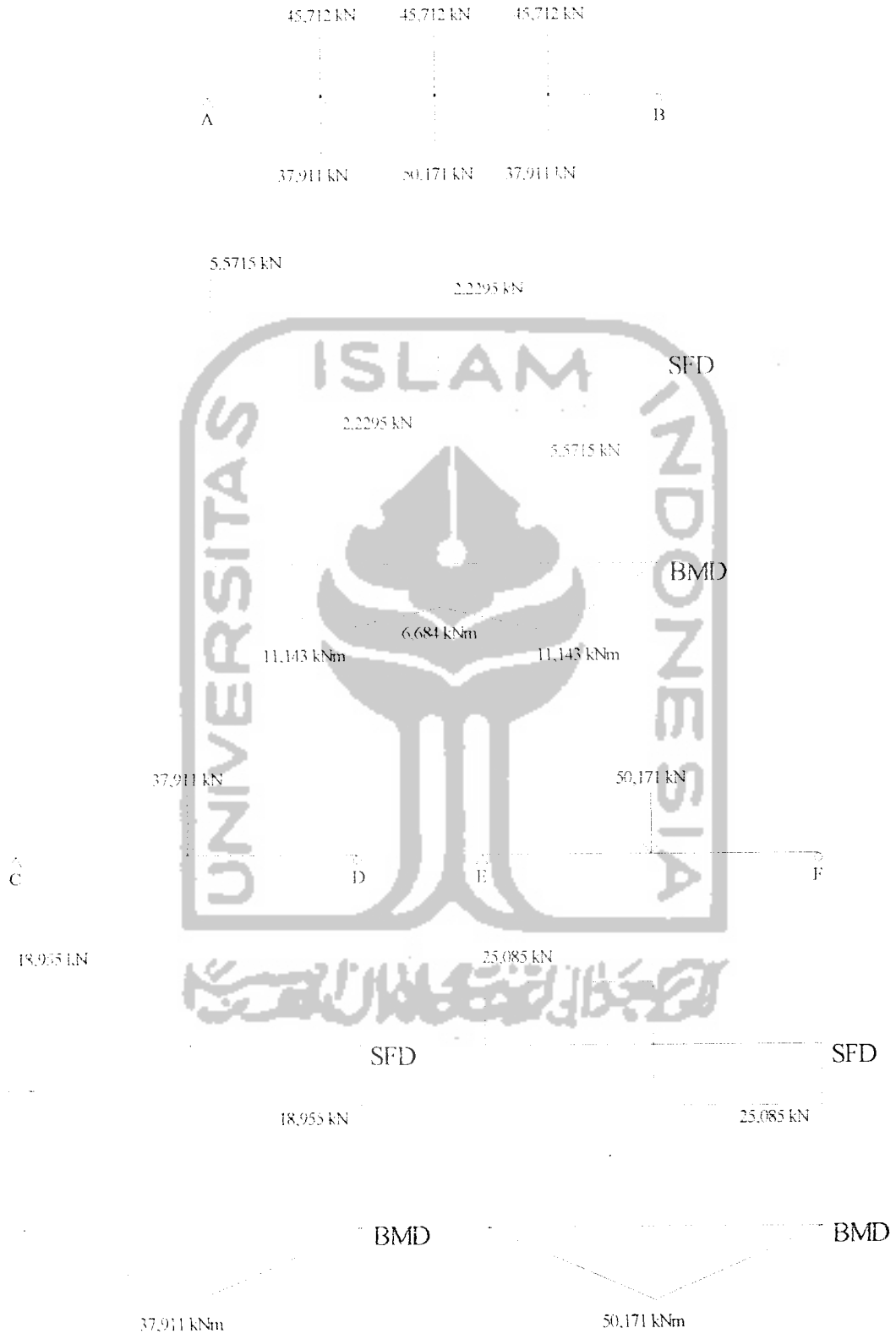
substitusi pers. (1) dan (2) :

$12,0056x_1+7,337x_2 = 823,227$

$14,674x_1+12,005x_2 = 1158,612$

didapat : $x_1 = 50,171$ kN

$x_2 = 37,911$ kN



Gambar 4.13 Grafik SFD dan BMD Balok Grid 3

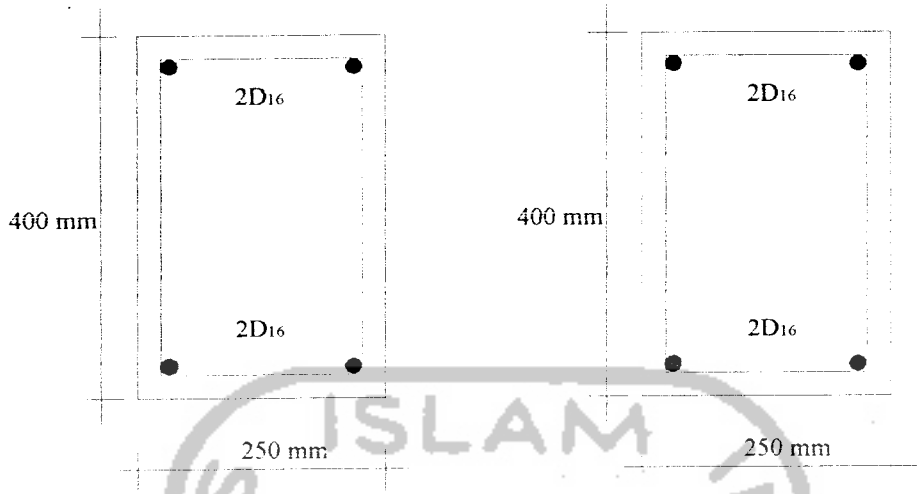
4.3.5.1 Perencanaan Tulangan Balok Grid 3 (BG 3)

Mmaks = 11,143 kN (Dari diagram BMD bentang AB hal 123)

Mu = 2/3. 11,143 = 7,429 kNm

Tabel 4.10 Perencanaan Balok Grid 3 (BG 3)

	Tumpuan	Lapangan
Mu (kNm)	7,429	7,429
Mu/Φ (kNm)	9,28625	9,28625
f'c (Mpa)	28	28
fy (Mpa)	400	400
β_1	0,85	0,85
m	16,80672269	16,80672269
ρ_b	0,030345	0,030345
ρ_{maks}	0,02275875	0,02275875
ρ_{min}	0,0035	0,0035
ρ_{pakai}	0,011379375	0,011379375
Rn (Mpa)	4,116488906	4,116488906
$b.d^2$ perlu	2255866,641	2255866,641
b (mm)	250	250
d_{perlu} (mm)	94,99192893	94,99192893
h (mm)	400	400
d_{pakai} (mm)	344	328
	dpakai > dperlu	
Perencanaan	Tul.sebelah	Tul.sebelah
Rn_{baru} (Mpa)	0,313894335	0,345265095
ρ_{baru}	0,000867711	0,00095443
$1,33.\rho_{baru}$	0,001154055	0,001269392
ρ_{pakai}	0,001154055	0,001269392
As_{perlu} (mm^2)	99,2487464	104,0901487
Ótul (mm)	**16**	**16**
$A_{1Ótul}$ (mm^2)	200,96	200,96
n perlu	0,493873141	0,517964514
jumlah tul. Pakai	**2**	**2**
As_{ada} (mm^2)	401,92	401,92
s > 25 (mm)	154	154
a (mm)	27,01983193	27,01983193
Mn (kNm)	53,13222983	50,55994183
Kontrol	Ok !	Ok !



Gambar 4.14 Tulangan Balok Grid 3 (BG 3)

4.3.5.2 Perencanaan Tulangan Geser Balok Grid 3 (BG 3)

Vumaks = 5,5715 kN (Dari diagram SFD bentang AB hal 123)

Tabel 4.11 Penulangan Geser Balok Grid 3

L (m)	8
b (mm)	250
h (mm)	400
d (mm)	328
f'c (Mpa)	28
fy (Mpa)	240
Vu (kN)	5,5715
Vu/Φ (kN)	9,285833333
Vu/Φkritis (kN)	8,524395
Vc (kN)	72,3172025
0,5. Vc (kN)	36,15860125
Kondisi	0,5.Vc <(Vu/Φkritis)<Vc
	sengkang minimum
Øsengkang (mm)	**8**
Av (mm^2)	100,48
s (mm)	289,3824
d/2 (mm)	164
Penulangan	**P8-160**

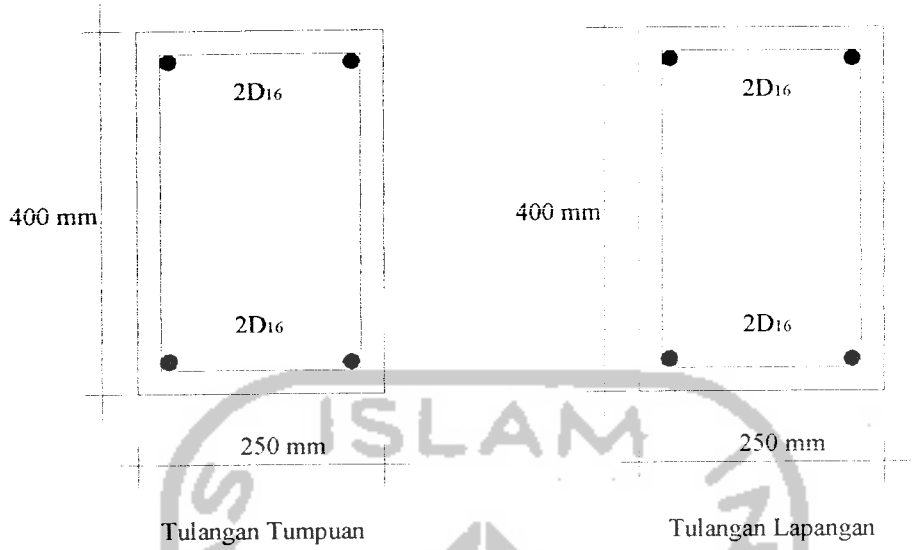
4.3.5.3 Perencanaan Tulangan Balok Grid 3' (BG 3')

Maks = 50,171 kNm (Dari diagram BMD EF hal 123)

Mu = 2/3. 50,171 = 33,447 kNm

Tabel 4.12 Perencanaan Balok Grid 3' (BG 3')

	Tumpuan	Lapngan
Mu (kNm)	33,447	33,447
Mu/Φ (kNm)	41,80875	41,80875
f'c (Mpa)	28	28
fy (Mpa)	400	400
β_1	0,85	0,85
m	16,80672269	16,80672269
ρ_b	0,030345	0,030345
ρ_{maks}	0,02275875	0,02275875
ρ_{min}	0,0035	0,0035
ρ_{pakai}	0,011379375	0,011379375
Rn (Mpa)	4,116488906	4,116488906
$b.d^2$ perlu	10156410,22	10156410,22
b (mm)	250	250
d_{perlu} (mm)	201,5580335	201,5580335
h (mm)	400	400
d_{pakai} (mm)	344	328
	dpakai > dperlu	
Perencanaan	Tul.sebelah	Tul.sebelah
Rn_{baru} (Mpa)	1,413221674	1,554459771
ρ_{baru}	0,003906625	0,004297055
$1,33.\rho_{baru}$	0,005195812	0,005715084
ρ_{pakai}	0,003906625	0,004297055
As_{perlu} (mm^2)	335,969769	352,3585383
Ótul (mm)	**16**	**16**
$A_{1Ötul}$ (mm^2)	200,96	200,96
n perlu	1,67182409	1,753376484
jumlah tul. Pakai	**2**	**2**
As_{ada} (mm^2)	401,92	401,92
s >> 25 (mm)	154	154
a (mm)	27,01983193	27,01983193
Mn (kNm)	53,13222983	50,55994183
Kontrol	Ok !	Ok !



Gambar 4.15 Tulangan Balok Grid 3' (BG 3')

4.3.5.4 Perencanaan Tulangan Geser Balok Grid 3' (BG 3')

Vumaks = 25,085 kN (Dari diagrm SFD EF hal 123)

Tabel 4.13 Penulangan Geser Balok Grid 3'

L (m)	4
b (mm)	250
h (mm)	400
d (mm)	328
fc (Mpa)	28
fy (Mpa)	240
Vu (kN)	25,085
Vu/Φ (kN)	41,80833333
Vu/Φkritis (kN)	34,95176667
Vc (kN)	72,3172025
0,5. Vc (kN)	36,15860125
Kondisi	0,5.Vc <(Vu/Φkritis)<Vc
	sengkang minimum
Ø sengkang (mm)	**8**
Av (mm^2)	100,48
s (mm)	289,3824
d/2 (mm)	164
Penulangan	**P8-160**

4.4 Perencanaan Struktur Portal dengan Daktilitas Penuh

4.4.1 Perhitungan Gaya Geser Dasar Horisontal Total Akibat Gempa

1. Beban mati yang digunakan

- Perhitungan pembebanan pelat lantai untuk beban mati per m^2

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

  berat mati total (qD) = 4,51 kN/m^2

- Perhitungan pembebanan pelat atap untuk beban mati per m^2

  1. berat pelat (taksiran) : 0,10 x 24 = 2,40 kN/m^2
  2. lapisan kedap air (tebal 3 cm) : 0,03 x 23 = 0,66 kN/m^2 +

  beban mati total (qD) = 3,09 kN/m^2

2. Beban hidup yang digunakan

Beban hidup pelat lantai untuk ruang kuliah = 2,5 kN/m^2

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

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

4.4.1.1 Arah X_1 (As – A)

A. Berat Total Bangunan

Lantai 1 s.d 4

- Beban mati

  - Pelat = 232 . 0,12 . 24 = 668,16 kN

- Balok induk = {0,4.(0,6 – 0,12)}.102 . 24 = 470 kN
- Balok anak = {0,25.(0,4 – 0,12)}.106 . 24 = 178,1 kN

  = ½ .{(0,25.(0,4 – 0,12)).58.24}= 48,7 kN
- Kolom = 12 . 3,5 . 0,6 . 0,6 . 24 = 362,9 kN
- Dinding = 74 . 3,5 . 2,5 = 647,5 kN
- Keramik = 232 . 0,24 = 55,7 kN
- Pasir = 232 . 0,05 . 18 = 208,8 kN +

  W_D = 2639,9 kN

- Beban hidup
  - Beban hidup lantai :

    Wh = 232 . 0,5 . 2,5 = 290 kN

Wtotal = 2639,9 + 290 = 2929,9 kN

Lantai atap

- Beban mati
  - Balok atap = (0,3 . 0,4 . 102).24 = 293,8 kN
  - Kuda-kuda = ½ . 2,8952 = 1,5 kN

    2 . 1,6068 = 3,2 kN

    ½ . (16 . 3,0627) = 24,5 kN

    ½ . 3,8619 = 1,9 kN

    (1/2 . 3,8619) . 1 = 1,9 kN
  - Berat baut = 0,2 . Berat total kuda-kuda = 6,6 kN
  - Plafond = 298 . 0,18 = 53,6 kN

- Genteng + reng + usuk = (60 . 6,105) . 0,5 = 183,1 kN +

W_D = 570,1 kN

Wtotal = 570,1 + 0 = 570,1 kN

W total bangunan = 4.(2929,9) + 570,1

= 12289,7 kN

B Waktu getar (T)

$Tx = Ty = 0{,}06 \cdot H^{\frac{2}{3}} = 0{,}06 \cdot 18^{\frac{2}{3}} = 0{,}412$ dt **(3.5.5)**

C. Koefisien gempa dasar

Tx = Ty = 0,412 dt ; Zona tiga dan tanah lunak diperoleh C = 0,07

D. Faktor keutamaan gedung (I) dan factor jenis bangunan (k)

I = 1,0 ; k = 1,0

E. Gaya geser horizontal akibat gempa (V) **(3.5.3)**

V = C . I . k . Wt = 0,07 . 1 . 1 . 12289,7

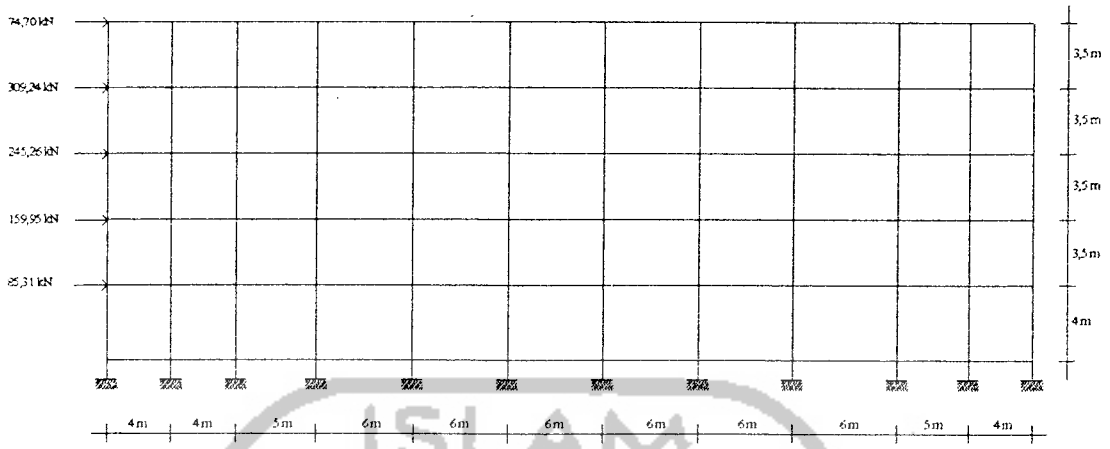
= 860,3 kN

F. Distribusi gaya geser dasar horizontal total akibat gempa (Fi)

$$Fi = \frac{Wi.Hi}{\sum Wi.Hi} V \qquad \textbf{(3.5.4)}$$

Tabel 4.14 Portal X1

Lantai	hi (m)	Wi (kN)	Wi.hi (kN)	Fix total (kN)
atap	18	570,1	10261,8	74,70
4	14,5	2929,9	42483,55	309,24
3	11,5	2929,9	33693,85	245,26
2	7,5	2929,9	21974,25	159,95
1	4	2929,9	11719,6	85,31
		Σ	120133,1	874,45



Gambar 4.16 Distribusi Beban Gempa Portal Arah X_1

4.4.1.2 Arah X_2 (arah As – C)

A. Berat total bangunan

Lantai 1 s.d 4

- Beban mati

- Pelat	= 348. 0,12 . 24	= 1002,2 kN
- Balok induk	= {0,4.(0,6 – 0,12)}.124 . 24	= 571,4 kN
- Balok anak	= {0,25.(0,4 – 0,12)}.130 . 24	= 218,4 kN
	= ½ .{(0,25.(0,4 – 0,12)).58.24}	= 48,7 kN
- Kolom	= 11 . 3,5 . 0,6 . 0,6 . 24	= 332,6 kN
- Dinding	= 66 . 3,5 . 2,5	= 577,5 kN
- Keramik	= 348 . 0,24	= 83,5 kN
- Pasir	= 348 . 0,05 . 18	= 313,2 kN +
		W_D = 3147,5 kN

- Beban hidup

  - Beban hidup lantai :

    Wh = 348 . 0,5 . 2,5 = 435 kN

- Beban hidup selasar :

Wh = 50 .2. 3 . 0,5 = 150 kN +

Wh = 585 kN

Wtotal = 3147,5 + 585 = 3732,5 kN

Lantai atap

- Beban mati

- Pelat	= 50 . 2 . .0,1 . 24	= 240 kN
- Balok atap	= (0,3 . 0,4 . 124).24	= 357,1 kN
- Balok anak	= {0,25.(0,4 – 0,10).50}.24	= 84 kN
- Kuda-kuda	= ½ . 2,8952	= 1,4 kN
	2 . 1,6068	= 3,2 kN
	½ . (16 . 3,0627)	= 24,5 kN
	½ . 3,8619	= 1,9 kN
	(1/2 . 3,8619) . 1	= 1,9 kN
- Berat baut	= 0,2 . Berat total kuda-kuda	= 6,6 kN
- Plafond	= 300 . 0,18	= 54 kN
- Genting + reng + usuk = (60 . 6,102) . 0,5		=183,1 kN +
		W_D = 957,7 kN

- Beban hidup

- Pelat atap :

Wh = 50 . 2 . 1 . 0,5 = 50 kN

Wtotal = 957,7 + 50 = 1007,7 kN

W total bangunan = 4.(3732,5) + 1007,7

= 15937,7 kN

B. Waktu getar (T)

$$Tx = Ty = 0{,}06 \cdot H^{\frac{2}{3}} = 0{,}06 \cdot 18^{\frac{2}{3}} = 0{,}412 \text{ dt} \qquad \textbf{(3.5.5)}$$

C. Koefisien gempa dasar

Tx = Ty = 0,412 dt ; Zona tiga dan tanah lunak diperoleh C = 0,07

D. Faktor keutamaan gedung (I) dan faktor jenis bangunan (k)

I = 1,0 ; k = 1,0

E. Gaya geser horizontal akibat gempa

$$V = C \cdot I \cdot k \cdot Wt = 0{,}07 \cdot 1 \cdot 1 \cdot 15937{,}7 \qquad \textbf{(3.5.3)}$$

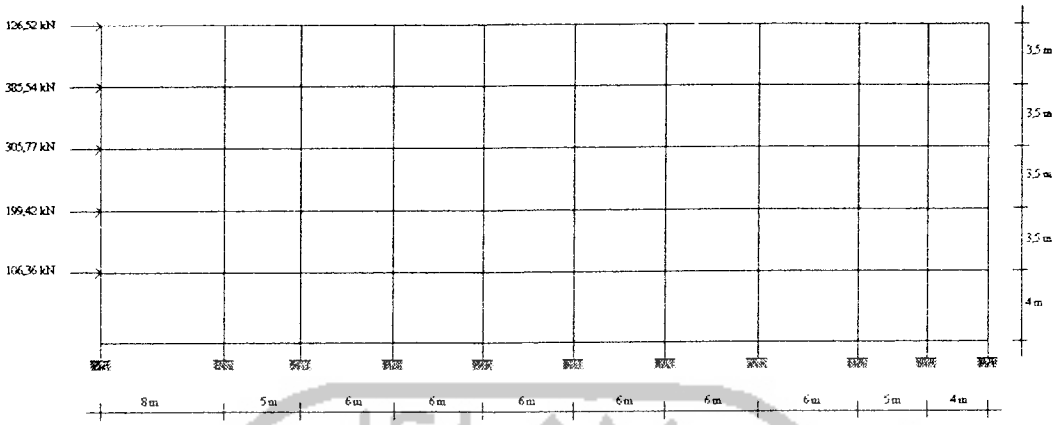
= 1115,6 kN

F. Distribusi gaya geser dasar horizontal total akibat gempa

$$Fi = \frac{Wi.Hi}{\sum Wi.Hi}.V \qquad \textbf{(3.5.4)}$$

Tabel 4.15 PORTAL X2

Lantai	hi (m)	Wi (kN)	Wi.hi (kN)	Fix total (kN)
atap	18	1007,7	18138,6	126,52
4	14,5	3732,5	54121,25	385,54
3	11,5	3732,5	42923,75	305,77
2	7,5	3732,5	27993,75	199,42
1	4	3732,5	14930	106,36
		Σ	158107,4	1123,61



Gambar 4.17 Distribusi Beban Gempa Portal Arah X_2

4.4.1.3 Arah X_3 (As – E) = arah Y_3 (As – 9)

A. Berat total bangunan

Lantai 1 s.d 4

- Beban mati

- Pelat	= 60 . 0,12 . 24	= 172,8 kN
- Balok induk	= {0,4.(0,6 – 0,12)}.22 . 24	= 101,4 kN
- Balok anak	= {0,25.(0,4 – 0,12)}.34 . 24	= 57,1 kN
- Kolom	= 2 . 3,5 . 0,6 . 0,6 . 24	= 60,5 kN
- Dinding	= 20 . 3,5 . 2,5	= 175 kN
- Keramik	= 60 . 0,24	= 14,4 kN
- Pasir	= 60 . 0,05 . 18	= 54 kN +
		W_D = 635,2 kN

- Beban hidup

  - Beban hidup lantai :

    Wh = 48 . 0,5 . 2,5 = 60 kN

  - Beban hidup selasar :

Wh = 6 .2. 3 . 0,5 = 18 kN +

Wh = 78 kN

Wtotal = 635,2 + 78 = 713,2 kN

Lantai atap

- Beban mati

- Pelat	= 6 . 2 . .0,1 . 24	= 28,8 kN
- Balok atap	= (0,3 . 0,4 . 22).24	= 63,4 kN
- Balok anak	= {0,25.(0,4 – 0,10).6}.24	= 10,8 kN
- Kuda-kuda	= 3,0627	= 3,1 kN
- Berat baut	= 0,2 . Berat total kuda-kuda	= 0,6 kN
- Plafond	= 10 . 0,18	= 1,8 kN
- Genteng + reng + usuk = 2.(6 . 6,102).0,5		= 36,6 kN +
		W_D = 126,8 kN

- Beban hidup

- Pelat atap :

Wh = 6 . 2 . 1 . 0,5 = 6 kN

Wtotal = 126,8 + 6 = 132,8 kN

W total bangunan = 4.(713,2) + 132,8

= 2985,6 kN

B. Waktu getar (T)

Tx = Ty = 0,06 . $H^{2/3}$ = 0,06 . $18^{2/3}$ = 0,412 dt

C. Koefisien gempa dasar

Tx = Ty = 0,412 dt ; Zona tiga dan tanah lunak diperoleh C = 0,07

D. Faktor keutamaan gedung (I) dan faktor jenis bangunan (k)

I = 1,0 ; k = 1,0

E. Gaya geser horizontal akibat gempa

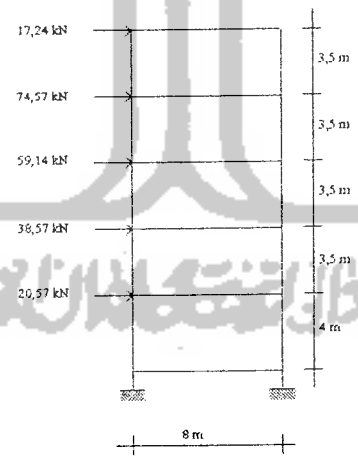
V = C . I . k . Wt = 0,07 . 1 . 1 . 2985,6

= 208,9 kN

F. Distribusi gaya geser dasar horizontal total akibat gempa

Tabel 4.16 PORTAL X3 =Y3

Lantai	hi (m)	Wi (kN)	Wi.hi (kN)	Fix total (kN)
atap	18	132,8	2390,4	17,24
4	14,5	713,2	10341,4	74,57
3	11,5	713,2	8201,8	59,14
2	7,5	713,2	5349	38,57
1	4	713,2	2852,8	20,57
		∑	29135,4	210,09



Gambar 4.18 Distribusi Beban Gempa Portal Arah X_3 & Y_3

4.4.1.4 Arah Y_1 (As – 1)

A. Berat total bangunan

Lantai 1 s.d 4

- Beban mati

- Pelat = 124 . 0,12 . 24 = 357,1 kN
- Balok induk = {0,4.(0,6 – 0,12)}.59 . 24 = 271,9 kN
- Balok anak = {0,25.(0,4 – 0,12)}.59 . 24 = 99,1 kN

  = ½ .{(0,25.(0,4 – 0,12)).31.24}= 26,0 kN
- Kolom = 8 . 3,5 . 0,6 . 0,6 . 24 = 241,9 kN
- Dinding = 43 . 3,5 . 2,5 = 376,2 kN
- Keramik = 124 . 0,24 = 29,8 kN
- Pasir = 124 . 0,05 . 18 = 111,6 kN +

W_D = 1513,7 kN

- Beban hidup
  - Beban hidup lantai :

    Wh = 124 . 0,5 . 2,5 = 155 kN

Wtotal = 1513,7 + 155 = 1668,7 kN

Lantai atap

- Beban mati
  - Balok atap = (0,3 . 0,4 . 59).24 = 169,9 kN
  - Kuda-kuda = ½ . 2,8952 = 1,4 kN

    2 . 1,6068 = 3,2 kN

    ½ . (7 . 3,0627) = 10,7 kN

    ½ . 3,8619 = 1,9 kN

    (1/2 . 3,8619) . 1 = 1,9 kN
  - Berat baut = 0,2 . Berat total kuda-kuda = 3,8 kN
  - Plafond = 163 . 0,18 = 29,3 kN

- Genteng + reng + usuk = (6,102 . 33) . 0,5 = 100,7 kN +

W_D = 322,8 kN

Wtotal = 322,8 + 0 = 322,8 kN

W total bangunan = 4.(1668,7) + 322,8

= 6997,6 kN

B. Waktu getar (T)

$Tx = Ty = 0{,}06 \cdot H^{\frac{2}{3}} = 0{,}06 \cdot 18^{\frac{2}{3}} = 0{,}412$ dt **(3.5.5)**

C. Koefisien gempa dasar

Tx = Ty = 0,412 dt ; Zona tiga dan tanah lunak diperoleh $C = 0{,}07$

D. Faktor keutamaan gedung (I) dan faktor jenis bangunan (k)

I = 1,0 ; k = 1,0

E. Gaya geser horizontal akibat gempa

$V = C \cdot I \cdot k \cdot Wt = 0{,}07 \cdot 1 \cdot 1 \cdot 6997{,}6$ **(3.5.3)**

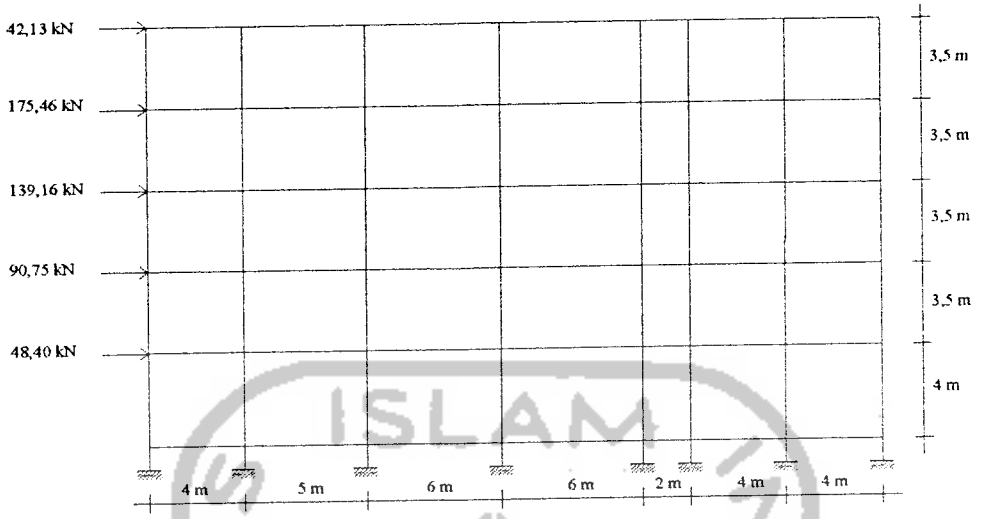
= 489,8 kN

F. Distribusi gaya geser dasar horizontal total akibat gempa

$$Fi = \frac{Wi.Hi}{\sum Wi.Hi} V \quad \textbf{(3.5.4)}$$

Tabel 4.17 PORTAL Y1

Lantai	hi (m)	Wi (kN)	Wi.hi (kN)	Fix total (kN)
atap	18	322,8	5810,4	42,13
4	14,5	1668,7	24196,15	175,46
3	11,5	1668,7	19190,05	139,16
2	7,5	1668,7	12515,25	90,76
1	4	1668,7	6674,8	48,40
		Σ	68386,65	495,91



Gambar 4.19 Distribusi Beban Gempa Portal Arah Y_1

4.4.1.5 Arah Y_2 (As – 3)

A. Berat total bangunan

Lantai 1 s.d 4

- Beban mati

- Pelat	= 191. 0,12 . 24	= 550,08 kN
- Balok induk	= {0,4.(0,6 – 0,12)}.72. 24	= 331,76 kN
- Balok anak	= {0,25.(0,4 – 0,12)}.90. 24	= 151,2 kN
	= ½ .{(0,25.(0,4 – 0,12)).31.24}=	26,04 kN
- Kolom	= 7 . 3,5 . 0,6 . 0,6 . 24	= 211,68 kN
- Dinding	= 48 . 3,5 . 2,5	= 420 kN
- Keramik	= 191 . 0,24	= 45,84 kN
- Pasir	= 191 . 0,05 . 18	= 171,9 kN +
		W_D = 1908,5 kN

- Beban hidup

- Beban hidup lantai :

  Wh = 134 . 0,5 . 2,5 = 167,5 kN

- Beban hidup selasar :

  Wh = 46 . 3 . 0,5 = 69 kN +

  Wh = 236,5 kN

Wtotal = 1908,5 + 236,5 = 2145 kN

Lantai atap

- Beban mati

  - Pelat = 47 . 0,1 . 24 = 112,8 kN
  - Balok atap = (0,3 . 0,4 .72).24 = 207,4 kN
  - Balok anak = {0,25.(0,4 – 0,10).23,5}.24 = 42,3 kN
  - Kuda-kuda = 1,5 . 2,8952 = 4,3 kN

    1 . 1,6068 = 1,61 kN

    ½ . (7 . 3,0627) = 10,72 kN

    ½ . 3,8619 = 1,93 kN

    (1/2 . 3,8619) . 1 = 1,93 kN
  - Berat baut = 0,2 . Berat total kuda-kuda = 4,12 kN
  - Plafond = 175 . 0,18 = 31,5 kN
  - Genteng + reng + usuk = (6,102 . 33) . 0,5 = 100,7 kN +

    W_D = 519,2 kN

- Beban hidup

  - Pelat atap :

    Wh = 23. 2 . 1 . 0,5 = 23 kN

Wtotal = 519,2 + 23 = 542,2 kN

W total bangunan = 4.(2145) + 542,2

= 9122,2 kN

B. Waktu getar (T)

$$Tx = Ty = 0{,}06 \cdot H^{\frac{2}{3}} = 0{,}06 \cdot 18^{\frac{2}{3}} = 0{,}412 \text{ dt} \qquad \textbf{(3.5.5)}$$

C. Koefisien gempa dasar

Tx = Ty = 0,412 dt ; Zona tiga dan tanah lunak diperoleh C = 0,07

D. Faktor keutamaan gedung (I) dan faktor jenis bangunan (k)

I = 1,0 ; k = 1,0

E. Gaya geser horizontal total akibat gempa

$$V = C \cdot I \cdot k \cdot Wt = 0{,}07 \cdot 1 \cdot 1 \cdot 9122{,}2 \qquad \textbf{(3.5.3)}$$

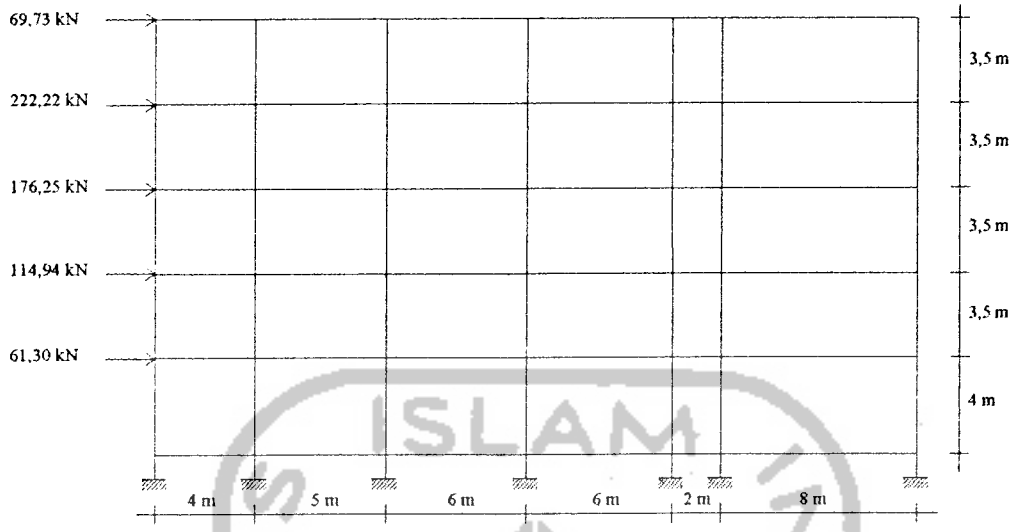
= 638,5 kN

F. Distribusi gaya geser dasar horizontal total akibat gempa

$$Fi = \frac{Wi.Hi}{\sum Wi.Hi} V \qquad \textbf{(3.5.4)}$$

Tabel 4.18 PORTAL Y2

Lantai	hi (m)	Wi (kN)	Wi.hi (kN)	Fix total (kN)
atap	18	542,2	9759.6	69,73
4	14,5	2145	31102.5	222,22
3	11,5	2145	24667.5	176,25
2	7,5	2145	16087.5	114,94
1	4	2145	8580	61,30
		∑	90197.1	644,45



Gambar 4.20 Distribusi Beban Gempa Portal Arah Y_2

4.4.2 Desain Balok

4.4.2.1 Desain Tulangan Lentur Balok

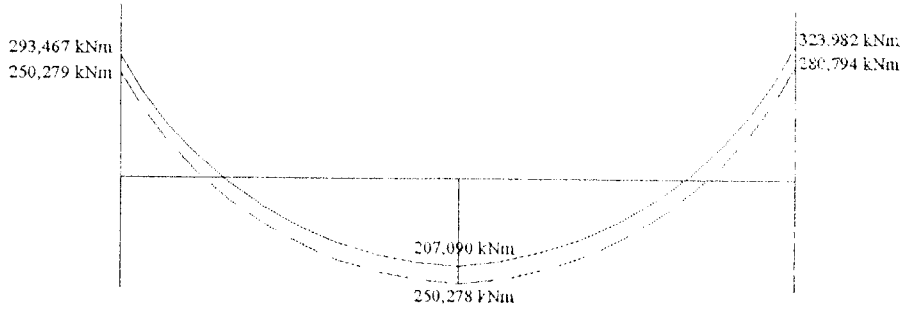
A. Momen Rencana Balok

Momen rencana balok diambil yang terbesar setelah dikombinasikan sebagai berikut :

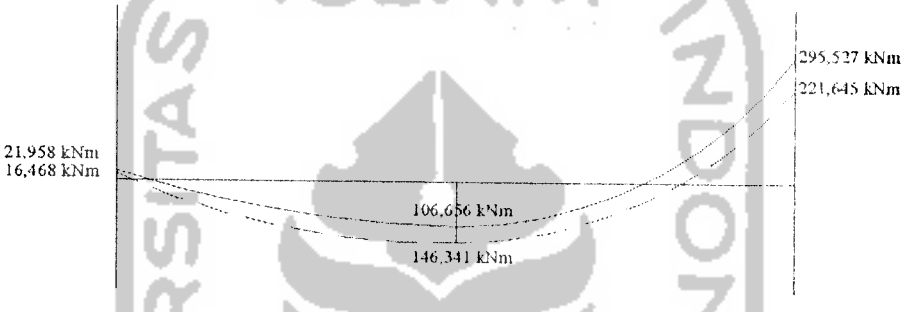
1. $1{,}2.M_D + 1{,}6.M_L$
2. $0{,}9.M_D \pm M_E$
3. $1{,}05\ (M_D + 0{,}6.M_L \pm M_E)$

Berikut diberikan contoh perhitungan balok pada portal As-E lantai 1. Untuk hasil perhitungan balok lainnya dapat dilihat pada lampiran.

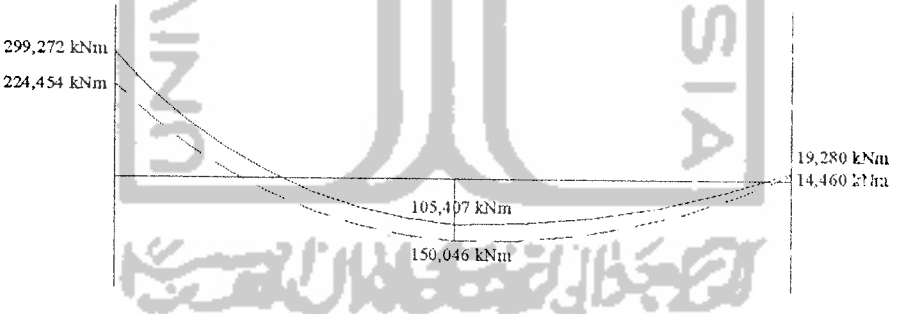
1,2MD + 1,6ML



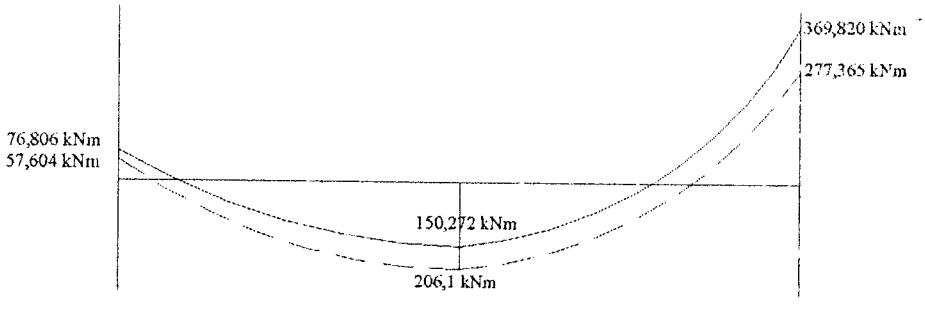
0,9MD ± ME_{kiri}



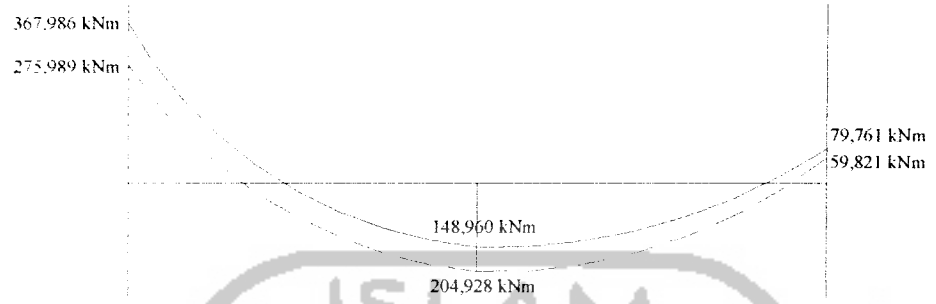
0,9MD ± ME_{kanan}



1,05(MD + 0,6ML ± ME_{kiri})



1,05(MD + 0,6ML ± ME_{kanan})



Gambar 4.21 Diagram Momen

	Tumpuan kiri	Lapangan	Tumpuan kanan
Mu_{maks}	-275,9890 kNm	250,2780 kNm	-280,794 kNm

B. Tulangan Tumpuan Kiri **(Perhitungan menggunakan rumus 3.3.10 – 3.3.22)**

Mu maks = 275,9890 kNm

$$Mn = \frac{Mu}{\phi} = \frac{275,9890}{0,8} = 344,98625 \text{ kNm}$$

Perencanaan balok dipakai ukuran 400/600

$$\rho_b = \frac{0,85.f'c}{fy}\beta 1\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{240}.0,85\left(\frac{600}{600+400}\right) = 0,0303$$

$\rho_{maks} = 0,75.\ \rho_b = 0,75.\ 0,0303 = 0,0227$

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

$\rho_{pakai} = 0,5.\ \rho_{maks} = 0,5.\ 0,0227 = 0,0114$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,8067$$

$Rn = \rho_{pakai}.\ fy.\ (1\text{-}0,5.\ \rho_{pakai}.m) = 0,0114.\ 400\ (1\text{-}0,5.0,0114.16,8067)$

$= 4,1165$ MPa

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{344{,}98625.10^6}{4{,}1165} = 83805946{,}73 \text{ mm}^2$$

diambil b = 400 mm, maka :

$$d_{perlu} = \sqrt{\frac{83805946{,}73}{400}} = 457{,}7279 \text{ mm}$$

d_{pakai} = h-pb-Ósengkang-1/2.Ótul.pokok

= 600-40-10-1/2.22 = 539 mm > d_{perlu} = 457,7279 mm

Karena $d_{pakai} > d_{perlu}$, maka direncanakan sebagai tulangan sebelah.

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{344{,}98625.10^6}{400.539^2} = 2{,}9687 \text{ MPa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho_{pakai} = \frac{2{,}9687}{4{,}1165}.0{,}0114 = 0{,}00821 > \rho_{min} = 0{,}0035$$

$\rho_{pakai} = \rho_{baru} = 0{,}00821$

$As_{perlu} = \rho_{pakai}.b.d = 0{,}00821.\ 400.\ 539 = 1769{,}312 \text{ mm}^2$

A1Ó22 = 379,94 mm^2

$$n = \frac{1769{,}312}{379{,}94} = 4{,}66 \approx 5 \text{ buah}$$

dipakai 5Ó22, maka :

$As_{tul} = 5 \times 379{,}94 = 1899{,}7 \text{ mm}^2 > As_{perlu} = 1769{,}312 \text{ mm}^2$

$$s = \frac{b - 2.pb - 2.\phi sengkang - n.\phi tul}{(n-1)} = \frac{400 - 2.40 - 2.10 - 5.22}{5-1}$$

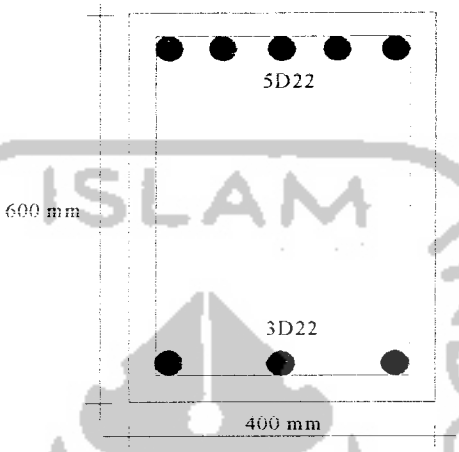
= 47,5 mm ≥ 25 mm

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{1899{,}7.400}{0{,}85.28.400} = 79{,}82mm$$

$$Mn = As_{pakai}.fy.\ (d - \frac{a}{2}) = 1899{,}7.\ 400\ (539 - \frac{79{,}82}{2})$$

$$= 379{,}249\ \text{KNm} > \frac{Mu}{\phi} = 344{,}986\ \text{KNm}\ldots\ldots(\text{OK})$$



Gambar 4.22 Tulangan Tumpuan kiri

C. Tulangan Lapangan **(Perhitungan menggunakan rumus 3.3.10 – 3.3.22)**

Mu maks = 250,2780 kNm

$$Mn = \frac{Mu}{\phi} = \frac{250{,}2780}{0{,}8} = 312{,}8475\ \text{kNm}$$

Perencanaan balok dipakai ukuran 400/600

$$\rho_b = \frac{0{,}85.f'c}{fy}\beta 1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28}{240}.0{,}85\left(\frac{600}{600+400}\right) = 0{,}0303$$

$$\rho_{maks} = 0{,}75.\ \rho_b = 0{,}75.\ 0{,}0303 = 0{,}0227$$

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

$$\rho_{pakai} = 0{,}5.\ \rho_{maks} = 0{,}5.\ 0{,}0227 = 0{,}0114$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.28} = 16{,}8067$$

$Rn = \rho_{pakai}.\ fy.\ (1\text{-}0{,}5.\ \rho_{pakai}.m) = 0{,}0114.\ 400\ (1\text{-}0{,}5.0{,}0114.16{,}8067)$

$= 4{,}1165$ MPa

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{312{,}8475.10^6}{4{,}1165} = 75998625{,}8 \text{ mm}^2$$

diambil b = 400 mm, maka :

$$d_{perlu} = \sqrt{\frac{75998625{,}8}{400}} = 435{,}8859 \text{ mm}$$

d_{pakai} = h-pb-Ósengkang-1/2.Ótul.pokok

$= 600\text{-}40\text{-}10\text{-}1/2.22 = 539$ mm > $d_{perlu} = 435{,}8859$ mm

Karena $d_{pakai} > d_{perlu}$, maka direncanakan sebagai tulangan sebelah.

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{312{,}8475.10^6}{400.539^2} = 2{,}6921 \text{ MPa}$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho_{pakai} = \frac{2{,}6921}{4{,}1165}.0{,}0114 = 0{,}00744 > \rho_{min} = 0{,}0035$$

$\rho_{pakai} = \rho_{min} = 0{,}00744$

$As_{perlu} = \rho_{pakai}.b.d = 0{,}00744.\ 400.\ 539 = 1604{,}484 \text{ mm}^2$

A1Ó22 = 379,94 mm^2

$$n = \frac{1604{,}484}{379{,}94} = 4{,}223 \approx 5 \text{ buah}$$

dipakai 5Ó22, maka :

$As_{tul} = 5 \times 379{,}94 = 1899{,}7 \text{ mm}^2 > As_{perlu} = 1604{,}484 \text{ mm}^2$

$$s = \frac{b - 2.pb - 2.\phi sengkang - n.\phi tul}{(n-1)} = \frac{400 - 2.40 - 2.10 - 5.22}{5-1}$$

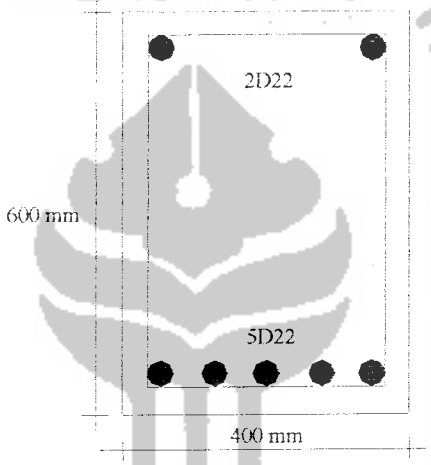
$= 47{,}5 \text{ mm} \geq 25 \text{ mm}$

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{1899{,}7.400}{0{,}85.28.400} = 79{,}82mm$$

$$Mn = As_{pakai}.fy.\ (d - \frac{a}{2}) = 1899{,}7.\ 400\ (539 - \frac{79{,}82}{2})$$

$$= 379{,}249\ KNm > \frac{Mu}{\phi} = 312{,}8475\ KNm\ldots\ldots(OK)$$



Gambar 4.23 Tulangan Lapangan

D.Tul. Tumpuan Kanan **(Perhitungan menggunakan rumus 3.3.10 – 3.3.22)**

Mu maks = 280,794 kNm

$$Mn = \frac{Mu}{\phi} = \frac{280{,}794}{0{,}8} = 350{,}9925\ kNm$$

Perencanaan balok dipakai ukuran 400/600

$$\rho_b = \frac{0{,}85.f'c}{fy}\beta 1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28}{240}.0{,}85\left(\frac{600}{600+400}\right) = 0{,}0303$$

$$\rho_{maks} = 0{,}75.\ \rho_b = 0{,}75.\ 0{,}0303 = 0{,}0227$$

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

$\rho_{pakai} = 0,5.\ \rho_{maks} = 0,5.\ 0,0227 = 0,0114$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,8067$$

$Rn = \rho_{pakai}.\ fy.\ (1-0,5.\ \rho_{pakai}.m) = 0,0114.\ 400\ (1-0,5.0,0114.16,8067)$

$= 4,1165$ MPa

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{350,9925.10^6}{4,1165} = 85265017,83\ mm^2$$

diambil b = 400 mm, maka :

$$d_{perlu} = \sqrt{\frac{85265017,83}{400}} = 461,695\ mm$$

d_{pakai} = h-pb-Ósengkang-1/2.Ótul.pokok

$= 600-40-10-1/2.22 = 539$ mm > $d_{perlu} = 461,695$ mm

Karena $d_{pakai} > d_{perlu}$, maka direncanakan sebagai tulangan sebelah.

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{350,9925.10^6}{400.539^2} = 3,02037\ MPa$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho_{pakai} = \frac{3,02037}{4,1165}.0,0114 = 0,00835 > \rho_{min} = 0,0035$$

$\rho_{pakai} = \rho_{baru} = 0,00835$

$As_{perlu} = \rho_{pakai}.b.d = 0,00835.\ 400.\ 539 = 1800,116\ mm^2$

A1Ó22 = 379,94 mm^2

$$n = \frac{1800,116}{379,94} = 4,74 \approx 5 \text{ buah}$$

dipakai 5Ó22, maka :

$As_{tul} = 5 \times 379,94 = 1899,7\ mm^2 > As_{perlu} = 1800,116\ mm^2$

$$s = \frac{b - 2.pb - 2.\phi sengkang - n.\phi tul}{(n-1)} = \frac{400 - 2.40 - 2.10 - 5.22}{5-1}$$

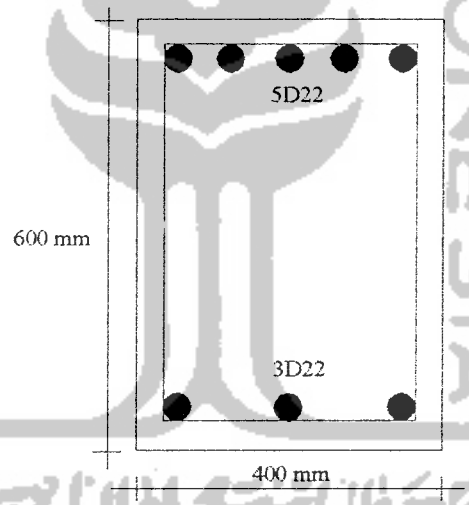
$$= 47{,}5 \text{ mm} \geq 25 \text{ mm}$$

Kontrol Mn :

$$a = \frac{As_{pakai}.fy}{0{,}85.f'c.b} = \frac{1899{,}7.400}{0{,}85.28.400} = 79{,}82mm$$

$$Mn = As_{pakai}.fy.\ (d - \frac{a}{2}) = 1899{,}7.\ 400\ (539 - \frac{79{,}82}{2})$$

$$= 379{,}249 \text{ KNm} > \frac{Mu}{\phi} = 350{,}9925 \text{ KNm}\ldots\ldots(OK)$$



Gambar 4.24 Tulangan Tumpuan Kanan

4.4.2.2 Momen Nominal Aktual Balok Tumpuan

Tulangan atas = 5D22, As = 1899,7 mm^2, $\rho = \frac{As}{b.d_{pakai}} = \frac{1899{,}7}{400.539} = 0{,}00881$

Tulangan bawah= 3D22, As'= 1140,4 mm^2, $\rho' = \frac{As'}{b.d_{pakai}} = \frac{1140{,}4}{400.539} = 0{,}0053$

$$fs' = \left(1 - \frac{0{,}85.f'c.\beta_1.d'}{(\rho - \rho').fy.d}\right).600$$

$$= \left(1 - \frac{0{,}85.28.0{,}85.61}{(0{,}00881 - 0{,}0053).400.539}\right).600$$

= 378,41 MPa < fy = 400 MPa, maka baja desak belum leleh

$fs'_{pakai} = fs' = 378{,}41$ MPa

$$a = \frac{As.fy - As'.fs'}{0{,}85.f'c.b} = \frac{1899{,}7.400 - 1140{,}4.378{,}41}{0{,}85.28.400} = 34{,}49 \text{ mm}$$

Mn1 = (As.fy-As'.fs').(d-a/2)

$$= (1899{,}7.400\text{-}1140{,}4.378{,}41).(539\text{-}\frac{34{,}49}{2}) = 171{,}314 \text{ kNm}$$

Mn2 = As'.fs'.(d – d')

= 1140,4.378,41.(539-61) = 206,275 kNm

$Mnak^{-}$ = Mn1 + Mn2 = 171,314 + 206,275 = 377,589 kNm

4.4.2.3 Momen Nominal Aktual Balok Daerah Lapangan

$$\rho = \frac{As}{b.d_{pakai}} = \frac{1140{,}4}{400.539} = 0{,}0053$$

Rn = ρ.fy.(1- ½ .ρ.m) = 0,0053. 400. (1- ½. 0,0053. 16,8067) = 2,0256

$Mnak^{+}$ = Rn. b. d^2 = 2,0256. 400. 539^2 = 235,392 KNm

4.4.2.4 Desain Tulangan Geser Balok

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut :

$$Vu,b = 0{,}7\ \phi_0 \left[\frac{M_{nak.b} + M_{nak.b}}{Ln}\right] + 1{,}05.Vg \qquad \textbf{(3.6.6)}$$

Tetapi tidak lebih besar dari

$$Vu,b = 1{,}05\ (\ V_{D,b} + V_{L,b} + 4/k\ V_{E,b}\ ) \qquad \textbf{(3.6.7)}$$

V_D = 121,549 kN V_L = 40,6476 kN V_E = 38,9112 kN

$$Vu,b = 0,7\ \Phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln} \right] + 1,05.Vg$$

$$Vu,b = 0,7\ 1,25 \left[\frac{377,589 + 235,392}{7,4} \right] + 1,05.(121,549 + 40,6476)$$

$$= 242,787 \text{ kN}$$

Dengan syarat tidak lebih besar dari :

$$Vu,b = 1,05\ (121,549 + 40,6476 + 4/1\ .\ 38,9112\ )$$

$$= 333,7335 \text{ kN}$$

Vu,b pakai

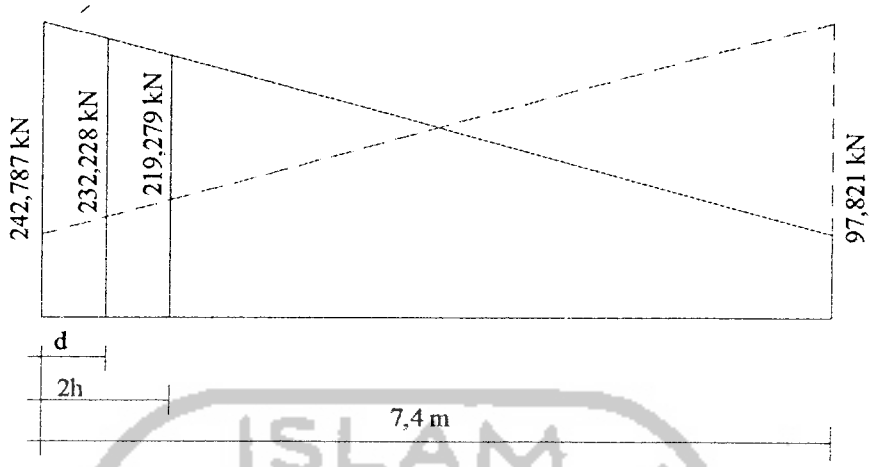
$$= \left[1,05.Vg - 0,7.\phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln} \right] \right] + \frac{Ln - d}{Ln}$$

$$\left[V_{u,b} - \left[1,05.Vg - 0,7.\phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln} \right] \right] \right]$$

$$= \left[1,05.(121,549 + 40,6476) - 0,7.1,25.\left(\frac{377,589 + 235,392}{7,4} \right) \right] + \frac{7,4 - 0,539}{7,4}$$

$$\left[242,787 - \left[1,05.(121,549 + 40,6476) - 0,7.1,25.\left[\frac{377,589 + 235,392}{7,4} \right] \right] \right]$$

$$= 232,228 \text{ kN}$$



Gambar 4.25 Gaya geser pada penampang kritis dan sendi plastis

1. Di dalam daerah sendi plastis

   Vu,b = 232,228 kN

   Vc = 0

   Vs = Vu,b/ϕ = 232,228/0,6 = 387,047 kN

   $$S \leq \frac{Av.fy.d}{Vs} = \frac{(2.\frac{1}{4}.\pi.10^2).240.539}{387,047.10^3} = 52,5mm$$

   $$d/4 \leq \frac{539}{4} = 134,75mm$$

   dipakai $P_{10\text{-}50}$

2. Di luar sendi plastis

   $$Vu,b = 97,821 + \frac{7,4 - 2.0,6}{7,4}.(242,787 - 97,821) = 219,279 \text{ kN}$$

   $$Vc = \frac{1}{6}\sqrt{f'c}.b.d = \frac{1}{6}.\sqrt{28}.400.539 = 190,141 \text{ kN}$$

   $$Vs = \frac{Vu,b}{\phi} - Vc = \frac{219,279}{0,6} - 190,141 = 175,324 \text{ kN}$$

   Syarat spasi

$$S \leq d/2 = \frac{539}{2} = 269{,}5 \text{ mm}$$

$$S \leq 600 \text{ mm}$$

$$S \leq \frac{Av.fy.d}{Vs} = \frac{(2.\frac{1}{4}.\pi.10^2).240.539}{175{,}324.10^3} = 115{,}9 \text{ mm}$$

Dipakai $P_{10\text{-}110}$

4.5 Perencanaan Kolom

4.5.1 Perhitungan Momen dan Gaya Aksial Rencana

a. Momen untuk portal arah X

Data momen :

$M_{Dy\ atas}$ = 3,40 kNm

$M_{Dy\ bawah}$ = -7,34 kNm

$M_{Ly\ atas}$ = 1,58 kNm

$M_{Ly\ bawah}$ = -2,9 kNm

$M_{Ey\ atas}$ = 127,05 kNm

$M_{Ey\ bawah}$ = -221,17 kNm

Daerah atas

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2 . 3,40 + 1,6 . 1,58 = 6,608 kNm

$1,05\ (M_{Dy} + M_{Ly})$ = 1,05 . (3,40 + (1,58 . 0,6))

Mby = 4,5654 kNm

$1,05\ M_{Ey}$ = 1,05 . 127,05 = 133,4025 kNm

Msy = 133,4025 kNm

Mby + Msy = 4,5654 + 133,4025 = 137,9679 kNm

Tetapi tidak perlu lebih besar dari :

$1,05\ (M_{Dy} + M_{Ly} + 4/k.M_{Ey})$ = 1,05 (3,40 + 1,58 + 4/1 . 127,05)

= 538,839 kNm

Daerah bawah

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2 . -7,34 + 1,6 . -2,9 = -13,448 kNm

$1,05\ (M_{Dy} + M_{Ly})$ = 1,05 . (-7,34 + (-2,9 . 0,6))

Mby = -9,534 kNm

1,05 M_{Ey} = 1,05 . -221,17 = -232,2285 kNm

Msy = -232,2285 kNm

Mby + Msy = (-9,534) + (-232,2285) = -241,7625 kNm

Tetapi tidak perlu lebih besar dari :

1,05 ($M_{Dy} + M_{Ly} + 4/k.M_{Ey}$) = -939,666 kNm

M pakai : atas = 137,9679 kNm

bawah = -214,7625 kNm

b. Momen untuk portal arah Y

Data momen :

$M_{Dx\ atas}$ = 88,04 kNm

$M_{Dx\ bawah}$ = -43,25 kNm

$M_{Lx\ atas}$ = 30,62 kNm

$M_{Lx\ bawah}$ = -14,95 kNm

$M_{Ex\ atas}$ = 61,98 kNm

$M_{Ex\ bawah}$ = -211,48 kNm

Daerah atas

1,2 M_{Dx} + 1,6 M_{Lx} = 1,2 . 88,04 + 1,6 . 30,62 = 154,64 kNm

1,05 ($M_{Dx} + M_{Lx}$) = 1,05 . (88,04 + (30,62 . 0,6))

Mbx = 111,7326 kNm

1,05 M_{Ex} = 1,05 . 61,98 = 65,079 kNm

Msx = 65,079 kNm

Mbx + Msx = 111,7326 + 65,079 = 176,8116 kNm

Tetapi tidak perlu lebih besar dari :

$1{,}05\ (M_{Dx} + M_{Lx} + 4/k.M_{Ex})$ = 366,58 kNm

Daerah bawah

$1{,}2\ M_{Dx} + 1{,}6\ M_{Lx}$ = 1,2 . -43,25 + 1,6 . -14,95 = -75,82 kNm

$1{,}05\ (M_{Dx} + M_{Lx})$ = 1,05 . (-43,25 + (-14,95 . 0,6))

Mbx = -54,831 kNm

$1{,}05\ M_{Ex}$ = 1,05 . -221,48 = -222,054 kNm

Msx = -222,054 kNm

Mbx + Msx = (-54,831) + (-222,054) = -276,885 kNm

Tetapi tidak perlu lebih besar dari :

$1{,}05\ (M_{Dx} + M_{Lx} + 4/k.M_{Ex})$ = -949,326 kNm

M pakai : atas = 176,8116 kNm

bawah = -276,885 kNm

c . Gaya aksial

Data gaya aksial :

$P_{D\ atas}$ = -1189,84 kN

$P_{D\ bawah}$ = -1224,4 kN

$P_{L\ atas}$ = -233,32 kN

$P_{L\ bawah}$ = -233,32 kN

$P_{E\ atas}$ = -127,44 kN

$P_{E\ bawah}$ = -127,44 kN

Daerah atas

$1{,}2\ P_D + 1{,}6\ P_L$ = 1,2 . -1189,84 + 1,6 . -233,32 = -1801,12 kN

$1{,}05\ (P_D + P_L) = 1{,}05\ .\ (-1189{,}84 + (-233{,}32\ .\ 0{,}6))$

$Pb = -1396{,}3236$ kN

$1{,}05\ P_E = 1{,}05\ .\ -127{,}44 = -133{,}812$ kN

$Ps = 133{,}812$ kN

$Pb + Ps = (-1396{,}3236) + (-133{,}812) = -1530{,}1356$ kN

Tetapi tidak perlu lebih besar dari :

$1{,}05\ (P_D + P_L + 4/k.P_E) = -2029{,}671$ kN

Daerah bawah

$1{,}2\ P_D + 1{,}6\ P_L = 1{,}2\ .\ -1224{,}4 + 1{,}6\ .\ -233{,}32 = -1842{,}592$ kN

$1{,}05\ (P_D + P_L) = 1{,}05\ .\ (-1224{,}4 + (-233{,}32\ .\ 0{,}6))$

$Pb = -1432{,}6116$ kN

$1{,}05\ P_E = 1{,}05\ .\ -127{,}44 = -133{,}812$ kN

$Ps = -133{,}6116$ kN

$Pb + Ps = (-1432{,}6116) + (-133{,}812) = -1566{,}4236$ kN

Tetapi tidak perlu lebih besar dari :

$1{,}05\ (P_D + P_L + 4/k.P_E) = -2065{,}854$ kN

P pakai : atas $= -1530{,}1356$ kN

bawah $= -1566{,}4236$ kN

4.5.2 Kriteria Kolom dan Pembesaran Momen

Menghitung kekakuan kolom

a. Arah X

$Ec = Eg = 4700\ .\ \sqrt{f'c} = 4700\ .\ \sqrt{28} = 24870$ MPa

Dicoba dimensi kolom 600 x 600

Ic (Inersia kolom) = 1/12 . 600 . 600^3 = 1,08.10^{10} mm^4

$$\beta_d = \frac{1,2M_D}{1,2M_D + 1,6M_L} = \frac{1,2.3,40}{1,2.3,40 + 1,6.1,58} = 0,617$$

$$EI = \frac{E_c.I_c}{2,5(1+\beta_d)} = \frac{24870.1,08.10^{10}}{2,5(1+0,617)} = 6,644.10^{13}\,Nmm^2$$

Menghitung momen inersia balok di kanan kiri kolom. Dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok di kanan kiri ujung atas kolom yaitu :

$$I_{cr} = \frac{Ig}{2} = \frac{1}{2}\left[\frac{1}{12}.400.600^3\right] = 3,6.10^9\,mm^4$$

2. Momen inersia balok di kanan kiri ujung bawah kolom = 0, karena ujung jepit.

Panjang bersih kolom : Lc_1 = 3,7 m

Lc_2 = 2,9 m

Panjang bersih balok : Lg = 7,4 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{E_c.I_{cr}}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{6,644.10^{13}}{3700}\right) + \left(\frac{6,644.10^{13}}{2900}\right)}{\left(\frac{24870.3,6.10^9}{7400}\right)} = 3,38$$

$$\psi_{bawah} = 0$$

Dari nomogram portal tanpa pengaku, didapat k = 1,4

$$\frac{k.lu}{r} = \frac{1,4.3700}{0,3.600} = 28,78 > 22 \text{ (termasuk kolom panjang)}$$

Beban tekuk Euler yang terjadi adalah :

$$P_c = \frac{\pi^2.EI}{(k.lu)^2} = \frac{\pi^2.6,644.10^{13}}{(1,4.3700)^2} = 24438235,73 \text{ N}$$

menghitung factor pembesaran momen δb_y

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

Cm = 1 (portal tanpa pengaku)

$$\delta b_y = \frac{1}{1-\left(\frac{1566424}{0,65.24438235,73}\right)} = 1,11 > 1$$

$\sum Pc = 24438235,73 + 22788045,83 = 47226281,56$ N

$\sum Pu = 1566424 + 1649508 = 3215932$ N

$$\delta s_y = \frac{Cm}{1-\left(\frac{\sum Pu}{\sum Pc}\right)} \geq 1$$

Cm = 1 (portal tanpa pengaku)

$$\delta b_y = \frac{1}{1-\left(\frac{3215932}{0,65.47226281,56}\right)} = 1,12 > 1$$

Momen akibat pembesaran momen :

Mux bawah $= \delta_{by}$ Mbx + δ_{sy} Msx

$= 1,11 . (-54,831) + 1,12 . (-222,054)$

$= -309,563$ kNm

Mux atas $= \delta_{by}$ Mbx + δ_{sy} Msx

= 1,11 . 111,7326 + 1,12 . 65,079

= 196,912 kNm

b. Arah Y

Ec = Eg = 4700 . $\sqrt{f'c}$ = 4700 . $\sqrt{28}$ = 24870 MPa

Dicoba dimensi kolom 600 x 600

Ic (Inersia kolom) = 1/12 . 600^3 . 600 = $1{,}08.10^{10}$ mm^4

$$\beta_d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.88{,}04}{1{,}2.88{,}04 + 1{,}6.30{,}62} = 0{,}683$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta_d)} = \frac{24870.1{,}08.10^{10}}{2{,}5(1+0{,}683)} = 6{,}384.10^{13}\ Nmm^2$$

Menghitung momen inersia balok di kanan kiri kolom. Dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok di kanan kiri ujung atas kolom yaitu :

$$I_{cr} = \frac{Ig}{2} = \frac{1}{2}\left[\frac{1}{12}.400.600^3\right] = 3{,}6.10^9\ mm^4$$

2. Momen inersia balok di kanan kiri ujung bawah kolom = 0, karena ujung jepit.

Panjang bersih kolom : Lc_1 = 3,7 m

Lc_2 = 2,9 m

Panjang bersih balok : Lg_1 = 5,4 m

Lg_2 = 5,4 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{E_c.I_{cr}}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{6{,}384.10^{13}}{3700}\right)+\left(\frac{6{,}384.10^{13}}{2900}\right)}{\left(\frac{24870.3{,}6.10^{9}}{5400}\right)+\left(\frac{24870.3{,}6.10^{9}}{5400}\right)} = 1{,}184$$

$$\psi_{bawah} = 0$$

Dari nomogram portal tanpa pengaku, didapat k = 1,19

$$\frac{k.lu}{r} = \frac{1{,}19.3700}{0{,}3.600} = 24{,}46 > 22 \text{ (termasuk kolom panjang)}$$

Beban tekuk Euler yang terjadi adalah :

$$P_c = \frac{\pi^2.EI}{(k.lu)^2} = \frac{\pi^2.6{,}384.10^{13}}{(1{,}19.3700)^2} = 32500889{,}93 \text{ N}$$

menghitung factor pembesaran momen δb_y

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

Cm = 1 (portal tanpa pengaku)

$$\delta b_y = \frac{1}{1-\left(\frac{1566424}{0{,}65.32500889{,}93}\right)} = 1{,}08 > 1$$

$\sum$Pc = 32500889,93 + 29320418,48 + 38922196,12 + 38824106,36 +

33436861,48 + 35672460,04 + 29619203,74

= 238296136,1 N

$\sum Pu$ = 1566424 + 1072436 + 1320967 + 1373429 + 1656833 + 1340991 + 620964

= 8952044 N

$$\delta s_y = \frac{Cm}{1-\left(\frac{\sum Pu}{\sum Pc}\right)} \geq 1$$

Cm = 1 (portal tanpa pengaku)

$$\delta b_y = \frac{1}{1-\left(\frac{8952044}{0{,}65.238296136{,}1}\right)} = 1{,}06 > 1$$

Momen akibat pembesaran momen :

Mux bawah = δ_{by} Mby + δ_{sy} Msy

= 1,08 . (-9,534) + 1,06 . (-232,2285)

= -256,459 kNm

Mux atas = δ_{by} Mby + δ_{sy} Msy

= 1,08 . 4,565 + 1,06 . 133,4025

= 146,337 kNm

4.5.3 Analisis Gaya Aksial dan Momen akibat Kekuatan Balok

h = 3,5 m

hn = 2,9 m

Rv = 1 (jumlah lantai diatasnya ; n = 3)

ωd = 1 (untuk lantai dasar)

k = 1

a. Perhitungan arah X

$Mkap_{(kiri)} = 0$

$Mkap_{(kanan)} = 1,25 \cdot Mnak = 1,25 \cdot 377,589 = 471,986$ kNm

menghitung gaya aksial rencana :

$$Nu,k_x = 0,7.Rv.\frac{Mkap_{kiri} + Mkap_{kanan}}{l} + 1,05.Ng$$

$$= 0,7.1.\frac{471,986}{8} + 1,05.(1224,4 + 233,32)$$

$$= 1571,905 \text{ kN}$$

tidak perlu melebihi :

$$Nu,k_x = 1,05\ (N_D + N_L + 4.N_E)$$

$$= 1,05\ (1224,4 + 233,32 + 127,44)$$

$$= 1664,418 \text{ kN}$$

dipakai $Nu,k_x = 1571,905$ kN

menghitung α :

$M_{E,K\ atas} = 127,05$ kNm

$M_{E,K\ bawah} = -221,17$ kNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{127,05}{127,05 + 221,17} = 0,365$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = -$$

menghitung momen rancang kolom :

$$Mu,k_x atas = \frac{hn}{h}.\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l''_{ki}}Mkap,ki + \frac{l_{kka}}{l''_{ka}}Mkap,ka\right)$$

$$= \frac{2,9}{3,5}.1.0,365.0,7.\left(\frac{8}{7,4}.471,986\right)$$

$$= 108,021 \text{ kNm}$$

$$\text{Mu,k}_x\text{bawah} = \frac{hm}{h}.\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l''_{ki}}Mkap,ki + \frac{l_{kka}}{l''_{ka}}Mkap,ka\right)$$

$$= \frac{2,9}{3,5}.1.0,7.\left(\frac{8}{7,4}.471,986\right)$$

$$= 295,848 \text{ kNm}$$

tidak perlu melebihi :

$$\text{Mu,k}_x = 1,05\ (\text{M}_{Dy} + \text{M}_{Ly} + \frac{4}{k}.\text{M}_{Ey})$$

$$= 1,05\ (7,34 + 2,9 + \frac{4}{1}.221,17)$$

$$= 939,666 \text{ kNm}$$

Mu,k_x pakai = 295,848 kNm

b. Perhitungan arah Y

$\text{Mkap}_{(\text{kiri})}$ = 1,25 . Mnak = 1,25 . 225,99 = 282,4875 kNm

$\text{Mkap}_{(\text{kanan})}$ = 1,25 . Mnak = 1,25 . 225,99 = 282,4875 kNm

menghitung gaya aksial rencana :

$$\text{Nu,k}_y = 0,7.Rv.\frac{Mkap_{kiri} + Mkap_{kanan}}{l} + 1,05.Ng$$

$$= 0,7.1.\frac{282,4875}{6} + 1,05.(1224,4 + 233,32)$$

$$= 1563,56 \text{ kN}$$

tidak perlu melebihi :

$Nu,k_y = 1,05\ (N_D + N_L + 4.N_E)$

$= 1,05\ (1224,4 + 233,32 + 127,44)$

$= 1664,418$ kN

dipakai $Nu,k_x = 1563,56$ kN

menghitung α :

$M_{E,K\ atas} = 127,05$ kNm

$M_{E,K\ bawah} = -221,17$ kNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{127,05}{127,05 + 221,17} = 0,365$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = -$$

menghitung momen rancang kolom :

$$Mu,k_y atas = \frac{hn}{h}.\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l''_{ki}}Mkap,ki + \frac{l_{kka}}{l''_{ka}}Mkap,ka\right)$$

$$= \frac{2,9}{3,5}.1.0,365.0,7.\left(\frac{6}{5,4}.282,4875 + \frac{6}{5,4}.282,4874\right)$$

$= 132,895$ kNm

$$Mu,k_y bawah = \frac{hn}{h}.\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l''_{ki}}Mkap,ki + \frac{l_{kka}}{l''_{ka}}Mkap,ka\right)$$

$$= \frac{2,9}{3,5}.1.0,7.\left(\frac{6}{5,4}.282,4875 + \frac{6}{5,4}.282,4875\right)$$

$= 364,095$ kNm

tidak perlu melebihi :

$$Mu,k_x = 1,05 \left(M_{Dx} + M_{Lx} + \frac{4}{k} . M_{Ex} \right)$$

$$= 1,05 \left(88,04 + 30,62 + \frac{4}{1} . 211,48 \right)$$

$$= 1012,809 \text{ kNm}$$

Mu,k_x pakai = 364,095 kNm

4.5.4 Perencanaan Penulangan Kolom

Berdasarkan analisa struktur diperoleh :

Mux = 276,885 kNm

Muy – 214,7625 kNm

Pu = 1566,4236 kN

Berdasarkan kekuatan balok diperoleh :

Mux – 295,848 kNm

Muy = 364,095 kNm

Nux = 1571,905 kN

Nuy = 1563,56 kN

Dalam perencanaan kolom digunakan momen dan gaya aksial yang terbesar.

Digunakan Moy untuk perencanaan.

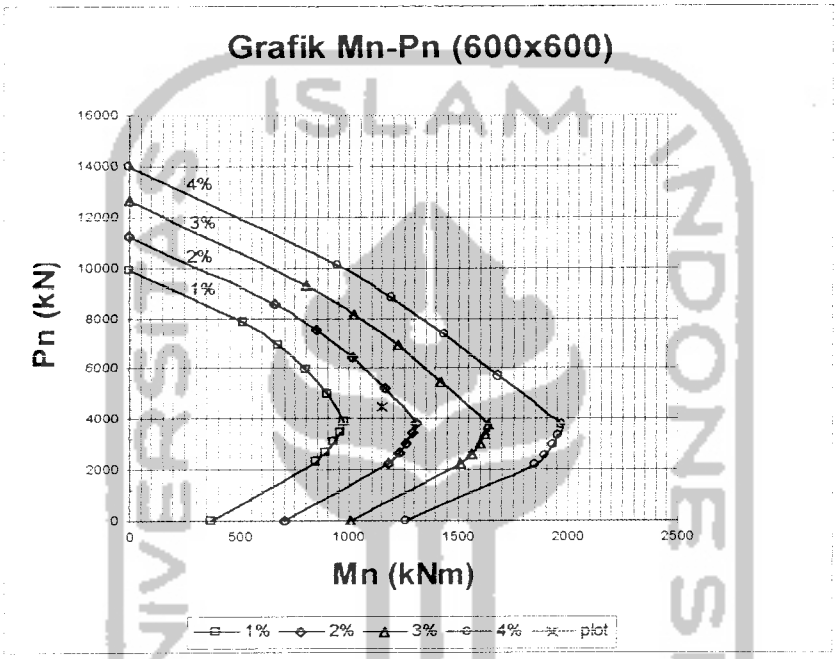
$$\text{Moy perlu} = Mny + Mnx. \left(\frac{b}{h} \right) \left(\frac{1-\beta}{\beta} \right)$$

$$= 560,1461 + 455,1507. \left(\frac{0,6}{0,6} \right) \left(\frac{1-0,65}{0,65} \right)$$

= 805,227 kNm

$$Pn = \frac{1571,905}{0,65} = 2418,44 \text{ kN}$$

$$e = \frac{Moy}{Pn} = \frac{805,227}{2418,44} = 0,333m$$



Gambar 4.26 Diagram Pn-Mn kolom

Dari grafik Pn vs Mn dengan nilai Pn = 2418,44 kN dan Mn = 805,227 kNm, didapat Ast = 1,0 %. Ag

Ast = 0,01. 600. 600 = 3600 mm^2

As = As' = 0,5. Ast = 1800 mm^2

dipakai tulangan D22 dengan As_{1tul} = 379,94 mm^2

jumlah tulangan (n) = $\frac{1800}{379,94} = 4,74$

Dipakai 5D22

As_{pakai} = 5. 379,94 = 1899,7 mm^2

Cek eksentrisitas balance (e_b)

$$c_b = \frac{600}{600 + fy}.d = \frac{600}{600 + 400}.539 = 323{,}4mm$$

$a_b = \beta_1 . c_b = 0{,}85 . 323{,}4 = 274{,}89$ mm

$$f's = 600. \frac{(c_b - d')}{c_b} = 600.\frac{(323{,}4 - 61)}{323{,}4} = 486{,}827MPa > fy = 400 \text{ MPa}$$

Dengan demikian digunakan f's = fy = 400 MPa

$Pn_b = 0{,}85 . f'c . b . a_b + As' . (f's - 0{,}85 . f'c) - As . fy$

$= 0{,}85 . 28 . 600 . 274{,}89 + 1899{,}7 . (400 - 0{,}85 . 28) - 1899{,}7 . 400$

$= 3880{,}216$ kN

$$Mnb = 0{,}85.f'c.b.a_b.(\frac{h}{2} - \frac{a_b}{2}) + A's.f's.\left(\frac{h}{2} - d'\right) + As.fy.\left(d - \frac{h}{2}\right)$$

$$= 0{,}85.28.600.274{,}89.\left(\frac{600}{2} - \frac{274{,}89}{2}\right) + 1899{,}7.400.\left(\frac{600}{2} - 61\right)$$

$$+ 1899{,}7.400.\left(539 - \frac{600}{2}\right)$$

$= 1001{,}321$ kNm

$$eb = \frac{Mn_b}{Pn_b} = \frac{1001{,}321}{3880{,}216} = 0{,}258$$

karena $e_b < e$, maka termasuk patah tarik

$$Pn = 0{,}85.f'c.b.d.\left[\left(\frac{h - 2.e}{2.d}\right) + \sqrt{\left(\left(\frac{h - 2.e}{2.d}\right)^2 + 2.\rho\left(\frac{fy}{0{,}85.f'c}\right)\left(1 - \frac{d'}{d}\right)\right)}\right]$$

$=0{,}85.28.600.539.$

$$\left[\left(\frac{600-2.333}{2.539}\right)+\sqrt{\left(\left(\frac{600-2.333}{2.539}\right)^2+2.0{,}00587.\left(\frac{400}{0{,}85.28}\right)\left(1-\frac{61}{539}\right)\right)}\right]$$

= 2782,767 kN > 2418,44 kN

Moyn = Pn. e

= 2782,767. 0,333

= 926,661 kNm > Moyperlu = 805,227 kNm

Penampang diasumsikan bujur sangkar sehingga :

Moyn = Moxn = 922,661 kNm

$$\frac{Mny}{Moyn}=\frac{560{,}1461}{926{,}661}=0{,}604$$

Dari diagram factor kontur β untuk kolom segiempat yang mengalami lentur biaksial, dicoba β = 0,65 dengan $\frac{Mny}{Moyn}=0{,}604$ diperoleh :

$$\frac{Mnx}{Moxn}=0{,}73$$

Mnx = 0,73. 926,661 = 673,632 kNm > 455,1507 kNm

4.5.5 Perencanaan Penulangan Geser

$$V_{u,k}=\frac{Mu,k_{atas}+Mu,k_{bawah}}{h'_k}$$

$$=\frac{132{,}895+364{,}095}{2{,}9}$$

= 171,376 kN

tidak perlu melebihi :

$$V_{u,k}=1{,}05\,(V_{D,k}+V_{L,k}+\frac{4}{K}\,V_{E,k}$$

= 1,05 (32,82 + 11,39 + 4 . 68,37)

= 333,5745 kN

$V_{U,k}$ terpakai = 171,376 kN

4.5.5.1 Daerah sendi plastis

d = 0,539 m

Vc = 0 (pada daerah sendi plastis, Vc dianggap 0)

$V_{U,k}$ terhitung = $\frac{h'_k - d}{h'_k} V_{U,k}terpakai$

$= \frac{2,9 - 0,539}{2,9}.171,376$

= 139,524 kN

$Vs = \frac{V_{U,k}terhitung}{\phi}$

$= \frac{139,524}{0,6}$

= 232,54 kN

Dipakai sengkang 2P10 dengan Av = 157,08 mm^2

Jarak (s) $< \frac{Av.fy.d}{Vs} = \frac{157,08.240.539}{232,54.10^3} = 87,38mm$

$< \frac{d}{4}$ = 134,75 mm

maka dipakai P10 – 80 mm

4.5.5.2 Daerah luar sendi plastis

$V_{U,k}$ terhitung = 139,524 kN

$$\frac{V_{U,k}terhitung}{0,6} = \frac{139,524}{0,6} = 232,54kN$$

Nu,k = 1571,986 kN

$$Vc = \left(1 + \frac{Nu,k}{14.Ag}\right)\frac{1}{6}.\sqrt{f'c}.b.d$$

$$= \left(1 + \frac{1571986}{14.600.600}\right)\frac{1}{6}.\sqrt{28}.600.539$$

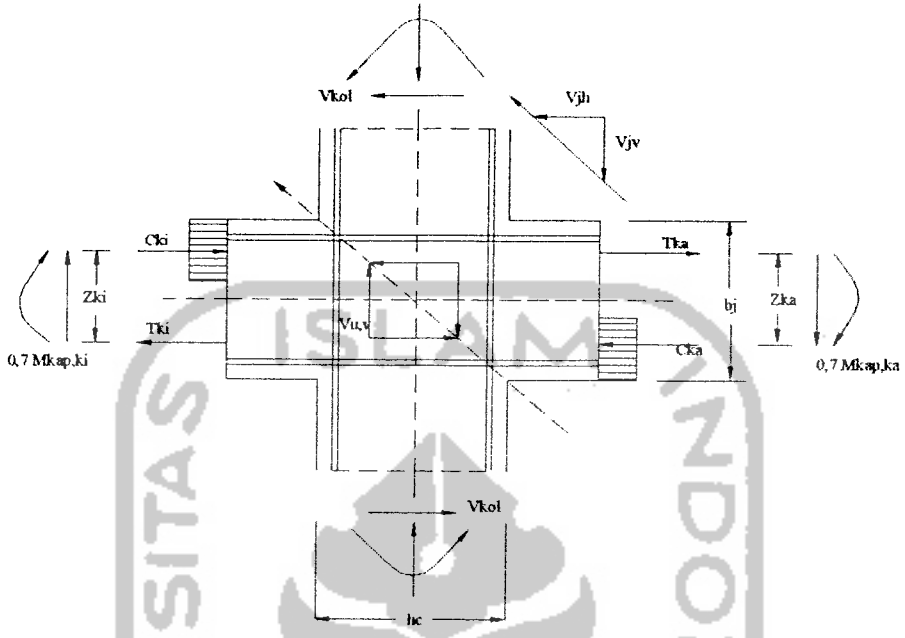
$$= 374,17 \text{ kN} > \frac{V_{U,k}terhitung}{0,6} = 232,54kN$$

karena $Vc > \frac{Vu,kterhitung}{0,6}$, maka digunakan tulangan geser minimum

dengan jarak (s) $< \frac{d}{2} = \frac{539}{2} = 269,5$ mm

Dipakai P10-200

4.5.6 Pertemuan Balok Kolom



Gambar 4.27 Joint Balok Kolom Dalam

a. Perhitungan gaya-gaya dalam

1. Arah X

$b_j = bc = 600$ mm

$= bb + 0{,}5.hc = 400 + 0{,}5{,}600 = 700$ mm

b_j pakai = 600 mm

$h_c = 600$ mm

$$V_{kol,x} = \frac{0{,}7.\phi_0.\left(\sum \frac{l_x}{l_{nx}} M_{nak,bx} + 0{,}3.\sum \frac{l_y}{l_{ny}} M_{nak,by}\right)}{1/2.(h_a + h_b)}$$

$$V_{kol,x} = \frac{0{,}7.1{,}25.\left[\left(0 + \frac{8}{7{,}4}.377{,}589\right) + 0{,}3\left(\frac{6}{5{,}4}.225{,}99 + \frac{6}{5{,}4}.225{,}99\right)\right]}{0{,}5.(3{,}5 + 4)}$$

$V_{kol,x}$ = 130,4016 kN

$z_{ki,x}$ = 0

$z_{ka,x}$ = 0,9.d = 0,9. 539 = 485 mm = 0,485 m

$C_{ki,x} = T_{ki,x} = 0$

$C_{ka,x} = T_{ka,x} = 0,7.\emptyset_0.(M_{nak,bx\text{-}ka})/z_{ka,x}$

$$= \frac{0,7.1,25.377,589}{0,485}$$

= 681,217 kN

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x}$

= 0 + 681,217 – 130,4016

= 550,8154 kN

Kontrol tegangan geser horizontal :

$$v_{jh,x} = \frac{V_{jh,x}}{b_j.h_c} \le 1,5\sqrt{f'c}$$

$$= \frac{550,8154}{0,6.0,6}$$

= 1530,043 kN/m^2 = 1,53 N/mm^2 < 1,5.$\sqrt{28}$ =7,9 N/mm^2 .....(OK)

$$V_{ch,x} = 2/3.\sqrt{\left\{\left(\frac{Nu,k}{Ag}\right) - 0,1.f'c\right\}}.b_j h_c$$

$$= 2/3.\sqrt{\left\{\left(\frac{1571,905.10^3}{600.600}\right) - 0,1.28\right\}}.600.600$$

= 300,374 kN

$V_{sh,x} = V_{jh,x} - V_{ch,x}$

= 550,815 – 300,374 = 250,441 kN

2). Arah Y

$b_j = b_c = 600$ mm

$= bb + 0{,}5.h_c = 400 + 0{,}5.\ 600 = 700$ mm

b_j pakai = 600 mm

$h_c = 600$ mm

$$V_{kol,y} = \frac{0{,}7.\phi_0.\left(0{,}3.\sum\frac{l_x}{l_{nx}}.M_{nak,bx} + \sum\frac{l_y}{l_{ny}}.M_{nak,by}\right)}{1/2.(h_a + h_b)}$$

$$V_{kol,y} = \frac{0{,}7.1{,}25.\left[0{,}3.\left(0 + \frac{8}{7{,}4}.377{,}589\right) + \left(\frac{6}{5{,}4}.225{,}99 + \frac{6}{5{,}4}.225{,}99\right)\right]}{0{,}5.(3{,}5 + 4)}$$

$V_{kol,x} = 145{,}754$ kN

$z_{ki,x} = 0{,}9.d = 0{,}9.539 = 485$ mm $= 0{,}485$ m

$z_{ka,x} = 0{,}9.d = 0{,}9.\ 539 = 485$ mm $= 0{,}485$ m

$$C_{ki,x} = T_{ki,x} = \frac{0{,}7.\phi_0.(M_{nak,by-ki})}{z_{ki,y}}$$

$$= \frac{0{,}7.1{,}25.225{,}99}{0{,}485} = 407{,}714 \text{ kN}$$

$$C_{ka,x} = T_{ka,x} = \frac{0{,}7.\phi_0.(M_{nak,by-ka})}{z_{ka,y}}$$

$$= \frac{0{,}7.1{,}25.225{,}99}{0{,}485}$$

$= 407{,}714$ kN

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x}$

$= 407{,}714 + 407{,}714 - 145{,}745 = 669{,}674$ kN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j . h_c} \leq 1{,}5\sqrt{f'c}$$

$$= \frac{669{,}674}{0{,}6.0{,}6}$$

$= 1860{,}205$ kN/m^2 = 1,53 N/mm^2 < 1,5.$\sqrt{28}$ =7,9 N/mm^2……(OK)

$$V_{ch,y} = 2/3.\sqrt{\left\{\left(\frac{Nu,k}{Ag}\right) - 0{,}1.f'c\right\}}.b_j.h_c$$

$$= 2/3.\sqrt{\left\{\left(\frac{1563{,}56.10^3}{600.600}\right) - 0{,}1.28\right\}}.600.600$$

$= 298{,}143$ kN

$V_{sh,y} = V_{jh,y} - V_{ch,y}$

$= 669{,}674 - 298{,}143 = 371{,}531$ kN

b. Penulangan Geser Horisontal

$V_{sh,mak} = V_{sh,y} = 371{,}531$ kN

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{371{,}531}{400} = 928{,}8275 mm^2$$

Digunakan sengkang P10 dengan Av = 157 mm^2

$$\text{Jumlah lapis sengkang} = \frac{928{,}8275}{157} = 5{,}916 \text{ lapis}$$

digunakan sengkang 8P10

c. Penulangan geser vertical

$$V_{cv} = \frac{As_c'}{As_c} . V_{jh,mak} . \left(0{,}6 + \frac{Nu,k}{Ag.f'c}\right)$$

$$= 1.\ 669{,}674.10^3.\left(0{,}6 + \frac{1563{,}56.10^3}{600.600.28}\right)$$

$$= 505681\ \text{N} = 505{,}681\ \text{kN}$$

$$V_{jv} = \frac{bj}{h_c} V_{jh,mak}$$

$$= \frac{0{,}6}{0{,}6}.669{,}674 = 669{,}674\ \text{kN}$$

$$V_{sv} = V_{jv} - V_{cv} = 669{,}674 - 505{,}681 = 163{,}993\ \text{kN}$$

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{163{,}674}{400} = 409{,}9825\ \text{mm}^2$$

Digunakan sengkang P10 dengan Av = 157 mm^2

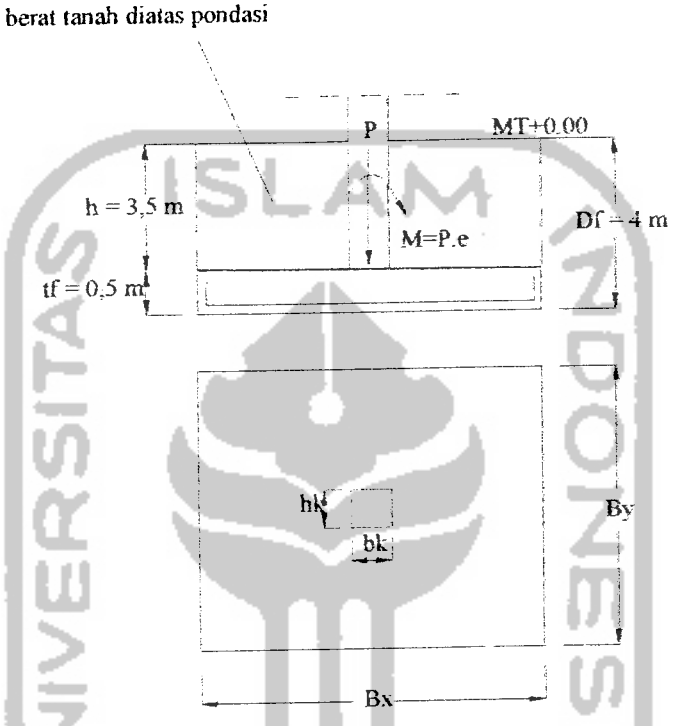
Jumlah lapis sengkang = $\frac{409{,}9825}{157} = 2{,}6$ lapis

Digunakan sengkang 3P10

4.6 Perencanaan Pondasi

4.6.1 Perencanaan Pondasi Telapak Setempat (PS1)

A. Perencanaan Dimensi Pondasi



Gambar 4.28 Pondasi telapak setempat

σ tanah = 400 kN/m^2

F'c = 28 Mpa

fy = 400 Mpa

P = 1842,592 kN

Mx tetap = 75,82 kNm

My tetap = 13,45 kNm

Mx sementara = 276,885 kNm

My sementara = 241,76 kNm

γbtanah = 1,744 kN/m^3

γbeton = 24 kN/m^3

Asumsi tebal pelat (tf) = 500 mm

Ukuran kolom :

hk = 600 mm

bk = 600 mm

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

$= 400 - (0,5.24) - (3,5.1,744)$

$= 381,896\ kN/m^2$

1. Tinjauan Terhadap Beban Tetap

Digunakan pondasi penampang bujur sangkar, dicoba dengan nilai Bx=By= 2,4 m

Luas penampang pelat pondasi :

$A = Bx \,.\, By = 2,4 \times 2,4 = 5,76\ m^2$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$$\sigma_{maks} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By} \pm \frac{6.Mx}{By^2.Bx} \qquad \textbf{(3.7.7)}$$

$$\sigma_{maks} = \frac{1842,592}{5,76} + \frac{6.13,45}{2,4^2.2,4} + \frac{6.75,82}{2,4^2.2,4}$$

$= 358,640\ kN/m^2 < \sigma_{nettotanah} = 381,896\ kN/m^2$ **.....Ok !**

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{My}{P} = \frac{241,76}{1842,592} = 0,131\ m$$

$$ey = \frac{Mx}{P} = \frac{276,886}{1842,592} = 0,15\ m$$

$B/6 = 2,4/6 = 0,4 >$ ex dan ey (beban eksentrisitas di dalam teras), maka :

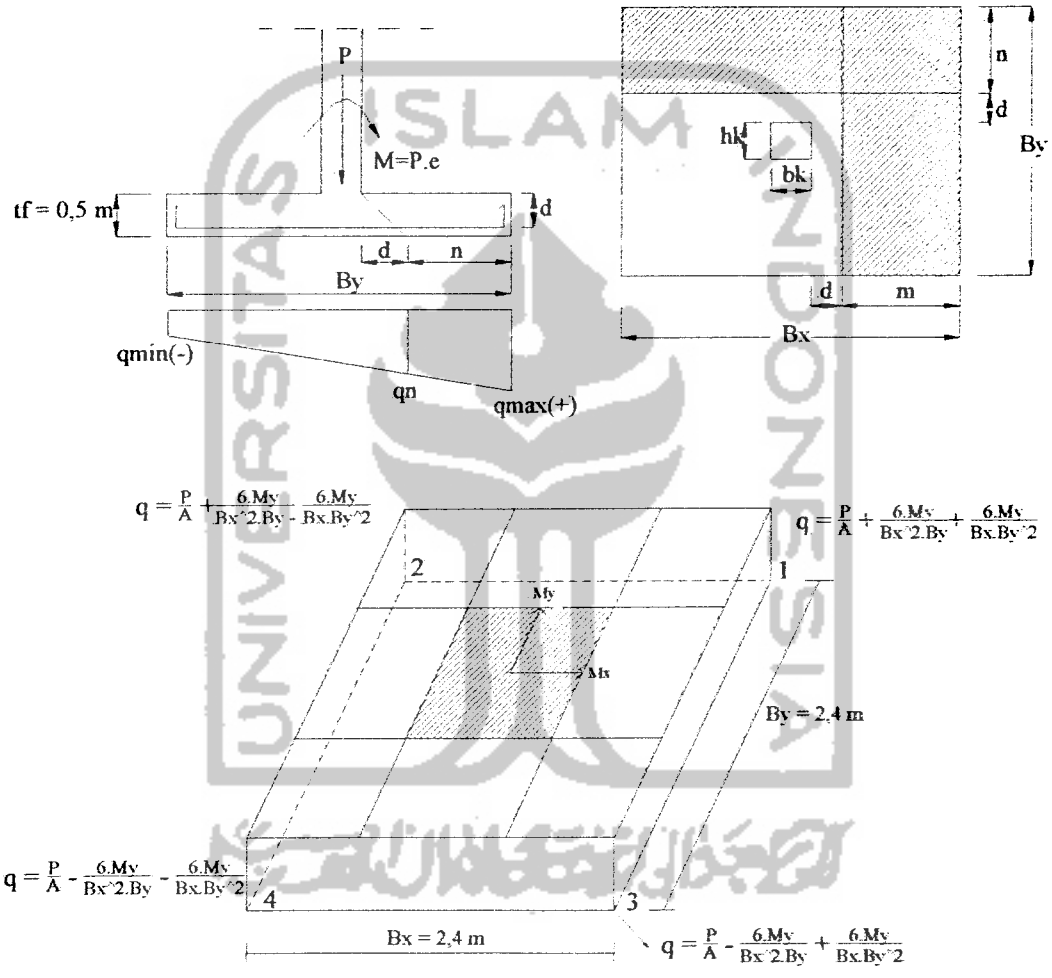
Kontrol tegangan yang terjadi :

$$\sigma_{maks} = \frac{P}{A}\left(1 + \frac{6.ex}{Bx} + \frac{6.ey}{By}\right)$$

$$= \frac{1842,592}{5,76}\left(1+\frac{6.0,131}{2,4}+\frac{6.0,15}{2,4}\right)$$

$= 545 \text{ kN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 381,896 = 572,844 \text{ kN/m}^2$ **.....Ok !**

B. Perencananaan Geser Satu Arah



Gambar 4.29 Pondasi dengan geser satu arah

Arah x

P = 1842,592 kN

My = 241,76 kNm

Mx = 276,885 kNm

• Tegangan kontak yang terjadi :

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

$$qu_1 = \frac{1842,592}{5,76} + \frac{6.241,76}{2,4.2,4^2} + \frac{6.276,885}{2,4^2.2,4}$$

$$= 545 \text{ kN/m}^2$$

$$qu_2 = \frac{1842,592}{5,76} + \frac{6.241,76}{2,4.2,4^2} - \frac{6.276,885}{2,4^2.2,4}$$

$$= 304,65 \text{ kN/m}^2$$

$$qu_3 = \frac{1842,592}{5,76} - \frac{6.241,76}{2,4.2,4^2} + \frac{6.276,885}{2,4^2.2,4}$$

$$= 335,14 \text{ kN/m}^2$$

$$qu_4 = \frac{1842,592}{5,76} - \frac{6.241,76}{2,4.2,4^2} - \frac{6.276,885}{2,4^2.2,4}$$

$$= 94,788 \text{ kN/m}^2$$

$$qu_a = \frac{(545 - 304,65).(2,4 - 0,481)}{2,4} + 304,65$$

$$= 496,83 \text{ kN/m}^2$$

$$qu_b = \frac{(335,14 - 94,788).(2,4 - 0,481)}{2,4} + 94,788$$

$$= 286,97 \text{ kN/m}^2$$

$$qu_c = \frac{545 + 496,83}{2} = 520,915 \text{ kN/m}^2$$

$$qu_d = \frac{335,14 + 286,97}{2} = 311,055 \text{ kN/m}^2$$

$$qu_{pakai} = \frac{520{,}915 + 311{,}055}{2} = 415{,}985 \text{ kN/m}^2$$

Jarak pusat tulangan tarik ke serat tekan beton :

$d = tf - Pb - \frac{1}{2}.\varnothing_{tul.\ pokok} = 500 - 70 - \frac{1}{2}.\ 22 = 419 \text{ mm} = 0{,}419 \text{ m}$ **(3.7.12)**

$$m = \frac{Bx - bk - 2.d}{2} = \frac{2{,}4 - 0{,}6 - 2.0{,}419}{2} = 0{,}481 \text{ m} \qquad \textbf{(3.7.14)}$$

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

$Vu = qu_{pakai} \ .\ m\ .\ Bx = 415{,}985\ .\ 0{,}481.\ 2{,}4 = 480{,}213 \text{ kN}$ **(3.7.13)**

$$Vu/\phi = 480{,}213/0{,}6 = 800{,}4 \text{ kN}$$

- Kekuatan beton menahan geser:

$Vc = 1/6.\ \sqrt{f'c}\ .\ Bx.\ d = 1/6.\ \sqrt{28}\ \ 2{,}4.\ 0{,}419\ .\ 10^3 = 886{,}856 \text{ kN}$ **(3.7.17)**

- Kontrol gaya geser :

$Vc = 886{,}856 \text{ kN} \geq Vu/\phi = 800{,}4 \text{ kN}$ **.....Ok !**

Arah y

$$qu_a = \frac{(545 - 335{,}14).(2{,}4 - 0{,}481)}{2{,}4} + 335{,}14$$

$$= 502{,}94 \text{ kN/m}^2$$

$$qu_b = \frac{(304{,}65 - 94{,}788).(2{,}4 - 0{,}481)}{2{,}4} + 94{,}788$$

$$= 262{,}59 \text{ kN/m}^2$$

$$qu_c = \frac{545 + 502{,}94}{2} = 523{,}97 \text{ kN/m}^2$$

$$qu_d = \frac{304{,}65 + 262{,}59}{2} = 283{,}62 \text{ kN/m}^2$$

$$qu_{pakai} = \frac{523{,}97 + 283{,}62}{2} = 403{,}795 \text{ kN/m}^2$$

Jarak pusat tulangan tarik ke serat tekan beton :

$$d = tf - Pb - \tfrac{1}{2}.\varnothing_{tul.\ pokok} = 500 - 70 - \tfrac{1}{2}.\ 22 = 419 \text{ mm} = 0{,}419 \text{ m} \quad \textbf{(3.7.12)}$$

$$n = \frac{Bx - bk - 2.d}{2} = \frac{2{,}4 - 0{,}6 - 2.0{,}419}{2} = 0{,}481 \text{ m} \quad \textbf{(3.7.16)}$$

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

$$Vu = qu_{pakai} \cdot n \cdot By = 403{,}795 \cdot 0{,}481 \cdot 2{,}4 = 466{,}141 \text{ kN} \quad \textbf{(3.7.15)}$$

$$Vu/\phi = 466{,}141/0{,}6 = 776{,}9 \text{ kN}$$

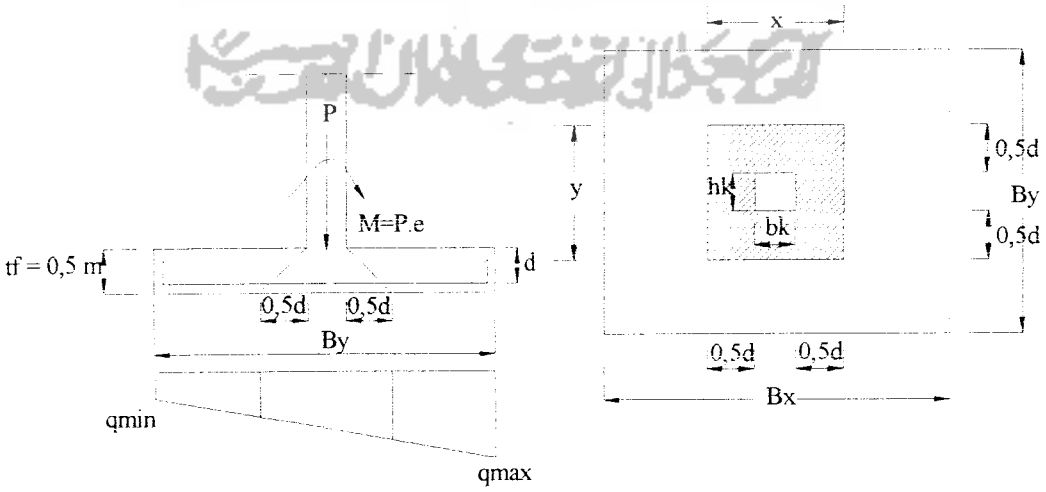
- Kekuatan beton menahan geser:

$$Vc = 1/6.\ \sqrt{f'c}\ .\ Bx.\ d = 1/6.\ \sqrt{28}\ \ 2{,}4.\ 0{,}419\ .\ 10^3 = 886{,}856 \text{ kN} \quad \textbf{(3.7.18)}$$

- Kontrol gaya geser :

$$Vc = 886{,}856 \text{ kN} \geq Vu/\phi = 776{,}9 \text{ kN} \textbf{ .....Ok !}$$

C. Perencananaan Geser Dua Arah



Gambar 4.30 Pondasi dengan geser dua arah

$x = bk + d$ **(3.7.20)**

$= 600 + 419$

$= 1019 \text{ mm} = 1{,}019 \text{ m}$

$y = hk + d$ **(3.7.21)**

$= 600 + 419$

$= 1019 \text{ mm} = 1{,}019 \text{ m}$

- Tegangan kontak yang terjadi :

$$qu_{pakai} = \frac{qu_1 + qu_2 + qu_3 + qu_4}{4}$$

$$= \frac{545 + 304{,}65 + 335{,}14 + 94{,}788}{4}$$

$$= 319{,}8945 \text{ kN/m}^2$$

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

$Vu = qu_{pakai} \cdot ((Bx.By) - (x.y))$ **(3.7.19)**

$= 319{,}8945 . ((2{,}4 . 2{,}4) - (1{,}019 . 1{,}019)) = 1510{,}426 \text{ kN}$

$$Vu/\phi = 1510{,}426/0{,}6 = 2517{,}377 \text{ kN}$$

- Kekuatan beton menahan geser :

$$\beta c = \frac{sisipanjang}{sisipendek} = \frac{y}{x} = \frac{1{,}019}{1{,}019} = 1$$

$bo = 2 . (x + y) = 2 . (1019 + 1019) = 4076 \text{ mm}$ **(3.7.24)**

$Vc_1 = (1 + 2/\beta c) . (2 . \sqrt{f'c}) . bo . d$ **(3.7.23)**

$= (1 + 2/1) . (2 . \sqrt{28}) . 4076 . 419 . 10^{-3} = 54222 \text{ kN}$

$Vc_2 = 4 . \sqrt{f'c} . bo . d$ **(3.7.22)**

$= 4.\ \sqrt{28}\ .\ 4076.\ 419.\ 10^{-3} = 36148\ kN$

$Vc = 36148\ kN \geq {}^{Vu}/_{\phi} = 2517{,}377\ kN$**.....Ok !**

D. Kuat Tumpuan Pondasi

• Kuat tumpuan Pondasi :

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

Luas penampang kolom (A_1) = bk . hk = 0,60. 0,60 = 0,36 m^2

Luas pelat pondasi (A_2) = Bx . By = 2,4 . 2,4 = 5,76 m^2

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{5{,}76}{0{,}36}} = 4 > 2$ (jika lebih besar dari 2, dipakai nilai 2)

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1.2)$

$= 0{,}7.\ (0{,}85\ .\ 28\ .\ 0{,}36\ .\ 2).10^3 = 11995\ kN$

• Kuat tumpuan kolom :

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1)$

$= 0{,}7.\ (0{,}85.\ 28.\ 0{,}36).10^3 = 5997\ kN$

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 11995\ kN > \phi.Pn_{kolom} = 5997\ kN$ **.....Ok !**

E. Perencanaan Tulangan Lentur Pondasi

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

$$L = \frac{Bx - bk}{2} = \frac{2{,}4 - 0{,}60}{2} = 0{,}9\ m$$

$qu = 545\ kN/m^2$

Mu = 0,5. qu. L^2 = 0,5 . 545 . $0,9^2$ = 220,571 kNm **(3.7.26)**

$$Mu/\phi = \frac{220,571}{0,8} = 275,714 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, maka : $A_{1\varnothing}$ = 380,133 mm^2
- Tebal pelat pondasi : tf = 500 mm, selimut beton (Pb) = 70 mm

d = tf – Pb – 0,5. $\varnothing_{tul.\ pokok}$ = 500 – 70 – 0,5. 22 = 419 mm

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,807 \quad \textbf{(3.7.29)}$$

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

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{275,714.10^6}{1000.419^2} = 1,57 \text{ MPa} \quad \textbf{(3.7.30)}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035 \quad \textbf{(3.7.32)}$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.28.0,85}{400}.\left(\frac{600}{600+400}\right) = 0,0303$$

ρ_{mak} = 0,75. ρb = 0,75. 0,0303 = 0,0227

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

$$= \frac{1}{16,807}.\left(1-\sqrt{1-\frac{2.16,807.1,57}{400}}\right) = 0,00406 \quad < \rho_{mak} = 0,0227$$

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

$\rho_{perlu} = \rho_{ada}$ = 0,00406

$AS_{perlu} = \rho_{perlu}$. b . d = 0,00406 . 1000 . 419 = 1701,14 mm^2 **(3.7.33)**

$0{,}002.b.h = 0{,}002.1000.500 = 1000 mm^2 < As_{perlu}$, maka, $As_{perlu} = 1701{,}14\ mm^2$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{380{,}133.1000}{1701{,}14} = 223{,}458\ mm \qquad \textbf{(3.7.35)}$$

$$s \le 2.h = 2.500 = 1000\ mm$$

$$s \le 250\ mm$$

→ Dipakai Tulangan Pokok : $D_{22} - 220$ mm

$$As_{ada} = \frac{A_{10}.1000}{s} = \frac{380{,}133.1000}{220} = 1727{,}877\ mm^2 \qquad \textbf{(3.7.36)}$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1727{,}877.400}{0{,}85.28.1000} = 29{,}04\ mm \qquad \textbf{(3.7.37)}$$

$$Mn = As_{ada}.fy.\left(d - \tfrac{a}{2}\right) \qquad \textbf{(3.7.38)}$$

$$= 1727{,}877 . 400\ (419 - \tfrac{29{,}04}{2})$$

$$= 279{,}557\ kNm \ge \tfrac{Mu}{\phi} = 275{,}714\ kNm \text{ .....Ok !}$$

Perencanaan Tulangan Bagi Pondasi

$As_{bagi} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 500 = 1000\ mm^2$

• Digunakan tulangan bagi ∅12 mm, maka: $A_{1\varnothing} = 113{,}097\ mm^2$

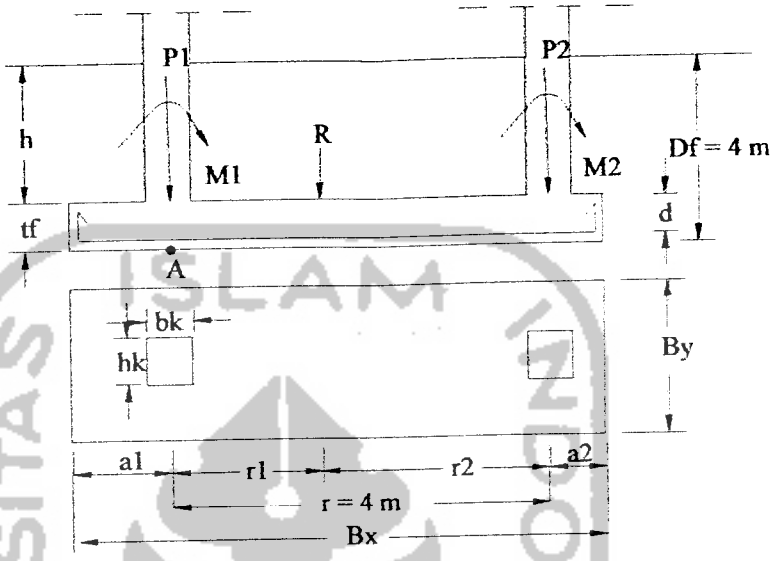
Jarak antar tulangan susut :

$$s \le \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113{,}097.1000}{1000} = 113{,}097\ mm \approx 110\ mm$$

→ Dipakai Tulangan Susut : $P_{12} - 110$ mm

4.5.2 Perencanaan Pondasi Gabungan

A. Perencanaan Dimensi Pondasi



Gambar 4.31 Pondasi telapak gabungan

σ tanah = 400 kN/m^2

Fiqih'c = 28 Mpa

fy = 400 Mpa

P_1 = 1360,768 kN

Mx_1 tetap = 39,42 kNm

My_1 tetap = 26,26 kNm

Mx_1 sementara = 260,15 kNm

My_1 sementara = 260,24 kNm

P_2 = 651,224 kN

Mx_2 tetap = 10,76 kNm

My_2 tetap = 27,12 kNm

γbtanah = 1,744 kN/m^3

γbeton = 24 kN/m^3

Asumsi tebal pelat (tf) = 600 mm

Ukuran kolom :

P1 : 600/600

P2 : 600/600

Mx_2 sementara = 237,67 kNm

My_2 sementara = 233,52 kNm

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

$= 400 - (0,6.24) - (3,4.1,744)$

$= 379,67\ kN/m^2$

1. Tinjauan Terhadap Beban Tetap

$R = P_1 + P_2 = 1360,768 + 651,224 = 2011,992$ kN

Luas minimum pondasi $A = \frac{R}{\sigma_{netto\ tanah}} = \frac{2011,992}{379,67} = 5,3\ m^2$

Pusat berat beban kolom :

$\sum M_A = 0$

$R.\ r_1 + P_2\ .\ r + M_1 + M_2 = 0$

$2011,992.\ r_1 + 651,224.\ 4 + 39,42 + 10,76 = 0$

$r_1 = 1,32$ m

$r_2 = 4 - 1,32 = 2,68$ m

Panjang pondasi : $r_2 + a_2 = ½\ .Bx$

$Bx = 2\ (r_2 + a_2) = 2\ (2,68 + 0,5) = 6,36$ m

$a_1 = 6,36 - 4 - 0,5 = 1,86$ m

Lebar pondasi : $By = \frac{A}{Bx} = \frac{5,3}{6,41} = 0,83$ m

Diambil nilai Bx = 6,5 m dan By = 2 m

$A = Bx\ .\ By = 6,5\ .\ 2 = 13\ m^2$

$Mx_{tot} = 39,42 + 10,76 = 50,18$ kNm

$My_{tot} = 26,26 + 27,12 = 53,38$ kNm

$$e_x = \frac{M_x}{R} = \frac{50,18}{2011,992} = 0,025m$$

$$e_y = \frac{M_y}{R} = \frac{53,38}{2011,992} = 0,026m$$

$$\sigma_{maks} = \frac{R}{A}\left(1 + \frac{6.ex}{Bx} + \frac{6.ey}{By}\right)$$

$$= \frac{2011,992}{13}\left(1 + \frac{6.0,025}{6,5} + \frac{6.0,026}{2}\right)$$

$$= 170,412 \text{ kN/m}^2 \leq 379,67 \text{ kN/m}^2 \text{..... Ok !}$$

2. Tinjauan Terhadap Beban Sementara

$Mx_{tot} = 260,15 + 237,67 = 497,82$ kNm

$My_{tot} = 160,24 + 233,52 = 493,76$ kNm

Eksentrisitas yang terjadi :

$$cx = \frac{My_{tot}}{R} = \frac{493,76}{2011,992} = 0,245 \text{ m}$$

$$ey = \frac{Mx_{tot}}{R} = \frac{497,82}{2011,992} = 0,247 \text{ m}$$

Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{R}{A}\left(1 + \frac{6.ex}{Bx} + \frac{6.ey}{By}\right)$$

$$= \frac{2011,992}{13}\left(1 + \frac{6.0,245}{6,5} + \frac{6.0,247}{2}\right)$$

$$= 304,45 \text{ kN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 379,67 = 569,505 \text{ kN/m}^2 \text{ .....Ok !}$$

B. Perencananaan Geser Satu Arah

→ Ditinjau pada arah memanjang

P1 = 1360,768 kN P2 = 651,224 kN

Mx1 = 260,15 kNm Mx2 = 237,67 kNm

R = 2011,992 kN Mx_{tot} = 497,82 kNm

• Tegangan yang terjadi :

$$q_{tjd} = \frac{R}{A} \pm \frac{6.Mx_{tot}}{By^2.Bx}$$

$$= \frac{2011,992}{13} \pm \frac{6.497,82}{6,5.2^2}$$

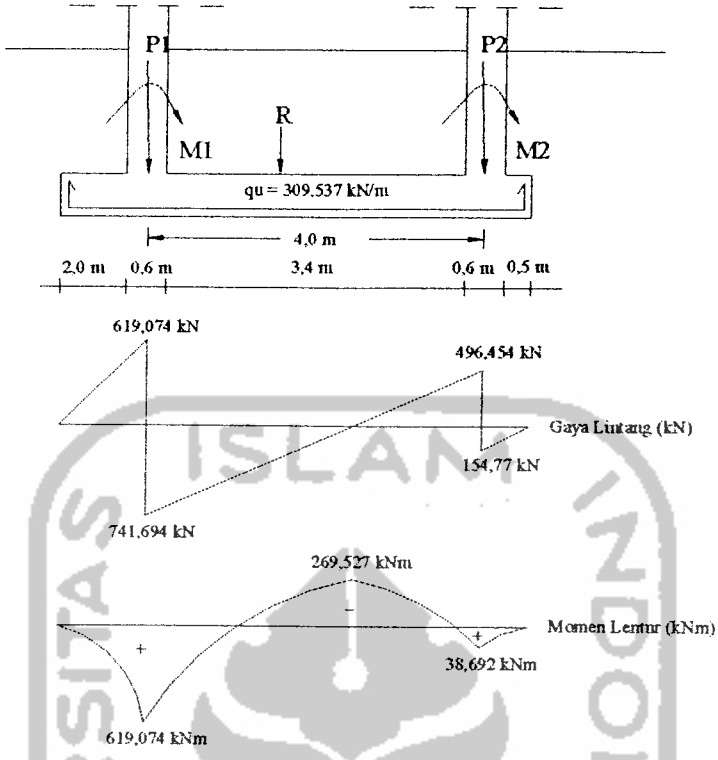
$q_{tjd\ max}$ = 269,65 kN/m^2

$q_{tjd\ min}$ = 39,887 kN/m^2

q_{tjd} = ½ (269,65 + 39,887) = 154,7685 kN/m^2

qu = 154,7685 .2 = 309,537 kN/m

Pada gambar dibawah ini tampak bahwa kedudukan kolom relative dekat dengan ujung pondasi. dengan demikian pada arah memanjang struktur pondasi gabungan dapat berlaku sebagai balok persegi lebar. Dengan menganggap kolom – kolom sebagai penopang dan pondasi akan menerima beban merata keatas yang berasal dari tekanan tanah. Dan untuk analisis geser dan momen yang terjadi dapat dianalisis dengan analisis balok sederhana.



Gambar 4.32 Diagram Geser dan Momen

- Gaya geser terbesar akibat beban luar yang bekerja pada penampang kritis pondasi sejauh d dari muka kolom:

$$Vu_d = Vmak - qu.\left(d + \frac{h_{klm}}{2}\right) = 741{,}694 - 309{,}537 .\left(0{,}519 + \frac{0{,}6}{2}\right) = 488{,}183 \text{ kN}$$

$$Vu/\phi = 488{,}183/0{,}6 = 813{,}638 \text{ kN}$$

- Kekuatan beton menahan geser:

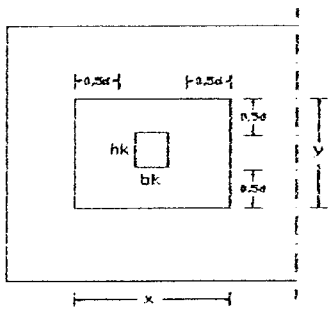
$$Vc = 1/6.\ \sqrt{f'c}\ .\ Bx.\ d = 1/6.\ \sqrt{28}\ .\ 2\ .\ 0{,}519\ .\ 10^3$$

$$= 915{,}429 \text{ kN} \geq Vu/\phi = 813{,}638 \text{ kN} \textbf{ .....Ok !}$$

C. Perencananaan Geser Dua Arah

→ Ditinjau masing-masing kolom pada arah pendek.

1. Kolom P1



Gambar 4.33 Bidang Geser 2 Arah kolom P1

x = bk + d = 600 + 519

= 1119 mm = 1,119 m

y = hk + d = 600 + 519

= 1119 mm = 1,119 m

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

$Vu = P_1 - \sigma_{terjadi} (x \,.\, y)$

$= 1360{,}768 - 304{,}45.(1{,}119 \,.\, 1{,}119) = 979{,}547$ kN

$Vu/\phi = 979{,}547/0{,}6 = 1632{,}579$ kN

• Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{1{,}119}{1{,}119} = 1{,}0$

bo = 2 . (x + y) = 2 . (1119 + 1119) = 4476 mm

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

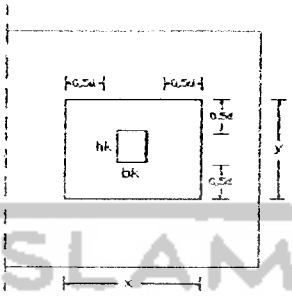
$= (1 + 2/1{,}0).(2.\sqrt{28}).\ 4476.\ 519.\ 10^{-3} = 73754{,}36$ kN

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

$= 4.\sqrt{28}\ .\ 4476\ .\ 519.\ 10^{-3} = 49169{,}57$ kN

Jadi Vc = 49169 kN ≥ Vu/ϕ = 1632,579 kN .....**Ok !**

2. Kolom P2



Gambar 4.34 Bidang Geser 2 Arah kolom P2

x = bk + d = 600 + 519

= 1119 mm = 1,119 m

y = hk + d = 600 + 519

= 1119 mm = 1,119 m

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

$Vu = P_2 - \sigma_{terjadi}(x \,.\, y)$

= 651,224 – 304,45.(1,119 . 1,119) = 270,003 kN

$Vu/\phi = 270{,}003/0{,}6 = 450$ kN

• Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{1{,}119}{1{,}119} = 1{,}0$

bo = 2 . (x + y) = 2 . (1119 + 1119) = 4476 mm

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

$= (1 + 2/1{,}0).(2.\sqrt{28}).\, 4476.\, 519.\, 10^{-3} = 73754{,}36$ kN

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

$= 4.\ \sqrt{28}\ .\ 4476.\ 519.\ 10^{-3} = 49169{,}57\ kN$

Jadi $Vc = 49169\ kN \geq {}^{Vu}/_{\phi} = 450\ kN$ **.....Ok !**

D. Kuat Tumpuan Pondasi

- Kuat tumpuan Pondasi :

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

Luas pelat pondasi (A_2) $= Bx\ .\ By = 6{,}5\ .\ 2{,}0 = 13\ m^2$

Luas penampang kolom $(A_1) = (bk_1\ .\ hk_1) + (bk_2\ .\ hk_2) = (0{,}6.\ 0{,}6) + (0{,}6\ .\ 0{,}6)$

$= 0{,}72\ m^2$

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{13}{0{,}72}} = 4{,}2 > 2$ (jika lebih besar dari 2, dipakai nilai 2)

- Kuat tumpuan pondasi :

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1.2)$

$= 0{,}7.\ (0{,}85.\ 28\ .\ 720000.\ 2).10^{-3} = 23990{,}4\ kN$

- Kuat tumpuan kolom :

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1)$

$= 0{,}7.\ (0{,}85.\ 28.\ 720000).10^{-3} = 11995{,}2\ kN$

- Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 23990{,}4\ kN > \phi.Pn_{\ kolom} = 11995{,}2\ kN$ **.....Ok !**

E. Perencanaan Tulangan Lentur Pondasi Gabungan

1. Arah Memanjang (x)

M^{+}_{mak} = 619,074 kNm

M^{-}_{mak} = 269,527 kNm

a. Momen Positif

Mu = 619,074 kNm

$$Mu/\phi = \frac{619,074}{0,8} = 773,8425 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, maka : $A_{1\varnothing} = 380,133 \text{ mm}^2$
- Tebal pelat pondasi : tf = 600 mm, selimut beton (Pb) = 70 mm

d = tf – Pb – 0,5. $\varnothing_{tul.\ pokok}$ = 600 – 70 – 0,5. 22 = 519 mm

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,807$$

Koefisien ketahanan (Rn), b = 2000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{773,8425.10^6}{2000.519^2} = 1,436 \text{ MPa}$$

Rasio Tulangan :

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{400}.0,85\left(\frac{600}{600+400}\right) = 0,0303$$

ρ_{maks} = 0,75 . ρb = 0,75 . 0,0303 = 0,0227

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

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

$$= \frac{1}{16,807}\left(1-\sqrt{1-\frac{2.16,807.1,436}{400}}\right) = 0,00371 < \rho_{mak} = 0,0227$$

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

$\rho_{perlu} = 0,00371$

$As_{perlu} = \rho_{perlu} . b . d = 0,00371. 2000 . 519 = 3847,404 \text{ mm}^2$

$0,002.b.h = 0,002.2000.600 = 2400 \text{ mm}^2 < As_{perlu}$, maka :

$As_{perlu} = 3847,404 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{380,133.2000}{3847,404} = 197,605 \text{ mm}$$

$s \leq 2.h = 2 .600 = 1200 \text{ mm}$

$s \leq 250 \text{ mm}$

→ Dipakai Tulangan Pokok : D_{22} – 190 mm

$$As_{ada} = \frac{A_{1\theta}.b}{s} = \frac{380,133.2000}{190} = 4001,4 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{4001,4.400}{0,85.28.2000} = 33,625 \text{ mm}$$

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

$= 4001,4 . 400 (519 - {}^{33,625}/_{2})$

$= 803,781 \text{ kNm} \geq {}^{Mu}/_{\phi} = 773,8425 \text{ kNm}$ **.....Ok !**

b. Momen Negatif

$Mu = 269,527 \text{ kNm}$

$$Mu/\phi = \frac{269,527}{0,8} = 336,909 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, maka : $A_{1\varnothing} = 380,133 \text{ mm}^2$
- Tebal pelat pondasi : tf = 600 mm, selimut beton (Pb) = 70 mm

d = tf – Pb – 0,5. $\varnothing_{tul.\ pokok}$ = 600 – 70 – 0,5. 22 = 519 mm

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,807$$

Koefisien ketahanan (Rn), b = 2000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{336,909.10^6}{2000.519^2} = 0,625 \text{ MPa}$$

Rasio Tulangan :

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

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{400}.0,85\left(\frac{600}{600+400}\right) = 0,0303$$

$$\rho_{maks} = 0,75 \,.\, \rho b = 0,75 \,.\, 0,0303 = 0,0227$$

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

$$= \frac{1}{16,807}.\left(1-\sqrt{1-\frac{2.16,807.0,625}{400}}\right) = 0,00158 < \rho_{mak} = 0,0227$$

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

1,33. ρ_{ada} = 0,00211 < ρ_{min} = 0,0035 , maka

ρ_{perlu} = 0,00211

$As_{perlu} = \rho_{perlu} \,.\, b \,.\, d = 0,00211.\ 2000 \,.\, 519 = 2186,186 \text{ mm}^2$

0,002.b.h = 0,002.2000.600 = 2400 mm^2 > As_{perlu}, maka :

$As_{perlu} = 2400\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{380{,}133.2000}{2400} = 316{,}777\ mm$$

$$s \leq 2.h = 2 .600 = 1200\ mm$$

$$s \leq 250\ mm$$

→ Dipakai Tulangan Pokok : D_{22} – 250 mm

$$As_{ada} = \frac{A_{1\theta}.b}{s} = \frac{380{,}133.2000}{250} = 3041{,}064\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

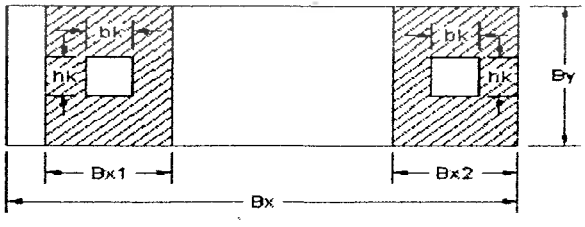
$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{3041{,}064.400}{0{,}85.28.2000} = 77{,}578\ mm$$

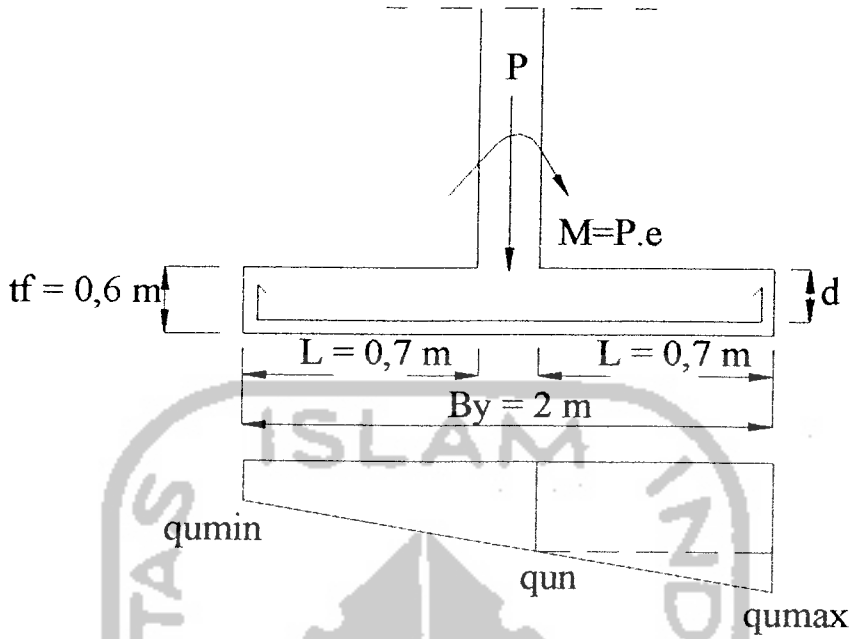
$$Mn = As_{ada}.fy.\ (d - {}^{a}/_{2})$$

$$= 3041{,}064\ .\ 400\ (519 - {}^{77{,}578}/_{2})$$

$$= 584{,}141\ kNm \geq {}^{Mu}/_{\phi} = 336{,}909\ kNm \ \ldots\ldots\ldots\ldots \textbf{Ok !}$$

2. Arah Memendek (y)





Gambar 4.35 Penampang pondasi gabungan arah y

Bx1 = Bx2 = bk + d = 0,6 + 0,519 = 1,119 m

a. Kolom P1

$$L = \frac{By - hk}{2} = \frac{2 - 0,6}{2} = 0,7 \text{ m}$$

$$qu_{max} = \frac{P1}{By.Bx1} + \frac{6.Mx1}{By^2.Bx1}$$

$$= \frac{1360,768}{2.1,119} + \frac{6.260,15}{2^2.1,119}$$

$$= 956,755 \text{ kN/m}^2$$

$$qu_{min} = \frac{P1}{By.Bx1} - \frac{6.Mx1}{By^2.Bx1}$$

$$= \frac{1360,768}{2.1,119} - \frac{6.260,15}{2^2.1,119}$$

$$= 259,302 \text{ kN/m}^2$$

$$\frac{(956{,}755 - qu_n)}{0{,}7} = \frac{(956{,}755 - 259{,}302)}{2}$$

$qu_n = 712{,}646\ kN/m^2$

$Mu = 0{,}5 \cdot qu_n \cdot L^2 = 0{,}5 \cdot 712{,}646 \cdot 0{,}7^2 = 174{,}598\ kNm$

$$Mu/\phi = \frac{174{,}598}{0{,}8} = 218{,}248\ kNm$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, maka : $A_{1\varnothing} = 380{,}133\ mm^2$
- Tebal pelat pondasi : tf = 600 mm, selimut beton (Pb) = 70 mm

$d = tf - Pb - 0{,}5.\ \varnothing_{tul.\ pokok} = 600 - 70 - 0{,}5.\ 22 = 519$ mm

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.28} = 16{,}807$$

Koefisien ketahanan (Rn), diambil b tiap 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{218{,}248.10^6}{1000.519^2} = 0{,}8102\ MPa$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0303$$

$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0303 = 0{,}0227$

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

$$= \frac{1}{16{,}807}.\left(1 - \sqrt{1 - \frac{2.16{,}807.0{,}8102}{400}}\right) = 0{,}00206 < \rho_{mak} = 0{,}0227$$

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

$1{,}33\ \rho_{ada} = 0{,}00274 < \rho_{min} = 0{,}0035$, maka : $\rho_{perlu} = 0{,}00274$

$As_{perlu} = \rho_{perlu} . b . d = 0{,}00274 . 1000 . 519 = 1422{,}864\ mm^2$

$0{,}002.b.h = 0{,}002.1000.600 = 1200\ mm^2 < As_{perlu}$, maka :

$As_{perlu} = 1422{,}864\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{380{,}133.1000}{1422{,}864} = 267{,}160\ mm$$

$$s \leq 2.h = 2 .600 = 1200\ mm$$

$$s \leq 250\ mm$$

→ Dipakai Tulangan Pokok : D_{22} – 250 mm

$$As_{ada} = \frac{A_{1\theta}.b}{s} = \frac{380{,}133.1000}{250} = 1520{,}532\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1520{,}532.400}{0{,}85.28.1000} = 25{,}555\ mm$$

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

$$= 1520{,}532.\ 400\ (519 - {}^{25{,}555}/_{2})$$

$$= 307{,}891\ kNm \geq {}^{Mu}/_{\phi} = 218{,}248\ kNm$$ **.....Ok !**

b. Kolom P2

$$L = \frac{By - hk}{2} = \frac{2 - 0{,}6}{2} = 0{,}7\ m$$

$$qu_{max} = \frac{P2}{By.Bx2} + \frac{6.Mx2}{By^2.Bx2}$$

$$= \frac{651{,}224}{2.1{,}119} + \frac{6.237{,}67}{2^2.1{,}119}$$

$$= 609{,}577 \text{ kN/m}^2$$

$$qu_{min} = \frac{P2}{By.Bx2} - \frac{6.Mx2}{By^2.Bx2}$$

$$= \frac{651{,}224}{2.1{,}119} - \frac{6.237{,}67}{2^2.1{,}119}$$

$$= -27{,}607 \text{ kN/m}^2$$

$$\frac{(609{,}577 - qu_n)}{0{,}7} = \frac{(609{,}577 - (-27{,}607))}{2}$$

$qu_n = 386{,}563 \text{ kN/m}^2$

$Mu = 0{,}5 \cdot qu_n \cdot L^2 = 0{,}5 \cdot 386{,}563 \cdot 0{,}7^2 = 94{,}708 \text{ kNm}$

$$Mu/\phi = \frac{94{,}708}{0{,}8} = 118{,}385 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, maka : $A_{1\varnothing} = 380{,}133 \text{ mm}^2$
- Tebal pelat pondasi : tf = 600 mm, selimut beton (Pb) = 70 mm

  $d = tf - Pb - 0{,}5. \varnothing_{tul.\ pokok} = 600 - 70 - 0{,}5. 22 = 519 \text{ mm}$

  $$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.28} = 16{,}807$$

Koefisien ketahanan (Rn), diambil b tiap 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{118{,}385.10^6}{1000.519^2} = 0{,}4395 \text{ MPa}$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.28.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0303$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0303 = 0{,}0227$$

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

$$= \frac{1}{16{,}807}.\left(1-\sqrt{1-\frac{2.16{,}807.0{,}4395}{400}}\right) = 0{,}00111 < \rho_{mak} = 0{,}0227$$

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

$1{,}33\ \rho_{ada} = 0{,}00147 < \rho_{min} = 0{,}0035$, maka : $\rho_{perlu} = 0{,}00147$

$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00147.\ 1000\ .\ 519 = 765{,}569\ mm^2$

$0{,}002.b.h = 0{,}002.1000.600 = 1200\ mm^2 > As_{perlu}$, maka :

$As_{perlu} = 765{,}569\ mm^2$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{380{,}133.1000}{765{,}569} = 496{,}536\ mm$$

$$s \le 2.h = 2\ .600 = 1200\ mm$$

$$s \le 250\ mm$$

→ **Dipakai Tulangan Pokok : D_{22} – 250 mm**

$$As_{ada} = \frac{A_{\theta}.b}{s} = \frac{380{,}133.1000}{250} = 1520{,}532\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1520{,}532.400}{0{,}85.28.1000} = 25{,}55\ mm$$

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

$$= 1520{,}532.\ 400\ (519 - {}^{25{,}55}/_{2})$$

$$= 307{,}89\ \text{kNm} \ \geq\ {}^{Mu}/_{\phi} = 118{,}385\ \text{kNm}\ \textbf{.....Ok !}$$

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 600 = 1200\ mm^2$

- Digunakan tulangan bagi ∅12 mm, maka: $A_{1\varnothing} = 113{,}097\ mm^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113{,}097.1000}{1200} = 94{,}25\ \text{mm} \approx 90\ \text{mm}$$

→ **Dipakai Tulangan Susut : P_{12} – 90 mm**

4.7 Perencanaan Tangga

4.7.1 Spesifikasi Struktur

a. Tinggi antar lantai (h) = 4 m = 400 cm

b. Lebar bordes (Lb) = 2 m = 200 cm

c. Tinggi optrede rencana diambil 20 cm

jumlah optrede = 400/20 = 20 buah

timggi optrede pakai (h'_o) = 400/20 = 20 cm

jumlah antrede = 20 – 2 = 18 buah

dipakai lebar antrede (La) = 30 cm

d. dimensi tangga :

tangga dibagi menjadi dua bagian, sehingga panjang bentang tangga (Pt) :

Pt = (La x jumlah antrede/2) + Lb

= (30 x 18/2) + 200

= 470 cm = 4,7 m

Lebar bersih tangga (Lt) = 0,5.(3,4 - 3. 0,15) = 1,475 m = 147,5 cm

e. Beban sandaran tangga :

tinggi sandaran = 1 m

tebal sandaran = 0,12 m

berat sandaran tangga = (0,12 . 1 . 24 . 2)/1,475 = 3,905 KN/m^2

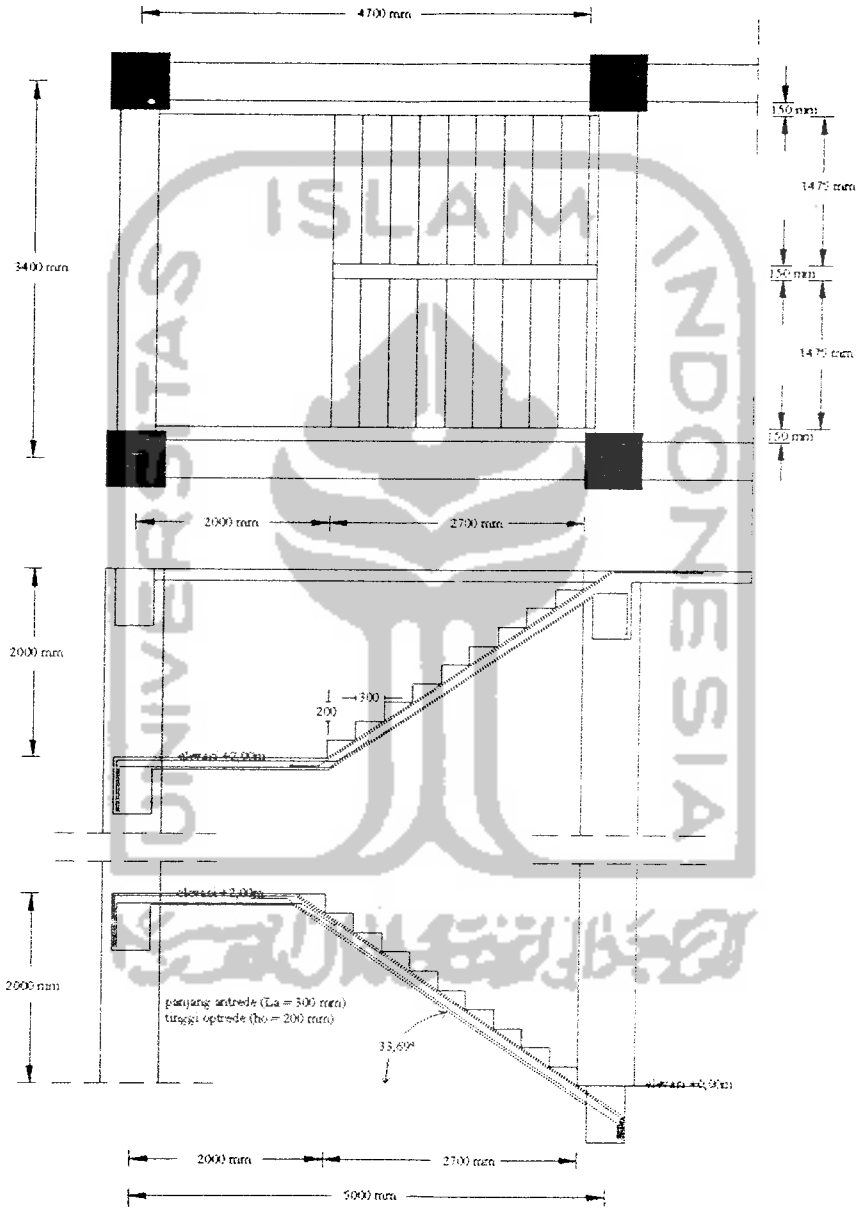
berat sandaran bordes = (0,12 . 1 . 24)/1,475 = 1,952 KN/m^2

f. Sudut kemiringan tangga (α) :

$$\alpha = \text{arc tgn}\frac{h'_o}{La} = arc\,tgn\frac{20}{30} = 33{,}69°$$

g. Tebal pelat tangga dan bordes diambil = 12 cm

$$\text{tebal pelat sisi miring (h')} = \frac{12}{\cos 33{,}69^\circ} = 14{,}422 \text{ cm}$$



Gambar 4.36 Dimensi Tangga

4.7.2 Pembebanan

a. Pembebanan bordes

Beban mati :

- Berat sendiri pelat = 0,12 x 24 = 2,88 KN/m^2
- Berat spesi = 0,03 x 21 = 0,63 KN/m^2
- Berat keramik = 0,01 x 20 = 0,20 KN/m^2
- Berat sandaran = 1,952 KN/m^2 +

Q_D = 5,662 KN/m^2

untuk lebar 1,475 m, maka qD = 1,475 x 5,168 = 8,351 KN/m'

Beban hidup: Q_L = 300 kg/cm^2 = 3 KN/m^2

qL = 1,475 x 3 = 4,425 KN/m'

q_U = 1,2qD + 1,6qL = 1,2.8,351 + 1,6.4,425= 17,101 KN/m'

b. Pembebanan tangga

Beban mati :

- Berat sendiri = (0,14422 + 0,2/2) x 24 = 5,861 KN/m^2
- Berat spesi = 0,03 x 21 = 0,63 KN/m^2
- Berat keramik = 0,01 x 20 = 0,20 KN/m^2
- Berat sandaran = 3,905 KN/m^2 +

Q_D = 10,596 KN/m^2

qD = 1,475 x 9,607 = 15,629 KN/m'

Beban hidup : Q_L = 300kg/cm^2 = 3 KN/m^2

qL = 1,475 x 3 = 4,425 KN/m'

qu = 1,2qD + 1,6qL = 1,2.15,629 + 1,6.4,425= 25,835 KN/m

4.7.3 Penulangan Tangga

Perhitungan Tulangan Pelat Bordes

Mu maks = 25,33 KNm

$$Mu/\phi = \frac{25,33}{0,8} = 31,6625 \text{ KNm}$$

Digunakan tulangan Ø13 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1Ø} = ¼ . \pi . D^2 = ¼ . \pi . 13^2 = 132,732 \text{ mm}^2$

tebal pelat bordes (h) = 120 mm, selimut beton (Pb) = 20 mm, maka :

$d = h - Pb - 0,5.Ø_{tul\ pokok} = 120 - 20 - 0,5.13 = 93,5$ mm

Rasio tulangan :

$$\rho b = \frac{0,85.f'c}{fy}\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{400}0,85.\left(\frac{600}{600+400}\right) = 0,0303$$

$\rho maks = 0,75.\rho b = 0,75 . 0,0303 = 0,0227$

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

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{31,6625.10^6}{1475.93,5^2} = 2,255 \text{ MPa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.28} = 16,8067$$

$$\rho_{perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.Rn}{fy}}\right) = \frac{1}{16,8067}\left(1-\sqrt{1-\frac{2.16,8067.2,255}{400}}\right)$$

$= 0,00649 < \rho maks = 0,0227$

$> \rho min = 0,0035$

sehingga $\rho_{pakai} = \rho_{perlu} = 0{,}00649$

$As = \rho_{pakai} \cdot b \cdot d \geq 0{,}002 \cdot b \cdot h$

$= 0{,}00649 \cdot 1475 \cdot 93{,}5 \geq 0{,}002 \cdot 1475 \cdot 120$

$= 895{,}4659 \text{ mm}^2 > 354 \text{ mm}^2$ (Ok!)

$$\text{jarak tulangan (s)} = \frac{A_1\phi.b}{As} = \frac{132{,}732.1475}{895{,}4659} = 218{,}635 \text{ mm}$$

$\leq 2.h = 2.\ 120 = 240$ mm

≤ 250 mm

Dipakai tulangan pokok : D13 – 210 mm

Kontrol kapasitas lentur pelat bordes yang terjadi :

$$As_{ada} = \frac{A_1\phi.b}{s} = \frac{132{,}732.1475}{210} = 932{,}284 \text{ mm}^2$$

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{932{,}284.400}{0{,}85.28.1475} = 10{,}623 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \left(d - {}^{a}/_{2}\right) \geq {}^{Mu}/_{\phi}$$

$$= 932{,}284 \cdot 400 \left(93{,}5 - {}^{10{,}623}/_{2}\right)$$

$= 32{,}887$ kNm $> 31{,}6625$ kNm ..... (Ok)

Tulangan Bagi Pelat Bordes

$As_{bagi} = 0{,}002 \cdot b \cdot h = 0{,}002 \cdot 1475 \cdot 120 = 354\ mm^2$

Dipakai tulangan bagi Ø8, maka $A_1Ø = 50{,}265\ mm^2$

$$\text{jarak tulangan bagi (s)} = \frac{A_1\phi.b}{As_{bagi}} = \frac{50{,}265.1475}{354} = 209{,}437\ mm$$

$$\leq 2.\ h = 2.\ 120 = 240\ mm$$

$$\leq 250\ mm$$

Dipakai tulangan bagi P8-200

4.7.4 Perencanaan Balok Bordes

Dimensi rencana balok :

tinggi (h) = 400 mm

lebar (b) = 250 mm

tinggi efektif (d) = 400 – 70 (dianggap tulangan sebelah 1 lapis)

= 330 mm

Pembebanan :

- beban akibat tangga : 38,94/1,475 = 26,4 kN/m

- berat sendiri = 1,2.0,25.0,4.24 = 2,880 kN/m +

qu = 29,28 kN/m

Momen tumpuan :

$$Mu = -\frac{1}{16} qu.L^2 = -\frac{1}{16}.29,28.4^2 = -29,28 \text{ kNm}$$

Momen lapangan :

$$Mu = \frac{1}{11} qu.L^2 = \frac{1}{11}.29,28.4^2 = 42,589 \text{ kNm}$$

a. Perencanaan tulangan lentur balok bordes

Tulangan lapangan

$$Mu/\phi = 53,236 \text{ kNm}$$

$$\rho b = \frac{0,85.f'c}{fy}\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0,85.28}{400}0,85.\left(\frac{600}{600+400}\right) = 0,0303$$

$$\rho maks = 0,75.\rho b = 0,75 \;.\; 0,0303 = 0,0227$$

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

$\rho pakai = 0{,}5.\rho maks = 0{,}5 \;.\; 0{,}0227 = 0{,}01135$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.28} = 16{,}807$$

$Rn = \rho.fy\,(1 - \tfrac{1}{2}\,\rho.m) = 0{,}01135 \;.\; 400\,(1 - \tfrac{1}{2}\,.0{,}01135 \;.\; 16{,}807) = 4{,}107$ MPa

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{53{,}236.10^6}{4{,}107.250}} = 227{,}442 \text{ mm} < d = 330 \text{ mm}$$, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^{\;2}} = \frac{53{,}236.10^6}{250.330^2} = 1{,}955 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{1{,}077}{4{,}107}0{,}01135 = 0{,}0054 > \rho_{min} = 0{,}0035$$

$\rho_{perlu} = \rho_{ada} = 0{,}0054$

$As = \rho_{perlu} \;.\; b \;.\; d = 0{,}0054 \;.\; 250 \;.\; 330 = 445{,}95 \text{ mm}^2$

Dipakai diameter tulangan D16, maka : $A1Ø = 201{,}96 \text{ mm}^2$

$$n = \frac{As}{A1\phi} = \frac{445{,}95}{201{,}96} = 2{,}21 \text{ batang}$$

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

$As_{ada} = 3 \;.\; 201{,}96 = 602{,}88 \text{ mm}^2 > As = 445{,}95 \text{ mm}^2$

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{602{,}88.400}{0{,}85.28.250} = 40{,}53 \text{ mm}$$

$$Mn = As_{ada}\, fy\,(d - a/2) \geq Mu/\phi$$

$= 602,88 \; . \; 400 \; (330 - {}^{40,53}/_{2})$

$= 74,693$ kNm $> {}^{Mu}/_{\phi} = 53,236$ kNm

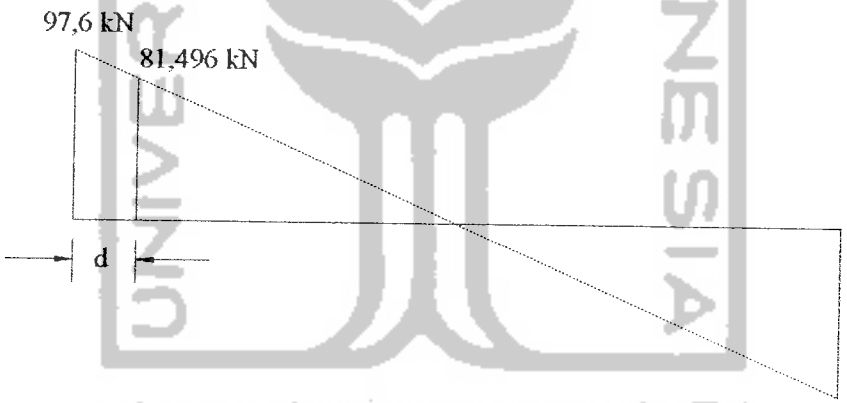
b. Perencanaan tulangan geser balok bordes

Gaya geser dukungan

Vu dukungan $= \frac{1}{2}.qu.L$

$= \frac{1}{2}.29,28.4 = 58,56$ kN

maka ${}^{Vu}/_{\phi} = \frac{58,56}{0,6} = 97,6$ kN



Gambar 4.37 Diagram geser balok bordes

Vu pakai $= \left(\frac{2-0,33}{2}\right).97,6 = 81,496$ kN

Tegangan geser beton (Vc) :

$Vc = \left(\frac{1}{6}\sqrt{f'c}\right)b.d = \left(\frac{1}{6}\sqrt{28}\right)250.330 = 72758$ N $= 72,758$ kN

Vu pakai = 81,496 > Vc = 72,758 , maka perlu tulangan geser.

Vs = Vu pakai – Vc = 81,496 – 72,758 = 8,738

Dipakai sengkang P10 , maka Av = 2 .¼.π.10^2 = 157 mm^2

$$s \leq \frac{Av.fy.d}{Vs}$$

$$\leq \frac{157.240.330}{8{,}738.10^3}$$

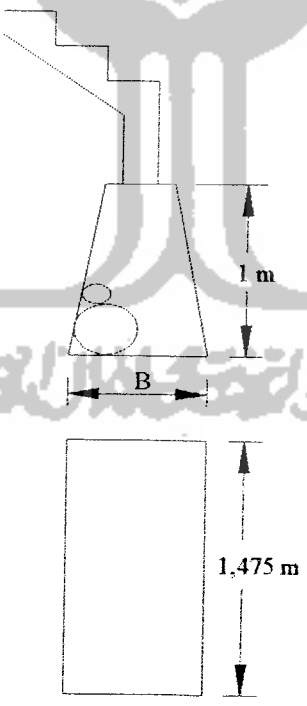
$$\leq 1423 \text{ mm}$$

$$\leq d/2 = 330/2 = 165 \text{ mm}$$

$$\leq 600 \text{ mm}$$

jadi dipakai sengkang **P10 – 165 mm**

4.7.5 Perencanaan Pondasi Tangga



Gambar 4.38 Pondasi Tangga

Diketahui : σ tanah = 100 kN/m^2

γ batu = 22 kN/m^3

Balok diatas pondasi 20/40

Tinjauan untuk lebar 1,475 m

Pembebanan :

- Akibat beban tangga = 145,71 kN

- Berat balok diatas pondasi = 1,2x0,2x0,4x1,475x24 = 3,398 kN +

Pu = 149,108 kN

σ ijin = σ tanah – γ batu . h

= 100- 22.1 = 78 kN/m^2

$$\sigma \text{ ijin} = \frac{Pu}{A}$$

$$A = \frac{Pu}{\sigma_{ijin}} = \frac{149{,}108}{78} = 1{,}912 \text{ m}^2$$

A = B x L , dimana L = 1,475 m, maka :

$$B = \frac{1{,}912}{1{,}475} = 1{,}296 \text{ , dipakai } 1{,}4 \text{ m}$$

A = 1,4 x 1,475 = 2,065 m^2

Kontrol tegangan yang terjadi :

$$\sigma \text{ terjadi} = \frac{Pu}{A} \leq \sigma_{ijin}$$

$$= \frac{149{,}108}{2{,}065}$$

= 72,207 kN/m^2 < 78 kN/m^2 ..... (OK)