BAB IV

PERHITUNGAN KONSTRUKSI

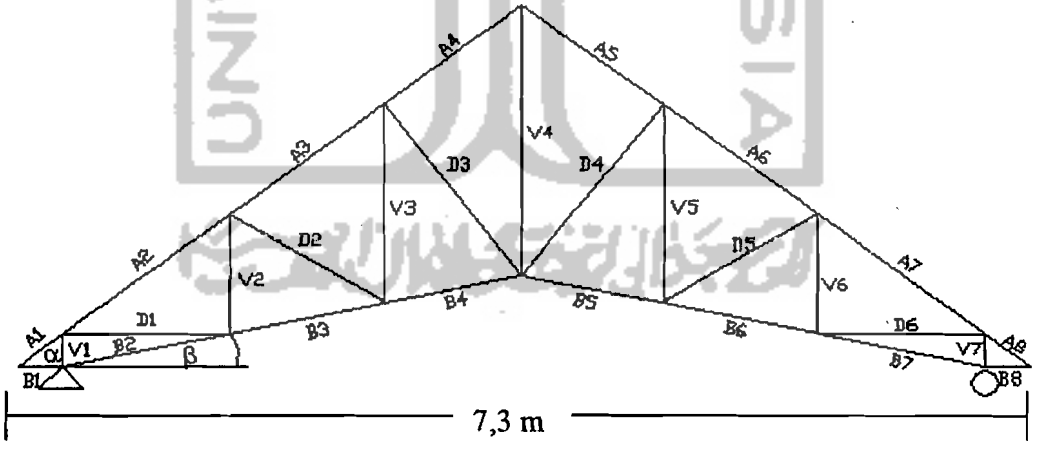
4.1 Perencanaan Atap

Pada perencanaan atap ini didasarkan pada spesifikasi dari AISC yaitu salah satunya adalah metode Allowable Stress Design. Prinsip metode ini, elemen structural harus direncanakan sedemikian rupa hingga tegangan yang dihitung akibat beban kerja tidak melampaui tegangan ijin yang ditentukan.

Pada perencanaan ulang ini tipe dari struktur kuda-kuda adalah sebagai berikut :

1. Atap KK-1

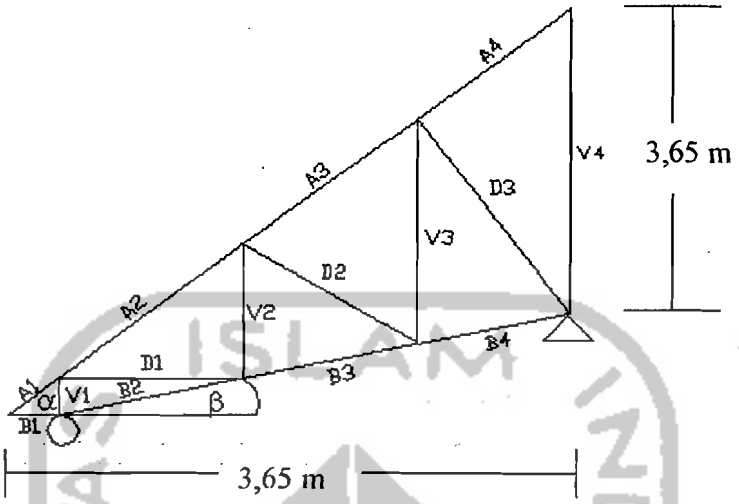
$\alpha = 45^0 \quad \beta = 18{,}25^0$



Gambar 4.1 Rencana Atap Kuda-Kuda 1

2. Atap KK-2

$\alpha = 45^0 \quad \beta = 18{,}25^0$



Gambar 4.2 Rencana Atap Kuda-Kuda 2

Di bawah ini akan diberikan contoh perhitungan untuk tipe kuda-kuda KK-1

4.1.1 Perencanaan Gording

Perencanaan gording diasumsikan seperti halnya merencanakan balok yaitu dengan menganggap bahwa gording mengalami lentur dua arah bila beban bekerja pada salah satu arah utamanya.

1. Data-data

   Jarak antar kuda-kuda = 4,5 m

   Mutu baja profil fy = 2400 kg/cm^2

   Kuat tarik fu = 3600 kg/cm^2

   Mutu baut non full drat ; fy = 2050 kg/cm^2

   Fu = 8250 kg/cm^2

   Atap genteng, Usuk dan reng kayu, Goding baja (light channel)

   Angin direncanakan pada daerah daratan.

Rumus yang digunakan untuk kontrol tegangan dan lendutan perencanaan gording pada BAB III rumus 3.2.1 s/d 3.2.6

Pembebanan gording

A. Beban tetap

- Berat penutup atap = 50 x 1,7 = 85 kg/m'
- Beban hidup = 20 x 1,7 = 34 kg/m'
- Beban gording = 10 kg/m' +

q = 129 kg/m'

$q \perp = q \cos \alpha = 129 \cos 45° = 91{,}217$ kg/m'

$q // = q \sin \alpha = 129 \sin 45° = 91{,}217$ kg/m'

B. Beban angin

Pada daerah daratan w = 25 kg/cm^2

1. angin tekan (wt) untuk $\alpha < 65^0$

diketahui $\alpha = 45°$

$C_1 = 0{,}02\ \alpha - 0{,}4 = 0{,}02.45° - 0{,}4 = 0{,}5$

Wt = C_1. w . jarak gording = 0,5 . 25 . 1,7 = 21,25 kg/m'

2. angin hisap (wh)

$C_2 = -0{,}4$

Wh = C_2 . w. jarak gording = -0,4 . 25 . 1,7 = -17 kg/m'

C. Perhitungan momen

- akibat beban tetap

$q \perp = Mmaks = 1/8.\ q\perp.\ b^2 = 1/8 . 91{,}217 . 4{,}5^2$

= 230,893 kgm' = 23089,3 kgcm'

q // = Mmaks = 1/32 . q// . b^2 = 1/32 . 91,217 . $4,5^2$

= 51,723 kgm' = 5172,3 kgcm'

- akibat beban angin

M maks = 1/8 W.b^2 = 1/8 . 21,25.$4,5^2$ = 53,789 kgm' = 5378,9 kgcm'

D. Dimensi gording

Dicoba profil 150 x 50 x 20 x 2,3 (*Light Lip Channel)*

A = 6,322 cm^2 Sx = 28,0 cm^3

W = 4,96 kg/m' < 10 kg/m' Sy = 6,33 cm^3

Ix = 210 cm^4

Iy = 21,9 cm^4

- Kontrol tegangan

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

$$fbx=\frac{M\perp}{Sx}=\frac{23089,3+5378,9}{28,0}=1016,72kg/cm^2$$

$$fby=\frac{M//}{Sy}=\frac{5172,3}{6,33}=817,109kg/cm^2$$

$$\frac{1016,72}{0,66.2400}+\frac{817,109}{0,75.2400}=1,0\le 1,0\Rightarrow ok!$$

- Kontrol lendutan

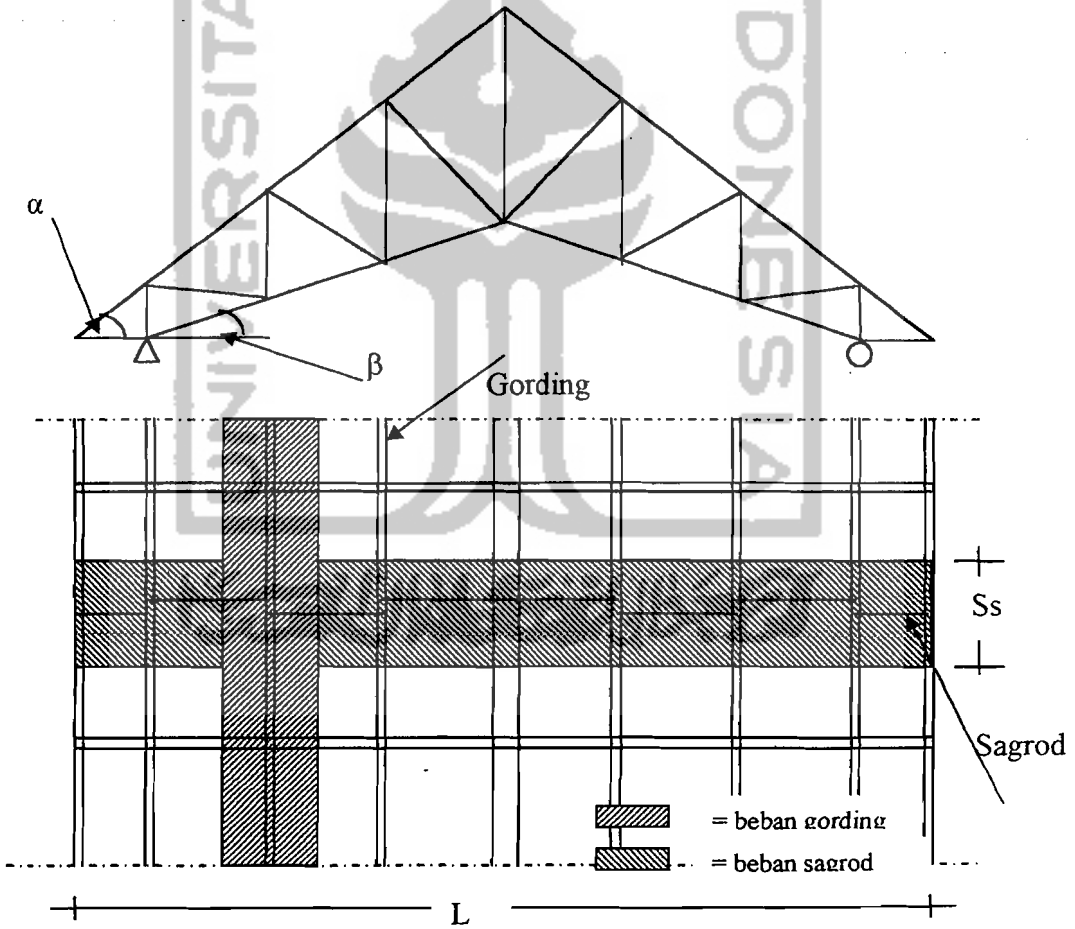
$$\delta\perp=\frac{5}{384}\frac{q\perp.b^4}{E.Ix}=\frac{5}{384}.\frac{0,91217.450^4}{2,1.10^6.210}=1,1044\le b/360=1,25cm\Rightarrow ok!$$

$$\delta // = \frac{5}{384}\frac{q//.[b/(a+1)]^4}{E.Iy} = \frac{5}{384}\frac{0{,}91217[450/(1+1)]}{2{,}1.10^6.21{,}9} = 0{,}662cm < b/360 \Rightarrow ok!$$

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

E. Perencanaan sagrod dan tierod

Denah sagrod dan tierod terletak pada gambar di bawah ini sedangkan rumus yang digunakan mengacu pada landasan teori dari rumus perencanaan sagrod dan tierod 3.2.7 s/d 3.2.14.



Gambar 4.3 Pembebanan Atap

1. Beban sagrod

- berat penutup atap x (½. L/cosα) = 50 x (½..7,3/cos 45°)

= 258,094 kg/m'

- beban hidup x (½.. L/cos α) = 20 x (½..7,3/cos 45°) = 103,238 kg/m'

- jml gording satu sisi miring x berat gording = 4 x 4,96 = 19,84 kg/m'

P = 258,094 + 103,238 + 19,84 = 381,172 kg/m'

P// = P. sin α . Ss = 381,172 . sin 45° . 2,25 = 606,441 kg

2. dimensi sagrod

$$\text{Asagrod} = \frac{P//}{0{,}33Fu} = \frac{606{,}441}{0{,}33.3600} = 0{,}5105\,\text{cm}^2$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4.0{,}5105}{\pi}} = 0{,}81\,\text{cm} \rightarrow \text{dipakai } \varnothing \text{ ½ in}$$

3. dimensi tierod

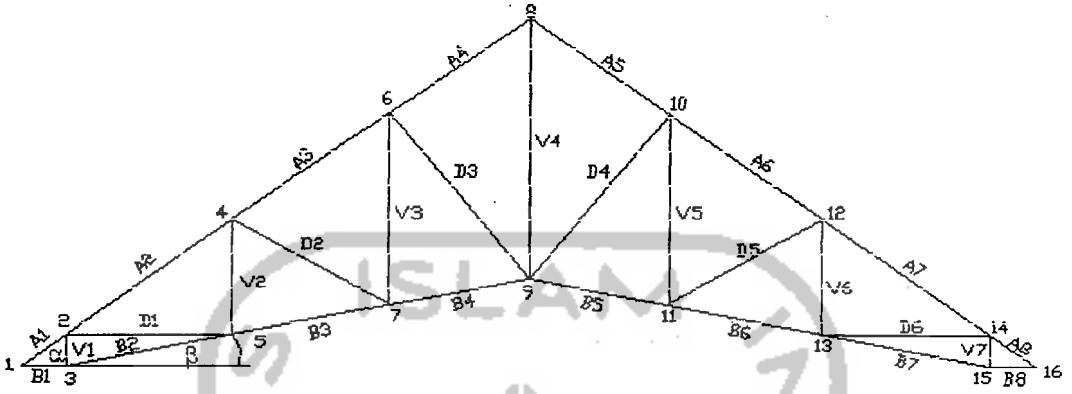
Beban tierod : T = P//.cos α = 606,441.cos 45° = 428,819 kg

Atierod = T/0,33 Fu = 0,361 cm^2

$$D = \sqrt{\frac{4.A}{\pi}} = \sqrt{\frac{4.0{,}361}{\pi}} = 0{,}6782\,\text{cm}$$

Tierod terpakai = D + 0,3 = 0,9782 cm→ dipakai ∅ ½ in

4.1.2 Perencanaan Kuda-Kuda KK-1



L = 7,3 m α = 45° β = 18,25°

Gambar 4.4 Rencana Kuda-Kuda KK1

- Pembebanan

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

$= \left(10 \pm \left(\frac{7{,}3-12}{3}\right).5\right).4{,}5 = 80{,}25$ kg/m'

a. Beban tetap :

- berat gording = 4,96 kg/m'
- berat eternit+penggantung = 18 kg/m'
- berat penutup atap = 50 kg/m'
- beban hidup = 20 kg/m'
- taksiran beban kuda-kuda = 80,25 kg/m'

- Beban masing-masing joint

P1 = P9 (joint 1 & 16)

Berat gording = 4,96.4,5 = 22,32 kg

Berat penutup atap = 50x4,5x0,5.0,424 = 47,7 kg

Beban hidup = 20x4,5x0,5.0,424 = 19,08 kg +

P1=P9 = 89,1 kg

P2 = P8 (joint 2 & 14)

Berat gording = 4,96.4,5 = 22,32 kg

Berat penutup atap = 50x4,5x(0,5.0,424+0,5.1,287) = 192,488 kg

Beban hidup = 20x4,5x(0,5.0,424+0,5.1,287) = 76,995 kg +

P2=P8 = 291,803 kg

P3 = P7 (joint 4 & 12)

Berat gording = 4,96.4,5 = 22,32kg

Berat penutup atap = 50x4,5x(0,5.1,287+0,5.1,711) = 337,275 kg

Beban hidup = 20x4,5x (0,5.1,287+0,5.1,711) = 134,91 kg +

P3 = P7 = 494,505 kg

P4 = P6 (joint 6 & 10)

Berat gording = 4,96.4,5 = 22,32 kg

Berat penutup atap = 50x4,5x(0,5.1,711+0,5.1,74) = 388,238 kg

Beban hidup = 20x4,5x (0,5.1,711+0,5.1,74) = 155,295 kg +

P4 = P6 = 565,853 kg

P5 (joint 8)

Berat gording = 2x4,96x4,5 = 44,64 kg

Berat penutup atap = 50x4,5x1,74 = 391,5 kg

Beban hidup = 20x4,5x1,74 = 156,6 kg +

P5 = 592,774 kg

P1' =P9' (joint 1 & 16)

Berat eternit+plafond=18x4,5x0,5.0,3 = 12,15 kg

Berat taksiran kuda-kuda=80,25x0,5.0,3 = 12,038 kg +

P1' =P9' = 24,188 kg

P3' = P7' (joint 5 & 13)

Berat eternit+plafond=18x4,5x(0,5.0,96+0,5.1,274) = 90,477 kg

Berat taksiran kuda-kuda=80,25x(0,5.0,96+0,5.1,274) = 89,64 kg +

P3' = P7' = 180,117 kg

P4' = P6' (joint 7 & 11)

Berat eternit+plafond=18x4,5x(0,5.1,274+0,5.1,297) = 104,126 kg

Berat taksiran kuda-kuda=80,25x(0,5.1,274+0,5.1,297) = 103,1614kg +

P4' = P6' = 207,2874 kg

P5' (joint 9)

Berat eternit+plafond=18x4,5x(0,5.1,297+0,5.1,297) = 105,057 kg

Berat taksiran kuda-kuda=80,25x(0,5.1,297+0,5.1,297) = 104,084 kg +

P5' = 209,141 kg

b. beban angin

Muatan angin di darat = 25 kg/m^2

Koefisien beban angin menurut Peraturan Pembebanan Indonesia untuk Gedung 1983, untuk $\alpha<65^0$

Diketahui : $\alpha = 45°$

Tekan = $C_1 = 0{,}02.\ \alpha - 0{,}4 = 0{,}5$

Hisap = $C_2 = -0{,}4$

Beban yang bekerja

$W_t = C_1 x\ w = 0{,}5.25 = 12{,}5\ kg/m^2$

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

- sisi kiri

$Wt_1 = 12{,}5.0{,}5.0{,}424.4{,}5 = 11{,}925$ kg

$Wt_2 = 12{,}5\ (0{,}5.0{,}424 + 0{,}5.1{,}287).4{,}5 = 48{,}122$ kg

$Wt_3 = 12{,}5.(0{,}5.1{,}287 + 0{,}5.1{,}711).4{,}5 = 84{,}319$ kg

$Wt_4 = 12{,}5.(0{,}5.1{,}711 + 0{,}5.1{,}74).4{,}5 = 97{,}06$ kg

$Wt_5 = 12{,}5\ .\ 0{,}5\ .\ 1{,}74\ .\ 4{,}5 = 48{,}938$ kg

- sisi kanan

$Wh_5 = -10\ .\ 0{,}5\ .\ 1{,}74\ .\ 4{,}5 = 39{,}15$ kg

$Wh_4 = -10.(0{,}5.1{,}711 + 0{,}5.1{,}74).4{,}5 = 77{,}648$ kg

$Wh_3 = -10.(0{,}5.1{,}711 + 0{,}5.1{,}287).4{,}5 = 67{,}455$ kg

$Wh_2 = -10.(0{,}5.1{,}287 + 0{,}5.0{,}424).4{,}5 = 38{,}498$ kg

$Wh_1 = -10.(0{,}5.0{,}424).4{,}5 = 9{,}54$ kg

Hasil analisa rangka menggunakan SAP2000 dapat dilihat pada tabel berikut :

Tabel 4.1 Beban rencana KK-1

Element	Panjang (m)	B.Tetap (kg)	B.Angin Kiri (kg)	B.Angin Kanan (kg)	0.3Beban Tetap (kg)	Beban Angin (kg)	Beban Rencana (kg)
A1	0.424	160.21	65.7	-9.54	48.063	56.16	216.37
A2	1.287	-2561.67	-143.15	100.31	-768.501	-42.84	-2561.67
A3	1.711	-2523.39	-125.91	89.78	-757.017	-36.13	-2523.39
A4	1.74	-1985.69	-93.25	64.96	-595.707	-28.29	-1985.69
A5	1.74	-1985.69	-181.34	153.06	-595.707	-28.28	-1985.69
A6	1.711	-2523.39	-138.8	104.4	-757.017	-34.4	-2523.39
A7	1.287	-2561.67	-84.86	48.95	-768.501	-35.91	-2561.67
A8	0.424	160.21	-9.54	11.92	48.063	2.38	160.21
B1	0.3	-113.29	-101.35	13.49	-33.987	-87.86	-201.15
B2	0.96	-119.29	397.7	-379.15	-35.787	18.55	-119.29
B3	1.274	1907.78	450.82	-432.39	572.334	18.43	1907.78
B4	1.297	1882.2	376.09	-374.89	564.66	1.2	1882.2
B5	1.297	1882.2	189.68	-189.78	564.66	-0.1	1882.2
B6	1.274	1907.78	98.98	-85.39	572.334	13.59	1907.78
B7	0.96	-119.29	14.21	-17.76	-35.787	-3.55	-119.29
B8	0.3	-113.29	13.49	-16.86	-33.987	-3.37	-113.29
D1	0.91	1924.66	50.34	-50.45	577.398	-0.11	1924.66
D2	1.3131	-29.37	-77.61	59.84	-8.811	-17.77	-47.14
D3	1.797	-554.95	-133.85	105.73	-166.485	-28.12	-554.95
D4	1.797	-554.95	124	-150.34	-166.485	-26.34	-554.95
D5	1.3131	-29.37	93.15	-107.25	-8.811	-14.1	-43.47
D6	0.91	1924.66	80.48	-64.21	577.398	16.27	1924.66
V1	0.3	-2216.46	-245.03	104.9	-664.938	-140.13	-2216.46
V2	0.91	-456.03	-16.98	17.01	-136.809	0.03	-456.03
V3	1.72	218.38	51.93	-39.63	65.514	12.3	218.38
V4	2.54	2215.65	187.26	-147.26	664.695	40	2215.65
V5	1.72	218.38	-65.49	75.26	65.514	9.77	218.38
V6	0.91	-456.03	-26.62	21.24	-136.809	-5.38	-456.03
V7	0.3	-2216.46	-26.04	-11.84	-664.938	-37.88	-2216.46

Syarat :

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

→ Beban rencana = beban tetap.

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

→ Beban rencana = beban tetap + beban angin

4.1.3. Perencanaan Dimensi Batang

Perencanaan dimensi batang didasarkan pada hasil dari analisa SAP 2000 yang didapatkan beban rencana pada masing-masing batang menurut perilaku batang yang mengalami tarik atau desak.

A. Perencanaan Batang Tarik

Untuk perencanaan batang tarik mengacu pada landasan teori pada rumus 3.2.15 s/d 3.2.22.

➢ Batang Atas

- Gaya tarik maksimal = 216,37 kg
- Panjang batang maksimal = 0,424 m = 42,4 cm

  Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

- Syarat batang tarik :

$$\frac{L}{r} \leq 240 \text{ s/d } 300 \quad \Rightarrow \quad r\,\text{min} = \frac{l}{240} = \frac{42,4}{240} = 0,177 \text{ cm}$$

- Luas tampang perlu :

$$Ag_1 = \frac{T}{0,6.Fy} = \frac{216,37}{0,6.2400} = 0,1503\,\text{cm}^2$$

$$Ag_2 = \frac{T}{0,5.Fu.0,85} + \left(\frac{1}{8}'' + \phi baut\right).tp.n = \frac{216,37}{0,5.3600.0,85} + \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2$$

$$= 3,166 \text{ cm}^2$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1.51 cm ≥ r min = 0,177 cm → dipakai r = 1,51 cm

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

$$\text{Alubang} = \left(\frac{1}{8}'' + \phi baut\right).tp.n = \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2 = 3{,}0242\ cm^2$$

Anetto = Abruto – Alubang = 9,60 cm^2 - 3,0242 cm^2

= 6,5758 cm^2 > 0,1503 cm^2

Aeffektif = 0,85.Anetto = 0,85 x 6,5758 = 5,59 cm^2

Kontrol tegangan :

- $\frac{T}{Aprofil} \le 0{,}6.Fy \Rightarrow \frac{216{,}37}{9{,}60} \le 0{,}6.2400$

  22,539 Kg/cm^2 ≤ 1440 Kg/cm^2 .....Ok

- $\frac{T}{Aeffektif} \le 0{,}5.Fu \Rightarrow \frac{216{,}37}{5{,}59} \le 0{,}5.3600$

  38,707 kg/cm^2 ≤ 1800 kg/cm^2 .....Ok

⇒ Profil yang digunakan 2L 50x50x5

➢ Batang Bawah

- Gaya tarik maksimal = 1907,78 kg
- Panjang batang maksimal = 1,274 m = 127,4 cm

  Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

- Syarat batang tarik :

$$\frac{L}{r} \le 240 \text{ s/d } 300 \quad \Rightarrow \quad r\ min = \frac{l}{240} = \frac{127{,}4}{240} = 0{,}531\ cm$$

• Luas tampang perlu :

$$Ag_1 = \frac{T}{0{,}6.Fy} = \frac{1907{,}78}{0{,}6.2400} = 1{,}325\ cm^2$$

$$Ag_2 = \frac{T}{0{,}5.Fu.0{,}85} + \left(\frac{1}{8}'' + \phi baut\right).tp.n = \frac{1907{,}78}{0{,}5.3600.0{,}85} + \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2$$

$= 4{,}271\ cm^2$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,531 cm → dipakai r = 1,51 cm

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

Alubang = $\left(\frac{1}{8}'' + \phi baut\right).tp.n = \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2 = 3{,}0242\ cm^2$

Anetto = Abruto – Alubang = 9,60 cm^2 - 3,0242 cm^2

$= 6{,}5758\ cm^2 > 1{,}325\ cm^2$

Aeffektif = 0,85.Anetto = 0,85 x 6,5758 = 5,59 cm^2

Kontrol tegangan :

° $\frac{T}{Aprofil} \le 0{,}6.Fy \Rightarrow \frac{1907{,}78}{9{,}60} \le 0{,}6.2400$

198,73 Kg/cm^2 ≤ 1440 Kg/cm^2 .....Ok

° $\frac{T}{Aeffektif} \le 0{,}5.Fu \Rightarrow \frac{1907{,}78}{5{,}59} \le 0{,}5.3600$

341,284 kg/cm^2 ≤ 1800 kg/cm^2 .....Ok

⇒ Profil yang digunakan 2L 50x50x5

➢ Batang Diagonal

• Gaya tarik maksimal = 1924,66 Kg

• Panjang batang maksimal = 0,91 m =91 cm

Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

• Syarat batang tarik :

$$\frac{L}{r} \leq 240 \text{ s/d } 300 \quad \Rightarrow \quad \text{r min} = \frac{l}{240} = \frac{91}{240} = 0{,}38 \text{ cm}$$

• Luas tampang perlu :

$$Ag_1 = \frac{T}{0{,}6.Fy} = \frac{1924{,}66}{0{,}6.2400} = 1{,}337 \text{ cm}^2$$

$$Ag_2 = \frac{T}{0{,}5.Fu.0{,}85} + \left(\frac{1}{8}'' + \phi baut\right) tp.n = \frac{1924{,}66}{0{,}5.3600.0{,}85} + \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2$$

$$= 4{,}282 \text{ cm}^2$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,38 cm → dipakai r = 1,51 cm

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

$$\text{Alubang} = \left(\frac{1}{8}'' + \phi baut\right) tp.n = \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2 = 3{,}0242 \text{ cm}^2$$

Anetto = Abruto – Alubang = 9,60 cm^2 – 3,0242 cm^2 = 6,5758 cm^2 ≥1,337

A effektif = 0,85.Anetto = 0,85. 6,5758 = 5,59 cm^2

Kontrol tegangan :

$\circ \quad \frac{T}{Aprofil} \leq 0{,}6.Fy \quad \Rightarrow \quad \frac{1924{,}66}{9{,}6} \leq 0{,}6.2400$

$200{,}49\ Kg/cm^2 \leq 1440\ Kg/cm^2$ ..... Ok

$\circ \quad \frac{T}{Aeffektif} \leq 0{,}5.Fu \Rightarrow \frac{1924{,}66}{5{,}59} \leq 0{,}5.3600$

$344{,}304\ kg/cm^2 \leq 1800\ kg/cm^2$ .....Ok

$\Rightarrow$ Profil yang digunakan 2L 50x50x5

➢ Batang Vertikal

- Gaya tarik maksimal = 2215,65 Kg
- Panjang batang maksimal = 2,54 m =254 cm

Fy – 2400 Kg/cm^2 Fu – 3600 Kg/cm^2

- Syarat batang tarik :

$\frac{L}{r} \leq 240$ s/d 300 $\Rightarrow$ r min = $\frac{l}{240} = \frac{254}{240} = 1{,}06$ cm

- Luas tampang perlu :

$$Ag_1 = \frac{T}{0{,}6.Fy} = \frac{2215{,}65}{0{,}6.2400} = 1{,}539\ cm^2$$

$$Ag_2 = \frac{T}{0{,}5.Fu.0{,}85} + \left(\frac{1}{8}'' + \phi baut\right).tp.n = \frac{2215{,}65}{0{,}5.3600.0{,}85} + \left(\frac{1}{8}'' + \frac{1}{2}''\right).1/8''.2$$

$$= 4{,}472\ cm^2$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 1,06 cm → dipakai r = 1,51 cm

Abruto – 2 x 4,80 cm^2 = 9,60 cm^2

$$\text{Alubang} = \left(\frac{1}{8}'' + \phi baut\right).tp.n = \left(\frac{1}{8}'' + \frac{1}{2}''\right).3/8''.2 = 3{,}0242 \text{ cm}^2$$

Anetto = Abruto –Alubang = 9,60 cm^2 – 3,0242 cm^2 = 6,576cm^2 > 1,539 cm^2

Aeffektif = 0,85.Anetto = 0,85. 6,576 = 5,59 cm^2

Kontrol tegangan :

$$\circ\ \frac{T}{Aprofil} \le 0{,}6.Fy \Rightarrow \frac{2215{,}65}{9{,}6} \le 0{,}6.2400$$

230,797 Kg/cm^2 ≤ 1440 Kg/cm^2 .....Ok

$$\circ\ \frac{T}{Aeffektif} \le 0{,}5.Fu \Rightarrow \frac{2215{,}65}{5{,}59} \le 0{,}5.3600$$

396,36 kg/cm^2 ≤ 1800 kg/cm^2 ..... Ok

⇒ Profil yang digunakan 2L 50x50x5

B. Perencanaan Batang Tekan

Untuk perencanaan batang tekan mengacu pada landasan teori pada rumus 3.2.23 s/d 3.2.29.

- Batang Atas
  - Gaya tekan maksimal = -2561,67 Kg

• Panjang batang maksimal = 1,287 m =128,7 cm

Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

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

• Syarat batang tarik :

$$\frac{KL}{r} \le 200 \Rightarrow r\,min = \frac{KL}{200} = \frac{1.128,7}{200} = 0,644 \text{ cm}$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,644 cm → dipakai r = 1,51 cm

W = 3,77 Kg/m

Ix = Iy =11,0 cm^4 rx = ry = 1,51 cm e = 1,40

x = e + ½.tp = 1,40 + ½.1 = 1,90 cm

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

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

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

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

→ dipakai r = rx.gab = 1,51 cm

Syarat :

$$\frac{KL}{r} \le Cc = \sqrt{\frac{2\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}}$$

$$\frac{1.128,7}{1,51} \leq \frac{6400}{\sqrt{2400}}$$

$85,232 \leq 130,64$

sehingga digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3}{8}\left[\frac{85,232}{130,64}\right] - \frac{1}{8}\left[\frac{85,232}{130,64}\right]^3 = 1,88$$

$$Fa = \frac{2400}{1,88}\left[1 - 0.5\left(\frac{85,232}{130,64}\right)^2\right] = 1004,7 \text{ kg/cm}^2$$

Kontrol kapasitas :

P = Fa . Atotal > P terjadi

= 1004,7 . 9,60 > 2561,67 kg

= 9645,12 kg > 2561,67 kg.....Ok

⇒ Profil yang digunakan 2L 50x50x5

➢ Batang Bawah

- Gaya tekan maksimal = -201,15 Kg
- Panjang batang maksimal = 0,3 m = 30 cm

Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

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

- Syarat batang tarik :

$\frac{KL}{r} \leq 200$ sehingga : r min = $\frac{KL}{200} = \frac{1:30}{200} = 0,15$ cm

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,15 cm → dipakai r = 1,51 cm

W = 3,77 Kg/m

Ix = Iy =11,0 cm^4 rx = ry = 1,51 cm e = 1,40

x = e + ½.tp = 1,40 + ½.1 = 1,90 cm

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

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

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

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

→ dipakai r = rx.gab = 1,51 cm

Syarat :

$$\frac{KL}{r} \le Cc = \sqrt{\frac{2\pi^2.E}{Fy}} - \frac{6400}{\sqrt{Fy}}$$

$$\frac{1.30}{1,51} \le \frac{6400}{\sqrt{2400}}$$

19,868 ≤ 130,64

sehingga digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3}{8}.\frac{KL/r}{Cc} - \frac{1}{8}.\frac{(KL/r)^3}{Cc^3} = \frac{5}{3} + \frac{3}{8}.\frac{19,868}{130,64} - \frac{1}{8}.\frac{(19,868)^3}{(130,64)^3} = 1,7233$$

$$Fa = \frac{Fy}{Fs}\left(1 - \frac{(KL/r)^2}{2.Cc^2}\right) = \frac{2400}{1,7233}\left(1 - \frac{(19,868)^2}{2.(130,64)^2}\right) = 1375,965\ kg/cm^2$$

Kontrol kapasitas :

P = Fa . Atotal > P terjadi

= 1375,965 . 9,60 > 201,15 kg

= 13209,264 kg > 201,15 kg.....Ok

⇒ Profil yang digunakan 2L 50x50x5

➢ Batang Diagonal

• Gaya tekan maksimal = - 554,95 Kg

• Panjang batang maksimal = 1,797 m =179,7 cm

Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

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

• Syarat batang tarik :

$$\frac{KL}{r} \leq 200 \Rightarrow \quad r\ min = \frac{KL}{200} = \frac{1.179,7}{200} = 0,8985\ cm$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,8985 cm → dipakai r = 1,51 cm

W = 3,77 Kg/m

Ix = Iy =11,0 cm^4 rx = ry = 1,51 cm e = 1,40

x = e + ½.tp = 1,40 + ½.1 = 1,90 cm

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

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

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

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

→ dipakai r = rx.gab = 1,51 cm

Syarat :

$$\frac{KL}{r} \le Cc = \sqrt{\frac{2\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}}$$

$$\frac{1.179,7}{1,51} \le \frac{6400}{\sqrt{2400}}$$

$119,01 \le 130,64$

sehingga digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3}{8}.\frac{KL/r}{Cc} - \frac{1}{8}.\frac{(KL/r)^3}{Cc^3} = \frac{5}{3} + \frac{3}{8}.\frac{119,01}{130,64} - \frac{1}{8}.\frac{(119,01)^3}{(130,64)^3} = 1,9142$$

$$Fa = \frac{Fy}{Fs}\left(1 - \frac{(KL/r)^2}{2.Cc^2}\right) = \frac{2400}{1,9142}\left(1 - \frac{(119,01)^2}{2.(130,64)^2}\right) = 1107,1\ kg/cm^2$$

Kontrol kapasitas :

P = Fa . Atotal > P terjadi

= 1107,1 . 9,6 > 554,95 kg

= 10628 kg > 554,95 kg.....Ok

⇒ Profil yang digunakan 2L 50x50x5

➢ Batang Vertikal

• Gaya tekan maksimal = -2216,46 Kg

• Panjang batang maksimal = 0,3 m =30 cm

Fy = 2400 Kg/cm^2 Fu = 3600 Kg/cm^2

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

• Syarat batang tarik :

$$\frac{KL}{r} \le 200 \Rightarrow \quad r\,min = \frac{KL}{200} = \frac{1.30}{200} = 0{,}15 \text{ cm}$$

⇒ Dicoba Profil 2L 50x50x5

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

r = 1,51 cm ≥ r min = 0,15 cm → dipakai r = 1,51 cm

W = 3,77 Kg/m

Ix = Iy =11,0 cm^4 rx = ry = 1,51 cm e = 1,40

x = e + ½.tp = 1,40 + ½.1 = 1,90 cm

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

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

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

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

→ dipakai r = rx.gab = 1,51 cm

Syarat :

$$\frac{KL}{r} \le Cc = \sqrt{\frac{2\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}}$$

$$\frac{1.30}{1,51} \le \frac{6400}{\sqrt{2400}}$$

$19,868 \le 130,64$

sehingga digunakan rumus :

$$Fs = \frac{5}{3} + \frac{3}{8}.\frac{KL/r}{Cc} - \frac{1}{8}.\frac{(KL/r)^3}{Cc^3} = \frac{5}{3} + \frac{3}{8}.\frac{19,868}{130,64} - \frac{1}{8}.\frac{(19,868)^3}{(130,64)^3} = 1,7233$$

$$Fa = \frac{Fy}{Fs}\left(1 - \frac{(KL/r)^2}{2.Cc^2}\right) = \frac{2400}{1,7233}\left(1 - \frac{(19,868)^2}{2.(130,64)^2}\right) = 1375,965 \text{ kg/cm}^2$$

Kontrol kapasitas :

P = Fa . Atotal > P terjadi

= 1375,965 . 9,6 > 2216,46 kg

= 13209,264 kg > 2216,46 kg .....Ok

⇒ Profil yang digunakan 2L 50x50x5

Tabel 4.2 Profil terpakai dan berat profil terpakai

Batang	Profil (mm)	Berat Profil (kg/m)	Panjang (m)	Berat (kg)
Batang Atas	2L 50x50x5	2 x 3,77 = 7,54	10,324	77,843
Batang Bawah	2L 50x50x5	2 x 3,77 = 7,54	7,662	57,7715
Batang Diagonal	2L 50x50x5	2 x 3,77 = 7,54	8,0402	60,6231
Batang Vertikal	2L 50x50x5	2 x 3,77 = 7,54	8,4	63,336
				259,574

Kontrol berat kuda-kuda :

- Berat total kuda-kuda = 259,574 kg
- Berat baut, plat sambung Ø baut =(20%.berat total kuda-kuda) = 51,9148 kg

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

  = 259,574 kg + 51,9148 kg = 311,4888 kg
- Panjang rangka kuda-kuda = L = 7,3 m

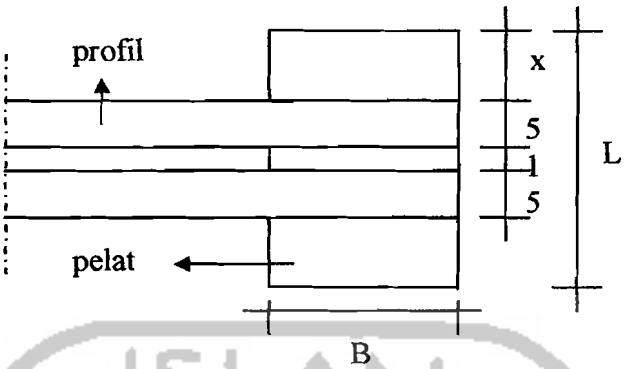
  $\frac{\Sigma}{L}$ < Berat taksiran kuda-kuda

  $\frac{311,4888}{7.3} < 80,25$ kg/m

  42,67 kg/m < 80,25 kg/m.....Ok

4.1.4 Perencanaan Pelat Kuda-kuda

Untuk perencanaan pelat kuda-kuda digunakan rumus-rumus pada landasan teori 3.2.34 s/d 3.2.36.



Gambar 4.5 Pelat Kuda-Kuda 1

P = 2253,8109 kg ; f'c = 25 Mpa = 250 kg/cm^2

$$\text{A perlu} = \frac{P}{0{,}33.f'c} = \frac{2253{,}8109}{0{,}33.250} = 27{,}319 \text{ cm}^2$$

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

$$q = \frac{P}{BxL} = \frac{2253{,}8109}{15x20} = 7{,}513 \text{ kg/cm}$$

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

M = ½ . q . x^2 = ½ . 7,513.4,5^2 = 76,07 kg.cm

Syarat :

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

$$tp = \sqrt{\frac{10M}{Fy}} = \sqrt{\frac{10.76{,}07}{2400}} = 0{,}563 \text{ cm} \approx 1 \text{ cm}$$

Sehingga dipakai pelat dengan tebal 1 cm

Pelat kuda-kuda berukuran : 15 x 20 x 1

4.1.5 Perencanaan dukungan arah lateral

Untuk perencanaan dukungan arah lateral dapat digunakan rumus pada landasan teori dari 3.2.37 dan 3.2.38.

Diketahui :

Lb = jarak antar gording = 1,7 m

Lc = jarak antar kuda-kuda = 4,5 m

$L = \sqrt{Lb^2 + Lc^2} = \sqrt{(1,7)^2 + (4,5)^2} = 4,81\,m$

Syarat : L/r ≤ 300 schingga :

$r\ min \geq \frac{L}{300} = \frac{4,81m}{300} = \frac{481cm}{300} = 1.603\ cm$

Keterangan :

1. L < 3 m → dipakai baja tulangan ∅12 mm
2. L > 5 m → dipakai baja tulangan ∅19 mm
3. 3 m ≤ L = 4,81 m ≤ 5 m → dipakai baja tulangan ∅16 mm

⇒ Sehingga dipakai baja tulangan ∅16 mm > r min = 1,385 cm .....Ok.

4.1.6 Perencanaan Sambungan

Perhitungan sambungan dilakukan pada setengah bentang pada tiap joint, diambil tebal pelat sambungan = 1 cm.

Mutu pelat → Fy = 2800 kg/cm^2 → Fu = 4200 kg/cm^2

Mutu baut non-full drat dari AISC $A_{325\,X}$

→ Fu = 8250 kg/cm^2 → Fy = 2050 kg/cm^2

Rumus pada landasan teori pada 3.2.30 s/d 3.2.32

Tegangan tumpu yang terjadi pada pelat :

Fa tumpu = 1,2 x Fu pelat = 1,2 x 4200 = 5040 kg/cm^2

Diameter baut dipakai 1/2" = 1,27 cm

→ P tumpu = tp x ∅ baut x Fa tumpu x n = 1 cm x 1, 27 cm x 5040 kg/cm^2 x n

= 6400,8.n kg

→ P geser = ¼ π D^2. 0,33 . Fu baut . 2.n = ¼ x 3,14 x (1,27)^2 x 0,33 . 8250 x 2 x n

= 6897,556 n kg.

maka : $n = \frac{P}{6400,8}$ buah.

Jarak penggunaan baut 1/2"

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

  diambil = 1,5 D = 1,5 x 1,27= 1,905 ~ 2 cm

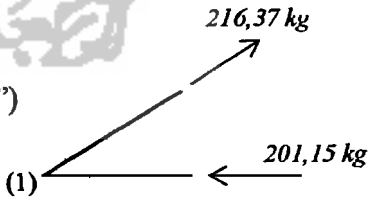
- Jarak antar baut (2D s/d 7D)

  diambil = 3D = 3 x 1,27 = 3,81 ~ 4 cm

1. Joint (1) = Joint (16)

   a. Batang (A1) = Batang (A8) = 216,37 kg

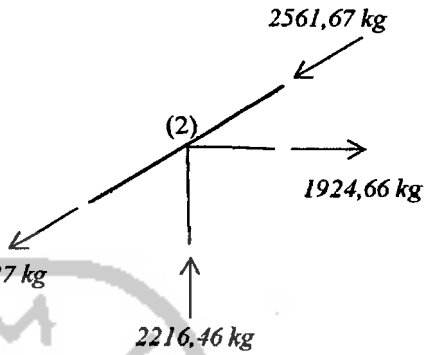
   $n = \frac{216,37}{6400,8} = 0,034$ ~ dipakai 2 buah baut (∅1/2")



   b. Batang (B1) = Batang (B8) = -201,15 kg

   $n = \frac{201,15}{6400,8} = 0,031$ ~ dipakai 2 buah baut (∅1/2")

2. Joint (2) = Joint (14)



a. Batang (A1) = Batang (A8) = 216,37 kg

~ dipakai 2 buah baut (∅1/2")

b. Batang (A2) = Batang (A7) = -2561,67 kg

$$n = \frac{2561,67}{6400,8} = 0,4$$

~ dipakai 2 buah baut (∅1/2")

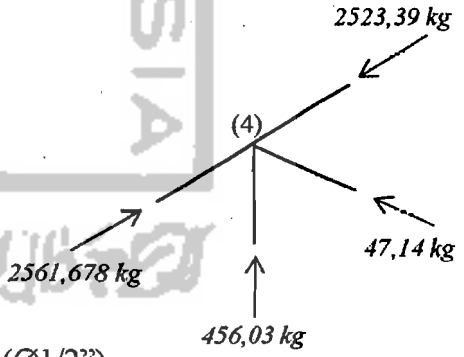
c. Batang (V1) = Batang (V7) = -2216,46 kg

$$n = \frac{2216,46}{6400,8} = 0,35$$ ~ dipakai 2 buah baut (∅1/2")

d. Batang (D1) = Batang (D6) = 1924,66 kg

$$n = \frac{1924,66}{6400,8} = 0,301$$ ~ dipakai 2 buah baut (∅1/2")

3. Joint (4) = Joint (12)



a. Batang (A2) = Batang (A7) = -2561,67 kg

~ dipakai 2 buah baut (∅1/2")

b. Batang (A3) = Batang (A6) = -2523,39 kg

$$n = \frac{2523,39}{6400,8} = 0,394$$ ~ dipakai 2 buah baut (∅1/2")

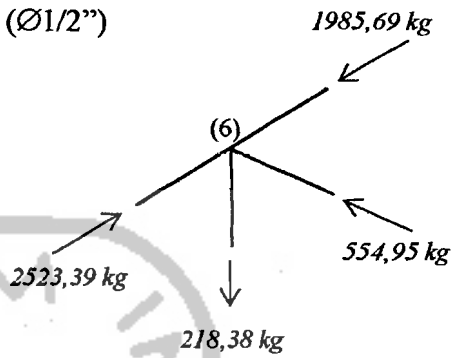
c. Batang (V2) = Batang (V6) = -456,03 kg

$$n = \frac{456,03}{6400,8} = 0,071$$ ~ dipakai 2 buah baut (∅1/2")

d. Batang (D2) = Batang (D5) = -47,14 kg

$$n = \frac{47,14}{6400,8} = 0,0074 \sim \text{dipakai 2 buah baut } (\varnothing 1/2")$$

4. Joint (6) = Joint (10)



a. Batang (A3) = Batang (A6) = -2523,39 kg

~ dipakai 2 buah baut (Ø1/2")

b. Batang (A4) = Batang (A5) = -1985,69 kg

$$n = \frac{1985,69}{6400,8} = 0,31 \sim \text{dipakai 2 buah baut } (\varnothing 1/2")$$

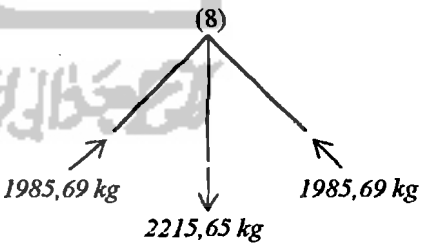
c. Batang (V3) = Batang (V5) = 218,38 kg

$$n = \frac{218,38}{6400,8} = 0,034 \sim \text{dipakai 2 buah baut } (\varnothing 1/2")$$

d. Batang (D3) = Batang (D4) = -554,95 kg

$$n = \frac{554,95}{6400,8} = 0,087 \sim \text{dipakai 2 buah baut } (\varnothing 1/2")$$

5. Joint (8)



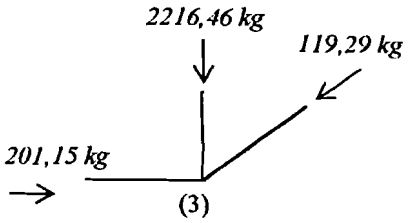
a. Batang (A4) = Batang (A5) = -1985,69 kg

~ dipakai 2 buah baut (Ø1/2")

b. Batang (V4) = 2215,65 kg

$$n = \frac{2215,65}{6400,8} = 0,3462 \sim \text{dipakai 2 buah baut } (\varnothing 1/2")$$

6. Joint (3) = Joint (15)



a. Batang (B1) = Batang (B8) = -201,15 kg

~ dipakai 2 buah baut (∅1/2")

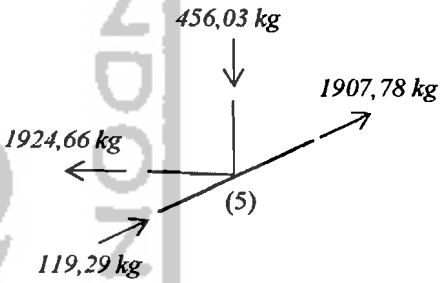
b. Batang (B2) = Batang (B7) = -119,29 kg

$$n = \frac{119,29}{6400,8} = 0,019$$ ~ dipakai 2 buah baut (∅1/2")

c. Batang (V1) = Batang (V7) = -2216,46 kg

~ dipakai 2 buah baut (∅1/2")

7. Joint (5) = Joint (13)



a. Batang (B2) = Batang (B7) = -119,29 kg

~ dipakai 2 buah baut (∅1/2")

b. Batang (B3) = Batang (B6) = 1907,78 kg

$$n = \frac{1907,78}{6400,8} = 0,2981$$ ~ dipakai 2 buah baut (∅1/2")

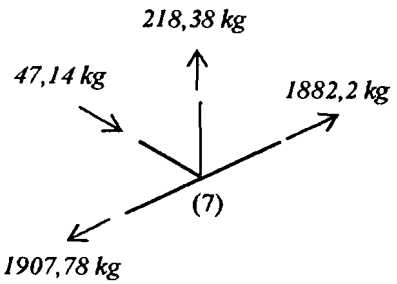
c. Batang (V2) = Batang (V6) = -456,03 kg

~ dipakai 2 buah baut (∅1/2")

d. Batang (D1) = Batang (D6) = 1924,66 kg

~ dipakai 2 buah baut (∅1/2")

8. Joint (7) = Joint (11)



a. Batang (B3) = Batang (B6) = 1907,78 kg

~ dipakai 2 buah baut (∅1/2")

b. Batang (B4) = Batang (B5) = 1882,2 kg

$$n = \frac{1882,2}{6400,8} = 0,2941 \sim \text{dipakai 2 buah baut (Ø1/2")}$$

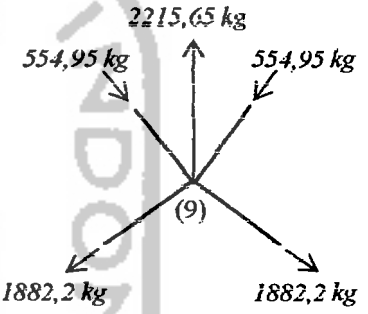
c. Batang (V3) = Batang (V5) = 218,38 kg

~ dipakai 2 buah baut (Ø1/2")

d. Batang (D2) = Batang (D5) = -47,14 kg

~ dipakai 2 buah baut (Ø1/2")

9. Joint (9)



a. Batang (B4) = Batang (B5) = 1882,2 kg

~ dipakai 2 buah baut (Ø1/2")

b. Batang (D3) = Batang (D4) = -554,95 kg

~ dipakai 2 buah baut (Ø1/2")

c. Batang (V4) = 2215,65 kg

~ dipakai 2 buah baut (Ø1/2")

4.2 Perencanaan Pelat Lantai dan Pelat Atap

Perencanaan pelat terdiri dari dua macam yaitu pelat satu arah dan pelat dua arah. Pelat satu arah di dasarkan pada perbandingan sisi pendek dan sisi panjang lebih dari 2 sehingga hamper semua beban dilimpahkan ke seisi pendek , sedangkan untuk pelat dua arah ly/lx kurang dari sama dengan 2. Pada perencanaan ini tumpuan pelat diasumsikan sebagai jepit elastis karena pelat merupakan kesatuan monolit dengan balok yang relative tidak terlalu kaku. Perhitungan pelat lantai dan pelat atap mengacu pada landasan teori rumus 3.3.4 s/d 3.3.20

4.2.1 Perencanaan Pelat Lantai PL 3

A. Pembebanan Pelat Lantai

-Fungsi bangunan : ruang kuliah (qL) = 2,5 KN/m^2

- Spesifikasi bahan : Mutu beton (f'c) = 22,5 Mpa

Mutu baja (fy) = 240 Mpa

- Perhitungan beban :

Tebal pelat lantai (h) = 120 mm

- Beban mati (qD)

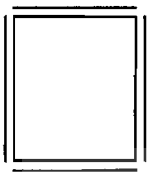
- Berat pelat beton = 0,12 . 24 = 2,88 KN/m^2
- Berat pasir (5cm) = 0,05 . 18 = 0,90 KN/m^2
- Berat spesi (2cm) = 2 . 0,21 = 0,42 KN/m^2
- Berat keramik = 0,01 .20 = 0,2 KN/m^2
- Eternit + penggantung = 0,18 KN/m^2 +

qD = 4,58 KN/m^2

- Beban hidup (qL) : qL = 2,5 KN/m^2

qU = 1,2.qD + 1,6.qL = 1,2.(4,58) + 1,6.(2,5) = 9,496 KN/m

- Menghitung distribusi momen



ly = 4,5m

lx = 3m

ly / lx = 1,5 pelat dua arah

Dari tabel 13.3.2 PBI 1971 (tumpuan tepi di anggap jepit elastis)

Didapat : clx = 56 ctx = 56

cly = 37 cty = 37

Mulx = 0,001 . qu . lx^2 . clx = 0,001.9,496.3^2.56 = 4,786 KNm

Mutx = -0,001 . qu . lx^2 . ctx = -0,001.9,496.3^2.37 = -4,786 KNm

Muly = 0,001 . qu . lx^2 . cly = 0,001.9,496.3^2.56 = 3,162 KNm

Muty = -0,001 . qu . lx^2 . cty = -0,001.9,496.3^2.37 = -3,162 KNm

B. Perhitungan Tulangan Pelat Lantai

❖ Perencanaan tulangan lx = tx

h = 120 mm

d = h - pb – ½.Øtul.pokok = 120-20-½.10 = 95 mm (digunakan tul. Ø10 mm)

Mu = 4,786 KNm

Mu/Ø = 5,9825 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{5{,}9825.10^6}{1000.95^2} = 0{,}663\ \text{Mpa}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}55$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2m.Rn}{fy}}\right] = \frac{1}{12{,}55}\left[1 - \sqrt{1 - \frac{2.12{,}55.0{,}663}{240}}\right] = 0{,}00281$$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,00281 = 0,00374 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,00374

As perlu = ρ terpakai . b . d = 0,00374 . 1000 . 95 = 355,245 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

Jarak tulangan : $S \leq \frac{A1\phi.}{As.perlu} = \frac{78{,}5.1000}{355{,}245} = 220{,}974$ mm

Dipakai jarak (S) = 200 mm

As ada = $\frac{A1\phi.1000}{S.terpakai} = \frac{78{,}5.1000}{200} = 392{,}5$ mm^2 > As perlu = 355,245 mm^2

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{392{,}5.240}{0{,}85.22{,}5.1000} = 4{,}9255 \text{ mm}$$

Mn = As.fy.(d – a/2) = 392,5.240.(95 – 4,9255/2)

=8,717 KNm ≥ Mu/∅ = 5,9825 KNm .....Ok

⇒ Dipakai tulangan P_{10-200}

❖ Perencanaan tulangan ly

h = 120 mm

d = h - pb – 10-1/2.10 = 120-20-10-5 = 85 mm (digunakan tulangan ∅10 mm)

Mu = 3,162 KNm

Mu/∅ = 3,9525 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{3{,}9525.10^6}{1000.85^2} = 0{,}5471\,\text{Mpa}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}55$$

$$\rho\,\text{perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{12{,}55}\left[1-\sqrt{1-\frac{2.12{,}55.0{,}5471}{240}}\right] = 0{,}002303$$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,002303 = 0,00306 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,00306

As perlu = ρ terpakai . b . d = 0,00306 . 1000 . 85 = 260,3 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

$$\text{Jarak tulangan : } S \le \frac{A1\phi.}{As.perlu} = \frac{78{,}5.1000}{260{,}3} = 301{,}58\ \text{mm}$$

Dipakai jarak (S) = 200 mm

As ada = $\frac{A1\phi.1000}{S.terpakai} = \frac{78,5.1000}{200} = 392,5\ mm^2 >$ As perlu = 260,3 mm^2

Kontrol kapasitas momen (Mn)

a = $\frac{Asada.fy}{0,85.f'c.b} = \frac{392,5.240}{0,85.22,5.1000} = 4,9255\ mm$

Mn = As.fy.(d – a/2) = 392,5.240.(85 – 4,9255/2)

= 7,775 KNm ≥ Mu/∅ = 3,9525 KNm.....Ok

⇒ Dipakai tulangan pokok P_{10-200}

❖ Perencanaan tulangan ty

h = 120 mm

d = h - pb – ½.Øtul.pokok = 120-20-½.10 = 95 mm (digunakan tulangan ∅10 mm)

Mu = 3,162 KNm

Mu/∅ = 3,9525 KNm

Rn = $\frac{Mu/\phi.}{b.d^2} = \frac{3,9525.10^6}{1000.95^2} = 0,438\ Mpa$

m = $\frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,55$

ρ perlu = $\frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{12,55}\left[1-\sqrt{1-\frac{2.12,55.0,438}{240}}\right] = 0,00185$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,00185 = 0,00246 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,00246

As perlu = ρ terpakai . b . d = 0,00246 . 1000 . 95 = 233,27 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

Jarak tulangan : $S \leq \frac{A1\phi.}{As.perlu} = \frac{78,5.1000}{233,27} = 336,73$ mm

Dipakai jarak (S) = 200 mm

As ada = $\frac{A1\phi.1000}{S.terpakai} = \frac{78,5.1000}{200} = 392,5\ mm^2 >$ As perlu = 233,27 mm^2

Kontrol kapasitas momen (Mn) :

$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{392,5.240}{0,85.22,5.1000} = 4,9255$ mm

Mn = As.fy.(d – a/2) = 392,5.240.(95 – 4,9255/2)

= 8,717 KNm ≥ Mu/∅ = 3,9525 KNm .....Ok

⇒ Dipakai tulangan P_{10-200}

❖ Perhitungan Tulangan Bagi Pelat Lantai

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

Dipakai tulangan pokok ∅8 mm dengan A1∅ = 50,24 mm^2

Jarak antar tulangan pokok : $S \leq \frac{A1\phi.}{As._{bagi}} = \frac{50,24.1000}{240} = 209,333$ mm

Tulangan bagi dipakai P_{8-200}

4.2.2 Perencanaan Pelat Atap PA 4

A. Pembebanan Pelat Atap

-Spesifikasi bahan : Mutu beton (f'c) = 22,5 Mpa

Mutu baja (fy) = 240 Mpa

Tebal pelat dak (h) = 100 mm

- Beban mati (qD)

  - Berat pelat beton = 0,1 . 24 = 2,4 KN/m^2

  - Lapisan kedap air = 0,02 . 24 = 0,48 KN/m^2

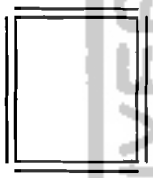
  - Eternit + penggantung = 0,18 KN/m^2 +

  qD = 3,06 KN/m^2

- Beban hidup (qL) : qL = 150 kg/m^2 = 1,5 KN/m^2

qu = 1,2.qD + 1,6.qL = 1,2.(3,06) + 1,6.(1,5) = 6,072 KN/m

- Menghitung distribusi momen



ly = 4,5m

lx = 2,5m

ly / lx = 1,8

Dari tabel 13.3.2 PBI 1971 (tumpuan tepi di anggap jepit elastis)

Didapat : clx = 60 ctx = 60

cly = 35 cty = 35

Mulx = 0,001 . qu . lx^2 . clx = 0,001.6,072.2,5^2.60 = 2,277 KNm

Mutx = -0,001 . qu . lx^2 . ctx = -0,001.6,072.2,5^2.60 = -2,277KNm

Muly = 0,001 . qu . lx^2 . cly = 0,001.6,072.2,5^2.35 = 1,3283 KNm

Muty = -0,001 . qu . lx^2 . cty = -0,001.6,072.2,5^2.35 = -1,3283KNm

B. Perhitungan Tulangan Pelat Atap

❖ Perencanaan tulangan lx = tx

h = 100 mm

Tinggi effektif (d)

d = h - pb – ½.∅ tulangan pokok = 100-20-½.10 = 75 mm

Mu = 2,277 KNm

Mu/∅ = 2,8463 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{2{,}8463.10^6}{1000.75^2} = 0{,}506 \text{ Mpa}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}55$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2m.Rn}{fy}}\right] = \frac{1}{12{,}55}\left[1 - \sqrt{1 - \frac{2.12{,}55.0{,}506}{240}}\right] = 0{,}00214$$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,00214 = 0,00284 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,00284

As perlu = ρ terpakai . b . d = 0,00284 . 1000 . 75 = 213,165 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1\phi.}{As.perlu} = \frac{78{,}5.1000}{213{,}165} = 368{,}26 \text{ mm}$$

Dipakai jarak (S) = 200 mm

$$\text{As ada} = \frac{A1\phi.1000}{S.terpakai} = \frac{78,5.1000}{200} = 392,5\ mm^2 > \text{As perlu} = 213,165\ mm^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{392,5.240}{0,85.22,5.1000} = 4,9255\ mm$$

Mn = As.fy.(d – a/2) = 392,5.240.(75 – 4,9255/2)

= 6,833 KNm ≥ Mu/∅ = 2,8463 KNm.....Ok

⇒ Dipakai tulangan P_{10-200}

❖ Perencanaan tulangan ly

h = 100 mm

Tinggi effektif (d)

d = h – pb – 10-1/2.10 = 100 – 20 -10-5 = 65 mm

Mu = 1,3283 KNm

Mu/∅ = 1,6603 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{1,6603.10^6}{1000.65^2} = 0,393\ Mpa$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,55$$

$$\rho\ perlu = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2m.Rn}{fy}}\right] = \frac{1}{12,55}\left[1 - \sqrt{1 - \frac{2.12,55.0,393}{240}}\right] = 0,00166$$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,00166 = 0,002201 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,002201

As perlu = ρ terpakai . b . d = 0,002201 . 1000 . 65 = 143,04 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1\phi.b}{As.perlu} = \frac{78,5.1000}{143,04} = 548,81 \text{ mm}$$

Dipakai jarak (S) = 200 mm

$$\text{As ada} = \frac{A1\phi.1000}{S.terpakai} = \frac{78,5.1000}{200} = 392,5 \text{ mm}^2 > \text{ As perlu} = 143,04 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{392,5.240}{0,85.22,5.1000} = 4,9255 \text{ mm}$$

Mn = As.fy.(d – a/2) = 392,5.240.(65 – 4,9255/2)

= 5,891 KNm ≥ Mu/∅ = 1,6603 KNm.....Ok

⇒ Dipakai tulangan P_{10-200} mm

❖ Perencanaan tulangan ty

h = 100 mm

Tinggi effektif (d)

d = h - pb – ½.∅ tulangan pokok = 100-20-½.10 = 75 mm

Mu = 1,3283 KNm

Mu/∅ = 1,6603 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{1{,}6603.10^6}{1000.75^2} = 0{,}2952 \text{ Mpa}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.22{,}5} = 12{,}55$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2m.Rn}{fy}}\right] = \frac{1}{12{,}55}\left[1 - \sqrt{1 - \frac{2.12{,}55.0{,}2952}{240}}\right] = 0{,}00124$$

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

ρ maks = 0,75.ρb = 0,75 . 0,0484 = 0,0363

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

1,33.ρ perlu = 1,33 . 0,00124 = 0,00165 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,00165

As perlu = ρ terpakai . b . d = 0,00165 . 1000 . 75 = 123,654 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1\phi.}{As.perlu} = \frac{78{,}5.1000}{123{,}654} = 634{,}834 \text{ mm}$$

Dipakai jarak (S) = 200 mm

$$\text{As ada} = \frac{A1\phi.1000}{S.terpakai} = \frac{78{,}5.1000}{200} = 392{,}5 \text{ mm}^2 > \text{ As perlu} = 123{,}654 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{392{,}5.240}{0{,}85.22{,}5.1000} = 4{,}9255 \text{ mm}$$

Mn = As.fy.(d – a/2) = 392,5.240.(75 – 4,9255/2)

= 6,833 KNm ≥ Mu/∅ = 1,6603 KNm.....Ok

⇒ Dipakai tulangan P_{10-200}

❖ Perhitungan Tulangan Bagi Pelat Atap

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

Dipakai tulangan pokok ∅8 mm dengan $A1\varnothing = 50{,}24\ mm^2$

Jarak antar tulangan pokok : $S \leq \frac{A1\phi.}{As._{bagi}} = \frac{50{,}24.1000}{200} - 251{,}2\ mm$

Tulangan bagi dipakai $P_{8\text{-}200}$

4.3 Perencanaan Tangga

4.3.1 Spesifikasi Struktur Tangga 1

Untuk spesifikasi struktur atau dimensi struktur tangga ditentukan berdasar gambar rencana ataupun dengan melihat ketentuan pada landasan teori untuk perencanaan tangga pada rumus 3.4.1 s/d 3.4.8.

1. Tinggi antar lantai = 4 m =400 cm
2. Jarak balok induk ke balok induk = 4,5 m = 450 cm
3. Panjang tangga keseluruhan = 6 m = 600 cm
4. Tinggi optrade diambil 18 cm
5. Panjang antrade diambil 30 cm

   Jumlah antrade = $\frac{400}{18} - 1 = 21{,}22 = 22$ buah
6. Sudut kemiringan tangga = 18/30 = arc tg $\alpha \rightarrow \alpha = 31^0$
7. Dimensi tangga
   - Panjang tangga = 0.5 x jumlah antrade x panjang antrade
     = 0,5 x 22 x 30 = 330 cm
   - Lebar bordes = 600 – 330 = 270 cm
   - di ambil tebal pelat bordes = 15 cm
   - tebal pelat sisi miring = 15 / cos31 = 17,321 cm
   - tebal eqifalen optrade = 0,5tinggi optrade = 9 cm

A. Pembebanan

1. Pembebanan bordes

Beban mati permeter lebar bordes

- Berat sendiri pelat = 0,15 x 24 x 1 = 3,6 KN/m'

- Berat spesi = 2 x 0,21 x 1 = 0,42 KN/m'
- Berat keramik = 1x 0,20 x 1 = 0,20 KN/m'
- Railing (ditaksir) = = 2,5 KN/m'+

Q_D = 6,72 KN/m'

Beban hidup

Q_L = 300 Kg/m^2 x 1 = 3 KN/m'

Qu = 1,2.Q_D + 1,6.Q_L = 1,2 . 6,72 + 1,6 . 3 = 12,864 KN/m'

2. Pembebanan Tangga

Beban mati

- Berat pelat = 0,17321 x 24 x 1 = 4,158 KN/m'
- Berat anak tangga = 0,09 x 17 x 1 = 1,530 KN/m'
- Spesi = 2 x 0,21 x 1 = 0,42 KN/m'
- Lantai keramik = 1 x 0,2 x 1 = 0,20 KN/m'
- Railing (ditaksir) = = 5 KN/m' +

Q_D = 11,348 KN/m'

Beban hidup

Q_L = 300 Kg/m^2 x 1 = 3 KN/m'

Qu = 1,2 Q_D + 1,6 Q_L = 1,2 . 11,348 + 1,6 . 3 = 18,418 KN/m'

B. Perencanaan Penulangan Tangga

f'c = 22,5 Mpa

fy = 400 Mpa

➢ Perencanaan tulangan tumpuan pelat tangga

h = 150 mm

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

Momen-momen pada tangga didapat dari analisis SAP 2000 untuk 2 Dimensi.

Mu = 22,9 KNm

Mu/Ø = 28,625 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{28{,}625.10^6}{1000.123{,}5^2} = 1{,}883\,Mpa$$

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

$$\rho\ perlu = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{20{,}915}\left[1-\sqrt{1-\frac{2.20{,}915.1{,}883}{400}}\right] = 0{,}005$$

$$\rho b = \frac{0{,}85.f'c.\beta}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}02438$$

ρ maks = 0,75.ρb = 0,75 . 0,02438 = 0,018288

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

ρ terpakai = ρ perlu = 0,005

As perlu = ρ terpakai . b . d = 0,005 . 1000 . 123,5 = 617,5 mm^2

Dipakai tulangan pokok Ø13 mm dengan A1Ø = 132,79 mm^2

$$\text{Jarak tulangan : } S \le \frac{A1\phi.}{As.perlu} = \frac{132{,}79.1000}{617{,}5} = 215{,}045\ mm$$

Dipakai jarak (S) = 200 mm

$$As\ ada = \frac{A1\phi.1000}{S.terpakai} = \frac{132{,}79.1000}{200} = 663{,}95\,mm^2 > \text{ As perlu} = 617{,}5\ mm^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0{,}85.f'c.b} = \frac{663{,}95.400}{0{,}85.22{,}5.1000} = 13{,}886\,mm$$

Mn = As.fy.(d – a/2) = 663,95.400.(123,5 – 13,886/2)

= 30,954 KNm ≥ Mu/∅ = 28,725 KNm .....Ok

⇒ Dipakai tulangan D_{13-200}

➢ Perencanaan tulangan lapangan pelat tangga

h = 150 mm

d = h - pb – ½.∅tul.pokok = 150-20-½.13 = 123,5 mm (digunakan tulangan ∅13 mm)

Mu = 11,08 KNm

Mu/∅ = 28,625 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{13{,}85.10^6}{1000.123{,}5^2} = 0{,}908\,Mpa$$

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

$$\rho\ perlu = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{20{,}915}\left[1-\sqrt{1-\frac{2.20{,}915.0{,}908}{400}}\right] = 0{,}0023$$

$$\rho b = \frac{0{,}85.f'c.\beta}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}02438$$

ρ maks = 0,75.ρb = 0,75 . 0,02438 = 0,018288

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

1,33 ρ perlu = 1,33 . 0,0023 = 0,003059 < ρ min

ρ terpakai = 1,33 . ρ perlu = 0,003059

As perlu = ρ terpakai . b . d = 0,003059 . 1000 . 123,5 = 377,787 mm^2

Dipakai tulangan pokok ∅13 mm dengan A1∅ = 132,79 mm^2

Jarak tulangan : $S \leq \frac{A1\phi.}{As.perlu} = \frac{132,79.1000}{377,787} = 351,4944$ mm

Dipakai jarak (S) = 200 mm

As ada = $\frac{A1\phi.1000}{S.terpakai} = \frac{132,79.1000}{200} = 663,95\,mm^2 >$ As perlu = 377,787 mm^2

Kontrol kapasitas momen (Mn) :

$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{663,95.400}{0,85.22,5.1000} = 13,886\,mm$

Mn = As.fy.(d – a/2) = 663,95.400.(123,79 – 13,886/2)

= 30,954 KNm ≥ Mu/∅ = 13,85 KNm .....Ok

⇒ Dipakai tulangan D_{13-200}

➢ Perhitungan Tulangan Bagi Pelat Tangga

As_{bagi} = 0,002 . b . h = 0,002 . 1000 . 150 = 300 mm^2

Dipakai tulangan pokok ∅8 mm dengan A1∅ = 50,24 mm^2

Jarak antar tulangan pokok : $S \leq \frac{A1\phi.}{As._{bagi}} = \frac{50,24.1000}{300} = 167,55$ mm

Tulangan bagi dipakai P_{8-150}

4.3.2 Spesifikasi Struktur Tangga 2

1. Tinggi tangga = 2 m =200 cm
2. Jarak balok induk ke balok sisi tangga = 1,25 m = 125 cm
3. Panjang tangga keseluruhan = 3 m = 300 cm

4. Tinggi optrade diambil 20 cm

5. Panjang antrade diambil 30 cm

   Jumlah antrade = $\frac{200}{20}$ = 10 buah

6. Sudut kemiringan tangga = 20/30 = arc tg $\alpha \rightarrow \alpha = 33{,}7^0$

8. Dimensi tangga

   - Panjang tangga = jumlah antrade x panjang antrade
     = 10 x 30 = 300 cm
   - tebal pelat sisi miring = 15 / cos31 = 17,321 cm
   - tebal eqifalen optrade = 0,5tinggi optrade = 9 cm

A. Pembebanan

1. Pembebanan Tangga

Beban mati per meter lebar tangga

- Berat pelat	= 0,17321 x 24 x 1	= 4,198 KN/m'
- Berat anak tangga	= 0,09 x 17 x 1	= 1,53 KN/m'
- Spesi	= 2 x 0,21 x 1	= 0,42 KN/m'
- Lantai keramik	= 1 x 0,2 x 1	= 0,20 KN/m'
- Railing (ditaksir)	=	= 2 KN/m' +
		Q_D = 8,348 KN/m'

Beban hidup

$Q_L = 300$ Kg/m^2 x 1 = 3 KN/m'

$Qu = 1{,}2\ Q_D + 1{,}6\ Q_L = 1{,}2 \cdot 8{,}348 + 1{,}6 \cdot 3 = 14{,}818$ KN/m'

B. Perencanaan Penulangan Tangga

f'c = 22,5 Mpa

fy = 250 Mpa

➢ Perencanaan tulangan tumpuan tangga

h = 150 mm

d = h - pb – ½.Øtul.pokok = 150-20-½.10 = 125 mm (digunakan tulangan ∅10 mm)

Momen-momen pada tangga didapat dari analisis SAP 2000 untuk 2 Dimensi.

Mu = 13,36 KNm

Mu/∅ = 16,7 KNm

$$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{16{,}7.10^6}{1000.125^2} = 1{,}069 \text{ Mpa}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{250}{0{,}85.22{,}5} = 13{,}072$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{13{,}072}\left[1-\sqrt{1-\frac{2.13{,}072.1{,}069}{250}}\right] = 0{,}0044$$

$$\rho b = \frac{0{,}85.f'c.\beta}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{250}\left(\frac{600}{600+250}\right) = 0{,}0459$$

ρ maks = 0,75.ρb = 0,75 . 0,0459 = 0,03442

$$\rho \text{ min} = \frac{1{,}4}{fy} = \frac{1{,}4}{250} = 0{,}0056$$

1,33 ρ perlu = 1,33 . 0,0044 = 0,00585 > ρ min

ρ terpakai = ρ min = 0,0056

As perlu = ρ terpakai . b . d = 0,0056 . 1000 . 125 = 700 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

Jarak tulangan : $S \leq \frac{Al\phi.}{As.perlu} = \frac{78,5.1000}{700} = 112,143$ mm

Dipakai jarak (S) = 100 mm

As ada = $\frac{Al\phi.1000}{S.terpakai} = \frac{78,5.1000}{100} = 785\,mm^2 >$ As perlu = 700 mm^2

Kontrol kapasitas momen (Mn) :

$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{785.250}{0,85.22,5.1000} = 10,3$ mm

Mn = As.fy.(d – a/2) = 785.250.(125 – 10,3/2)

= 23,521 KNm ≥ Mu/∅ = 16,7 KNm .....Ok

⇒ Dipakai tulangan P_{10-100}

➢ Perencanaan tulangan lapangan tangga

h = 150 mm

d = h - pb – ½.∅tul.pokok = 150-20-½.10 = 125 mm (digunakan tulangan ∅10 mm)

Mu = 6,68 KNm

Mu/∅ = 8,35 KNm

$Rn = \frac{Mu/\phi.}{b.d^2} = \frac{8,35.10^6}{1000.125^2} = 0,5344$ Mpa

$m = \frac{fy}{0,85.f'c} = \frac{250}{0,85.22,5} = 13,072$

$\rho\ perlu = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{13,072}\left[1-\sqrt{1-\frac{2.13,072.0,5344}{250}}\right] = 0,002142$

$\rho b = \frac{0,85.f'c.\beta}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{250}\left(\frac{600}{600+250}\right) = 0,0459$

ρ maks = 0,75.ρb = 0,75 . 0,0459 = 0,03442

$$\rho \text{ min} = \frac{1,4}{fy} = \frac{1,4}{250} = 0,0056$$

1,33 ρ perlu = 1,33 . 0,002142 = 0,00285 < ρ min

ρ terpakai = 1,33 ρ perlu = 0,00285

As perlu = ρ terpakai . b . d = 0,00285 . 1000 . 125 = 356,25 mm^2

Dipakai tulangan pokok ∅10 mm dengan A1∅ = 78,5 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1\phi.}{As.perlu} = \frac{78,5.1000}{356,25} = 214,921 \text{ mm}$$

Dipakai jarak (S) = 200 mm

$$\text{As ada} = \frac{A1\phi.1000}{S.terpakai} = \frac{78,5.1000}{200} = 392,5 \text{ mm}^2 > \text{ As perlu} = 356,25 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{Asada.fy}{0,85.f'c.b} = \frac{392,5.250}{0,85.22,5.1000} = 5,131 \text{ mm}$$

Mn = As.fy.(d – a/2) – 392,5.250.(125 – 5,131/2)

= 12,014 KNm ≥ Mu/∅ = 8,35 KNm .....Ok

⇒ Dipakai tulangan P_{10-200}

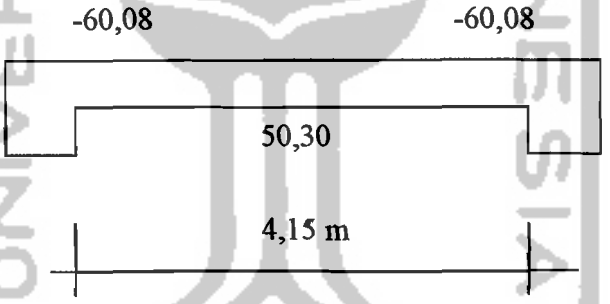
4.3.3 Perencanaan Balok Bordes

Balok bordes direncanakan bersamaan dengan analisis dari struktur portal 3 Dimensi dengan beban-beban yang ditumpunya sehingga hasil dari momen-momen juga berasal dari out put SAP 2000.

Pembebanan Balok Bordes :

- Beban Mati
  - Beban akibat reaksi tangga 1 = 7,0733 KN/m'
  - Beban akibat reaksi tangga 2 = 15,05 KN/m'
  - Beban pelat terusan pelat bordes = 4,58 KN/m'
  - Berat sendiri (0,25x 0,4 x 24) = 2,4 KN/m'
- Beban Hidup
  - Beban akibat reaksi tangga 1 = 3,7911 KN/m'
  - Beban akibat reaksi tangga 2 = 5,409 KN/m'
  - Beban pelat terusan pelat bordes = 2,5 KN/m'

Adapun momen-momen yang terjadi sebagai berikut :



Gambar 4.6 Nilai Momen Balok Bordes

➢ Penulangan balok

Ukuran balok 25/40

f'c = 22,5 Mpa

fy ulir = 400 Mpa

∅ tul pokok = 16 mm

∅ tulangan sengkang = 10 mm

untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0,85$

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

$$\rho_b = \frac{0,85.f'c}{fy}\beta\left(\frac{600}{600+fy}\right) = \frac{0,85.22.5}{400}0,85\left(\frac{600}{600+400}\right) = 0,0244$$

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

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

diambil $\rho_{pakai} = 0,5\ \rho_{maks} = 0,0092$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22.5} = 20,915$$

$$Rn = \rho.fy.(1-0,5.\ \rho.m) = 0,0092.400.(1-\frac{1}{2}.0,0092.20,915) = 3,326$$

➢ Penulangan untuk Mu tumpuan = 60,08 KNm

$$Mn = \frac{Mu_1}{\phi} = \frac{60,08}{0,8} = 75,1\ KNm$$

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{75,1.10^6}{3,326} = 22579675,29\ mm^2$$

diambil b = 250 mm

$250.d^2 = 22579675,29\ mm^2$

$d_{perlu} = 300,531$ mm

ambil h = 400 mm

d = h-pb-∅sengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 400 - 40 - 10 - \frac{1}{2}.16 = 342\ mm$$

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

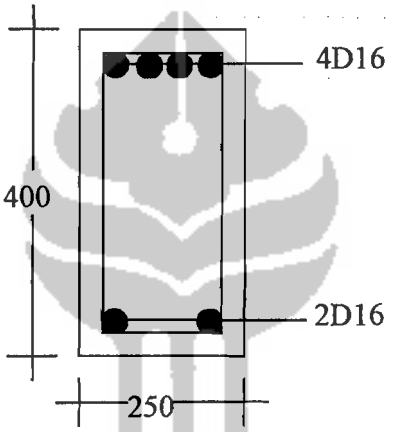
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{75{,}1.10^6}{250.342^2} = 2{,}57$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{2{,}57}{3{,}326}.0{,}0092 = 0{,}00711$$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00711.250.342 = 607{,}91\ mm^2$

$A\phi_{16} = 200{,}96\ mm^2$

dipakai $4\phi_{16}$ $As_{ada} = 803{,}84\ mm^2 > As_{perlu} = 607{,}91\ mm^2$



Gambar 4.7 Penampang Melintang Balok Bordes

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$

$$= \frac{250 - 2.40 - 2.10 - 4.16}{(4-1)} = 28{,}667\ mm > 25\ mm$$

Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f.c.b} = \frac{803{,}84.400}{0{,}85.22{,}5.250} = 67{,}25\ mm$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right) = 803{,}84.400\left(342 - \frac{67{,}25}{2}\right)$$

$$= 99{,}154 \text{ KNm} \geq \frac{Mu}{\phi} = 75{,}1 \text{ KNm}\text{.....}\textbf{OK!}$$

➢ Penulangan untuk Momen lapangan = 50,30 KNm

$$\frac{Mu}{\phi} = \frac{50{,}3}{0{,}8} = 62{,}875 \text{ KNm}$$

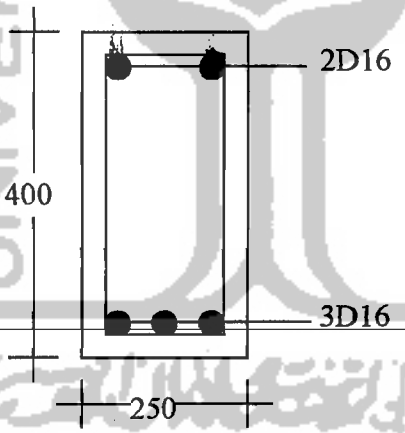
$$\text{Rn}_{\text{baru}} = \frac{Mu/\phi}{b.d^2} = \frac{62{,}875.10^6}{250.342^2} = 2{,}15$$

$$\rho_{\text{baru}} = \frac{Rn_{baru}}{Rn}\rho = \frac{2{,}15}{3{,}326}.0{,}0092 = 0{,}00595$$

$$\text{As}_{\text{perlu}} = \rho_{baru}.\text{b.d} = 0{,}00595.250.342 = 508{,}725 \text{ mm}^2$$

$$\text{A}\phi_{16} = 200{,}96 \text{ mm}^2$$

dipakai $3\phi_{16}$ $\text{As}_{\text{ada}} = 602{,}88 \text{ mm}^2 > \text{As}_{\text{perlu}} = 508{,}725 \text{ mm}^2$



Gambar 4.8 Penampang Melintang Balok Bordes

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi \text{ sengkang} - n.\phi \text{ tulangan}}{(n-1)}$

$$= \frac{250 - 2.40 - 2.10 - 3.16}{(3-1)} = 51 \text{ mm} > 25 \text{ mm}$$

Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{602{,}88.400}{0{,}85.22{,}5.250} = 50{,}44$$

$$Mn = As_{ada}.fy.(d - a/2) = 602{,}88.400\ (342 - \frac{50{,}44}{2})$$

$$= 76{,}392\ KNm \geq \frac{Mu}{\phi} = 62{,}875\ KNm \ldots .OK!$$

4.3.4 Perhitungan penulangan geser balok bordes

Prinsip dari perencanaan geser pada struktur terlentur didasarkan pada anggapan bahwa beton menahan sebagian gaya geser, sedangkan kelebihannya atau gaya geser diatas kemampuan beton menahan gaya geser ditahan oleh baja tulangan geser.

diketahui : b = 250 mm fy = 240 Mpa

h = 400 mm f'c = 22,5 Mpa d = 342 mm

Bentang bersih balok = 4,5 – 0,35 = 4,15 m

Perencanaan geser balok bordes ini dihitung berdasakan rumus 3.5.35 s/d 3.5.47 pada landasan teori bab III.

- Gaya geser pada tumpuan :

Vu = 110,99 KN

$$\frac{Vu}{\phi} = \frac{110{,}99}{0{,}6} = 184{,}98\ KN$$

- Gaya geser pada jarak 1,075 m dari tumpuan

Vu = 63,53 KN

$$\frac{Vu}{\phi} = 105{,}88\ KN$$

- Gaya geser pada penampang kritis sejauh d dari tumpuan :

$$Vu = \frac{(110,99.0,733)+(63,53.0,342)}{1,075} = 95,9\,KN$$

$$\frac{Vu}{\phi} = \frac{95,9}{0,6} = 159,833\ KN$$

- Gaya geser beton :

$$Vc = \frac{1}{6}\sqrt{f'c}\,.b.d = \frac{1}{6}\sqrt{22,5}\,.250.342 = 67,594\,KN$$

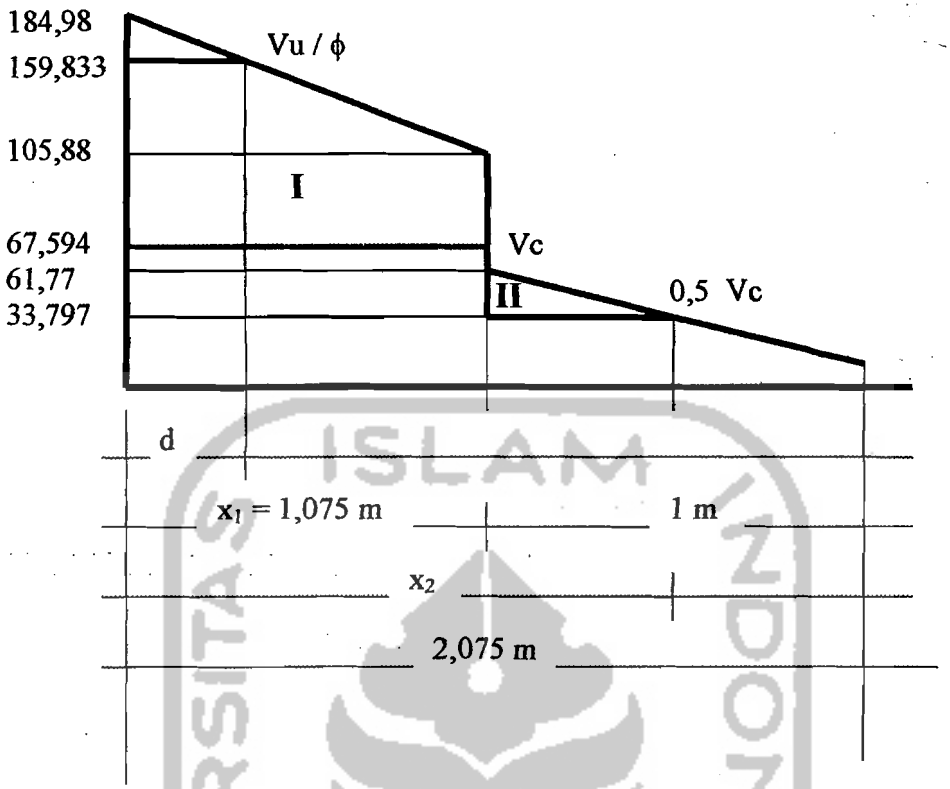
$$\frac{1}{2}Vc = 33,797\,KN$$

$$3Vc = 202,782\,KN$$

$$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3}.250.342 = 28,500KN$$

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

$(67,594 + 28,500) = 96,094\ KN < 159,833\ KN < 202,782\ KN$ maka ukuran balok dapat digunakan tetapi diperlukan tulangan geser.



Gambar 4.9 Diagram Geser Balok Bordes

- Titik dimana gaya geser = Vc = 67,594 KN

  $x_1 = 1{,}075$ m

- Titik dimana gaya geser = 0,5 Vc = 33,797 KN

  $x_2 = \frac{33{,}797}{61{,}77}.1 = 0{,}55$ m dari tengah bentang

  Daerah I

  digunakan sengkang Ø10mm

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

  $S_1 \le \frac{Av.fy.d}{Vs} = \frac{157.240.342}{(159{,}833-67{,}594).10^3} = 139{,}708$ mm

$S_2 \leq \frac{d}{2} = \frac{342}{2} = 171$ mm

$S_3 \leq 600$ mm

Dipakai P_{10-130}

Daerah II

yaitu daerah tulangan geser minimum

digunakan sengkang ∅10 mm, dengan Av = 157 mm^2

$S_1 \leq \frac{3.Av.fy}{bw} = \frac{3.157.240}{250} = 452{,}16$ mm

$S_2 \leq \frac{d}{2} = \frac{342}{2} = 171$ mm

$S_3 \leq 600$ mm

Dipakai P_{10-170}

Cek Pengaruh Torsi

Momen Torsi terfaktor Tu = 4,87 KNm

$$Tu_{maks} = \phi\left[\left(\frac{1}{24}\sqrt{f'c}\right)\sum x^2 y\right] = 0{,}6\left[\left(\frac{1}{20}\sqrt{22{,}5}\right)250^2 400\right] = 3{,}558 \text{ KNm}$$

Tu = 4,87 KNm > 3,558 KNm maka diperlukan tulangan torsi

Karena merupakan torsi keserasian, menurut SK-SNI boleh direncanakan terhadap momen torsi sebagai berikut :

$$Tu = \phi\left(\frac{1}{3}\sqrt{f'c}\right)\sum\frac{1}{3}x^2 y = 0{,}6\left(\frac{1}{3}\sqrt{22{,}5}\right)\sum\frac{1}{3}250^2 400 = 7{,}906 \text{ KNm} > \text{Tu} = 4{,}87 \text{ KNm}$$

dipakai Tu = 4,87 KNm

Sebagai tulangan geser dan tulangan torsi digunakan sengkang ϕ 10 mm

- Perencanaan sengkang torsi

d = 342 mm

$$Ct = \frac{b.d}{\sum x^2.y} = \frac{250.342}{25.10^6} = 0{,}00342 / \text{mm}$$

sumbangan beton dalam menahan torsi :

$$Tc = \frac{\left(\frac{1}{15}\sqrt{f'c}\right)\sum x^2 y}{\sqrt{1+\left(\frac{0{,}4.Vu}{Ct.Tu}\right)^2}} = \frac{\left(\frac{1}{15}\sqrt{22{,}5}\right)250^2 400}{\sqrt{1+\left(\frac{0{,}4.95{,}9.10^3}{0{,}00342.4{,}87.10^6}\right)^2}} = 3{,}148 \text{ KNm}$$

torsi yang ditahan tulangan torsi :

$$Ts = \frac{Tu}{\phi} - Tc = \frac{4{,}87}{0{,}6} - 3{,}148 = 4{,}969 \text{ KNm}$$

x_1 = 250 – 2 (pb + 0,5 ϕsengkang) = 160 mm

y_1 = 400 – 2 (pb + 0,5 ϕsengkang) = 310 mm

$$\alpha_t = \frac{1}{3}\left(2 + \frac{y_1}{x_1}\right) = \frac{1}{3}\left(2 + \frac{310}{160}\right) = 1{,}313 < 1{,}5$$

$$\frac{At}{s} = \frac{Ts}{\alpha_t.x_1.y_1.fy} = \frac{4{,}969.10^6}{1{,}313.160.310.400} = 0{,}191 \text{ mm}^2 / \text{mm jarak / kaki}$$

- Perencanaan sengkang geser

$$\frac{Vu}{\phi} = \frac{95{,}9}{0{,}6} = 159{,}833 \text{ KN}$$

sumbangan beton dalam menahan geser :

$$Vc = \frac{\frac{1}{6}\sqrt{f'c}.b.d}{\sqrt{1+\left(2{,}5.Ct.\frac{Tu}{Vu}\right)^2}} = \frac{\frac{1}{6}\sqrt{22{,}5}.250.342}{\sqrt{1+\left(2{,}5.0{,}00342\frac{4{,}87.10^6}{95{,}9.10^3}\right)^2}} = 61{,}989 \text{ KN}$$

geser yang ditahan tulangan geser :

$$Vs = \frac{Vu}{\phi} - Vc = 159{,}833 - 61{,}989 = 97{,}844 \text{ KN}$$

$$\frac{Av}{s} = \frac{Vs}{fy.d} = \frac{97{,}844.10^3}{400.342} = 0{,}715 \text{ mm}^2 \text{ / mm jarak / dua kaki}$$

- Perencanaan tulangan geser dan torsi (gabungan)

$$\frac{Avt}{s} = 2\frac{At}{s} + \frac{Av}{s} = 2.0{,}191 + 0{,}715 = 1{,}097 \text{ mm}^2$$

dipakai sengkang 10 mm, dengan luas dua kaki As = 157 mm^2

jarak sengkang : $s = \frac{As}{\frac{Avt}{s}} = \frac{157}{1{,}097} = 143{,}118$ mm

jarak sengkang maksimum $\frac{1}{4}(x_1 + y_1) = \frac{1}{4}(160 + 310) = 117{,}5$ mm < 143,118 mm

dipakai s = 100 mm

luas sengkang minimum perlu :

$$At + 2.Av = \frac{b.s}{3.fy} = \frac{250.100}{3.400} = 20{,}833 \text{ mm}^2 < 157 \text{ mm}^2$$

dipakai sengkang P10 – 100

4.4 Analisis Struktur Portal

Portal direncanakan dengan struktur 3 Dimensi, beban-beban yang bekerja disesuaikan dari gambar struktur Gedung D3 Fakultas Ekonomi. Adapun perincian dari beban-beban yang bekerja pada struktur sebagai berikut :

Perhitungan pembebanan pelat lantai untuk beban mati per m^2

- Pelat beton = 0,12 . 24 = 2,88 kN/m^2
- Keramik = 2 . 0,2 = 0,2 kN/m^2
- Spesi = 2 . 0,21 = 0,84 kN/m^2
- Pasir = 5 . 0,18 = 0,9 kN/m^2
- Plafond = 0,18 kN/m^2

4,58 kN/m^2

Perhitungan pembebanan pelat atap untuk beban mati per m^2

- Pelat beton = 0,1 . 24 = 2,4 kN/m^2
- Spesi = 0,02 . 24 = 0,48 kN/m^2
- Plafond = 0,18 kN/m^2

3,06 kN/m^2

Beban hidup yang digunakan

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

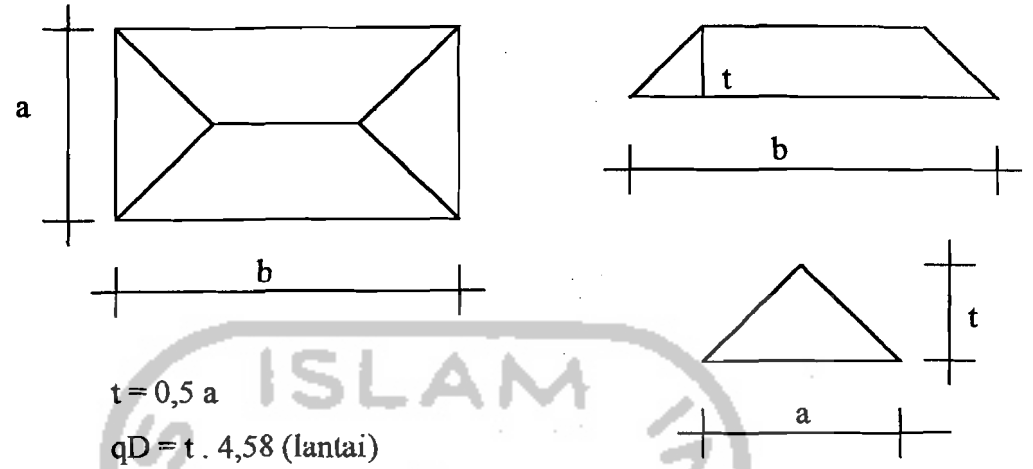
Beban hidup pelat selasar = 2,5 KN/m^2

Beban hidup pekerja atap = 1,5 KN/m^2

Pemodelan jenis beban pada SAP 2000 :

Beban yang terjadi dimodelkan sesuai dengan pendistribusian beban yang digunakan, seperti contoh berikut ini :

- Beban Pelat

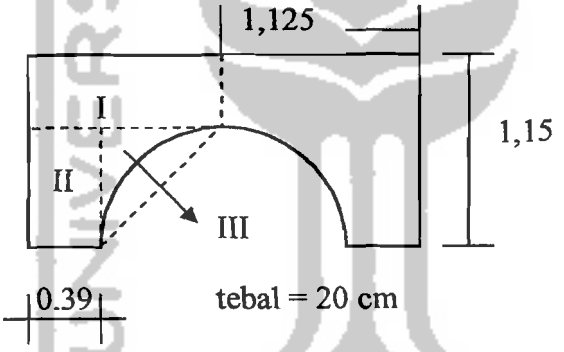


t = 0,5 a

qD = t . 4,58 (lantai)

qL = t. 2,5 (lantai)

- Beban Ornamen



Distribusi beban

berat I = 1,125 . 0,39 . 20 . 17 = 1,49 KN

berat II= 0,76 . 0,39 . 20 . 17 = 1,01 KN

berat III= 0,5 0,76 . 0,735 . 20 .17 = 0,95 KN

t1 = 1,49 / 1,125 = 1,326 KN/m'

t2 = 1,01 / 0,39 = 2,6 KN/m'

4.4.1 Perhitungan Beban Akibat Gravitasi

A. Portal As N

➢ Beban Mati

a. Beban merata lantai 1, 4

- Bentang 5-6 = 8-9

-Pelat lantai tipe 4	= 1,5 x 4,58	= 6,87 KN/m'
- Dinding	= 4 x 2,5	= 10 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 11-12

- Dinding	= 4 x 2,5	= 10 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6	= 1,5 x 4,58	= 6,87 KN/m'
- dinding	= 4 x 2,5	= 10 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

b. Beban merata lantai 2

- Bentang 5-6 = 8-9

-Pelat lantai tipe 4	= 1,5 x 4,58	= 6,87 KN/m'
- Pelat lantai tipe 11	= 1 x 4,58	= 4,58 KN/m'
- Dinding	= 4 x 2,5	= 10 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 11-12

- Pelat lantai tipe 11	= 1 x 4,58	= 4,58 KN/m'

- tangga 2 = reaksi tangga = 15,05 KN/m'
- Dinding = 2 x 2,5 = 5 KN/m'
- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11
  - Pelat lantai tipe 6 = 1,5 x 4,58 = 6,87 KN/m'
  - dinding = 4 x 2,5 = 10 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

c. Beban merata lantai 3

- Bentang 5-6 = 8-9
  - Pelat lantai tipe 4 = 1,5 x 4,58 = 4,58 KN/m'
  - Pelat lantai tipe 13 = 0,613 x 4,58 = 2,808 KN/m'
  - Dinding = 4 x 2,5 = 10 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'
- Bentang 11-12
  - Pelat lantai tipe 13 = 0,613 x 4,58 = 2,808 KN/m'
  - tangga 2 = reaksi tangga = 15,05 KN/m'
  - Dinding = 2 x 2,5 = 5 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'
- Bentang 6-7 = 7-8 = 9-10 = 10-11
  - Pelat lantai tipe 6 = 1,5 x 4,58 = 6,87 KN/m'
  - dinding = 4 x 2,5 = 10 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

d. Beban merata atap

- Bentang 5-6 = 11-12

- Pelat atap tipe d	= 0,5 x 3,06	= 1,53 KN/m'
- Pelat atap tipe b	= 0,5 x 3,06	= 1,53 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang 8-9

- Pelat atap tipe g	= 0,5 x 3,06	= 1,53 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat atap tipe f	= 0,5 x 3,06	= 1,53 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

e. Beban terpusat

- Pada as 7 dan 10 lantai 2, 3, 4 dan untuk atap as 5 s/d 12

- beban tritisan	= 5,64 KN

- Pada as 6, 8, 9, 11 lantai 2, 3, 4

- beban tritisan	= 0,5 x 5,64	= 2,82 KN

➢ Beban Hidup

a. Beban merata lantai 1, 4

- Bentang 5-6 = 8-9

- Pelat lantai tipe 4	= 1,5 x 2,5	= 3,75 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6	= 1,5 x 2,5	= 3,75 KN/m'

b. Beban merata lantai 2

- Bentang 5-6 = 8-9

-Pelat lantai tipe 4 = 1,5 x 2,5 = 3,75 KN/m'

- Pelat lantai tipe 11 = 1 x 2,5 = 2,5 KN/m'

- Bentang 11-12

- Pelat lantai tipe 11 = 1 x 2,5 = 2,5 KN/m'

- Tangga 2 = = 5,409 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6 = 1,5 x 2,5 = 3,75 KN/m'

c. Beban merata lantai 3

- Bentang 5-6 = 8-9

- Pelat lantai tipe 4 = 1,5 x 2,5 = 3,75 KN/m'

- Pelat lantai tipe 13 = 0,613 x 2,5 = 1,533 KN/m'

- Bentang 11-12

- Pelat lantai tipe 13 = 0,613 x 2,5 = 1,533 KN/m'

- Tangga 2 = = 5,409 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6 = 1,5 x 2,5 = 3,75 KN/m'

b. beban merata atap

- Bentang 5-6 = 11-12

- Beban pekerja (tipe d) = 0,5 x 1,5 = 0,75 KN/m'

- Beban pekerja (tipe b) = 0,5 x 1,5 = 0,75 KN/m'

- Bentang 8-9

- Beban pekerja (tipe g) = 0,5 x 1,5 = 0,75 KN/m'

- Bentang 6-7 = 7-8 = 9-10 =10-11

- Beban pekerja (tipe f) = 0,5 x 1,5 = 0,75 KN/m'

B. Portal As K[1]

➢ Beban Mati

a. Beban merata lantai 1, 2, 3, 4

- Bentang 5-6 = 8-9

- Pelat lantai tipe 4 = 1,5 x 4,58 = 6,87 KN/m'
- Pelat lantai tipe 3 = 1,25 x 4,58 = 5,725 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6 = 1,5 x 4,58 = 6,87 KN/m'
- Pelat lantai tipe 5 = 1,25 x 4,58 = 5,725 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

b. Beban merata lantai 1

- Bentang 11-12

- Pelat lantai tipe 3 = 1,25 x 4,58 = 5,725 KN/m'
- Pelat lantai tipe 4 = 1,5 x 4,58 = 6,87 KN/m'
- tangga 1 = reaksi tangga = 54,8599 KN/m'
- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

c. Beban merata lantai 2, 3

- Bentang 11-12

- Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Tangga 1	= reaksi tangga	= 54,8599 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

d. Beban merata lantai 4

- Bentang 11-12

- Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Tangga 1	=	= 54,8599 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

e. Beban merata atap

- Bentang 5-6 = 11-12

- Pelat atap tipe d	= 0,5 x 3,06	= 1,53 KN/m'
- Pelat atap tipe b	= 0,5 x 3,06	= 1,53 KN/m'
- Pelat atap tipe c	= 1,25 x 3,06	= 3,825 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang 8-9

- Pelat atap tipe g	= 0,5 x 3,06	= 1,53 KN/m'
- Pelat atap tipe c	= 1,25 x 3,06	= 3,825 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat atap tipe f	= 0,5 x 3,06	= 1,53 KN/m'
- Pelat atap tipe e	= 1,25 x 3,06	= 3,825 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang 5-6, 8-9

- Pelat lantai tipe 4 = 1,5 x 2,5 = 3,75 KN/m'

- Pelat lantai tipe 3 = 1,25 x 2,5 =3,125 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 6 = 1,5 x 2,5 = 3,75 KN/m'

- Pelat lantai tipe 5 = 1,25 x 2,5 = 3,125 KN/m'

b. beban merata lantai 1

- Bentang 11-12

- Pelat lantai tipe 3 = 1,25 x 2,5 = 3,125 KN/m'

- Pelat lantai tipe 4 = 1,5 x 2,5 = 3,75 KN/m'

- Tangga = = 15,8846 KN/m'

c. Beban merata lantai 2, 3

- Bentang 11-12

- Pelat lantai tipe 3 = 1,25 x 2,5 = 3,125 KN/m'

- Tangga = = 15,8846 KN/m'

d. Beban merata lantai 4

- Bentang 11-12

- Pelat lantai tipe 3 = 1,25 x 2,5 = 3,125 KN/m'

- Tangga = = 15,8846 KN/m'

e. beban merata atap

- Bentang 5-6, 11-12

  - Beban pekerja (tipe d) = 0,5 x 1,5 = 0,75 KN/m'
  - Beban pekerja (tipe b) = 0,5 x 1,5 = 0,75 KN/m'
  - Beban pekerja (tipe c) = 1,25 x 1,5 = 3,125 KN/m'

- Bentang 8-9

  - Beban pekerja (tipe g) = 0,5 x 1,5 = 0,75 KN/m'
  - Beban pekerja (tipe c) = 1,25 x 1,5 = 3,125 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

  - Beban pekerja (tipe f) = 0,5 x 1,5 = 0,75 KN/m'
  - Beban pekerja (tipe e) = 1,25 x 1,5 = 3,125 KN/m'

C. Portal As K

➢ Beban Mati

a. Beban merata lantai 1

- Bentang 8-9 = 11-12

  - Pelat lantai tipe 3 = 1,25 x 4,58 = 5,725 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

  - Pelat lantai tipe 5 = 1,25 x 4,58 = 5,725 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

- Bentang 5-6

  - Pelat lantai tipe 3 = 1,25 x 4,58 = 5,725 KN/m'
  - Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

b. Beban merata lantai 2

- Bentang 8-9

-Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 7-8 = 9-10 = 10-11

- Pelat lantai tipe 5	= 1,25 x 4,58	= 5,725 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 6-7

- Pelat lantai tipe 5	= 1,25 x 4,58	= 5,725 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 5-6

- Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Pelat lantai tipe 7	= 2,25 x 4,58	= 10,305 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 11-12

- Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

d. Beban merata lantai 3, 4

- Bentang 8-9 = 11-12

-Pelat lantai tipe 3	= 1,25 x 4,58	= 5,725 KN/m'
- Berat sendiri	= 0,3 x 0,45 x 24	= 3,24 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 5	= 1,25 x 4,58	= 5,725 KN/m'

- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

- Bentang 5-6

- Pelat lantai tipe 3 = 1,25 x 4,58 = 5,725 KN/m'

- Pelat lantai tipe 7 = 2,25 x 4,58 = 10,305 KN/m'

- Berat sendiri = 0,3 x 0,45 x 24 = 3,24 KN/m'

d. beban merata atap

- Bentang 5-6 = 8-9 = 11-12

- Pelat atap tipe c = 1,25 x 3,06 = 3,825 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat atap tipe e = 1,25 x 3,06 = 3,825 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban terpusat

- Untuk lantai 2 as 8 s/d 10 = lantai 3, 4 dan atap as 7 s/d 12

- Beban tritisan = 5,64 KN

- Untuk lantai 1, 2, 3, 4 pada balok selasar

- Beban railing = 2,115 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang 8-9 = 11-12

-Pelat lantai tipe 3 = 1,25 x 2,5 = 3,125 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Pelat lantai tipe 5 = 1,25 x 2,5 = 3,125 KN/m'

- Bentang 5-6

- Pelat lantai tipe 3 = 1,25 x 2,5 = 3,125 KN/m'

- Pelat lantai tipe 7 = 2,25 x 2,5 = 5,625 KN/m'

b. beban merata atap

- Bentang 5-6 = 8-9 = 11-12

- Beban pekerja (tipe c) = 1,25 x 1,5 = 1,875 KN/m'

- Bentang 6-7 = 7-8 = 9-10 = 10-11

- Beban pekerja (tipe e) = 1,25 x 1,5 = 1,875 KN/m'

D. Portal As 5

➢ Beban Mati

a. Beban merata lantai 1

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

- Pelat lantai tipe 2 = 1,5 x 4,58 = 6,87 KN/m'

- Dinding = 4 x 2,5 = 10 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang $K^1\text{-}K$

- Pelat lantai tipe 1 = 1,25 x 4,58 = 5,725 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K-J

- Pelat lantai tipe 7 = 2,25 x 4,58 = 10,305 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

- Pelat lantai tipe 2 = 1,5 x 4,58 = 6,87 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K
  - Pelat lantai tipe 1 = 1,5 x 4,58 = 6,87 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'
- Bentang K- J
  - Pelat lantai tipe 7 = 2,25 x 4,58 = 10,305 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban merata lantai 2

- Bentang N-O
  - Pelat lantai tipe 12 = 1 x 4,58 = 4,58 KN/m'
  - Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

d. Beban merata lantai 3

- Bentang N-O
  - Pelat lantai tipe 14 = 0,613 x 4,58 = 2,808 KN/m'
  - Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

e. Beban merata atap

- Bentang K^1-N
  - Pelat atap tipe h = 0,5 x 3,06 = 1,53 KN/m'
  - Pelat atap tipe b = 0,5 x 3,06 = 1,53 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K^1-K

  - Pelat atap tipe a = 1,25 x 3,06 = 3,825 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

  - Pelat lantai tipe 2 = 1,5 x 2,5 = 3,75 KN/m'

- Bentang K^1-K

  - Pelat lantai tipe 1 = 1,25 x 2,5 = 3,125 KN/m'

b. Beban merata lantai 2

- Bentang N-O

  - Pelat lantai tipe 12 = 1 x 2,5 = 2,5 KN/m'

c. Beban merata lantai 3

- Bentang N-O

  - Pelat lantai tipe 14 = 0,613 x 2,5 = 1,533 KN/m'

d. Beban merata atap

- Bentang K^1-N

  - Beban pekerja (tipe h) = 0,5 x 1,5 = 0,75 KN/m'
  - Beban pekerja (tipe b) = 0,5 x 1,5 = 0,75 KN/m'

- Bentang K^1-K

  - Beban pekerja (tipe a) = 1,25 x 1,5 = 1,875 KN/m'

E. Portal As 6

➢ Beban Mati

a. Beban merata lantai 1

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2	= 2 x 1,5 x 4,58	= 13,74 KN/m'
- Berat sendiri	= 0,35 x 0,7 x 24	= 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1	= 2 x 1,25 x 4,58	= 11,45 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang K-P

- Pelat lantai tipe 9	= 0,75 x 4,58	= 3,435 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2	= 2 x 1,5 x 4,58	= 13,74 KN/m'
- Berat sendiri	= 0,35 x 0,7 x 24	= 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1	= 2 x 1,5 x 4,58	= 13,74 KN/m'
- Beban ornament	=	= 3,93 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

c. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12	= 1 x 4,58	= 4,58 KN/m'
- Berat sendiri	= 0,15 x 0,3 x 24	= 1,08 KN/m'

- Bentang K- P

- Pelat lantai tipe 9 = 0,75 x 4,58 = 3,435 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

d. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 4,58 = 2,808 KN/m'
- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

e. Beban merata atap

- Bentang N^1-N = K^1-K^3

- Pelat atap tipe b = 2 x 0,5 x 3,06 = 3,06 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a = 2 x 1,25 x 3,06 = 7,65 KN/m'
- Beban ornament = = 3,39 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

f. Beban terpusat

- Pada kolom atap

- beban kuda-kuda 22,54 KN

- Pada as P dan J lantai 3, 4, atap

- bcban tritisan = 4,23 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 2,5 = 7,5 KN/m'

- Bentang K^1-K

  - Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

b. Beban merata lantai 2

- Bentang N-O

  - Pelat lantai tipe 12 = 1 x 2,5 = 2,5 KN/m'

c. Beban merata lantai 3

- Bentang N-O

  - Pelat lantai tipe 14 = 0,613 x 2,5 = 1,533 KN/m'

d. Beban merata atap

- Bentang K^1-K^3 = N-N^1

  - Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang K^1-K

  - Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 3,75 KN/m'

F. Portal As 7

➢ Beban Mati

a. Beban merata lantai 1

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

  - Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'
  - Dinding = 4 x 2,5 = 10 KN/m'
  - Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

  - Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

- Pelat lantai tipe 2	= 2 x 1,5 x 4,58	= 13,74 KN/m'
- Berat sendiri	= 0,35 x 0,7 x 24	= 5,88 KN/m'

- Bentang $K^1\text{-}K$

- Pelat lantai tipe 1	= 2 x 1,5 x 4,58	= 13,74 KN/m'
- Beban ornament	=	= 3,93 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

c. Beban merata atap

- Bentang $N^1\text{-}N = K^1\text{-}K^3$

- Pelat atap tipe b	= 2 x 0,5 x 3,06	= 3,06 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang $K^1\text{-}K$

- Pelat atap tipe a	= 2 x 1,25 x 3,06	= 7,65 KN/m'
- Beban ornament	=	= 3,39 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

d. Beban terpusat

- Pada kolom atap

- beban kuda-kuda	= 22,54 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

- Pelat lantai tipe 2	= 2 x 1,5 x 2,5	= 7,5 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

b. Beban merata atap

- Bentang K^1-K^3 = N-N^1

- Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang K^1-K

- Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 6,25 KN/m'

G. Portal As 8

➢ Beban Mati

a. Beban merata lantai 1

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'

- Dinding = 4 x 2,5 = 10 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'

- Dinding = 4 x 2,5 = 10 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,5 x 4,58 = 13,74 KN/m'
- Beban ornament = = 3,39 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 4,58 = 4,58 KN/m'
- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

d. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 4,58 = 2,808 KN/m'
- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

e. Beban merata atap

- Bentang N^1-N = K^1-K^3

- Pelat atap tipe b = 2 x 0,5 x 3,06 = 3,06 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a = 2 x 1,25 x 3,06 = 7,65 KN/m'
- Beban ornament = = 3,39 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

f. Beban terpusat

- Pada kolom atap

- beban kuda-kuda = 22,54 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

  - Pelat lantai tipe 2 = 2 x 1,5 x 2,5 = 7,5 KN/m'

- Bentang $K^1\text{-}K$

  - Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

b. Beban merata lantai 2

- Bentang N-O

  - Pelat lantai tipe 12 = 1 x 2,5 = 2,5 KN/m'

c. Beban merata lantai 3

- Bentang N-O

  - Pelat lantai tipe 14 = 0,613 x 2,5 = 1,533 KN/m'

d. Beban merata atap

- Bentang $K^1\text{-}K^3 = N\text{-}N^1$

  - Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang $K^1\text{-}K$

  - Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 3,75 KN/m'

H. Portal As 9

➢ Beban Mati

a. Beban merata lantai 1

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

  - Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'

  - Dinding = 4 x 2,5 = 10 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,5 x 4,58 = 13,74 KN/m'

- Beban ornament = = 3,93 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 4,58 = 4,58 KN/m'

- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

d. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 4,58 = 2,808 KN/m'

- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

e. Beban merata atap

- Bentang N^1-N = K^1-K^3

- Pelat atap tipe b = 2 x 0,5 x 3,06 = 3,06 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a = 2 x 1,25 x 3,06 = 7,65 KN/m'

- Beban ornament = = 3,39 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

f. Beban terpusat

- Pada kolom atap

- beban kuda-kuda = 22,54 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 2,5 = 7,5 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

b. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 2,5 = 2,5 KN/m'

c. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 2,5 = 1,533 KN/m'

d. Beban merata atap

- Bentang K^1-K^3 = N-N^1

- Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang K^1-K

  - Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 3,75 KN/m'

I. Portal As 10

➢ Beban Mati

a. Beban merata lantai 1

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

  - Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'
  - Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

  - Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang $N\text{-}L^2 = L^2\text{-}L^1 = L^1\text{-}K^1$

  - Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'
  - Dinding (hanya di lt 2) = 4 x 2,5 = 10 KN/m'
  - Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

  - Pelat lantai tipe 1 = 2 x 1,5 x 4,58 = 13,74 KN/m'
  - Beban ornament = = 3,93 KN/m'
  - Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban merata atap

- Bentang $N^1\text{-}N = K^1\text{-}K^3$

  - Pelat atap tipe b = 2 x 0,5 x 3,06 = 3,06 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a = 2 x 1,25 x 3,06 = 7,65 KN/m'

- Beban ornament = = 3,39 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

d. Beban terpusat

- Pada kolom atap

- beban kuda-kuda = 22,54 KN

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 2,5 = 7,5 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

b. Beban merata atap

- Bentang K^1-K^3 = N-N^1

- Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang K^1-K

- Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 3,75 KN/m'

J. Portal As 11

➢ Beban Mati

a. Beban merata lantai 1

- Bentang N-L^2

- Pelat lantai tipe 2 = 1,5 x 4,58 = 6,87 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 2 x 1,5 x 4,58 = 13,74 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2 = 1,5 x 4,58 = 6,87 KN/m'
- Dinding = 4 x 2,5 = 10 KN/m'
- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 4,58 = 11,45 KN/m'
- Beban ornament = = 3,93 KN/m'
- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

c. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 4,58 = 4,58 KN/m'
- Berat sendiri = 0,15 x 0,3 x 24 = 1,08 KN/m'

- Pelat lantai tipe 2 = 1,5 x 2,5 = 3,75 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 2 x 1,25 x 2,5 = 6,25 KN/m'

c. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 2,5 = 2,5 KN/m'

d. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 2,5 = 1,533 KN/m'

e. Beban merata atap

- Bentang K^1-K^3 = N-N^1

- Beban pekerja (tipe b) = 2 x 0,5 x 1,5 = 1,5 KN/m'

- Bentang K^1-K

- Beban pekerja (tipe a) = 2 x 1,25 x 1,5 = 3,75 KN/m'

K. Portal As 12

➢ Beban Mati

a. Beban merata lantai 1, 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Dinding = 4 x 2,5 = 10 KN/m'

- Berat sendiri = 0,35 x 0,7 x 24 = 5,88 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1 = 1,25 x 4,58 = 5,725 KN/m'

- Berat sendiri = 0,25 x 0,4 x 24 = 2,4 KN/m'

d. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14	= 0,613 x 4,58	= 2,808 KN/m'
- Berat sendiri	= 0,15 x 0,3 x 24	= 1,08 KN/m'

e. Beban merata atap

- Bentang N^1-N = K^1-K^3

- Pelat atap tipe b	= 2 x 0,5 x 3,06	= 3,06 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a	= 2 x 1,25 x 3,06	= 7,65 KN/m'
- Beban ornament	=	= 3,39 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

f. Beban terpusat

- Pada kolom atap

- beban kuda-kuda	= 22,54 KN

➢ Beban Hidup

a. Beban merata lantai 1

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

- Pelat lantai tipe 2	= 2 x 1,5 x 2,5	= 7,5 KN/m'

- Bentang K^1-K

- Pelat lantai tipe 1	= 2 x 1,25 x 2,5	= 6,25 KN/m'

b. Beban merata lantai 2, 3, 4

- Bentang N-L^2 = L^2-L^1 = L^1-K^1

b. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12	= 1 x 4,58	= 4,58 KN/m'
- Berat sendiri	= 0,15 x 0,3 x 24	= 1,08 KN/m'

c. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14	= 0,613 x 4,58	= 2,808 KN/m'
- Berat sendiri	= 0,15 x 0,3 x 24	= 1,08 KN/m'

d. Beban merata atap

- Bentang N^1-N = K^1-K^3

- Pelat atap tipe b	= 0,5 x 3,06	= 1,53 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang K^1-K

- Pelat atap tipe a	= 1,25 x 3,06	= 3,825 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

- Bentang K^3-N^1

- Pelat atap tipe h	= 0,5 x 3,06	= 1,53 KN/m'
- Berat sendiri	= 0,25 x 0,4 x 24	= 2,4 KN/m'

➢ Beban Hidup

a. Beban merata lantai 1, 2, 3, 4

- Bentang K^1-K

- Pelat lantai tipe 1	= 1,25 x 2,5	= 3,125 KN/m'

b. Beban merata lantai 2

- Bentang N-O

- Pelat lantai tipe 12 = 1 x 4,58 = 4,58 KN/m'

c. Beban merata lantai 3

- Bentang N-O

- Pelat lantai tipe 14 = 0,613 x 4,58 = 2,808 KN/m'

d. Beban merata atap

- Bentang K^1-K^3 = N-N^1

- Beban pekerja (tipe b) = 0,5 x 1,5 = 0,75 KN/m'

- Bentang K^1-K

- Beban pekerja (tipe a) = 1,25 x 1,5 = 1,875 KN/m'

- Bentang K^3-N^1

- Beban pekerja (tipe h) = 0,5 x 1,5 = 0,75 KN/m'

4.4.2 Perhitungan Gaya Geser Dasar Horizontal Total Akibat Gempa

Gaya geser dasar horizontal akibat gempa dipengaruhi oleh berat total dari keseluruhan struktur. Berat total struktur merupakan berat total dari massa struktur yang direncanakan ditambah dengan beban hidup yang bakerja. Untuk perencanaan beban gempa, beban hidup direduksi sebesar 0,5 menurut Peraturan Pembebanan Indonesi 1983.

A. Atap

a. Beban mati

- Berat atap = (192,5/cos 45) x 0,5 = 136,1 KN

- Berat kuda-kuda = 273,9042 x 0,0377 = 10,3262 KN

- Pelat PA1 dan PA3 = 29,5 x 1 x 2 x 3,06 = 180,54 KN
- Pelat PA2 dan PA4 = 29,5 x 2,5 x 3,06 = 225,675 KN
- Pelat PA5 = 7 x 1 x 2 x 3,06 = 42,84 KN
- Plaf. + penggantung = 192,5 x 0,18 = 34,65 KN
- Kolom 25x25 = 0,25 x 0,25 x 24 x19,2 = 28,8 KN
- Balok BA3 = 0,25 x 0,40 x 24 x 285,5 = 685,2 KN

wD = 1344,13 KN

b Beban hidup

- Pelat PA1 dan PA3 = 0,5 x 29,5 x 1 x 2 x 1,5 = 44,25 KN
- Pelat PA2 dan PA4 = 0,5 x 29,5 x 2,5 x 1,5 = 55.,3125 KN
- Pelat PA5 = 0,5 x 7 x 1 x 2 x 1,5 = 10,5 KN
- Pekerja atap = 0,5 x1 x (192,5/cos 45) = 136,118 KN

wL = 246.1805 KN

Wt_a = 1344,13 + 246,1805 = 1590,3105 KN

B. Lantai 4

a. Beban mati

- Pelat PA1 dan PA3 = 9 x 25 x 4,58 = 1030,5 KN
- Pelat PA2 dan PA4 = 29,5 x 2,5 x 4,58 = 337,775 KN
- Pelat PA5 dan PA6 = 4,5 x 4,5 x 4,58 = 92,745 KN
- Pelat PA3 = 3 x 2 x 4,58 = 27,48 KN
- Kolom KA1 = 0,45 x 0,60 x 24 x 64 = 414,72 KN

- Kolom KA2	= 0,4 x 0,4 x 24 x 44	= 168,96 KN
- Kolom KA3	= 0,25 x 0,35 x 24 x 12	= 25,2 KN
- Balok BA1	= 0,35 x 0,7 x 24 x 72	= 423,4 KN
- Balok BA2	= 0,3 x 0,45 x 24 x 59	= 191,16 KN
- Balok BA3	= 0,25 x 0,4 x 24 x 123,864	= 297,274 KN
- Dinding	= 4 x 95 x 2,5	= 950 KN
- Tangga	= 41,11 + 36,2 + 21,9	= 99,18 KN
- Railing (ditaksir)	= 2,115 x 10	= 21,15 KN
- Ornamen (ditaksir)	= 7 x 8	= 56 KN
	wD	= 4135,544 KN

b. Beban hidup

- Pelat PA1 dan PA3	= 0,5 x 9 x 25 x 2,5	= 281,25 KN
- Pelat PA2 dan PA4	= 0,5 x 29,5 x 2,5 x 2,5	= 92.1875 KN
- Pelat PA5 dan PA6	= 0,5 x 4,5 x 4,5 x 2,5	= 25,3125 KN
- Pelat PA3	= 0x5 x 3 x 2 x 2,5	= 7,5 KN
- Tangga	= 0,5 x 88,543	= 44,2715 KN
	wL	= 406,25 KN

Wt_4 = 4135,544 + 406,25 = 4541,794 KN

C. Lantai 3

a. Beban mati

- Pelat PA1 dan PA3	= 9 x 25 x 4,58	= 1030,5 KN
- Pelat PA2 dan PA4	= 29,5 x 2,5 x 4,58	= 337,775 KN
- Pelat PA5 dan PA6	= 4,5 x 4,5 x 4,58	= 92,745 KN

- Pelat PA3	= 3 x 2 x 4,58	= 27,48 KN
- Pelat PA7	= 1,225 x 4,5 x 3 x 4,58	= 75,742 KN
- Kolom KA1	= 0,45 x 0,60 x 24 x 64	= 414,7 KN
- Kolom KA2	= 0,4 x 0,4 x 24 x 44	= 168,96 KN
- Kolom KA3	= 0,25 x 0,35 x 24 x 12	= 25,2 KN
- Balok BA1	= 0,35 x 0,7 x 24 x 72	= 423,4 KN
- Balok BA2	= 0,3 x 0,45 x 24 x 59	= 191,16 KN
- Balok BA3	= 0,25 x 0,4 x 24 x 141,864	= 340,474 KN
- Balok BA4	= 0,15 x 0,30 x 24 x 7,35	= 7,938 KN
- Dinding	= 2 x 196 x 2,5	= 980 KN
- Railing (ditaksir)	= 2,115 x 10	= 21,15 KN
- Ornamen (ditaksir)	= 7 x 8	= 56 KN
- Tangga	= 278,699	= 278,699 KN
	wD	= 4471,923 KN

b. Beban hidup

- Pelat PA1 dan PA3	= 0,5 x 9 x 25 x 2,5	= 281,25 KN
- Pelat PA2 dan PA4	= 0,5 x 29,5 x 2,5 x 2,5	= 92.1875 KN
- Pelat PA5 dan PA6	= 0,5 x 4,5 x 4,5 x 2,5	= 25,3125 KN
- Pelat PA3	= 0,5 x 3 x 2 x 2,5	= 7,5 KN
- Pelat PA7	= 0,5 x 1,225 x 4,5 x 3 x 2,5	= 20,675 KN
- Tangga	= 0,5 x 88,543	= 44,2715 KN
	wL	= 471,1935 KN

$\mathbf{Wt_3}$ = 4471,923 + 471,1935 = 4943,1165 KN

D. Lantai 2

a. Beban mati

- Pelat PA1 dan PA3	= 9 x 25 x 4,58	= 1030,5 KN
- Pelat PA2 dan PA4	= 29,5 x 2,5 x 4,58	= 337,775 KN
- Pelat PA5 dan PA6	= 4,5 x 4,5 x 4,58	= 92,745 KN
- Pelat PA3	= 3 x 2 x 4,58	= 27,48 KN
- Pelat PA7	= 2 x 4,5 x 3 x 4,58	= 123,66 KN
- Kolom KA1	= 0,45 x 0,60 x 24 x 64	= 414,7 KN
- Kolom KA2	= 0,4 x 0,4 x 24 x 44	= 168,96 KN
- Kolom KA3	= 0,25 x 0,35 x 24 x 12	= 25,2 KN
- Balok BA1	= 0,35 x 0,7 x 24 x 72	= 423,4 KN
- Balok BA2	= 0,3 x 0,45 x 24 x 72,5	= 234,9 KN
- Balok BA3	= 0,25 x 0,4 x 24 x 119	= 285,6 KN
- Balok BA4	= 0,15 x 0,30 x 24 x 12	= 12,96 KN
- Balok BA5	= 0,20 x 0,35 x 24 x 13,243	= 22,25 KN
- Dinding	= 2 x 216 x 2,5	= 1078 KN
- Tangga	=	= 278,699KN
- Railing (ditaksir)	= 2,115 x 10	= 21,15 KN
- Ornamen (ditaksir)	= 7 x 8	= 56 KN
	wD	= 4633,979 KN

b. Beban hidup

- Pelat PA1 dan PA3	= 0,5 x 9 x 25 x 2,5	= 281,25 KN
- Pelat PA2 dan PA4	= 0,5 x 29,5 x 2,5 x 2,5	= 92,1875 KN

- Pelat PA5 dan PA6 = 0,5 x 4,5 x 4,5 x 2,5 = 25,3125 KN
- Pelat PA3 = 0,5 x 3 x 2 x 2,5 = 7,5 KN
- Pelat PA7 = 0,5 x 2 x 4,5 x 3 x 2,5 = 33,75 KN
- Tangga = 0,5 x 88,543 = 44,2715 KN

wL = 484,2715 KN

Wt_2 = 4633,979 + 484,2715 = 5118,2505 KN

E. Lantai 1

a. Beban mati

- Pelat PA1 dan PA3 = 9 x 25 x 4,58 = 1030,5 KN
- Pelat PA2 dan PA4 = 29,5 x 2,5 x 4,58 = 337,775 KN
- Pelat PA5 dan PA6 = 4,5 x 4,5 x 4,58 = 92,745 KN
- Pelat PA3 = 3 x 2 x 4,58 = 27,48 KN
- Kolom KA1 = 0,45 x 0,60 x 24 x 64 = 414,7 KN
- Kolom KA2 = 0,4 x 0,4 x 24 x 44 = 168,96 KN
- Kolom KA3 = 0,25 x 0,35 x 24 x 12 = 25,2 KN
- Balok BA1 = 0,35 x 0,7 x 24 x 72 = 423,4 KN
- Balok BA2 = 0,3 x 0,45 x 24 x 59 = 191,16 KN
- Balok BA3 = 0,25 x 0,4 x 24 x 117,5 = 282 KN
- Dinding = 4 x 115 x 2,5 = 1145 KN
- Tangga = = 278,699 KN
- Railing (ditaksir) = 2,115 x 10 = 21,15 KN
- Ornamen (ditaksir) = 7 x 8 = 56 KN

wD = 4493,769 KN

b. Beban hidup

- Pelat PA1 dan PA3 = 0,5 x 9 x 25 x 2,5 = 281,25 KN
- Pelat PA2 dan PA4 = 0,5 x 29,5 x 2,5 x 2,5 = 92,1875 KN
- Pelat PA5 dan PA6 = 0,5 x 4,5 x 4,5 x 2,5 = 25,315 KN
- Pelat PA3 = 0,5 x 3 x 2 x 2,5 = 7,5 KN
- Tangga = 0,5 x 88,543 = 44,2715 KN

wL = 450,5215 KN

$\mathbf{Wt_1}$ = 4493,769 + 450,5215 = 4944,2905 KN

Wtotal = $Wt_a + Wt_4 + Wt_3 + Wt_2 + Wt_1$ = 1590,3105 + 4541,794 + 4943,1165 + 5118,2505 + 4944,2905 = 21137,762 KN

F. Waktu getar bangunan (T)

Waktu getar alami struktur gedung untuk portal beton ditentukan dengan rumus pendekatan sebagai berikut :

$T = 0,06 . H^{3/4} = 0,06 . 22^{3/4} = 0,6095$ dt

G. Koefisien gempa dasar

Pada buku PPKGURG 1987 untuk jenis tanah keras dengan T = 0,6095 dt pada wilayah Zona 3 didapatkan koefisien gempa dasar C = 0,048

H. Faktor keutamaan I dan faktor jenis struktur K

Faktor keutamaan struktur diambil I = 1,5 didasrkan pada jenis dan fungsi dari gedung yang ditinjau agar waktu ulang dari kerusakan struktur akibat gempa dapat diperpanjang sehingga ada kesempatan untuk usaha penyelamatan.

Sedangkan untuk faktor jenis struktur diambil K = 1 yaitu untuk portal daktail.

I. Gaya geser horizontal akibat gempa

Dalam Pedoman Ketahanan Gempa, rumus untuk gaya geser dasar horizontal akibat gempa adalah

$V = C \cdot I \cdot K \cdot Wt = 0{,}048 \cdot 1{,}5 \cdot 1 \cdot 21137{,}762 = 1521{,}92$ KN

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

Tingkat	Hi (m)	Wi (kN)	V (kN)	Wi.hi (kN.m)	Fi (kN)
Atap	22	1590,31	1521,92	34986,83	200,7604
Lantai 4	17,92	4541,79	1521,92	81388,95	467,0236
Lantai 3	13,92	4943,12	1521,92	68808,18	394,8331
Lantai 2	9,92	5118,25	1521,92	50773,04	291,3444
Lantai 1	5,92	4944,29	1521,92	29270,2	167,9574

Distribusi Gaya Geser Dasar Horisontal Total ke Arah X dan Y

- Untuk gaya geser dasar horizontal arah X harus dibagi dengan jumlah portal arah X yaitu Fix = Fi / 8
- Untuk gaya geser dasar horizontal arah Y harus dibagi dengan jumlah portal arah Y yaitu Fiy = Fi / 3

Tingkat	Fix (KN)	Fiy (KN)
Atap	25,1	66,920
Lantai 4	58,378	155,675
Lantai 3	49,354	131,611
Lantai 2	36,42	97,115
Lantai 1	20,995	55,986

4.5 Perencanaan Balok Anak

Balok anak pada gedung Kampus D3 Ekonomi dalam perencanaan ini di asumsikan sebagai balok menerus dan dimodelkan menjadi bagian dari struktur portal, sehingga momen-momen yang didapat dari hasil analisis SAP 2000.

Untuk perhitungan perencanaan balok anak dapat dilihat berdasarkan landasan teori rumus 3.5.1 s/d 3.5.17.

Data-data :

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

b) qD pelat = 4,58 KN/m^2

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

d) perkiraan ukuran balok

$$h = \frac{1}{12} \cdot 4{,}5 = 0{,}375 \text{ m}$$

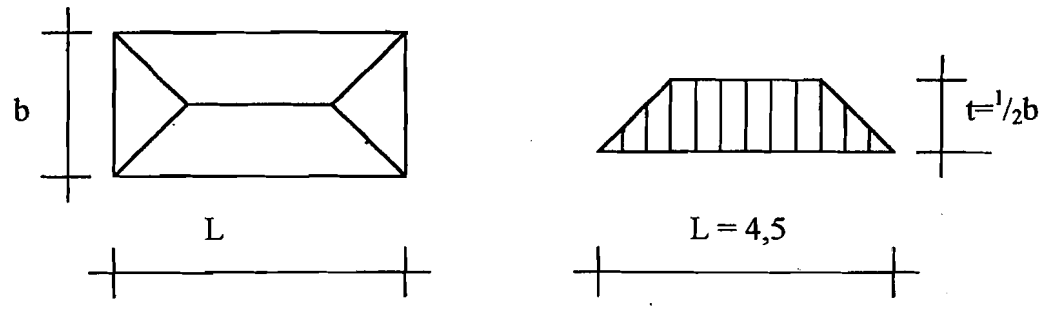
sehingga asumsi ukuran balok h = 0,4 m

b = 0,25 m

e) tinggi tembok (lantai 1) = 4 m

4.5.1 Perencanaan Balok Anak

A. Pembebanan Balok Anak



Gambar 4.10 Model Pembebanan Balok Anak

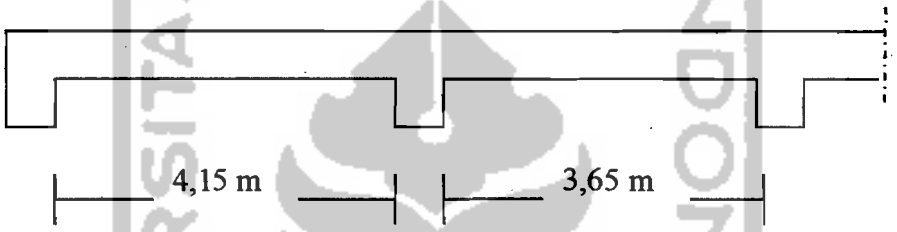
- Beban mati

beban pelat	= t.qD.n = 1,5.4,58.2	= 13,74 KN/m
berat balok	= b_{blk}. (h_{blk}- t_{pelat}).bj	
	= 0,25.(0,4 - 0,12).24	= 1,68 KN/m

- Beban hidup

qL balok anak	= t.qL.n = 1,5 . 2,5 . 2	= 7,5 KN/m

B. Momen-momen pada balok



Gambar 4.11 Balok Menerus Balok Anak

Dari hasil analisis SAP 2000, momen-momen yang didapat dianggap sama pada semua bentang dari lantai 1 sampai dengan lantai 4 dan diambil momen pada tumpuan dan lapangan yang paling besar. Perhitungan balok anak ini untuk semua As yaitu As L^1 dan L^2. Adapun besar momen pada tumpuan dan lapangan adalah :

Momen tumpuan = 50,47 KNm

Momen lapangan = 37,14 KNm

C. Penulangan Balok Anak

f'c = 22,5 Mpa

fy ulir = 400 Mpa

∅ tul pokok = 16 mm

∅ tulangan sengkang = 10 mm

untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0,85$

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

$$\rho_b = \frac{0,85.f'c}{fy}\beta\left(\frac{600}{600+fy}\right) = \frac{0,85.22.5}{400}0,85\left(\frac{600}{600+400}\right) = 0,0244$$

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

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

diambil $\rho_{pakai} = 0,5\ \rho_{maks} = 0,0092$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22.5} = 20,915$$

$$Rn = \rho.fy.(1\text{-}0,5.\ \rho.m) = 0,0092.400.(1-\frac{1}{2}.0,0092.20,915) = 3,326$$

➢ Penulangan untuk Mu tumpuan = Mu = 50,47 KNm

$$Mn = \frac{Mu_1}{\phi} = \frac{50,47}{0,8} = 63,088\ KN/m$$

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{60,088.10^6}{3,326} = 18066145,52\ mm^2$$

diambil b = 250 mm

$250.d^2 = 18066145,52\ mm^2$

$d_{perlu} = 268,821$ mm

ambil h = 400 mm

d = h-pb-∅sengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 400 - 40 - 10 - \frac{1}{2}.16 = 342\ mm$$

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

Karena ukuran balok yang dipakai sama dengan perkiraan awal di atas maka momen yang dipakai masih sama dengan yang di atas.

$$Mn = \frac{Mu_1}{\phi} = \frac{50,47}{0,8} = 63,088 \text{ KNm}$$

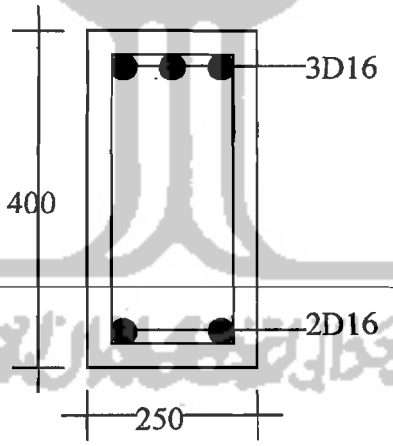
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{63,088.10^6}{250.342^2} = 2,158$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{2,158}{3,326}.0,0092 = 0,00597$$

$$As_{perlu} = \rho_{baru}.b.d = 0,00597.250.342 = 510,435 \text{ mm}^2$$

$$A\phi_{16} = 200,96 \text{ mm}^2$$

dipakai $3\phi_{16}$ $As_{ada} = 602,88 \text{ mm}^2 > As_{perlu} = 510,435 \text{ mm}^2$



Gambar 4.12 Penampang Melintang Balok Anak

$$\text{Jarak bebas datar} = \frac{b - 2.Pb - 2.\phi \text{ sengkang} - n.\phi \text{ tulangan}}{(n-1)}$$

$$= \frac{250 - 2.40 - 2.10 - 3.16}{(3-1)} = 51 \text{ mm} > 25 \text{ mm}$$

Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f.c.b} = \frac{602{,}88.400}{0{,}85.22{,}5.250} = 50{,}44 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right) = 602{,}88.400\left(342 - \frac{50{,}44}{2}\right)$$

$$= 76{,}4 \text{ KNm} \geq \frac{Mu}{\phi} = 63{,}088 \text{ KNm}\ldots\ldots\textbf{OK}$$

➢ Penulangan untuk Mu lapangan = 37,14 KNm

$$\frac{Mu}{\phi} = \frac{37{,}14}{0{,}8} = 46{,}425 \text{ KNm}$$

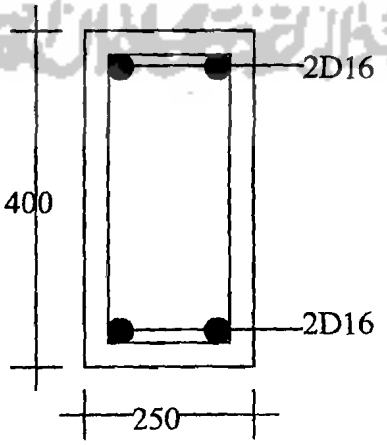
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{46{,}425.10^6}{250.342^2} = 1{,}588$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{1{,}588}{3{,}326}.0{,}0092 = 0{,}0044$$

$$As_{perlu} = \rho_{baru}.b.d = 0{,}0044.250.342 = 376{,}2 \text{ mm}^2$$

$$A\phi_{16} = 200{,}96 \text{ mm}^2$$

dipakai $2\phi_{16}$ $As_{ada} = 401{,}92 \text{ mm}^2 > As_{perlu} = 376{,}2 \text{ mm}^2$



Gambar 4.13 Penampang Melintang Balok Anak

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$

$= \dfrac{250 - 2.40 - 2.10 - 2.16}{(2-1)} = 118\ mm > 25\ mm$

Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{401{,}92.400}{0{,}85.22{,}5.250} = 33{,}625$$

$$Mn = As_{ada}.fy.\ (d - a/2) = 401{,}92.400\ (342 - \frac{33{,}625}{2})$$

$$= 52{,}28\ KNm > \frac{Mu}{\phi} = 46{,}425\ KNm\text{.....}\mathbf{OK!}$$

4.5.2 Perhitungan penulangan geser balok anak

diketahui : b = 250 mm fy = 240 Mpa

h = 400 mm f'c = 22,5 Mpa d = 342 mm

jarak bentang bersih = 4,15 m

Perencanaan geser balok anak berdasakan rumus 3.5.35 s/d 3.5.47 pada landasan teori bab III.

- Gaya geser pada tumpuan :

  Vu = 63,68 KN

  $\dfrac{Vu}{\phi} = \dfrac{63{,}68}{0{,}6} = 106{,}133\ KN$

- Gaya geser pada penampang kritis sejauh d dari tumpuan :

  Vu $= \dfrac{2{,}075 - 0{,}342}{2{,}075}.63{,}68 = 53{,}2\ KN$

  $\dfrac{Vu}{\phi} = \dfrac{53{,}2}{0{,}6} = 88{,}667\ KN$

- Gaya geser beton :

$$Vc = \frac{1}{6}\sqrt{f'c}\,.b.d = \frac{1}{6}\sqrt{22,5}\,.250.342 = 67,594\ KN$$

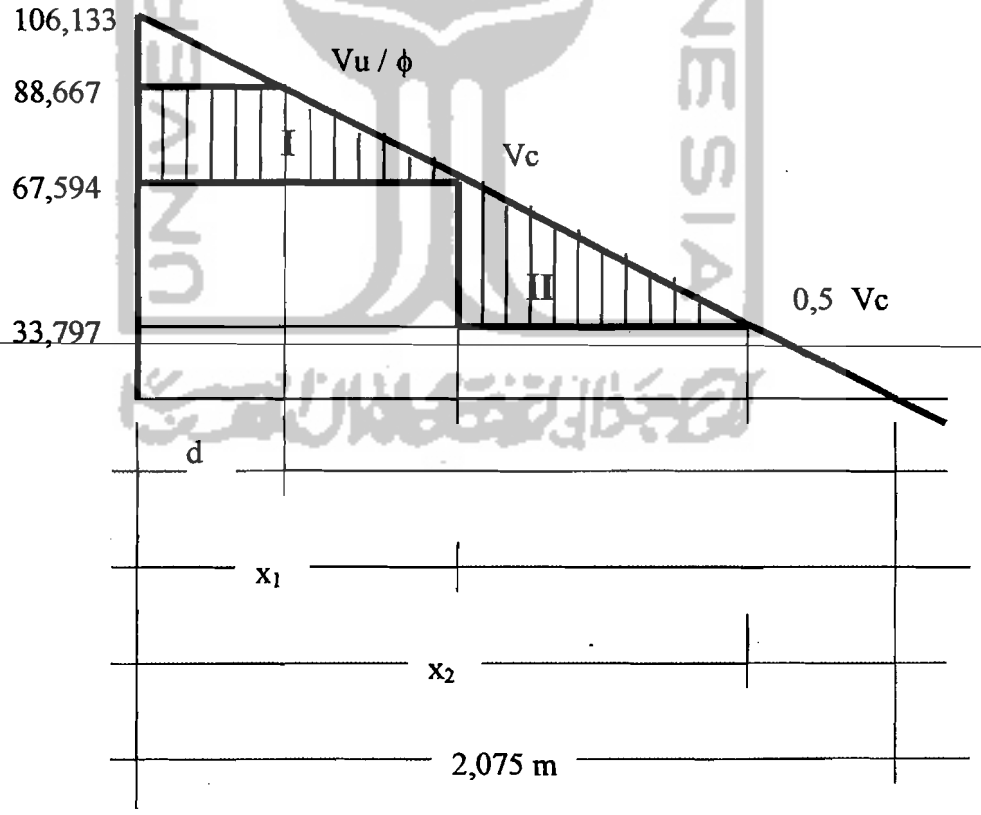
$$\frac{1}{2}Vc = 33,797\ KN$$

$$3Vc = 202,782\ KN$$

$$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3}.250.342 = 28,500KN$$

ternyata : $Vc < \frac{Vu}{\phi} \le (Vc + Vs_{min})$

67,594 KN < 88,667 KN < 96,094 KN maka ukuran balok dapat digunakan tetapi diperlukan tulangan geser.



Gambar 4.14 Diagram Geser Balok Anak

- Titik dimana gaya geser = Vc = 67,594 KN

$x_1 = \frac{67,594}{106,133}.2,075 = 1,322$ m dari tengah bentang

- Titik dimana gaya geser = 0,5 Vc = 33,797 KN

$x_2 = \frac{33,797}{106,133}.2,075 = 0,661$ m dari tengah bentang

Daerah I

digunakan sengkang ∅10mm

$Av = 2.\ 1/4\ \pi D^2 = 2.\ 1/4\ \pi 10^2\ mm^2 = 157\ mm^2$

$S_1 \leq \frac{Av.fy.d}{Vs} = \frac{157.240.342}{(88,667 - 67,594).10^3} = 611,52$ mm

$S_2 \leq \frac{d}{2} = \frac{342}{2} = 171$ mm

$S_3 \leq 600$ mm

Dipakai $P_{10\text{ - }170}$

Daerah II

digunakan sengkang ∅8 mm, dengan $Av = 100,48\ mm^2$

$S_1 \leq \frac{Av.fy.d}{Vs\min} = \frac{100,48.240.342}{28,500.10^3} = 289,382$ mm

$S_2 \leq \frac{d}{2} = \frac{342}{2} = 171$ mm

$S_3 \leq 600$ mm

Dipakai $P_{8\text{ - }170}$

4.6 Perencanaan Balok Portal Dan Kolom Portal

4.6.1 Perencanaan Balok Portal terhadap Beban Lentur

Dalam perencanaan balok portal, kuat lentur balok (Mu,b) dari hasil analisa SAP 2000 3 Dimensi dinyatakan berdasarkan berbagai kombinasi pembebanan sebagai berikut :

$M_{U,b} = 1{,}2\, M_{D,b} + 1{,}6\, M_{L,b}$

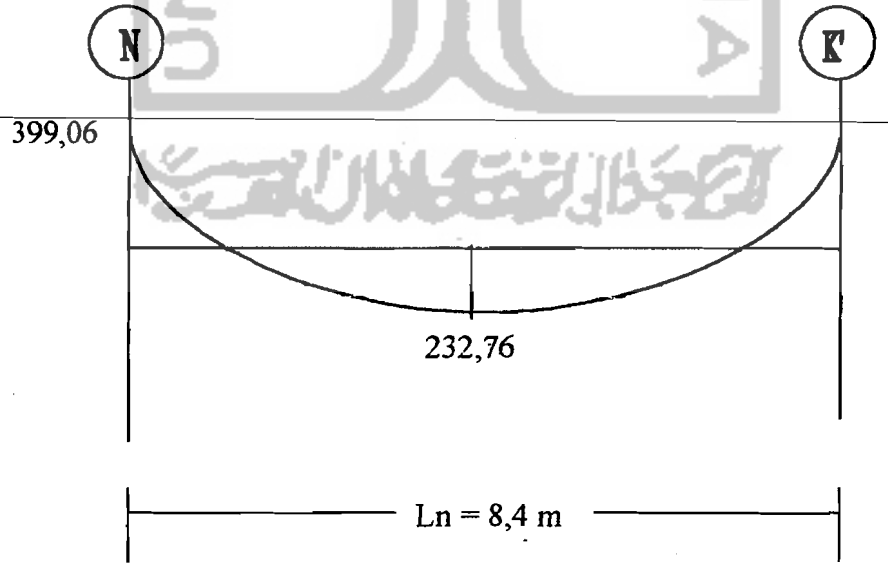
$M_{U,b} = 1{,}05\, (M_{D,b} + M_{L,bR} + M_{E,b})$

$M_{U,b} = 0{,}9.\, M_{D,b} + M_{E,b}$

Dalam hal ini akan diberikan contoh perhitungan dari balok portal, diambil portal As 8 balok BA1 dan BA3 lantai 2 bentang N – K, sedangkan untuk portal As lain dapat dilihat pada lampiran.

A. Momen-momen

Momen diambil yang terbesar dari analisis struktur pada daerah tumpuan dan lapangan sehingga didapatkan seperti terlihat pada gambar di bawah ini :



Gambar 4.15 Momen Balok Portal bentang N-K'

B. Penulangan balok bentang N-K'

Untuk prosedur perencanaan penulangan balok dapat dilihat pada landasan teori rumus 3.5.1 s/d 3.5.34.

perkiraan ukuran balok :

h = 700 mm = 0,7 m

b = 350 mm = 0,35

data :

f'c = 22,5 Mpa

fy ulir = 400 Mpa

∅ tul pokok = 22 mm

∅ tulangan sengkang = 10 mm

untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0,85$

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

$$\rho_b = \frac{0,85.f'c}{fy}\beta\left(\frac{600}{600+fy}\right) = \frac{0,85.22.5}{400}0,85\left(\frac{600}{600+400}\right) = 0,0244$$

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

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

diambil $\rho_{pakai} = 0,5\ \rho_{maks} = 0,0092$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22.5} = 20,915$$

$$Rn = \rho.fy.(1-0,5.\ \rho.m) = 0,0092.400.(1-\frac{1}{2}.0,0092.20,915) = 3,326$$

➢ Penulangan untuk Mu tumpuan terbesar = 399,06 KNm

$$Mn = \frac{Mu_1}{\phi} = \frac{399,06}{0,8} = 498,825 KNm$$

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{498,825.10^6}{3,326} = 149977450,4 \text{ mm}^2$$

diambil b = 350 mm

$350. d^2 = 149979690,3 \text{ mm}^2$

$d_{perlu} = 654,609$ mm

ambil h = 700 mm

d = h-pb-Øsengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 400 - 40 - 10 - \frac{1}{2}.22 = 639 \text{ mm}$$

$d < d_{perlu}$ maka dipakai tulangan rangkap

Untuk perencanaan tulangan rangkap

Di ambil :

d = 700 – 100 – 600 mm (tulangan tarik 2 lapis)

d' = 65 mm

$$As_1 = \rho_1.b.d = 0,0092.350.600 = 1932 \text{ mm}^2$$

$$a = \frac{As_1..fy}{0,85.f'c.b} = \frac{1932.400}{0,85.22,5.350} = 115,451 mm$$

$$Mn_1 = As_1.fy.(d - a/2) = 1932.400(600 - 115,451/2) = 419,07 \text{ KNm}$$

$$Mn_2 = Mn - Mn_1 = 498,825 - 419,07 = 79,755 \text{ KNm}$$

Kelebihan Mn harus ditahan oleh tulangan tekan dan tambahan tulangan tarik :

$$fs' = 600\left[1 - \frac{0{,}85.f'c.\beta_1}{(\rho - \rho')fy}.\frac{d'}{d}\right] = 600\left[1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}0092.400}.\frac{65}{600}\right] = 312{,}86 \text{ Mpa}$$

fs' < fy

dipakai fs' = 312,86 Mpa

Untuk tulangan tekan :

$$As' = \frac{Mn_2}{fs'(d - d')} = \frac{79{,}755.10^6}{312{,}86(600 - 65)} = 476{,}484 \text{ mm}^2$$

$\phi As_{22} = 379{,}94 \text{ mm}^2$

dipakai $2\phi_{22}$ $As_{ada} = 759{,}88 \text{ mm}^2 > As' = 476{,}484 \text{ mm}^2$

Untuk tulangan tarik :

$As = As_1 + As' = 1932 + 476{,}887 = 2408{,}484 \text{ mm}^2$

$\phi As_{22} = 379{,}94 \text{ mm}^2$

Dipakai $7\phi_{22}$ $As_{ada} = 2659{,}58 \text{ mm}^2$

Kontrol tegangan :

$$\rho = \frac{As_{ada}}{b.d} = \frac{2659{,}58}{350.600} = 0{,}0127$$

$$\rho' = \frac{As'_{ada}}{b.d} = \frac{759{,}88}{350.600} = 0{,}00362$$

$$\rho - \rho' = 0{,}0127 - 0{,}00362 = 0{,}00905$$

$$fs' = 600\left[1 - \frac{0{,}85.f'c.\beta_1}{(\rho - \rho')fy}.\frac{d'}{d}\right] = 600\left[1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}00908.400}.\frac{65}{600}\right] = 307{,}98 \text{ Mpa}$$

fs' < fy

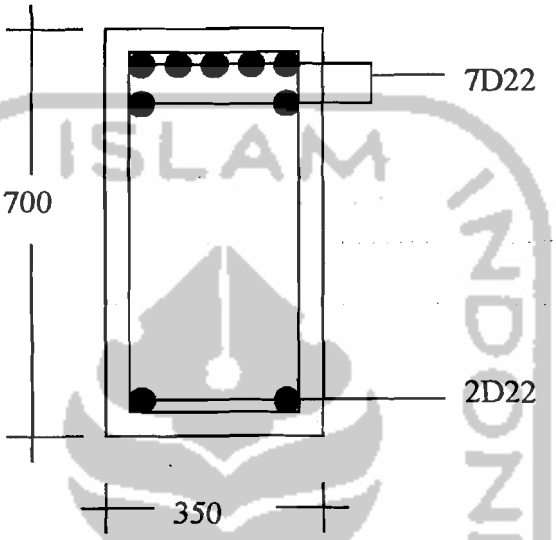
$$a = \frac{As.fy - As'.fs'}{0{,}85.f'c.b} = \frac{2659{,}58.400 - 759{,}88.307{,}98}{0{,}85.22{,}5.350} = 123{,}967 \text{ mm}$$

$$Mn_1 = As.fy - As'.fs'\left(d - a/2\right) = 2659{,}58.400 - 759{,}88.307{,}98\left(600 - 123{,}967/2\right)$$

$$= 446{,}447 \text{ KNm}$$

$$Mn_2 = As'.fs'.(d - d') = 759{,}88.307{,}983.(600 - 65) = 125{,}206 \text{ KNm}$$

$$\text{Mn} = \text{Mn}_1 + \text{Mn}_2 = 446{,}447 + 125{,}206 = 571{,}653 > \text{Mn} = 498{,}825 \text{ Mpa}$$



Gambar 4.16 Penampang Melintang Balok Portal N-K'

$$\text{Jarak bebas datar} = \frac{b - 2.Pb - 2.\phi \text{ sengkang} - n.\phi \text{ tulangan}}{(n-1)}$$

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

➢ Penulangan untuk Mu lapangan = 232,76 KNm

$$\text{Mn} = \frac{Mu_1}{\phi} = \frac{232{,}76}{0{,}8} = 290{,}95 \text{ KNm}$$

$$\text{b.d}^2 = \frac{Mu/\phi}{Rn} = \frac{290{,}95.10^6}{3{,}326} = 87478756{,}9 \text{ mm}^2$$

diambil b = 350 mm

$350.\ d^2 = 87478756{,}9 \text{ mm}^2$

d_{perlu} = 499,939 mm

ambil h = 700 mm

d = h-pb-Øsengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 400 - 40 - 10 - \frac{1}{2}.22 = 639 \text{ mm}$$

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

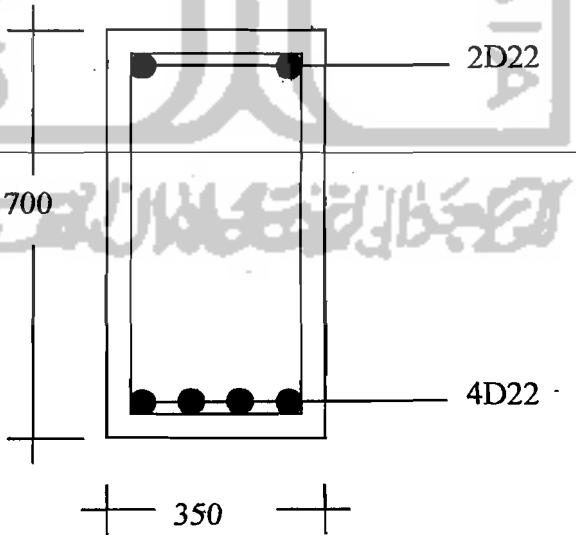
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{290{,}95.10^6}{350.639^2} = 2{,}036$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{2{,}036}{3{,}326}.0{,}0092 = 0{,}005632$$

$As_{perlu} = \rho_{baru}.b.d = 0{,}005632.350.639 = 1259{,}475 \text{ mm}^2$

$A\phi_{22} = 379{,}94 \text{ mm}^2$

dipakai $4\phi_{22}$ $As_{ada} = 1519{,}76 \text{ mm}^2 > As_{perlu} = 1259{,}475 \text{ mm}^2$



Gambar 4.17 Penampang Melintang Balok Portal N – K'

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$

$= \dfrac{350 - 2.40 - 2.10 - 4.22}{(4-1)} = 54$ mm

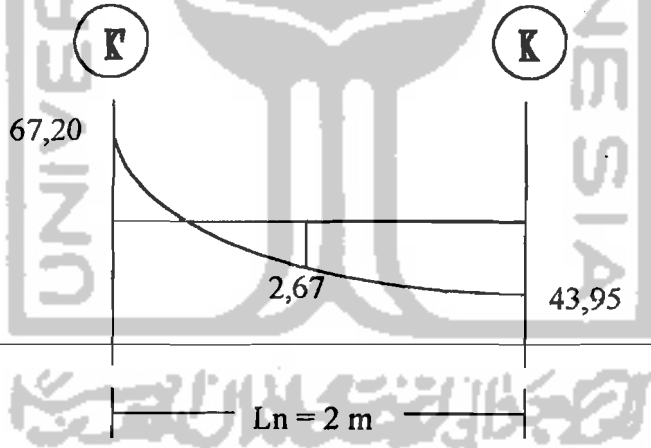
Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1519{,}76.400}{0{,}85.22{,}5.350} = 90{,}817$$

$$Mn = As_{ada}.fy.\ (d - a/2) = 1519{,}76.400\ (639 - \frac{90{,}817}{2})$$

$$= 360{,}85\ \text{KNm} > \frac{Mu}{\phi} = 290{,}95\ \text{KNm}$$

C. Penulangan Balok Portal Bentang K'-K



Gambar 4.18 Momen Bentang K'-K

perkiraan ukuran balok :

h = 400 mm = 0,4 m

b = 250 mm = 0,25

data :

f'c = 22,5 Mpa

fy ulir = 400 Mpa

∅ tul pokok = 22 mm

∅ tulangan sengkang = 10 mm

untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0,85$

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

$$\rho_b = \frac{0,85.f'c}{fy}\beta\left(\frac{600}{600+fy}\right) = \frac{0,85.22.5}{400}0,85\left(\frac{600}{600+400}\right) = 0,0244$$

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

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

diambil $\rho_{pakai} = 0,5\ \rho_{maks} = 0,0092$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.22.5} = 20,915$$

$$Rn = \rho.fy.(1-0,5.\ \rho.m) = 0,0092.400.(1-\frac{1}{2}.0,0092.20,915) = 3,326$$

➢ Penulangan untuk Mu (negative) tumpuan terbesar = 67,2 KNm

$$Mn = \frac{Mu_1}{\phi} = \frac{67,2}{0,8} = 84\ KNm$$

$$b.d^2 = \frac{Mu/\phi}{Rn} = \frac{84.10^6}{3,326} = 25255939,4\ mm^2$$

diambil b = 250 mm

$250.\ d^2 = 25255939,4\ mm^2$

$d_{perlu} = 317,842$ mm

ambil h = 400 mm

d = h-pb-Øsengkang-jarak pusat tulangan pokok kesisi dalam sengkang

$$= 400 - 40 - 10 - \frac{1}{2}.22 = 339 \text{ mm}$$

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

Karena ukuran balok yang dipakai sama dengan perkiraan awal di atas maka momen yang dipakai masih sama dengan yang di atas.

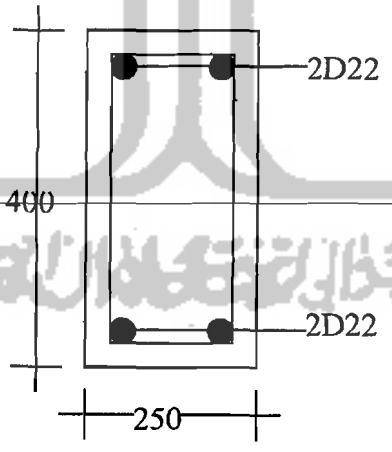
$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{84.10^6}{250.339^2} = 2{,}924$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{2{,}924}{3{,}326}.0{,}0092 = 0{,}00809$$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00809.250.339 = 685{,}412 \text{ mm}^2$

$A\phi_{22} = 379{,}94 \text{ mm}^2$

dipakai $2\phi_{22}$ $As_{ada} = 759{,}88 \text{ mm}^2 > As_{perlu} = 685{,}412 \text{ mm}^2$



Gambar 4.19 Penampang Melintang Balok Portal K-K'

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$

$$= \frac{250 - 2.40 - 2.10 - 2.22}{(2-1)} = 106 \text{ mm} > 25 \text{ mm}$$

Kontrol Mn :

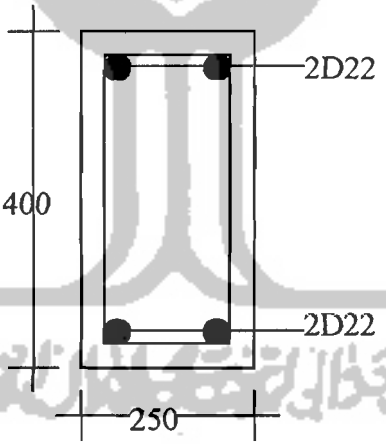
$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{759{,}88.400}{0{,}85.22{,}5.250} = 63{,}572 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right) = 759{,}88.400\left(339 - \frac{63{,}572}{2}\right)$$

$$= 93{,}378 \text{ KNm} \geq \frac{Mu}{\phi} = 84 \text{ KNm}\ldots\ldots\textbf{OK}$$

➢ Penulangan untuk Mu lapangan = 2,67 KNm

Pada penulangan lapangan ini karena momen yang terjadi sangat kecil, maka akan diberikan tulngan minimum.



Gambar 4.20 Penampang Melintang Balok Portal K-K'

➢ Penulangan untuk Mu (positif) tumpuan terbesar = 43,95 KNm

$$Mn = \frac{Mu_3}{\phi} = \frac{43{,}95}{0{,}8} = 54{,}938 \text{ KNm}$$

$$Rn_{baru} = \frac{Mu/\phi}{b.d^2} = \frac{54{,}938.10^6}{250.339^2} = 1{,}9122$$

$$\rho_{baru} = \frac{Rn_{baru}}{Rn}\rho = \frac{1{,}9122}{3{,}326}.0{,}0092 = 0{,}00529$$

$As_{perlu} = \rho_{baru}.b.d = 0{,}00529.250.339 = 448{,}27\ mm^2$

$A\phi_{22} = 379{,}94\ mm^2$

dipakai $2\phi_{22}$ $As_{ada} = 759{,}88\ mm^2 > As_{perlu} = 448{,}27\ mm^2$

Jarak bebas datar $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tulangan}{(n-1)}$

$$= \frac{250 - 2.40 - 2.10 - 2.22}{(2-1)} = 106\ mm > 25\ mm$$

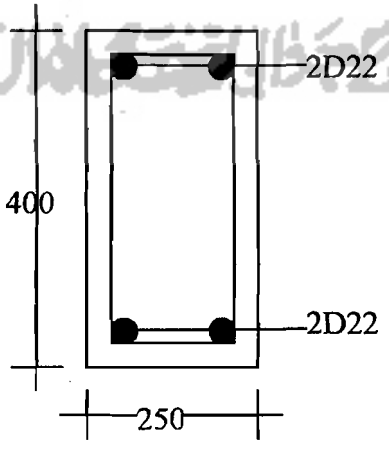
Kontrol Mn :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{759{,}88.400}{0{,}85.22{,}5.250} = 63{,}572\ mm$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right) = 759{,}88.400\left(339 - \frac{63{,}572}{2}\right)$$

$$= 93{,}378\ KNm \geq \frac{Mu}{\phi} = 54{,}938\ KNm\text{.....}\mathbf{OK}$$

Jadi tulangan terpakai pada penampang balok K-K' :



Gambar 4.21 Penampang Melintang Balok Portal K-K'

D. Momen Nominal Aktual Balok Negatif

- Untuk bentang N-K'

$$\rho_{aktual} = \frac{759,88}{350.600} = 0,0036$$

$$Rn = \rho.fy(1-1/2.\rho.m) = 0,0036.400.(1\text{-}1/2.0,0036.20,915) = 1,3926$$

$$Mnak^{-} = Rn.b.d^2 = 1,3926.350.600^2.10^{-6} = 175,47 \text{ KNm}$$

- Untuk bentang K'-K

$$\rho_{aktual} = \frac{759,88}{250.339} = 0,009$$

$$Rn = \rho.fy(1-1/2.\rho.m) = 0,009.400.(1\text{-}1/2.0,009.20,915) = 3,250$$

$$Mnak^{-} = Rn.b.d^2 = 3,250.250.339^2.10^{-6} = 93,378 \text{ KNm}$$

E. Momen Nominal Aktual Balok Positif

- Untuk bentang N-K'

$$\rho_{aktual} = \frac{2659,58}{350.600} = 0,0127$$

$$Rn = \rho.fy(1-1/2.\rho.m) = 0,0127.400.(1\text{-}1/2.0,0127.20,915) = 4,3949$$

$$Mnak^{+} = Rn.b.d^2 = 4,3949.350.600^2 = 553,762 \text{ KNm}$$

- Untuk bentang K'-K

$$\rho_{aktual} = \frac{759,88}{250.339} = 0,009$$

$$Rn = \rho.fy(1-1/2.\rho.m) = 0,009.400.(1\text{-}1/2.0,009.20,915) = 3,250$$

$$Mnak^{+} = Rn.b.d^2 = 3,250.250.339^2.10^{-6} = 93,378 \text{ KNm}$$

4.6.2 Perencanaan Balok Portal Terhadap Geser

A. Perencanaan Geser Balok bentang N-K'

Perencanaan geser balok menggunakan momen kapasitas dari balok yang terpasang, sedangkan rumus perinciannya ada pada landasan teori rumus 3.7.15 dan 3.7.16. Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

$$V_{U,b} = 0{,}7\left[\frac{M_{kap} + M'_{kap}}{Ln}\right] + 1{,}05.Vg$$

$$M_{kap} = \phi_0 . M_{nak,b}$$

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

Tetapi tidak lebih besar dari

$V_{U,b} = 1{,}05\ (\ V_{D,b} + V_{L,b} + 4/k\ V_{E,b}\ )$

$V_D = 140{,}96$ KN $\quad V_L = 38{,}47$ KN $\quad V_E = 37{,}34$ KN

Gaya geser pada muka tumpuan akibat beban kombinasi :

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

$$V_{U,b} = 0{,}7.1{,}25\left[\frac{175{,}47 + 553{,}762}{8{,}4}\right] + 1{,}05.(140{,}96 + 38{,}47) = 264{,}36 \text{ KN}$$

Dengan syarat tidak lebih besar dari :

$V_{U,b} = 1{,}05\ (\ 140{,}96 + 38{,}47 + 4/1\ .\ 37{,}34\ ) = 345{,}23$ KN

$V_{U,b}$ pada daerah sejauh 2,7 dari tumpuan yaitu dimana balok anak menumpu :

$$\left[1{,}05Vg - 0{,}7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] +$$

$$\frac{Ln-2,7}{Ln}\left[Vu,b-\left[1,05Vg-0,7\phi_0\left(\frac{Mnak,b+Mnak,b'}{Ln}\right)\right]\right]$$

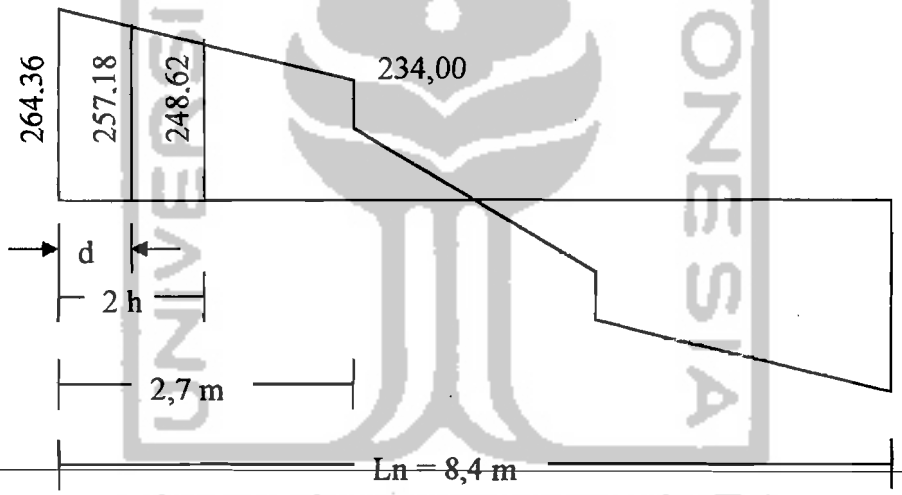
$$[1,05.178,43-75,962]+\frac{8,4-2,7}{8,4}[264,36-169,92]=234,00\ \text{KN}$$

$V_{u,b}$ pada daerah sejauh d :

$$\frac{264,36.2,061+0,600.234,00}{2,7}=257,18\ \text{KN}$$

$V_{u,b}$ pada daerah sejauh 2h :

$$\frac{264,36.1,3+1,4.234,00}{2,7}=248,62\ \text{KN}$$



Gambar 4.22 Diagram Gaya Geser Balok bentang N-K'

- Dalam daerah sendi plastis

  $V_{U,b}$ = 257,18 KN

  Vc = 0

  $$Vs=\frac{Vu,b}{\phi}=\frac{257,18}{0,6}=428,633\ \text{KN}$$

  $$S=\frac{Av.fy.d}{Vs}=\frac{2.157.240.600}{428,633.10^3}=112,35\ \text{mm}$$

$\frac{d}{4} = 159{,}75$ mm

dipakai $P_{10\text{-}100}$

- Diluar sendi plastis

$V_{U,b} = 248{,}62$ KN

$$Vc = \frac{1}{6}.\sqrt{fc'}.b.d = \frac{1}{6}\sqrt{22{,}5}.350.600 = 176{,}81\,KN$$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{248{,}62}{0{,}6} - 176{,}81 = 237{,}56\,KN$$

Syarat spasi

$S \leq d/2 = 319{,}5$ mm

$S \leq 600$ mm

$$S \leq \frac{Av.fy.d}{Vs} = \frac{2.157.240.600}{237{,}56.10^3} = 202{,}71\,mm$$

Dipakai $P_{10\text{-}200}$

B. Perencanaan Geser Balok bentang K'-K

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

$$V_{U,b} = 0{,}7\left[\frac{M_{kap} + M'_{kap}}{Ln}\right] + 1{,}05.Vg$$

$$M_{kap} = \phi_0.M_{nak,b}$$

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

Tetapi tidak lebih besar dari

$V_{U,b} = 1{,}05\ (\ V_{D,b} + V_{L,b} + 4/k\ V_{E,b}\ )$

V_D = 17,83 KN V_L = 4,93 KN V_E = 45,95 KN

Gaya geser balok pada muka tumpuan akibat beban kombinasi :

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

$$V_{U,b} = 0,7.1,25\left[\frac{93,378 + 93,378}{2}\right] + 1,05.(17,83 + 4,93) = 105,604 \text{ KN}$$

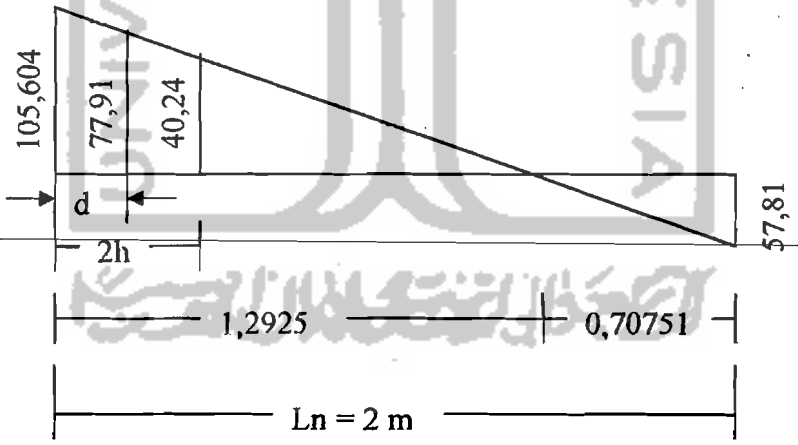
Dengan syarat tidak lebih besar dari :

$V_{U,b}$ = 1,05 (17,83 + 4,93 + 4/1 . 45,95) = 216,89 KN

Gaya geser balok pada muka tumpuan sisi yang lain :

$$\left[1,05Vg - 0,7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] = 1,05.22,76 - 81,706 = -57,81 \text{ KN}$$

maka bentuk diagram gesernya adalah :



Gambar 4.23 Diagram Gaya Geser Balok bentang K'-K

$V_{U,b}$ pada daerah sejauh d dari muka tumpuan :

$$\frac{1,2925 - 0,339}{1,2925}.105,604 = 77,91 \text{ KN}$$

$V_{U,b}$ pada daerah sejauh 2h dari muka tumpuan :

$$\frac{1,2925-0,8}{1,2925}.105,604 = 40,24 \text{ KN}$$

- Dalam daerah sendi plastis

  $V_{U,b}$ = 77,91 KN

  Vc = 0

  $$Vs = \frac{Vu,b}{\phi} = \frac{77,91}{0,6} = 129,85 \text{ KN}$$

  $$S = \frac{Av.fy.d}{Vs} = \frac{2.157.240.339}{129,85.10^3} = 196,75 \text{ mm}$$

  $$\frac{d}{4} = 84,75 \text{ mm}$$

  dipakai $P_{10\text{-}80}$

- Diluar sendi plastis

  $V_{U,b}$ = 40,24 KN

  $$Vc = \frac{1}{6}.\sqrt{fc'}.b.d = \frac{1}{6}\sqrt{22,5}.250.339 = 67 \text{ KN}$$

  $$Vs = \frac{Vu,b}{\phi} - Vc = \frac{40,24}{0,6} - 67 = 0,0717 \text{KN}$$

  Syarat spasi

  S ≤ d/2 = 169,5 mm

  S ≤ 600 mm

  $$S \leq \frac{Av.fy.d}{Vs} = \frac{2.157.240.339}{0,0717.10^3} = 356,3 \text{ mm}$$

  Dipakai $P_{10\text{-}150}$

Cek Pengaruh Torsi untuk balok bentang N – K'

Momen Torsi terfaktor Tu = 26,37 KNm

$$Tu_{maks} = \phi\left[\left(\frac{1}{20}\sqrt{f'c}\right)\sum x^2 y\right] = 0{,}6\left[\left(\frac{1}{20}\sqrt{22{,}5}\right)350^2 700\right] = 12{,}202 \text{ KNm}$$

Tu = 26,37 KNm > 12,202 KNm maka diperlukan tulangan torsi

Karena merupakan torsi keserasian, menurut SK-SNI boleh direncanakan terhadap momen torsi sebagai berikut :

$$Tu = \phi\left(\frac{1}{3}\sqrt{f'c}\right)\sum\frac{1}{3}x^2 y = 0{,}6\left(\frac{1}{3}\sqrt{22{,}5}\right)\sum\frac{1}{3}350^2 700 = 27{,}117 \text{ KNm} > \text{Tu} =$$

26,37 KNm

dipakai Tu = 26,37 KNm

❖ Untuk daerah dalam sendi plastis

Vu = 257,18 KN

Sebagai tulangan geser dan tulangan torsi digunakan sengkang ϕ 10 mm

- Perencanaan sengkang torsi

d = 600 mm

$$Ct = \frac{b.d}{\sum x^2.y} = \frac{350.600}{350^2.700} = 0{,}00286/\text{mm}$$

sumbangan beton dalam menahan torsi :

$$Tc = \frac{\left(\frac{1}{15}\sqrt{f'c}\right)\sum x^2 y}{\sqrt{1+\left(\frac{0{,}4.Vu}{Ct.Tu}\right)^2}} = \frac{\left(\frac{1}{15}\sqrt{22{,}5}\right)350^2 700}{\sqrt{1+\left(\frac{0{,}4.257{,}18.10^3}{0{,}00286.26{,}37.10^6}\right)^2}} = 16{,}022 \text{ KNm}$$

torsi yang ditahan tulangan torsi :

$$Ts = \frac{Tu}{\phi} - Tc = \frac{26,37}{0,6} - 16,022 = 27,928 \text{ KNm}$$

$x_1 = 350 - 2\ (pb + 0,5\ \phi sengkang\ ) = 260$ mm

$y_1 = 700 - 2\ (pb + 0,5\ \phi sengkang\ ) = 610$ mm

$$\alpha_t = \frac{1}{3}\left(2 + \frac{y_1}{x_1}\right) = \frac{1}{3}\left(2 + \frac{610}{260}\right) = 1,45 < 1,5$$

$$\frac{At}{s} = \frac{Ts}{\alpha_t . x_1 . y_1 . fy} = \frac{27,928.10^6}{1,45.260.610.400} = 0,3039 \text{ mm}^2 \text{ / mm jarak / kaki}$$

- Perencanaan sengkang geser

$$\frac{Vu}{\phi} = \frac{257,18}{0,6} = 428,633 \text{ KN}$$

sumbangan beton dalam menahan geser :

$$Vc = \frac{\frac{1}{6}\sqrt{f'c}.b.d}{\sqrt{1 + \left(2,5.Ct.\frac{Tu}{Vu}\right)^2}} = \frac{\frac{1}{6}\sqrt{22,5}.350.600}{\sqrt{1 + \left(2,5.0,00286\frac{26,37.10^6}{257,18.10^3}\right)^2}} = 142,645 \text{ KN}$$

geser yang ditahan tulangan geser :

$$Vs = \frac{Vu}{\phi} - Vc = 428,633 - 142,645 = 285,986 \text{ KN}$$

$$\frac{Av}{s} = \frac{Vs}{fy.d} = \frac{285,986.10^3}{400.600} = 1,119 \text{ mm}^2 \text{ / mm jarak / dua kaki}$$

- Perencanaan tulangan geser dan torsi (gabungan)

$$\frac{Avt}{s} = 2\frac{At}{s} + \frac{Av}{s} = 2.0,3039 + 1,119 = 1,727 \text{ mm}^2$$

dipakai sengkang 10 mm, dengan luas dua kaki As = 157 mm^2

jarak sengkang : $s = \frac{As}{\frac{Avt}{s}} = \frac{157}{1,727} = 90,928$ mm

jarak sengkang maksimum $\frac{1}{4}(x_1 + y_1) = \frac{1}{4}(260 + 610) = 217,5$ mm $> 90,928$ mm

dipakai s = 80 mm

luas sengkang minimum perlu :

$$At + 2.Av = \frac{b.s}{3.fy} = \frac{350.80}{3.400} = 23,33 \text{ mm}^2 < 157 \text{ mm}^2$$

dipakai sengkang P_{10-80}

❖ Untuk daerah diluar sendi plastis

Vu = 248,62 KN

Sebagai tulangan geser dan tulangan torsi digunakan sengkang ϕ 10 mm

- Perencanaan sengkang torsi

d = 600 mm

$$Ct = \frac{b.d}{\sum x^2.d} = \frac{350.600}{350^2.600} = 0,00286 / \text{mm}$$

sumbangan beton dalam menahan torsi :

$$Tc = \frac{\left(\frac{1}{15}\sqrt{f'c}\right)\sum x^2 y}{\sqrt{1+\left(\frac{0,4.Vu}{Ct.Tu}\right)^2}} = \frac{\left(\frac{1}{15}\sqrt{22,5}\right)350^2 700}{\sqrt{1+\left(\frac{0,4.248,62.10^3}{0,00286.26,37.10^6}\right)^2}} = 16,375 \text{ KNm}$$

torsi yang ditahan tulangan torsi :

$$Ts = \frac{Tu}{\phi} - Tc = \frac{26,37}{0,6} - 16,375 = 27,575 \text{ KNm}$$

x_1 = 350 – 2 (pb + 0,5 ϕsengkang) = 260 mm

y_1 = 700 – 2 (pb + 0,5 ϕsengkang) = 610 mm

$$\alpha_t = \frac{1}{3}\left(2 + \frac{y_1}{x_1}\right) = \frac{1}{3}\left(2 + \frac{610}{260}\right) = 1{,}45 < 1{,}5$$

$$\frac{At}{s} = \frac{Ts}{\alpha_t . x_1 . y_1 . fy} = \frac{27{,}38.10^6}{1{,}45.260.610.400} = 0{,}300 \text{ mm}^2 \text{ / mm jarak / kaki}$$

- Perencanaan sengkang geser

$$\frac{Vu}{\phi} = \frac{248{,}62}{0{,}6} = 414{,}367 \text{ KN}$$

sumbangan beton dalam menahan geser :

$$Vc = \frac{\frac{1}{6}\sqrt{f'c}.b.d}{\sqrt{1 + \left(2{,}5.Ct.\frac{Tu}{Vu}\right)^2}} = \frac{\frac{1}{6}\sqrt{22{,}5}.350.600}{\sqrt{1 + \left(2{,}5.0{,}00286\frac{26{,}37.10^6}{248{,}62.10^3}\right)^2}} = 140{,}933 \text{ KN}$$

geser yang ditahan tulangan geser :

$$Vs = \frac{Vu}{\phi} - Vc = 414{,}367 - 140{,}933 = 273{,}437 \text{ KN}$$

$$\frac{Av}{s} = \frac{Vs}{fy.d} = \frac{273{,}437.10^3}{400.600} = 1{,}07 \text{ mm}^2 \text{ / mm jarak / dua kaki}$$

- Perencanaan tulangan geser dan torsi (gabungan)

$$\frac{Avt}{s} = 2\frac{At}{s} + \frac{Av}{s} = 2.0{,}300 + 1{,}070 = 1{,}67 \text{ mm}^2$$

dipakai sengkang 10 mm, dengan luas dua kaki As = 157 mm^2

jarak sengkang : $s = \frac{As}{\frac{Avt}{s}} = \frac{157}{1{,}670} = 94{,}020$ mm

jarak sengkang maksimum $\frac{1}{4}(x_1 + y_1) = \frac{1}{4}(260 + 610) = 217{,}5$ mm > 94,020 mm

dipakai s = 80 mm

luas sengkang minimum perlu :

$$At + 2.Av = \frac{b.s}{3.fy} = \frac{350.80}{3.400} = 23{,}33 \text{ mm}^2 < 157 \text{ mm}^2$$

dipakai sengkang P_{10-80}

Cek Pengaruh Torsi untuk balok bentang K' - K

Momen Torsi terfaktor Tu = 0,3041 KNm

$$Tu_{maks} = \phi\left[\left(\frac{1}{20}\sqrt{f'c}\right)\sum x^2 y\right] = 0{,}6\left[\left(\frac{1}{20}\sqrt{22{,}5}\right)250^2.400\right] = 3{,}558 \text{ KNm}$$

Tu = 0,3041 KNm < 3,558 KNm maka torsi tidak diperhitungkan.

4.6.3 Perencanaan Kolom Portal terhadap Lentur dan Aksial

Di bawah ini akan diberikan contoh perhitungan kolom tipe KA1 pada portal As K[1]-8 lantai 2 dengan mengacu pada landasan teori rumus 3.6.1 s/d 3.6.23 dan juga rumus 3.7.6 s/d 3.7.19.

Diketahui data-data sebagai berikut :

hk = 0,60 m h = 4 m

bk = 0,45 m hn = 3,3 m

ωd = 1,3 Rv = 1

ϕ_o = 1,25 K = 1

1. Momen-momen :

Join atas

$M_{nak,bx\text{-}ki}$ = 741,086 kNm $l_{x\text{-}ki}$ = 9 m $l_{nx\text{-}ki}$ = 8,4 m

$M_{nak,bx\text{-}ka}$ = 186,756 kNm $l_{x\text{-}ka}$ = 2,5 m $l_{nx\text{-}ka}$ = 2 m

$M_{nak,by\text{-}ki}$ = 375,744 kNm $l_{y\text{-}ki}$ = 4,5 m $l_{ny\text{-}ki}$ = 4,05 m

$M_{nak,by\text{-}ka}$ = 375,744 kNm $l_{y\text{-}ka}$ = 4 m $l_{ny\text{-}ka}$ = 3,55 m

Join Bawah

$M_{nak,bx\text{-}ki}$ = 741,086 kNm $l_{x\text{-}ki}$ = 9 m $l_{nx\text{-}ki}$ = 8,4 m

$M_{nak,bx\text{-}ka}$ = 186,756 kNm $l_{x\text{-}ka}$ = 2,5 m $l_{nx\text{-}ka}$ = 2 m

$M_{nak,by\text{-}ki}$ = 375,744 kNm $l_{y\text{-}ki}$ = 4,5 m $l_{ny\text{-}ki}$ = 4,05 m

$M_{nak,by\text{-}ka}$ = 375,744 kNm $l_{y\text{-}ka}$ = 4 m $l_{ny\text{-}ka}$ = 3,55 m

2. Gaya Aksial

$N_{D,k}$ = 660,91 kN

$N_{L,k}$ = 135,39 kN

A. Perhitungan Momen dan Gaya Aksial Rencana

1. Momen Rencana

$$k_a = \frac{EI_{k-a}}{h_{n,k-a}} = \frac{73211,5385}{2,6} = 28158,2840 \text{ kNm}$$

$$k_b = \frac{EI_{k-b}}{h_{n,k-b}} = \frac{73211,5385}{2,6} = 28158,2840 \text{ kNm}$$

$$\alpha_a = \frac{k_a}{k_a + k_b} = \frac{28158,2840}{28158,2840 + 28158,2840} = 0,5$$

$$\alpha_b = \frac{k_b}{k_a + k_b} = \frac{28158,2840}{28158,2840 + 28158,2840} = 0,5$$

menghitung momen rancang kolom :

Join atas

$$M_{u,kx} = \frac{hn}{h} 0,7.\omega_d.\alpha.\phi_o \left(\Sigma \frac{l}{l_{nx}} M_{nak,bx} \right)$$

$$= \frac{3,3}{4}.0,7.1,3.0,5.1,25 \left(\frac{9}{8,4}.741,086 + \frac{2,5}{2}.186,756 \right)$$

$$= 482,1032 \text{ kNm}$$

tetapi tidak perlu lebih dari :

$$M_{u,kx} = 1,05.(M_{D,kx} + M_{L,kx} + \frac{4}{k} (M_{E,kx} \pm 0,3.M_{E,ky}))$$

$$= 1,05.((-77,79)+(-28,23) + \frac{4}{1}((-132,02) + 0,3.(-107,32)))$$

$$= -813,1788 \text{ kNm}$$

$$M_{u,ky} = \frac{hn}{h} 0,7.\omega_d.\alpha.\phi_o \left(\sum \frac{l_{ka}}{l_{ny}} M_{nak,by} \right)$$

$$= \frac{3,3}{4}.0,7.1,3.0,5.1,25.\left(\frac{4,5}{4,05} 375,744 + \frac{4}{3,55}.375,744 \right)$$

$$= 394,5505 \text{ kNm}$$

tetapi tidak perlu lebih dari :

$$M_{u,ky} = 1,05.(M_{D,ky} + M_{L,ky} + \frac{4}{k} (0,3.M_{E,kx} \pm M_{E,ky})$$

$$= 1,05 ((-6,47)+(-2,47) + \frac{4}{1}(0,3.(-132,02) + (-107,32)))$$

$$= -626,4762 \text{ kNm}$$

Join bawah

$$M_{u,kx} = \frac{hn}{h} 0,7.\omega_d.\alpha.\phi_o \left(\sum \frac{l}{l_{nx}} M_{nak,bx} \right)$$

$$= \frac{3,3}{4}.0,7.1,3.0,5.1,25 \left(\frac{9}{8,4}.741,086 + \frac{2,5}{2}.186,756 \right)$$

$$= 482,1062 \text{ kNm}$$

tetapi tidak perlu lebih dari :

$$M_{u,kx} = 1,05.(M_{D,kx} + M_{L,kx} + \frac{4}{k} (M_{E,kx} \pm 0,3.M_{E,ky})$$

$$= 1,05 ((72,29)+(27,66) + \frac{4}{1}(121,67 + 0,3.90,40))$$

$$= 729,8655 \text{ kNm}$$

$$M_{u,ky} = \frac{hn}{h} 0,7.\omega_d.\alpha.\phi_o \left(\sum \frac{l_{ka}}{l_{ny}} M_{nak,by} \right)$$

$$= \frac{3,3}{4}.0,7.1,3.0,5.1,25.\left(\frac{4,5}{4,05} 375,744 + \frac{4}{3,55}.375,744 \right)$$

$$= 394,5505 \text{ kNm}$$

tetapi tidak perlu lebih dari :

$$M_{u,ky} = 1,05.(M_{D,ky} + M_{L,ky} + \frac{4}{k} (0,3.M_{E,kx} \pm M_{E,ky})$$

$$= 1,05 ((6,44)+(2,63) + \frac{4}{1}(0,3.121,67 + 90,40))$$

$$= 542,5077 \text{ kNm}$$

2. Gaya Aksial Rencana

Join atas

$$N_{u,k_x} = 0,7.Rv.\phi_o.\left(\sum \frac{M_{nak,bx}}{l_x} + 0,3.\sum \frac{M_{nak,by}}{l_y} \right) + 1,05.Ng$$

$$= 0,7.1.1,25.\left[\left(\frac{741,086}{9} + \frac{186,756}{2,5} \right) + 0,3.\left(\frac{375,744}{4,5} + \frac{375,744}{4} \right) \right]$$

$$+ 1,05.(660,91 + 135,39)$$

$$= 1020,1062 \text{ kN}$$

tetapi tidak perlu lebih besar dari :

$$N_{u,kx} = 1,05 (N_{D,k} + N_{L,k} + \frac{4}{k}.(N_{E,kx} \pm 0,3.N_{E,ky})$$

$$= 1,05 ((-634,99)+(-135,39) + \frac{4}{1}((-21,27) + 0,3.(-9,39)))$$

= -910,0392 kNm

$$N_{u,ky} = 0,7.Rv.\phi_o.\left(0,3.\sum\frac{M_{nak,bx}}{l_x}+\sum\frac{M_{nak,by}}{l_y}\right) + 1,05.\ Ng$$

$$= 0,7.1.1,25.\left[0,3.\left(\frac{741,086}{9}+\frac{186,756}{2,5}\right)+\left(\frac{186,756}{4,5}+\frac{186,756}{4}\right)\right]$$

$$+ 1,05.(\ 660,91 + 135,39\ )$$

= 1032,5947 kN

tetapi tidak perlu lebih besar dari :

$$N_{u,ky} = 1,05\ (\ N_{D,k} + N_{L,k} + \frac{4}{k}.(\ 0,3.N_{E,kx} \pm N_{E,ky})$$

$$= 1,05\ ((-634,99)+(-135,39) + \frac{4}{1}(0,3.(-21,27)+(-9,37)))$$

= -875,0532 kNm

Join bawah

$$N_{u,kx} = 0,7.Rv.\phi_o.\left(\sum\frac{M_{nak,bx}}{l_x}+0,3.\sum\frac{M_{nak,by}}{l_y}\right) + 1,05.\ Ng$$

$$= 0,7.1.1,25.\left[\left(\frac{741,086}{9}+\frac{186,756}{2,5}\right)+0,3.\left(\frac{375,744}{4,5}+\frac{375,744}{4}\right)\right]$$

$$+ 1,05.(\ 660,91 + 135,39\ )$$

= 1020,1062 kN

tetapi tidak perlu lebih besar dari :

$$N_{u,kx} = 1,05\ (\ N_{D,k} + N_{L,k} + \frac{4}{k}.(N_{E,kx} \pm 0,3.N_{E,ky})$$

$$= 1{,}05\ ((-660{,}91)+(-135{,}39) + \frac{4}{1}((-21{,}27)+0{,}3.(-9{,}37)))$$

$$= -937{,}2552 \text{ kNm}$$

$$N_{u,ky} = 0{,}7.Rv.\phi_o.\left(0{,}3.\sum\frac{M_{nak,bx}}{l_x} + \sum\frac{M_{nak,by}}{l_y}\right) + 1{,}05.\ Ng$$

$$= 0{,}7.1.1{,}25.\left[0{,}3.\left(\frac{741{,}086}{9} + \frac{186{,}756}{2{,}5}\right) + \left(\frac{186{,}756}{4{,}5} + \frac{186{,}756}{4}\right)\right]$$

$$+ 1{,}05.(\ 660{,}91 + 135{,}39\ )$$

$$= 1032{,}5947 \text{ kN}$$

tetapi tidak perlu lebih besar dari :

$$N_{u,ky} = 1{,}05\ (\ N_{D,k} + N_{L,k} + \frac{4}{k}.(\ 0{,}3.N_{E,kx} \pm N_{E,ky})$$

$$= 1{,}05\ ((-660{,}91)+(-135{,}39) + \frac{4}{1}(0{,}3.(-21{,}27)+(-9{,}37)))$$

$$= -902{,}2692 \text{ kNm}$$

B. Perencanaan Tulangan Kolom

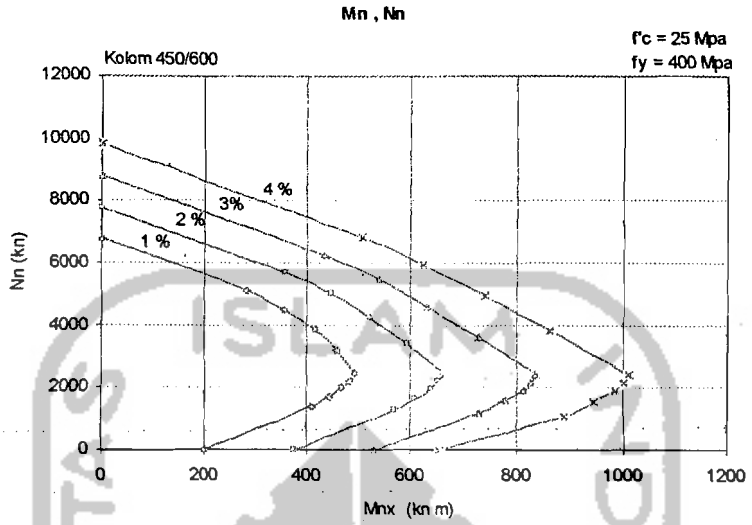
$$N_n = \frac{Nu,k}{\phi} = \frac{937{,}2552}{0{,}65} = 1441{,}9311 \text{ kN}$$

$$M_{n,x} = \frac{Mu,kx}{\phi} = \frac{482{,}1062}{0{,}65} = 741{,}7018 \text{ kNm}$$

$$M_{n,y} = \frac{Mu,kx}{\phi} = \frac{394{,}5505}{0{,}65} = 607{,}0008 \text{ kNm}$$

Kolom mengalami lentur dua arah (arah x dan arah y), sehingga kolom direncanakan terhadap kedua arah tersebut.

1. Arah X



Gambar 4.25 Diagram Interaksi M_{nx}-N_n

ρ_g = 1,6%

As,t = 0,016.450.600 = 4320 mm^2

coba pakai tulangan 10D25

As,t ada = 4985,7144 mm^2

As = As' = 0,5.As,t ada = 2455,3571 mm^2

Cek eksentrisitas balance (e_b)

$$cb = \frac{600.d}{600+fy} = \frac{600.537,5}{600+400} = 322,5 \text{ mm}$$

ab = β_1 . cb = 0,85 . 232,5 = 274,125 mm

$$f's = 600\frac{(cb-d')}{cb} = 600\frac{(322,5-62,5)}{322,5} = 483,7209 \text{ MPa} > fy = 400 \text{ MPa}$$

digunakan f's = fy = 400 MPa

Ccb = 0,85.f'.b.ab = 0,85.25.450.274,125 = 2621320,313 N

Csb = As'(fs'- 0,85.f'c) = 2495,8572.(400 – 0,85.25) = 929966,518 N

Tsb = As.fy = 2495,8572.400 = 982142,857 N

Nnb = Ccb + Csb – Tsb = 2621320,313 + 929966,518 – 982142,857

= 2569,144 kN

$$Mnb = \text{Ccb}\left[\frac{h}{2}-\frac{ab}{2}\right]+\text{Csb}\left(\frac{h}{2}-d'\right)+\text{Tsb}\left(d-\frac{h}{2}\right)$$

$$= 2621320{,}313.\left[\frac{450}{2}-\frac{274{,}125}{2}\right]+929966{,}518.\left(\frac{600}{2}-62{,}5\right)$$

$$+\ 982142{,}857.\left(387{,}5-\frac{450}{2}\right)$$

= 939,3602 kNm

$$\text{eb} = \frac{Mnb}{Nnb} = \frac{939{,}3602}{2569{,}144} = 0{,}3656$$

$$\text{e}_x = \frac{M_{n,kx}}{N_{n,k}} = \frac{741{,}7018}{1441{,}9311} = 0{,}5144$$

karena $e_x > e_b$,kolom mengalami patah tarik

Kontrol kekuatan kolom terhadap patah tarik

$$\frac{h-2.e}{2.d} = \frac{600-2.514{,}4}{2.537{,}5} = -0{,}3988$$

$$\text{m} = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}8235$$

$$\rho = \frac{As}{b.d} = \frac{2455{,}3571}{450.537{,}5} = 0{,}0102$$

$$1-\frac{d'}{d} = 1-\frac{62{,}5}{537{,}5} = 0{,}8837$$

$$N_{n,kx} = 0{,}85.\text{f'c.b.d.}\left[\frac{h-2.e}{2.d}+\sqrt{\left(\frac{h-2.e}{2.d}\right)^2+2.m.\rho.\left(1-\frac{d'}{d}\right)}\right]$$

$$= 0{,}85.25.450.537{,}5.\left[-0{,}3988+\sqrt{(0{,}3988)^2+2.18{,}8235.0{,}0102.0{,}8837}\right]$$

$$= 1572{,}7888 \text{ kN} > \frac{N_{u.k}}{\phi} = 1441{,}9311 \text{ kN}$$

Kontrol tegangan pada tulangan desak :

$$a = \frac{N_{n.kx}}{0{,}85.f'c.b} = \frac{1572{,}7888.10^6}{0{,}85.25.450} = 164{,}4746 \text{ mm}$$

$$c = \frac{a}{0{,}85} = \frac{164{,}4746}{0{,}85} = 193{,}4996 \text{ mm}$$

$$fs' = 600.\left(\frac{c-d'}{c}\right) = 600.\left(\frac{193{,}4996-62{,}5}{193{,}4996}\right)$$

$$= 406{,}2011 \text{ Mpa} < fy = 400 \text{ Mpa}$$

* anggapan awal benar, tulangan desak telah luluh

Kontrol momen nominal kolom:

$Cc = 0{,}85.f'c.b.a = 0{,}85.25.450.\ 164{,}4746 = 1572788{,}363$ N

$Cs = As'(fs' - 0{,}85.f'c) = 2455{,}3571\,(400 - 0{,}85.25)$

$= 929966{,}5016$N

$Ts = As.fy = 2455{,}3571.400 = 982142{,}84$ N

$M_{n.ky} = N_{n.ky}.e$

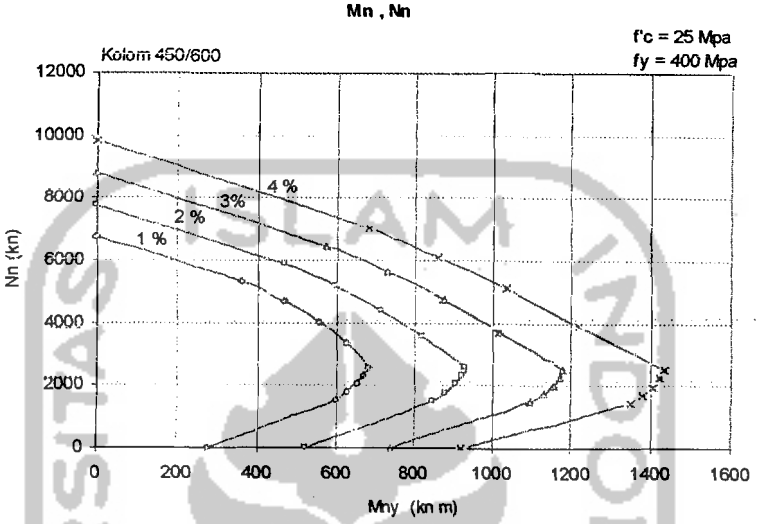
$$= Cc.\left[\frac{h}{2}-\frac{a}{2}\right] + Cs.\left(\frac{h}{2}-d'\right) + Ts.\left(d-\frac{h}{2}\right)$$

$$= 1572788{,}363.\left[\frac{600}{2}-\frac{164{,}4746}{2}\right] + 929966{,}5016.\left(\frac{600}{2}-62{,}5\right)$$

$$+ 982142{,}84.\left(537{,}5-\frac{600}{2}\right)$$

$$= 809{,}0126 \text{ kNm} > \frac{M_{u,ky}}{\phi} = 741{,}7018 \text{ kNm}$$

2. Arah Y



Gambar 4.25 Diagram Interaksi M_{ny}-N_n

$\rho_g = 2\%$

$As,t = 0{,}02.450.600 = 5400 \text{ mm}^2$

coba pakai tulangan 14D25

$As,t \text{ ada} = 6875 \text{ mm}^2$

$As = As' = 0{,}5.As,t \text{ ada} = 3437{,}5 \text{ mm}^2$

Cek eksentrisitas balance (e_b)

$$cb = \frac{600.d}{600 + fy} = \frac{600.387{,}5}{600 + 400} = 232{,}5 \text{ mm}$$

$ab = \beta_1 . cb = 0{,}85 . 232{,}5 = 197{,}625 \text{ mm}$

$$f'sb = 600\frac{(cb - d')}{cb} = 600\frac{(232{,}5 - 62{,}5)}{232{,}5} = 438{,}7096 \text{ MPa} > fy = 400 \text{ MPa}$$

digunakan $f'sb = fy = 400$ MPa

Ccb = 0,85.f'c.b.ab = 0,85.25.600. 197,625 = 2519718,75 N

Csb = As'(fs'- 0,85.f'c) = 3437,5.(400 – 0,85.25) = 1301953,125 N

Tsb = As.fy = 3437,5.400 = 1375000 N

Nnb = Ccb + Csb – Tsb = 2519718,75 + 1301953,125 – 1375000

= 2446,672 N

$$Mnb = \text{Ccb}\left[\frac{h}{2} - \frac{ab}{2}\right] + \text{Csb}\left(\frac{h}{2} - d'\right) + \text{Tsb}\left(d - \frac{h}{2}\right)$$

$$= 2519718{,}75.\left[\frac{450}{2} - \frac{197{,}625}{2}\right] + 1301953{,}125.\left(\frac{450}{2} - 62{,}5\right)$$

$$+ \ 1375000.\left(387{,}5 - \frac{450}{2}\right)$$

= 834,3339 kNm

$$\text{eb} = \frac{Mnb}{Nnb} = \frac{834{,}3339}{2446{,}672} = 0{,}341$$

$$\text{e}_y = \frac{M_{n,ky}}{N_{n,k}} = \frac{607{,}0007}{1441{,}9311} = 0{,}421$$

karena $e_y > e_b$,kolom mengalami patah tarik

Kontrol kapasitas kolom terhadap patah tarik

$$\frac{h - 2.e}{2.d} = \frac{450 - 2.421}{2.387{,}5} = -0{,}5057$$

$$\text{m} = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}8235$$

$$\rho = \frac{As}{b.d} = \frac{3437{,}5}{450.387{,}5} = 0{,}0148$$

$$1 - \frac{d'}{d} = 1 - \frac{62{,}5}{387{,}5} = 0{,}8387$$

$$N_{n,kx} = 0{,}85.f'c.b.d.\left[\frac{h-2.e}{2.d} + \sqrt{\left(\frac{h-2.e}{2.d}\right)^2 + 2.m.\rho.\left(1-\frac{d'}{d}\right)}\right]$$

$$= 0{,}85.25.600.387{,}5.\left[-0{,}5057 + \sqrt{(0{,}5057)^2 + 2.18{,}8235.0{,}0148.0{,}8387}\right]$$

$$= 1701{,}2243 \text{ kN} > \frac{N_{u,k}}{\phi} = 1441{,}9310 \text{ kN}$$

Kontrol tegangan pada tulangan desak :

$$a = \frac{N_{n,ky}}{0{,}85.f'c.b} = \frac{1701{,}2243.10^6}{0{,}85.25.600} = 133{,}4294 \text{ mm}$$

$$c = \frac{a}{0{,}85} = \frac{133{,}4294}{0{,}85} = 156{,}9757 \text{ mm}$$

$$fs' = 600.\left(\frac{c-d'}{c}\right) = 600.\left(\frac{156{,}9757-62{,}5}{156{,}9757}\right)$$

$= 361{,}1096$ Mpa < fy = 400 Mpa

* anggapan awal salah, tulangan desak belum luluh.

Tulangan desak diasumsikan belum luluh, sehingga nilai c dicari dengan persamaan :

$$(0{,}85.f'c.\beta_1.b).c^2 + (600.As' - As.fy).c - 600.As'.d' = 0$$

$$(0{,}85.25.0{,}85.600).c^2 + (600.3437{,}5 - 3437{,}5.400).c - 600.3437{,}5.62{,}5 = 0$$

$c = 81{,}862$ mm , $a = 0{,}85.c = 0{,}85.81{,}862 = 69{,}5827$ mnm

$$fs' = 600.\left(\frac{c-d'}{c}\right) = 600.\left(\frac{81{,}862-62{,}5}{81{,}862}\right)$$

$= 141{,}912$ Mpa < fy = 400 Mpa

fs' = 141,912 Mpa

* anggapan benar, tulangan desak belum leleh

Kontrol momen nominal kolom :

Cc = 0,85.f'c.b.a = 0,85.25.600.69,5827 = 8871779,425 N

Cs = As'(fs'- 0,85.f'c) = 3437,5.(141,912 – 0,85.25) = 414775,625 N

Ts = As.fy = 3422,5714.400 = 1375000 N

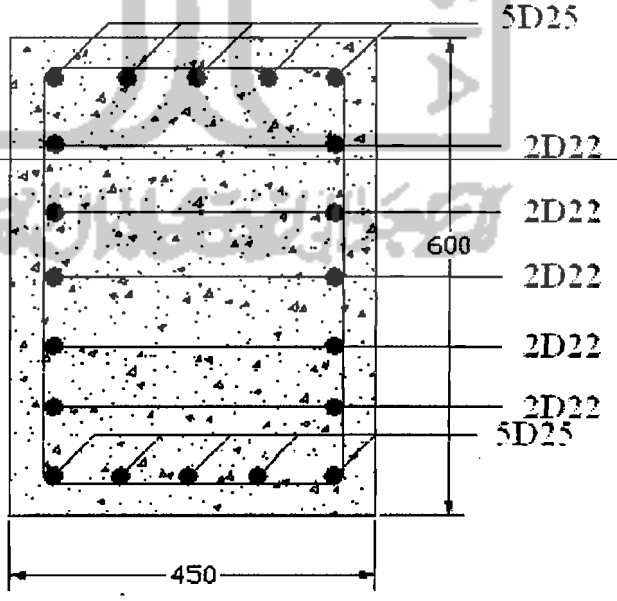
$M_{n,ky} = N_{n,ky}.e$

$$= Cc.\left[\frac{h}{2} - \frac{a}{2}\right] + Cs.\left(\frac{h}{2} - d'\right) + Ts.\left(d - \frac{h}{2}\right)$$

$$= 8871779{,}425.\left[\frac{450}{2} - \frac{69{,}5827}{2}\right] + 414775{,}625.\left(\frac{450}{2} - 62{,}5\right)$$

$$+ \ 1375000.\left(387{,}5 - \frac{450}{2}\right)$$

$$= 883{,}3692 \text{ kNm} > \frac{M_{u,ky}}{\phi} = 607{,}0008 \text{ kNm}$$



Gambar 4.26 Penampang Kolom dengan Tulangan

4.6.4 Perencanaan Kolom Portal terhadap Geser

Sumbu X

$$V_{u,kx} = \frac{Mu,kx_{atas} + Mu,kx_{bawah}}{h_n} = \frac{482{,}1062 + 482{,}1062}{3{,}3} = 292{,}1856 \text{ kN}$$

tetapi tidak perlu lebih besar dari :

$$V_{u,kx} = 1{,}05\,(V_{D,kx} + V_{L,kx} + \frac{4}{k}(V_{E,kx} \pm 0{,}3.V_{E,ky})$$

$$= 1{,}05\,(48{,}6 + 13{,}97 + \frac{4}{1}.(63{,}42 + 0{,}3.49{,}43)$$

$$= 394{,}3443 \text{ kN}$$

Sumbu Y

$$V_{u,ky} = \frac{Mu,ky_{atas} + Mu,ky_{bawah}}{h_n} = \frac{394{,}5505 + 394{,}5505}{3{,}3} = 239{,}1215 \text{ kN}$$

tetapi tidak perlu lebih besar dari :

$$V_{u,ky} = 1{,}05\,(V_{D,ky} + V_{L,ky} + \frac{4}{k}\,(0{,}3.V_{E,kx} \pm V_{E,ky})$$

$$= 1{,}05\,(3{,}23 + 1{,}28 + \frac{4}{1}.(0{,}3.62{,}59 + 63{,}42)$$

$$= 292{,}2507 \text{ kN}$$

$V_{u,k\ mak} = 292{,}1856$ kN

$$V_{u,k} \text{ pakai} = \frac{hn - d}{hn}.V_{u,k}mak = \frac{3{,}3 - 0{,}539}{3{,}3}.292{,}1856 = 244{,}462 \text{ kN}$$

$$Vc = \left(1 + \frac{N_{u,k}}{14.Ag}\right)\left(.\frac{1}{6}.\sqrt{f'c}.b.d\right) = \left(1 + \frac{937255{,}2}{14.600.450}\right)\left(\frac{1}{6}.\sqrt{25}.450.539\right)$$

$$= 50{,}1181 \text{ kN}$$

di luar daerah *lo*

$$Vs = \frac{V_{u,k}\,pakai}{\phi} - Vc = \frac{244{,}462}{0{,}6} - 50{,}1181 = 357{,}3186 \text{ kN}$$

$$\text{Jarak (S)} \quad < \frac{Av.fy.d}{Vs} = \frac{154.240.539}{357{,}3186.10^3} = 95{,}7526 \text{ mm}$$

$< b/4 = 112{,}5$ mm

$< 8.D = 176$ mm

< 100 mm

digunakan sengkang $P_{10\text{-}100}$

di daerah sejauh *lo*

kekuatan dalam menahan gaya geser dianggap 0 (Vc = 0)

$$Vs = \frac{V_{u,k}\,pakai}{\phi} = \frac{244{,}462}{0{,}6} = 407{,}4367 \text{ kN}$$

$$\text{Jarak (S)} \quad < \frac{Av.fy.d}{Vs} = \frac{154.240.539}{407{,}4367.10^3} = 48{,}9 \text{ mm}$$

$< b/4 = 112{,}5$ mm

$< 8.D = 176$ mm

< 100 mm

digunakan sengkang $P_{10\text{-}50}$

Syarat panjang *lo* tidak boleh kurang dari :

- h untuk Nu,k < 0,3.Ag.f'c
- 1,5.h untuk Nu,k > 0,3.Ag.f'c
- 1/6 bentang bersih elemen struktur
- 450 mm

4.6.5 Pertemuan Panel Balok Kolom

Rumus-rumus perencanaan pertemuan balok kolom dapat dilihat pada landasan teori dari rumus 3.7.20 s/d 3.7.33.

1. Perhitungan gaya-gaya dalam

a. Sumbu X

b_j = bc = 450 mm

= b + 0,5.bc = 350 + 0,5.600 = 750 mm

b_j pakai = 450 mm

h_c = 600 mm

$$V_{kol,x} = \frac{0,7.\phi_o.\left(\sum \frac{l_x}{l_{nx}}.M_{nak,bx} + 0,3.\sum \frac{l_y}{l_{ny}}.M_{nak,by}\right)}{\frac{1}{2}.(h_a + h_b)}$$

$$V_{kol,x} = 0,7.1,25.\left[\left(\frac{9}{8,4}.741,086 + \frac{2,5}{2}.186,756\right) + 0,3.\left(\frac{4,5}{4,05}.375,744 + \frac{4}{3,55}.375,744\right)\right] \Big/ \frac{1}{2}.(4+4)$$

= 279,94 kN

$z_{ki,x}$ = 0,9.d = 0,9.639 = 575 mm = 0,575 m

$z_{ka,x}$ = 0,9.d = 0,9.339 = 305 mm = 0,305 m

$C_{ki,x} = T_{ki,x} = 0,7.\phi_o.(M_{nak,bx\text{-}ki})/z_{ki,x}$

= 0,7.1,25.(741,086)/0,575 = 1127,7396 kN

$C_{ka,x} = T_{ka,x} = 0,7.\phi_o.(M_{nak,bx\text{-}ka})/z_{ka,x}$

= 0,7.1,25.(186,756)/0,305 = 535,7754 kN

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x} = 1127{,}7396 + 535{,}7754 - 279{,}94$

$= 1383{,}575$ kN

Kontrol tegangan geser horizontal :

$$v_{jh,x} = \frac{V_{jh,x}}{b_j . h_c} \leq 1{,}5\sqrt{f'c}$$

$$v_{jh,x} = \frac{847{,}7996}{0{,}45.0{,}6} = 3139{,}9985 \text{ kN/m}^2$$

$= 3{,}14 \text{ N/mm}^2 < 1{,}5.\sqrt{25} = 7{,}5 \text{ N/mm}^2$ .....Aman

$$V_{ch,x} = 2/3 \; . \sqrt{\left\{\left(Nu,k/Ag\right) - 0{,}1.f'c\right\}} . b_j . h_c$$

$$V_{ch,x} = 2/3 . \sqrt{\left\{\left(937255{,}5/600.450\right) - 0{,}1.25\right\}} .450.600$$

$= 177{,}3996$ kN

$V_{sh,x} = V_{jh,x} - V_{ch,x}$

$= 1383{,}575 - 177{,}3996 = 1206{,}1754$ kN

b. Arah Y

$b_j = bc = 600$ mm

$= b + 0{,}5.hc = 300 + 0{,}5.450 = 525$ mm

b_j pakai = 525 mm

$h_c = 450$ mm

$$V_{kol,y} = \frac{0{,}7.\phi_o \left(0{,}3.\sum \frac{l_x}{l_{nx}} . M_{nak,bx} + \sum \frac{l_y}{l_{ny}} . M_{nak,by} \right)}{1/2 . (h_a + h_b)}$$

$$V_{kol,y} = 0,7.1,25.\left[0,3.\left(\frac{9}{8,4}.741,086 + \frac{2,5}{2}.186,756\right)\right.$$

$$\left.+\left(\frac{4,5}{4,05}.375,744 + \frac{4}{3,55}.375,744\right)\right] \Big/ \frac{1}{2}.(4+4)$$

$= 251,285$ kN

$z_{ki,y} = 0,9.d = 0,9.389 = 350,1$ mm $= 0,35$ m

$z_{ka,y} = 0,9.d = 0,9.389 = 350,1$ mm $= 0,35$ m

$C_{ki,y} = T_{ki,y} = 0,7.\phi_o.(M_{nak,by\text{-}ki})/z_{ki,y}$

$= 0,7.1,25.(375,744)/0,35 = 939,36$ kN

$C_{ka,y} = T_{ka,y} = 0,7.\phi_o.(M_{nak,by\text{-}ka})/z_{ka,y}$

$= 0,7.1,25.(375,744)/0,35 = 939,36$ kN

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y}$

$= 939,36 + 939,36 - 251,285 = 1627,435$ kN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j.h_c} \le 1,5\sqrt{f'c}$$

$$v_{jh,y} = \frac{688,075}{0,525.0,45} = 2912,4868 \text{ kN/m}^2$$

$= 2,9125$ N/mm^2 $< 1,5.\sqrt{25} = 7,5$ N/mm^2 .....Aman

$$V_{ch,y} = 2/3\ .\sqrt{\left\{\left(Nu,k/Ag\right) - 0,1.f'c\right\}}.b_j.h_c$$

$$V_{ch,y} = 2/3.\sqrt{\left\{\left(937255,2/600.450 - 0,1.25\right)\right\}}.525.450$$

$= 155,2247$ kN

$V_{sh,y} = V_{jh,y} - V_{ch,y}$

$= 1627,435 - 155,2247 = 1472,2103$ kN

2. Penulangan Geser Horisontal

$V_{sh,mak} = V_{sh,y} = 1472,2103$ kN

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{1472210,3}{400} = 3680,5276 \text{ mm}^2$$

Digunakan sengkang rangkap P12 dengan Av = 452 mm^2

$$\text{Jumlah lapis sengkang} = \frac{3680,5276}{452} = 8,14 \text{ lapis}$$

digunakan sengkang rangkap 9P12

3. Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}} V_{jh.mak} \cdot \left(0,6 + {Nu,k}/{Ag.f'c}\right)$$

$$V_{cv} = 1.847,7996.10^3 \left(0,6 + {937,2552.10^3}/{600.450.25}\right)$$

$= 626398,9576$ N $= 626,399$ kN

$V_{jv} = d/h_c . V_{jh,mak}$

$= 0,7/0,6 . 847,7996 = 989,0995$ kN

$V_{sv} = V_{jv} - V_{cv} = 989,0995 - 626,399 = 362,7005$ kN

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{362700,5}{400} = 906,7513 \text{ mm}^2$$

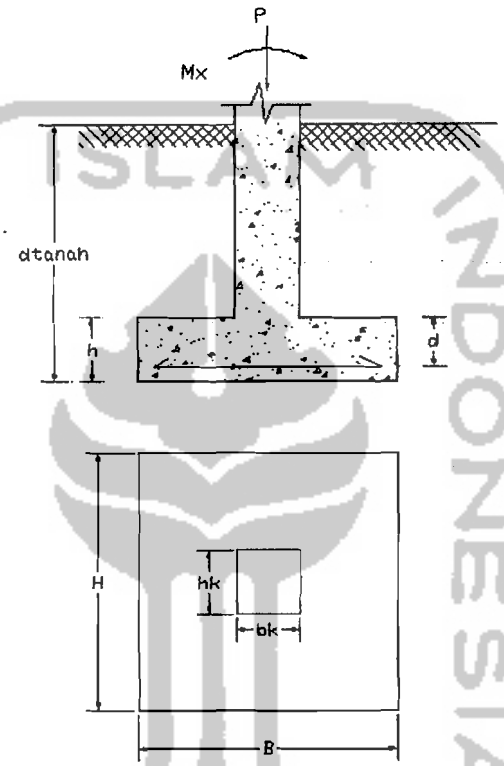
Tulangan lentur kolom terpasang sudah dapat menahan gaya geser vertikal sehingga tidak diperlukan tambahan tulangan geser vertikal.

4.7 Perencanaan Pondasi

4.7.1 Perencanaan Pondasi Telapak Setempat (P1)

A. Perencanaan Dimensi Pondasi

1. Tinjauan Terhadap Beban Tetap



Gambar 4.27 Penampang Pondasi Telapak Setempat

σ tanah	= 350 kN/m^2	γbtanah	= 16.68 KN/m^3
F'c	= 25 Mpa	γbeton	= 24 KN/m^3
Fy	= 400 Mpa	Asumsi tebal pondasi (h) = 500 mm	
P	= 931.8058 Kn	. Ukuran kolom :	
Mx	= 26.4437 kNm	hk	= 450 mm
My	= 5.1546 kNm	bk	= 600 mm

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

$= 350 - (0,5.24) - (2,05.16,68)$

$= 303,806 \text{ kN/m}^2$

Dimensi luas pelat pondasi : (terdapat momen yang bekerja pada arah x dan y)

$$\sigma_{\text{netto tanah}} = \frac{P}{A_{perlu}} + \frac{6.My}{By^2.Bx} + \frac{6.Mx}{Bx^2.By}$$

dicoba dengan nilai By = Bx = 1,9 m

$$A_{\text{perlu}} = \frac{P}{\sigma netto\tan ah - \left(\frac{6.My}{By^2.Bx}\right) - \left(\frac{6.Mx}{Bx^2.By}\right)}$$

$$= \frac{931,8058}{303,806 - \left(\frac{6.5,1546}{1.9^2.1.9}\right) - \left(\frac{6.26,4437}{1,9^2.1,9}\right)} = 3,3741 \text{ m}^2$$

Digunakan penampang bujur sangkar dengan :

$B = H = \sqrt{3,3741} = 1,8369 \text{ m} \longrightarrow B_{ada} = H_{ada} = 1,9 \text{ m}$

Luas penampang pelat pondasi : $A_{ada} = B \times H = 1,9 \times 1,9 = 3,61 \text{ m}^2$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$A_{ada} = 3,61 \text{ m}^2 > A_{perlu} = 3,3741 \text{ m}^2$ **.....Ok.**

Tegangan kontak yang terjadi di dasar pondasi :

$$\sigma_{\text{kontak}} = \frac{P}{A_{ada}} + \frac{6.My}{H^2.B} + \frac{6.Mx}{B^2.H}$$

$$\sigma_{\text{kontak}} = \frac{931,8058}{3,61} + \frac{6.5,1546}{1,9^2.1,9} + \frac{6.26,4437}{1,9^2.1,9}$$

$= 285,7590 \text{ kN/m}^2 < \sigma_{\text{nettotanah}} = 303,806 \text{ kN/ m}^2$ **.....Aman.**

Jarak pusat tulangan tarik ke serat tekan beton :

$d = h - Pb - \frac{1}{2}.\varnothing_{\text{tul. pokok}} = 500 - 75 - \frac{1}{2}.22 = 414 \text{ mm}$

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{26,4437}{931,8058} = 0,0284 \text{ m}$$

$$ey = \frac{My}{P} = \frac{5,1546}{931,8058} = 0,0055 \text{ m}$$

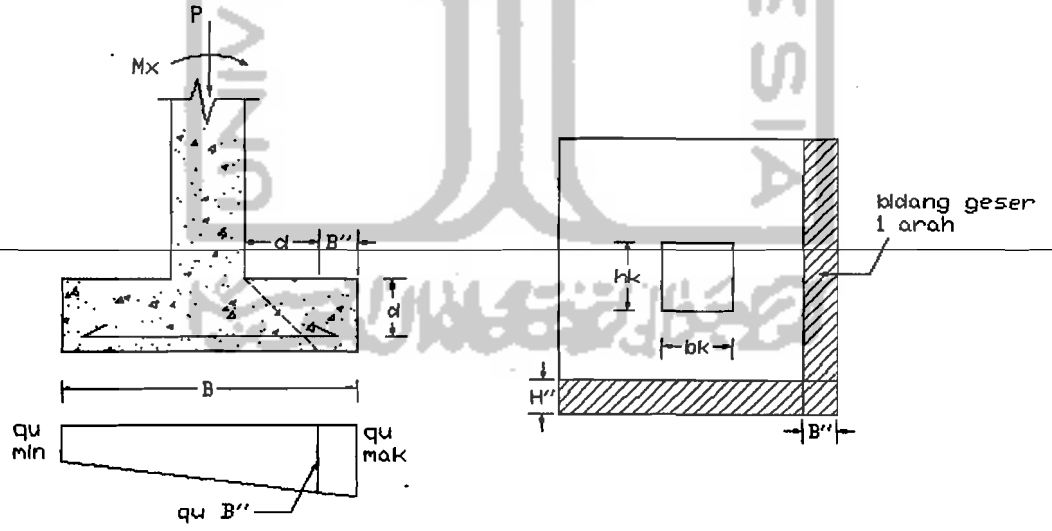
Kontrol tegangan yang terjadi :

$$\sigma = \frac{P}{(H.(B-2.ex))+(B.(H-2.ey))}$$

$$= \frac{931,8058}{(1,9.(1,9-2.0,0284))+(1,9.(1,9-2.0,0055))}$$

$= 131,4043 \text{ kNm} < 1,5.\sigma_{netto} = 1,5.\ 303,806 = 455,709 \text{ kNm}$ **.....Aman.**

B. Perencananaan Geser Satu Arah



Gambar 4.28 Pondasi dengan geser satu arah

→ Ditinjau pada arah momen terbesar .

Pu = 931,8058 kN

Mux = 26,4437 kNm

Muy = 5,1546 kNm

$$H'' = \frac{H - hk - 2.d}{2} = \frac{1{,}9 - 0{,}35 - 2.0{,}414}{2} = 0{,}361 \text{ m}$$

$$B'' = \frac{B - bk - 2.d}{2} = \frac{1{,}9 - 0{,}25 - 2.0{,}414}{2} = 0{,}411 \text{ m}$$

Arah B

- Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{6.Mx}{B^2.H}$$

$$= \frac{931{,}8058}{3{,}61} \pm \frac{6.26{,}4437}{1{,}9^2.1{,}9}$$

$qux_{mak} = 281{,}2499 \text{ kN/m}^2$

$qux_{min} = 234{,}9860 \text{ kN/m}^2$

$$qu_{B''} = \frac{(B - B'').qux_{mak} + B''.qux_{min}}{.B}$$

$$= \frac{(1{,}9 - 0{,}411).281{,}2499 + 0{,}411.234{,}9860}{1{,}9}$$

$= 271{,}2423 \text{ kN/m}^2$

$qux_{pakai} = ½.(qux_{mak} + qux_{B''}) = ½.(281{,}2499 + 271{,}2423)$

$= 276{,}2461 \text{ kN/m}^2$

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

$Vu = qu_{xpakai}.\ H''\ .\ B = 276{,}2461.\ 0{,}361\ .\ 1{,}9 = 189{,}4772 \text{ kN}$

$Vu/\phi = {}^{189{,}4772}/_{0{,}6} = 315{,}7953 \text{ kN}$

• Kekuatan beton menahan geser:

$$Vc = 1/6.\ \sqrt{f'c}\ .\ B.\ d = 1/6.\ \sqrt{25}\ \ 1{,}9.\ 0{,}414\ .\ 10^3 = 655{,}5\ \text{kN}$$

• Kontrol gaya geser :

$$Vc = 655{,}5\ \text{kN} \geq {}^{Vu}/_{\phi} = 315{,}7953\ \text{kN}$$ **.....Aman.**

Arah H

• Tegangan kontak yang terjadi :

$$qux = \frac{P}{A_{ada}} \pm \frac{6.My}{H^2.B}$$

$$= \frac{931{,}8058}{3{,}61} \pm \frac{6.5{,}1546}{1{,}9^2.1{,}9}$$

$$qux_{mak} = 262{,}6270\ \text{kN/m}^2$$

$$qux_{min} = 253{,}6089\ \text{kN/m}^2$$

$$qu_{H"} = \frac{(H - H").qux_{mak} + H".qux_{min}}{H}$$

$$= \frac{(1{,}9 - 0{,}361).262{,}6270 + 0{,}361.253{,}6089}{1{,}9} = 260{,}9136\ \text{kN/m}^2$$

$$qux_{pakai} = ½.\ (qux_{mak} + qux_{H"}) = ½.\ (262{,}6270 + 260{,}9136) = 261{,}7703\ \text{kN/m}^2$$

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

$$Vu = qu_{xpakai}.\ B" .\ H = 192{,}866.\ 0{,}411\ .\ 1{,}9 = 204{,}4164\ \text{kN}$$

$${}^{Vu}/_{\phi} = {}^{204{,}4164}/_{0{,}6} = 340{,}6940\ \text{kN}$$

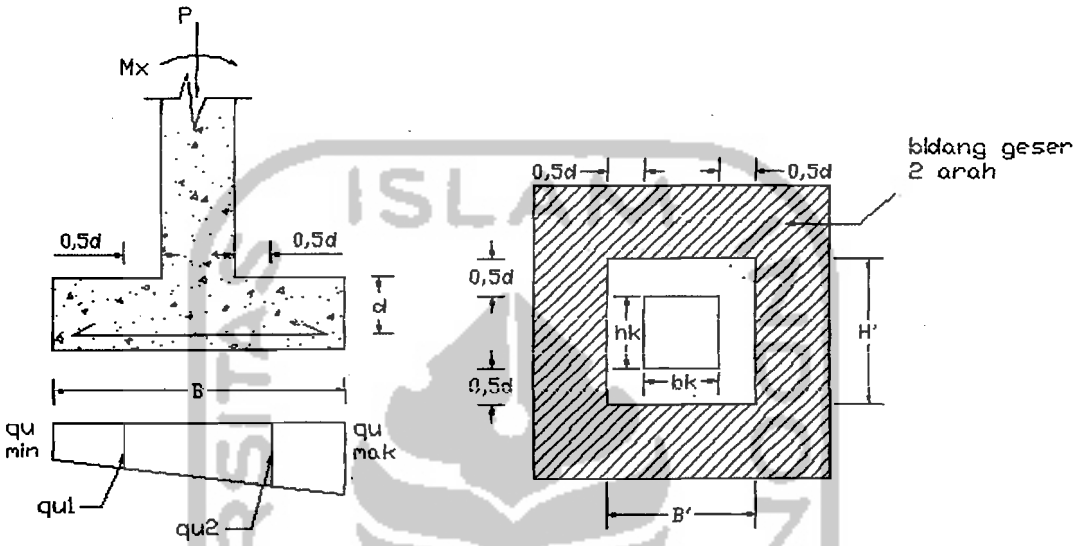
• Kekuatan beton menahan geser:

$$Vc = 1/6.\ \sqrt{f'c}\ .\ H.\ d = 1/6.\ \sqrt{25}\ \ 1{,}9.\ 0{,}414\ .\ 10^3 = 655{,}5\ \text{kN}$$

• Kontrol gaya geser :

Vc = 655,5 kN ≥ Vu/ϕ = 340,6940 kN .....**Aman.**

C. Perencananaan Geser Dua Arah



Gambar 4.29 Pondasi dengan geser dua arah

→ Ditinjau pada arah momen terbesar.

H' = hk + d

= 350 + 414

= 764 mm = 0,764 m

B' = bk + d

= 250 + 414

= 664 mm = 0,664 m

• Tegangan kontak yang terjadi :

$$qu = \frac{P}{A_{perlu}} \pm \frac{6.My}{Bx^2.By} \pm \frac{6.Mx}{By^2.Bx}$$

$$= \frac{931,8058}{3,61} \pm \frac{6.5,1546}{1,9^2.1,9} \pm \frac{6.26,4437}{1,9^2.1,9}$$

qu_{max} = 285,7590 kN/m^2

qu_{min} = 230,4769 kN/m^2

qu_{pakai} = ½ (qu_1 + qu_2) = ½ (275,2554 + 240,9805) = 258,1180 kN/m^2

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

Vu = qu_{pakai}. ((H .B) – (H'.B'))

=258,1180 . ((1,9.1,9) – (0,764 .0,664)) = 800,8636 kN

Vu/ϕ = $800,8636/0,6$ = 1334,7727 kN

• Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{hk}{bk} = \frac{0,35}{0,25} = 1,4$

bo = 2 . (H' + B') = 2 . (764 + 664) = 2856 mm

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

= $(1 + 2/1,4).(2.\sqrt{25}).$ 2856. 414. 10^{-3} = 28715,04 kN

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

= 4. $\sqrt{25}$. 2856. 414. 10^{-3} = 23647,68 kN

• Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc_1 dan Vc_2, yaitu Vc_2 = 23647 KN

Vc_2 = 23647 KN ≥ Vu/ϕ = 1334,7727 kN.....**Aman**.

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) = H . B = 1,9 . 1,9 = 3,61 m^2

Luas penampang kolom (A_1) = hk . bk = 0,35. 0,25 = 0,0875 m^2

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{3{,}61}{0{,}0875}} = 6{,}4232 > 2$ (jika lebih besar dari 2, dipakai nilai 2)

$\phi.Pn = \phi.(0{,}85.\ f'c.\ A_1.2)$

$= 0{,}7.\ (0{,}85.\ 25.\ 87{,}5.2).10^{-3} = 2603{,}1$ kN

• Kuat tumpuan kolom :

$\phi.Pn = \phi.(0{,}85.\ f'c.\ A_1)$

$= 0{,}7.\ (0{,}85.\ 25.\ 87.5).10^{-3} = 1301{,}6$ KN

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 2603{,}1$ kN > $\phi.Pn_{kolom} = 1301{,}6$ kKN **.....Aman.**

E. Perencanaan Tulangan Lentur Pondasi

Arah B

$l = \frac{B - bk}{2} = \frac{1{,}9 - 0{,}25}{2} = 0{,}83$ m

$qu_{mak} = 285{,}7590$ kN/m^2.

$Mu = 0{,}5.\ qu_{mak}.\ l^2 = 0{,}5\ .\ 285{,}7590\ .\ 0{,}83 = 97{,}2474$ kNm

$Mu/\phi = \frac{97{,}2474}{0{,}8} = 121{,}5592$ kNm

- Digunakan tulangan pokok $\varnothing_{13}$ mm, sehingga luas tampang 1 tulangan pokok :

  $A_{1\varnothing} = ¼ . \pi . D^2 = ¼ . \pi . 13^2 = 132{,}665 \text{ mm}^2$

- Tebal pelat pondasi : h = 500 mm, selimut beton (Pb) = 75 mm

  $d = h - Pb - 0{,}5 . \varnothing_{tul.\ pokok} = 500 - 75 - 0{,}5 . 13 = 418{,}5 \text{ mm}$

  $$m = \frac{fy}{0{,}85 . f'c} = \frac{400}{0{,}85 . 25} = 18{,}8235$$

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

  $$Rn = \frac{Mu/\phi}{b.d^2} = \frac{121{,}5592 . 10^6}{1000 . 418{,}5^2} = 0{,}6941 \text{ MPa}$$

  Rasio Tulangan :

  $$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}00350$$

  $$\rho b = \frac{0{,}85 . f'c . \beta_1}{fy} \left(\frac{600}{600 + fy} \right) = \frac{0{,}85 . 25 . 0{,}85}{400} \left(\frac{600}{600 + 400} \right) = 0{,}02709$$

  $$\rho_{mak} = 0{,}75 . \rho b = 0{,}75 . 0{,}02709 = 0{,}02032$$

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

  $$= \frac{1}{18{,}8235} \left(1 - \sqrt{1 - \frac{2 . 18{,}8235 . 0{,}6941}{400}} \right) = 0{,}00176 < \rho_{mak} = 0{,}02032$$

  $$< \rho_{min} = 0{,}00350$$

  $0{,}002 < 1{,}33\rho_{ada} = 0{,}00235 < \rho_{min}$

  sehingga dipakai : $\rho_{perlu} = 1{,}33\rho_{ada} = 0{,}00235$

  $As_{perlu} = \rho_{perlu} . b . d = 0{,}00235 . 1000 . 418{,}5 = 983{,}475 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{132,6650.1000}{983,475} = 134,8941 \text{ mm}$$

$s \leq 2.h = 2 .500 = 1000$ mm

$s \leq 250$ mm

Dipakai Tulangan Pokok : $D_{13} - 130$ mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{132,665.1000}{130} = 1020,5 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{1020,5.400}{0,85.25.1000} = 19,2094 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right)$$

$$= 1020,5 \,.\, 400 \left(418,5 - \frac{19,2094}{2}\right)$$

$$= 166,9111 \text{ kNm} \geq \frac{Mu}{\phi} = 121,5592 \text{ kNm} \text{ .....Aman.}$$

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0,002. b. h = 0,002. 1000. 500 = 1000 \text{ mm}^2$

Digunakan tulangan bagi Ø12 mm, sehingga luas tampang 1 tulangan susut :

$A_{1\varnothing} = ¼. \pi. D^2 = ¼. \pi. 12^2 = 113,04 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113,04.1000}{1000} = 113,04 \text{ mm} \approx 100 \text{ mm}$$

Dipakai Tulangan Susut : $P_{12} - 100$ mm

Arah H

Momen rencana :

$$l = \frac{H - hk}{2} = \frac{1{,}9 - 0{,}35}{2} = 0{,}78 \text{ m}$$

$$qu_{mak} = 285{,}7590 \text{ kN/m}^2$$

$$Mu = 0{,}5.\ qu_{mak}.\ l^2 = 0{,}5\ .\ 285{,}7590\ .\ 0{,}78^2 = 85{,}8170 \text{ kNm}$$

$$Mu/\phi = \frac{85{,}8170}{0{,}8} = 107{,}2713 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{13}$ mm, sehingga luas tampang 1 tulangan pokok :

$$A_{1\varnothing} = \tfrac{1}{4}.\ \pi.\ D^2 = \tfrac{1}{4}.\ \pi.\ 13^2 = 132{,}665 \text{ mm}^2$$

- Tebal pelat pondasi : h = 500 mm, selimut beton (Pb) = 75 mm

$$d = h - Pb - 0{,}5.\ \varnothing_{tul.\ pokok} = 500 - 75 - 0{,}5.\ 13 = 418{,}5 \text{ mm}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}8235$$

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

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{107{,}2712.10^6}{1000.418{,}5^2} = 0{,}6125 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}00350$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}02709$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}02709 = 0{,}02032$$

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

$$= \frac{1}{18,8235}\left(1-\sqrt{1-\frac{2.18,8235.0,6125}{400}}\right) = 0,00155 < \rho_{mak} = 0,02032$$

$$< \rho_{min} = 0,00350$$

$0,002 < 1,33\rho_{ada} = 0,00207 < \rho_{min}$

sehingga dipakai : $\rho_{perlu} = 1,33\rho_{ada} = 0,00207$

$As_{perlu} = \rho_{perlu} . b . d = 0,00207 . 1000 . 418,5 = 866,295 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{132,6650.1000}{866,295} = 153,1407 \text{ mm}$$

$s \leq 2.h = 2 .500 = 1000$ mm

$s \leq 250$ mm

Dipakai Tulangan Pokok : D_{13} – 130 mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{132,665.1000}{130} = 1020,5 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{1020,5.400}{0,85.25.1000} = 19,2094 \text{ mm}$$

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

$= 1020,5 . 400 (418,5 - {}^{19,2094}/_{2})$

$= 166,9111 \text{ kNm} \geq {}^{Mu}/_{\phi} = 121,5592 \text{ kNm}$ **.....Aman.**

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0,002. b. h = 0,002. 1000. 500 = 1000 \text{ mm}^2$

Digunakan tulangan bagi ∅12 mm, sehingga luas tampang 1 tulangan susut :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 12^2 = 113{,}04\ mm^2$

Jarak antar tulangan susut :

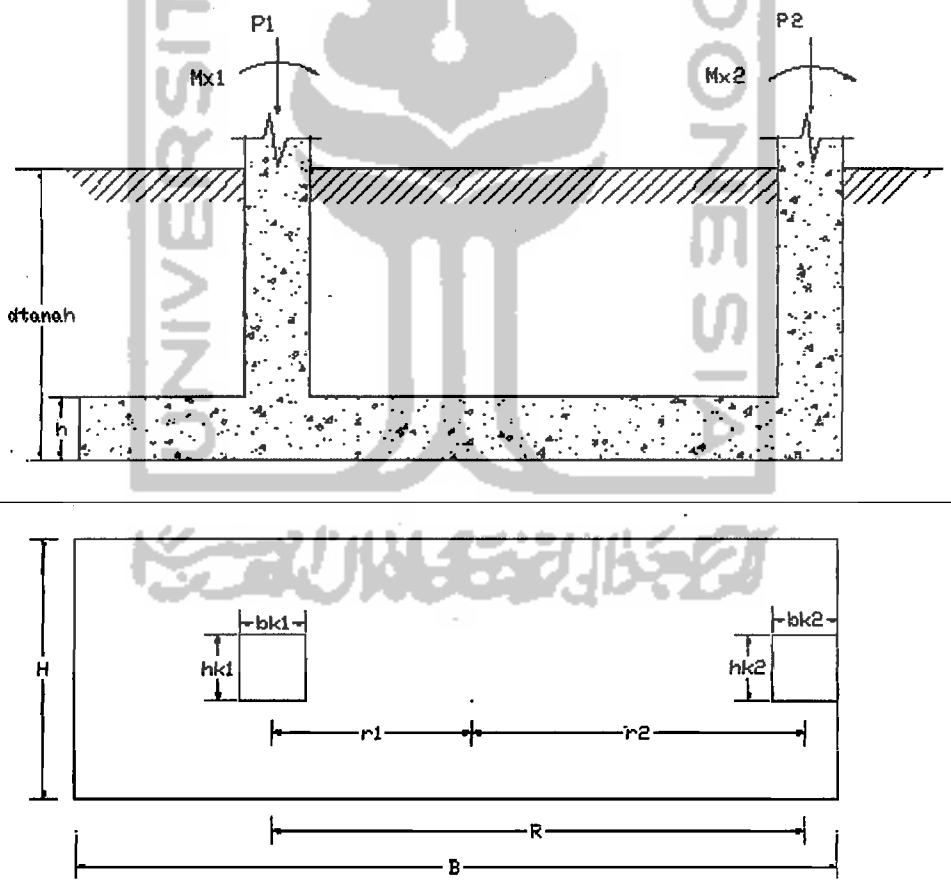
$$s \leq \frac{A_{\varnothing 1}.b}{As_{susut}} = \frac{113{,}04.1000}{1000} = 113{,}04\ mm \approx 100\ mm$$

Dipakai Tulangan Susut : P_{12} – 100 mm

4.7.2 Perencanaan Pondasi Gabungan

A. Perencanaan Dimensi Pondasi

1. Tinjauan Terhadap Beban Tetap



Gambar 4.30 Penampang Pondasi Gabungan

σ tanah = 350 kN/m^2 ybtanah = 16,68 KN/m^3

F'c = 25 Mpa ybeton = 24 KN/m^3

Fy = 400 Mpa Asumsi tebal pondasi (h) = 600 mm

P_1 = 1914,0612 kN Ukuran kolom :

Mx_1 = 144,1934 kNm P1 : 450/600

My_1 = 120,4858 kNm P2 : 400/400

P_2 = 716.3461 kN

Mx_2 = 79,8755 kNm

My_2 = 44,3815 kNm

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

$= 350 - (0,6.24) - (1,95.16,68)$

$= 303,047$ kN/m^2

Ptotal = $P_1 + P_2$ = 1914,0612 + 716,3461 = 2630,4073 kN

Ptotal . $r_1 = P_2 . R$

$r_1 = 716,3461.2,5 / 2630,4073 = 0,6808$ m

$r_2 = 2,5 - 0,6808 = 1,8192$ m

$B = r_2 . 2 + 0,5 . bk_2 = 2 . 1.8192 + 0,5 . 0,4 = 4,0383$

diambil B = 4 m

$Mx_{tot} = 144,1934 + 79,8755 = 224,0689$ kNm

$My_{tot} = 120,4858 + 44,3815 = 164,8673$ kNm

Dimensi luas pelat pondasi : (terdapat momen yang bekerja pada arah x dan y)

$$\sigma_{netto\ tanah} = \frac{P}{B.H} + \frac{6.My}{H^2.B} + \frac{6.Mx}{.B^2.H}$$

dicoba dengan nilai B = 4 m dan H = 3 m

$$\sigma_{netto\ tanah} = \frac{2630,4073}{4.3} + \frac{6.164,8673}{3^2.4} + \frac{6.224,0689}{4^2.3}$$

$$= 277,1538\ kN/m^2 \leq 303,074\ kN/m^2 \text{ ..... } \mathbf{Aman}$$

Jarak pusat tulangan tarik ke serat tekan beton :

$d = h - Pb - \frac{1}{2}.\varnothing_{tul.\ pokok} = 600 - 75 - \frac{1}{2}.22 = 514$ mm

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx_{tot}}{P_{tot}} = \frac{224,0689}{2630,4073} = 0,0852\ m$$

$$ey = \frac{My_{tot}}{P_{tot}} = \frac{164,8673}{2630,4073} = 0,0627\ m$$

Kontrol tegangan yang terjadi :

$$\sigma_{kontak} = \frac{P_{tot}}{(H.(B-2.ex)) + (B.(H-2.ey))}$$

$$= \frac{2630,4073}{(3.(4-2.0,0852)) + (4.(3-2.0,0627))}$$

$= 114,6524$ kNm $< 1,5.\sigma_{netto} = 1,5.\ 303,074 = 454,611$ kNm **..Aman.**

B. Perencananaan Geser Satu Arah

→ Ditinjau pada arah memanjang

P1 = 2421,114 kN P2 = 716,3461 kN

P_{tot} = 2630,4073

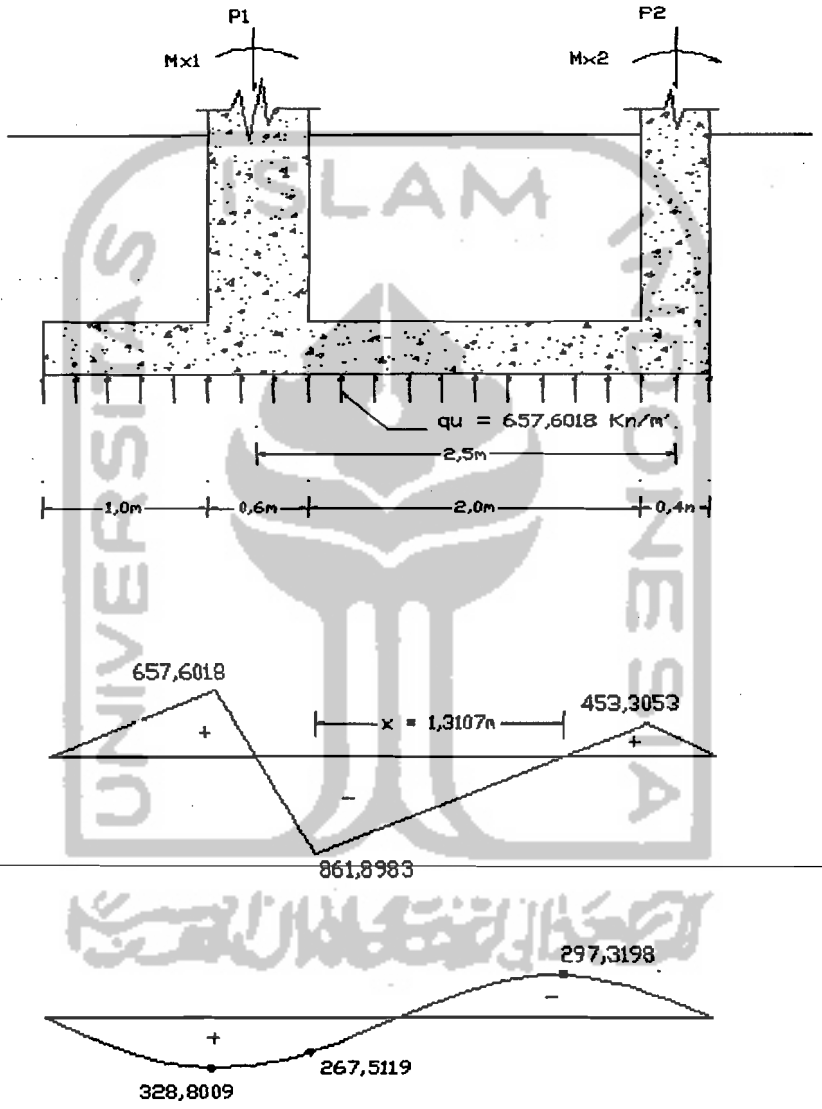
• Tegangan kontak yang terjadi :

$$qu = \frac{P_{tot}}{A_{ada}}$$

$$= \frac{2630,4073}{4.3}$$

qu $= 219,2006 \text{ kN/m}^2$

qu . H $= 219,2006 \cdot 3 = 657,6018 \text{ kN/m}$



Gambar 4.31 Diagram Geser dan Momen

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

Vu_d = Vmak – (qu.H). d = 861,8983 – 657,6018.0.514 = 523,8909 kN

$Vu/\phi = 523,8909/0,6 = 873,1516$ kN

• Kekuatan beton menahan geser:

$$Vc = 1/6.\ \sqrt{f'c}\ .\ B.\ d = 1/6.\ \sqrt{25}\ 4.\ 0{,}514\ .\ 10^3 = 1285\ kN$$

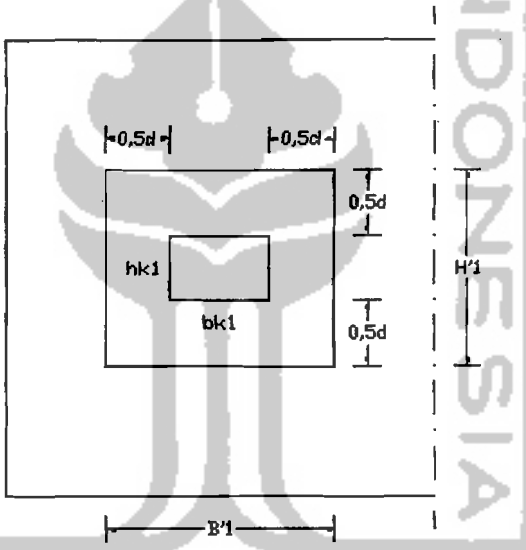
• Kontrol gaya geser :

$$Vc = 1285\ kN \geq {}^{Vu}/_{\phi} = 873{,}1516\ kN$$ **.....Aman.**

C. Perencananaan Geser Dua Arah

→ Ditinjau masing-masing kolom pada arah pendek (H).

1. Kolom P1



Gambar 4.32 Bidang Geser 2 Arah Kolom P1

$H'_1 = hk_1 + d = 450 + 514$

$= 964\ mm = 0{,}964\ m$

$B'_1 = bk_1 + d = 600 + 514$

$= 1114\ mm = 1{,}114\ m$

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

$Vu = P_1 - qu.\ .\ H'_1\ .\ B'_1$

$$=1914{,}0612 - 219{,}2006 \,.\, 0{,}964 \,.\, 1{,}114 = 1678{,}6625 \text{ kN}$$

$$Vu/\phi = 1678{,}6625/0{,}6 = 2797{,}7709 \text{ kN}$$

• Kekuatan beton menahan geser :

$$\beta c = \frac{sisipanjang}{sisipendek} = \frac{bk_1}{hk_1} = \frac{0{,}60}{0{,}45} = 1{,}3$$

$$bo = 2 \,.\, (H'_1 + B'_1) = 2 \,.\, (964 + 1114) = 4156 \text{ mm}$$

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

$$= (1 + 2/1{,}3).\, (2.\sqrt{25}).\, 4156.\, 514.\, 10^{-3} = 53404{,}6 \text{ kN}$$

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

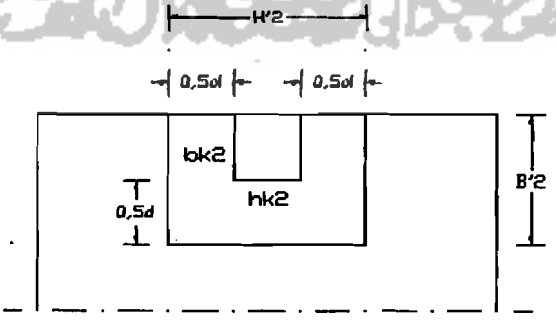
$$= 4.\sqrt{25} \,.\, 4156.\, 514.\, 10^{-3} = 42723{,}68 \text{ kN}$$

• Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc_1 dan Vc_2, yaitu Vc_2 = 42723,68 KN

$$Vc_2 = 42723{,}688 \text{ KN} \geq Vu/\phi = 2797{,}7709 \text{ kN}$$ **.....Aman.**

2. Kolom P2



Gambar 4.33 Bidang Geser 2 Arah Kolom P2

$$H'_2 = hk_2 + d = 400 + 514$$

$= 914 \text{ mm} = 0{,}964 \text{ m}$

$B'_2 = bk_2 + d = 400 + 1/2 \cdot 514$

$= 657 \text{ mm} = 0{,}657 \text{ m}$

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

$Vu = P_2 - qu \cdot H'_2 \cdot B'_2$

$= 716{,}3461 - 219{,}2006 \cdot 0{,}914 \cdot 0{,}657 = 584{,}7166 \text{ kN}$

$Vu/\phi = 584{,}7166/0{,}6 = 974{,}5276 \text{ kN}$

- Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{bk_2}{hk_2} = \frac{0{,}40}{0{,}40} = 1{,}0$

$bo = 2 \cdot (H'_2 + B'_2) = 2 \cdot (914 + 657) = 3142 \text{ mm}$

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

$= (1 + 2/1) \cdot (2 \cdot \sqrt{25}) \cdot 3142 \cdot 514 \cdot 10^{-3} = 48449{,}6 \text{ kN}$

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

$= 4 \cdot \sqrt{25} \cdot 3142 \cdot 514 \cdot 10^{-3} = 32299{,}76 \text{ kN}$

- Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc_1 dan Vc_2, yaitu $Vc_2 = 32299{,}76$ KN

$Vc_2 = 32299{,}76 \text{ KN} \geq Vu/\phi = 974{,}5276 \text{ kN}$**.....Aman.**

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) = B . H = 4 . 3 = 12 mm^2

Luas penampang kolom (A_1) = bk1 . hk1 + bk2 . hk2 = 0,6. 0,45 + 0,4 + 0,4

= 0,43 m^2

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{12}{0{,}43}} = 5{,}2827 > 2$$ (jika lebih besar dari 2, dipakai nilai 2)

• Kuat tumpuan pondasi :

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1.2)$

$= 0{,}7.\ (0{,}85.\ 25\ .\ 430000.\ 2).10^{-3} = 12792{,}5$ kN

• Kuat tumpuan kolom :

$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1)$

$= 0{,}7.\ (0{,}85.\ 25.\ 430000).10^{-3} = 6396{,}25$ KN

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 12792.5$ kN$> \phi.Pn_{kolom} = 6396{,}25$ kKN .....Aman.

E. Perencanaan Tulangan Lentur Pondasi

1. Arah Memanjang B

M^+_{mak} = 328,8009 kNm

M^-_{mak} = 297,3198 kNm

Momen Positif

Mu = 328,8009 kNm

$$Mu/\phi = \frac{328{,}8009}{0{,}8} = 411{,}0011 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, sehingga luas tampang 1 tulangan pokok :

  $A_{1\varnothing} = \frac{1}{4}.\ \pi.\ D^2 = \frac{1}{4}.\ \pi.\ 22^2 = 379{,}94 \text{ mm}^2$

- Tebal pelat pondasi : h = 600 mm, selimut beton (Pb) = 75 mm

  d $= h - Pb - 0{,}5.\ \varnothing_{tul.\ pokok} = 600 - 75 - 0{,}5.\ 22 = 514$ mm

  m $= \dfrac{fy}{0{,}85.f'c} = \dfrac{400}{0{,}85.25} = 18{,}8235$

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

  Rn $= \dfrac{Mu/\phi}{b.d} = \dfrac{411{,}0011.10^6}{1000.514^2} = 1{,}5557$ MPa

  Rasio Tulangan :

  $\rho_{min} = \dfrac{1{,}4}{fy} = \dfrac{1{,}4}{400} = 0{,}00350$

  $\rho b = \dfrac{0{,}85.f'c.\beta_1}{fy}.\left(\dfrac{600}{600+fy}\right) = \dfrac{0{,}85.25.0{,}85}{400}.\left(\dfrac{600}{600+400}\right) = 0{,}02709$

  $\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}02709 = 0{,}02032$

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

  $= \dfrac{1}{18{,}8235}.\left(1 - \sqrt{1 - \dfrac{2.18{,}8235.1{,}5557}{400}}\right) = 0{,}00404 < \rho_{mak} = 0{,}02032$

  $> \rho_{min} = 0{,}00350$

  sehingga dipakai : $\rho_{perlu} = \rho_{ada} = 0{,}00404$

  $As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00404.\ 1000\ .\ 514 = 2078{,}1087 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{379,94.1000}{2078,1087} = 182,8297 \text{ mm}$$

$s \leq 2.h = 2 .600 = 1200$ mm

$s \leq 250$ mm

Dipakai Tulangan Pokok : D_{22} – 150 mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{379,94.1000}{150} = 2532,9333 \text{ mm}^2$$

- Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2533,9333.400}{0,85.25.1000} = 47,6787 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right)$$

$$= 2533,9333 . 400 \left(514 - \frac{47,6787}{2}\right)$$

$$= 496,6177 \text{ kNm} \geq \frac{Mu}{\phi} = 411,0011 \text{ kNm} \text{ .....Aman.}$$

Momen Negatif

$Mu = 297,3198$ kNm

$$\frac{Mu}{\phi} = \frac{297,3198}{0,8} = 371,6498 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{16}$ mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\varnothing} = \frac{1}{4}. \pi. D^2 = \frac{1}{4}. \pi. 16^2 = 200,96 \text{ mm}^2$

- Tebal pelat pondasi : h = 600 mm, selimut beton (Pb) = 75 mm

$d = h - Pb - 0,5. \varnothing_{tul.\ pokok} = 600 - 75 - 0,5. 16 = 517$ mm

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}8235$$

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

$$Rn = \frac{Mu/\phi}{b.d} = \frac{371{,}6498.10^6}{1000.517^2} = 1{,}4067 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}00350$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}02709$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}02709 = 0{,}02032$$

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

$$= \frac{1}{18{,}8235}\left(1-\sqrt{1-\frac{2.18{,}8235.1{,}1{,}4067}{400}}\right) = 0{,}00364 < \rho_{mak} = 0{,}02032$$

$$> \rho_{min} = 0{,}00350$$

sehingga dipakai : $\rho_{perlu} = \rho_{ada} = 0{,}00364$

$$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00364.\ 1000\ .\ 517 = 1871{,}7886 \text{ mm}^2$$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{200{,}96.1000}{1871{,}7886} = 107{,}3625 \text{ mm}$$

$s \le 2.h = 2\ .600 = 1200$ mm

$s \le 250$ mm

Dipakai Tulangan Pokok : D_{16} – 100 mm

$$As_{ada} = \frac{A_{10}.1000}{s} = \frac{200,96.1000}{100} = 2009,6 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2009,6.400}{0,85.25.1000} = 37,8278 \text{ mm}$$

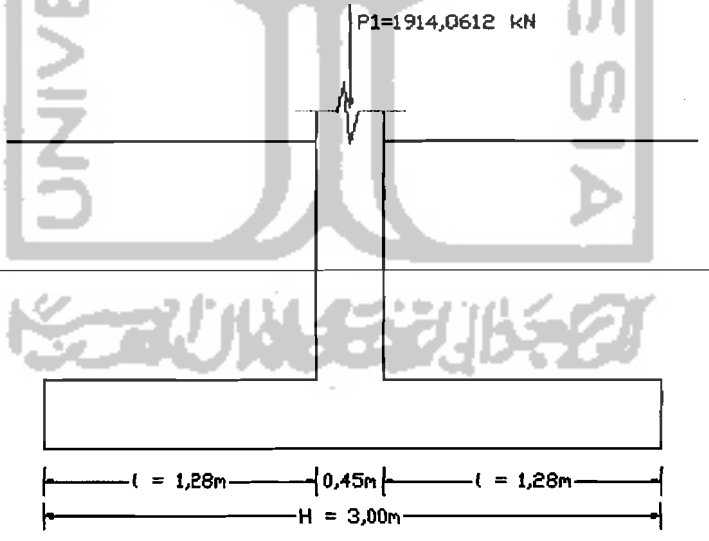
$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right)$$

$$= 2009,6 \,.\, 400 \left(517 - \frac{37,8278}{2}\right)$$

$$= 397,97 \text{ kNm} \geq \frac{Mu}{\phi} = 371,6498 \text{ kNm} \quad \textbf{.....Aman.}$$

2. Arah Memendek H

a. Kolom P1



Gambar 4.34 Perencanaan Momen Kolom P1

$$qu = \frac{P_1}{H} = \frac{1914,0612}{3} = 638,0204 \text{ kN/m}$$

$$l = \frac{H - hk_1}{2} = \frac{3 - 0{,}45}{2} = 1{,}28 \text{ m}$$

$$Mu = 0{,}5 \,.\, qu \,.\, l^2 = 0{,}5 \,.\, 638{,}0204 \,.\, 1{,}28^2 = 518{,}5910 \text{ kNm}$$

$$Mu/\phi = \frac{518{,}5910}{0{,}8} = 648{,}2387 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{22}$ mm, sehingga luas tampang 1 tulangan pokok :

$$A_{1\varnothing} = \tfrac{1}{4} . \pi . D^2 = \tfrac{1}{4} . \pi . 22^2 = 379{,}94 \text{ mm}^2$$

- Tebal pelat pondasi : h = 600 mm, selimut beton (Pb) = 75 mm

$$d = h - Pb - 0{,}5 . \varnothing_{tul.\ pokok} = 600 - 75 - 0{,}5 . 22 = 514 \text{ mm}$$

$$m = \frac{fy}{0{,}85 . f'c} = \frac{400}{0{,}85 . 25} = 18{,}8235$$

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

$$Rn = \frac{Mu/\phi}{b.d} = \frac{648{,}2387 . 10^6}{1000 . 514^2} = 2{,}4536 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}00350$$

$$\rho b = \frac{0{,}85 . f'c . \beta_1}{fy} \left(\frac{600}{600 + fy} \right) = \frac{0{,}85 . 25 . 0{,}85}{400} \left(\frac{600}{600 + 400} \right) = 0{,}02709$$

$$\rho_{mak} = 0{,}75 . \rho b = 0{,}75 . 0{,}02709 = 0{,}02032$$

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

$$= \frac{1}{18{,}8235} \left(1 - \sqrt{1 - \frac{2 . 18{,}8235 . 2{,}4536}{400}} \right) = 0{,}00654 < \rho_{mak} = 0{,}02032$$

$$> \rho_{min} = 0{,}00350$$

sehingga dipakai : $\rho_{perlu} = \rho_{ada} = 0{,}00654$

$As_{perlu} = \rho_{perlu} . b . d = 0{,}00654 . 1000 . 514 = 3359{,}5825 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{379{,}94.1000}{3359{,}5825} = 113{,}0914 \text{ mm}$$

$s \leq 2.h = 2 .600 = 1200$ mm

$s \leq 250$ mm

Dipakai Tulangan Pokok : $D_{22} - 100$ mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{379{,}94.1000}{100} = 3799{,}4 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{3799{,}4.400}{0{,}85.25.1000} = 71{,}5181 \text{ mm}$$

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

$= 3799{,}4 . 400 (514 - 71{,}5181/2)$

$= 726{,}8115 \text{ kNm} \geq Mu/\phi = 648{,}2387 \text{ kNm}$ **.....Aman.**

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002. b. h = 0{,}0018. 1000. 600 = 1080 \text{ mm}^2$

Digunakan tulangan bagi Ø13 mm, sehingga luas tampang 1 tulangan susut :

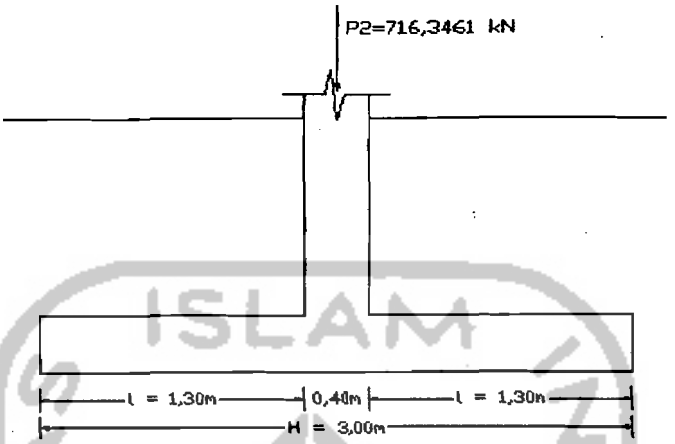
$A_{1\varnothing} = ¼. \pi. D^2 = ¼. \pi. 13^2 = 132{,}665 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{132{,}665.1000}{1080} = 122{,}838 \text{ mm} \approx 100 \text{ mm}$$

Dipakai Tulangan Susut : D_{13} – 100 mm

b. Kolom P2



Gambar 4.35 Perencanaan Momen Kolom P2

$$qu = \frac{P_2}{H} = \frac{716,3461}{3} = 238,7820 \text{ kN/m}$$

$$l = \frac{H - hk_2}{2} = \frac{3 - 0,40}{2} = 1,30 \text{ m}$$

$$Mu = 0,5 \cdot qu \cdot l^2 = 0,5 \cdot 638,0204 \cdot 1,30^2 = 201,7708 \text{ kNm}$$

$$Mu/\phi = \frac{201,7708}{0,8} = 252,2135 \text{ kNm}$$

- Digunakan tulangan pokok $\varnothing_{16}$ mm, sehingga luas tampang 1 tulangan pokok :

  $A_{1\varnothing} = \frac{1}{4} . \pi . D^2 = \frac{1}{4} . \pi . 16^2 = 200,96 \text{ mm}^2$

- Tebal pelat pondasi : h = 600 mm, selimut beton (Pb) = 75 mm

  $d = h - Pb - 0,5 . \varnothing_{tul. pokok} = 600 - 75 - 0,5 . 16 = 517 \text{ mm}$

  $$m = \frac{fy}{0,85 . f'c} = \frac{400}{0,85.25} = 18,8235$$

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

$$Rn = \frac{Mu/\phi}{b.d} = \frac{252,2135.10^6}{1000.517^2} = 0,9436 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,00350$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.25.0,85}{400}\left(\frac{600}{600+400}\right) = 0,02709$$

$$\rho_{mak} = 0,75.\ \rho b = 0,75.\ 0,02709 = 0,02032$$

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

$$= \frac{1}{18,8235}\left(1-\sqrt{1-\frac{2.18,8235.0,9436}{400}}\right) = 0,00241 < \rho_{mak} = 0,02032$$

$$> \rho_{min} = 0,00350$$

$0,0020 < 1,33\rho_{ada} = 0,00321 < \rho_{min}$

sehingga dipakai : $\rho_{perlu} = 1,33\rho_{ada} = 0,00321$

$As_{perlu} = \rho_{perlu}\ .\ b\ .\ d = 0,00321.\ 1000\ .\ 517 = 1659,7771 \text{ mm}^2$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{200,96.1000}{1659,7771} = 121,0765 \text{ mm}$$

$s \le 2.h = 2\ .600 = 1200$ mm

$s \le 250$ mm

Dipakai Tulangan Pokok : D_{16} – 100 mm

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{200,96.1000}{100} = 2009,6 \text{ mm}^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{2009{,}6.400}{0{,}85.25.1000} = 37{,}8278 \text{ mm}$$

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

$$= 2009{,}6\ .\ 400\ (517 - 37{,}8278/2)$$

$$= 335{,}7629 \text{ kNm} \geq Mu/\phi = 252{,}2135 \text{ kNm} \quad \textbf{.....Aman.}$$

Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002.\ b.\ h = 0{,}0018.\ 1000.\ 600 = 1080 \text{ mm}^2$

Digunakan tulangan bagi ∅13 mm, sehingga luas tampang 1 tulangan susut :

$A_{1\varnothing} = ¼.\ \pi.\ D^2 = ¼.\ \pi.\ 13^2 = 132{,}665 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{132{,}665.1000}{1080} = 122{,}838 \text{ mm} \approx 100 \text{ mm}$$

Dipakai Tulangan Susut : D_{13} – 100 mm