BAB IV

PERENCANAAN STRUKTUR

4.1 Rangka Atap Kuda-kuda Baja

a. Data Konstruksi Rangka Atap

- Jarak antar kuda-kuda maksimum b = 4.00
- Panjang bentang L = 11,20 m
- Mutu baja profil :

  Tegangan leleh (fy) = 36 Ksi = 2531 kg/cm^2

  Kuat tarik (fu) = 58 Ksi = 4077 kg/cm^2
- Mutu baut A325X (Non Full Draat) :

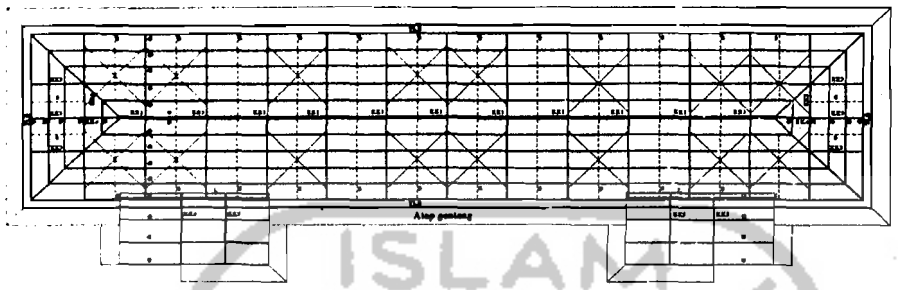
  Tegangan leleh (fy) = 44 Ksi = 3093 kg/cm^2

  Kuat tarik (fu) = 30 Ksi = 2109 kg/cm^2
- Usuk dan reng dipakai kayu sedangkan gording dipakai baja jenis Light Lip Channel.
- Jurai menggunakan profil Double Light Lip Channel dan rangka kuda-kuda menggunakan profil Double Angel.

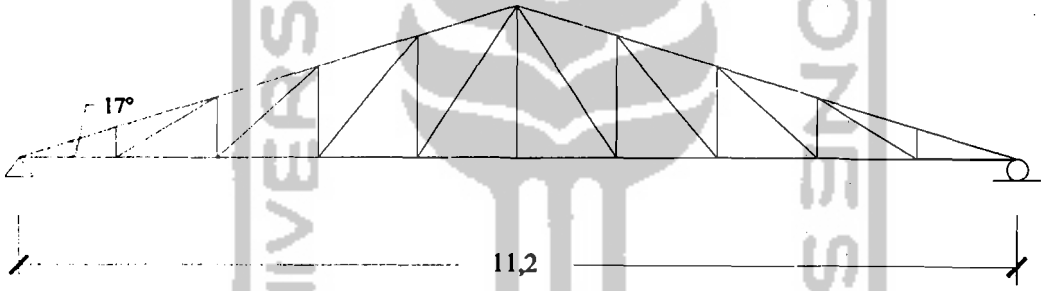
b. Jumlah dan Jarak Antar Gording

- Panjang sisi miring kuda-kuda (M) = $\frac{0{,}5.L}{\cos\alpha} = \frac{0{,}5.11{,}20}{\cos 20} = 5{,}85$ m
- Jumlah gording setengah bentang (n) = 6 buah

- Jarak antar gording (Lg) = $\frac{5{,}85}{5}$ = 1,17 m



Gambar 4.1 Rencana Kuda – Kuda



Gambar 4.2 Penampang Kuda-kuda

4.1.1 Perencanaan Gording

a. Pembebanan Gording

1. Beban Tetap

- Berat Asbes 5cm (table 2.1 PPIUG'83)= 11 x 1,17 = 12,87 Kg/m'
- Berat hidup (pasal 3.2.2.b. PPIUG'83) = 20 x 1,17 = 23,4 Kg/m'
- Berat Gording diperkirakan 7 ~ 10 Kg/m' = 10 Kg/m'

q_{total} = 46,27 Kg/m'

Mekanika gording

$q_{\perp}$ = q_{total} x cos α = 46,27 x cos 17° = 13,53 Kg/m'

$q_{//}$ = q_{total} x sin α = 46,27 x sin 17° = 44,25 Kg/m'

2. Beban Angin

Wa = 25 Kg/m^2 (pasal 4.2.1 PPIUG'83)

- Angin Tekan (Wt)

Wt = 0 (karena sudut yang terlalu landai α = 17°)

- Angin Hisap (Wh)

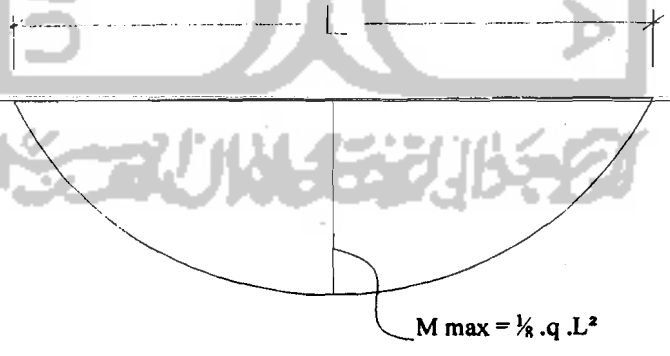
Wt = 0 (karena sudut yang terlalu landai α = 17°)

$W_{\perp}$ = 0 (karena sudut yang terlalu landai α = 17°)

W// = 0 (karena beban angin bekerja di atap PPIUG'83)

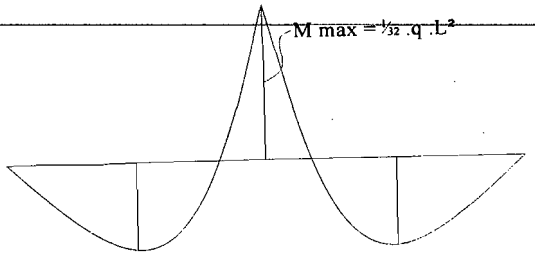
b. Momen yang terjadi

- Akibat beban tetap



Gambar 4.3 Momen akibat beban merata sepanjang bentang

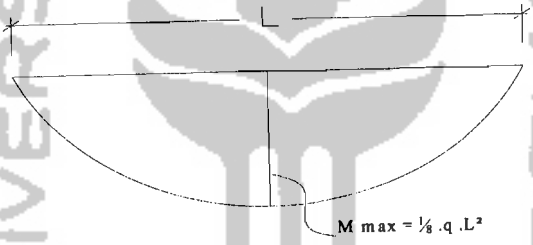
$$M_{\perp} \text{ maks} = \frac{1}{8} q_{\perp} L^2 = \frac{1}{8}.83{,}87.4^2 = 167{,}74 \text{ kgm'}$$



Gambar 4.4 Momen Akibat Sagrod menumpu di tengah bentang

$$M_{//}\ maks = \frac{1}{32} q_{//} L^2 = \frac{1}{32}.25{,}64 .4^2 = 12{,}82 \text{ kgm'}$$

- Akibat beban angin



Gambar 4.5 Momen Akibat Beban Angin

$$M_{\perp}\ maks = \frac{1}{8} W_{\perp} L^2 = \frac{1}{8}.0.4^2 = 0 \text{ kgm'}$$

c. Penentuan Profil Baja

Dicoba profil Light Lip Channel 125x50x20x3,2

Dari tabel Baja didapatkan :

Sx	= 29 cm^3	W	= 7,43 kg/m'
Sy	= 8,02 cm^3	fy	= 36 ksi = 2531 kg/cm^2
Ix	= 181 cm^4	E	= 2,1x10^6 kg/cm^2
Iy	= 26,6 cm^4	Fu	= 4078 kg/cm^2

d. Kontrol Penampang Kompak

$$\frac{bf}{2tf} \le \frac{65}{\sqrt{fy}} \rightarrow \frac{50}{2.3,2} = 7,81 \le 10,8$$

$$\frac{d}{tw} \le \frac{640}{\sqrt{fy}} \Rightarrow \frac{100}{3,2} = 31,25 \le 107$$

Jadi profil Light Lip Channel 125x50x20x3,2 → Kompak

$$Lc = \frac{76.bf}{\sqrt{fy}}(\text{in}) = \frac{76.1,97}{\sqrt{36}} = 24,95 \text{ m} > Lb = 4 \text{ m}$$

$$Lu = \frac{2000}{d/Af.fy} = \frac{2000}{4,92/(1,97.1,26).36} = 7,11 > Lb = 4 \text{ m}$$

Maka Fb dipakai = 0,66.Fy

e. Kontrol Tegangan Berdasar AISC

$$fbx = \frac{M_{\perp} maks}{Sx} = \frac{0,8.(167,74+0).100}{29} = 462,731 \text{ Kg/cm}^2$$

$$fbx = \frac{M_{\perp} maks}{Sx} = \frac{1.(167,74).100}{29} = 578,414 \text{ Kg/cm}^2$$

$$fby = \frac{M_{//} maks}{Sy} = \frac{12,82.100}{8,02} = 159,85 \text{ Kg/cm}^2$$

$$\frac{fbx}{0,66.fy} + \frac{fby}{0,75.fy} = \frac{462,731}{0,66.2531} + \frac{159,85}{0,75.2531} = 0,35 \le 1,0$$

$$\frac{fbx}{0,66.fy} + \frac{fby}{0,75.fy} = \frac{574,414}{0,66.2531} + \frac{159,85}{0,75.2531} = 0,42 \le 1,0$$

f. Kontrol Lendutan

$$\delta_{\perp} = \frac{5}{384}.\frac{q_{\perp}.L^4}{E.Ix} = \frac{5}{384}.\frac{0,8(13.53+0).4^4.10^6}{2,1.10^6.181}$$

$$= 0{,}1 \text{ cm} \leq \frac{L}{360} = \frac{4.100}{360} = 1{,}11 \text{ cm (ok)}$$

$$\delta_{\perp} = \frac{5}{384} \cdot \frac{q_{\perp} . L^4}{E.Ix} = \frac{5}{384} \cdot \frac{1(13{,}53).4^4.10^6}{2{,}1.10^6.181}$$

$$= 0{,}1 \text{ cm}$$

$$\delta_{//} = \frac{5}{384} \cdot \frac{q_{//} (.L/(a+1))^4}{E.Iy} = \frac{5}{384} \cdot \frac{44{,}25.(4/(1+1))^4.10^6}{2{,}1.10^6.181}$$

$$= 0{,}02 \text{ cm}$$

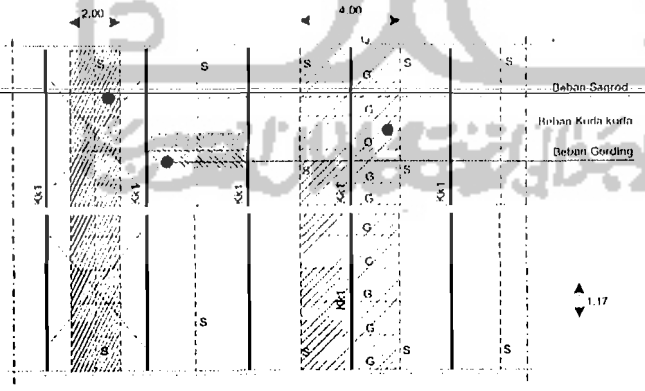
$$\delta = \sqrt{(\delta_{\perp}{}^2 + \delta_{//})} = \sqrt{(0{,}1^2 + 0{,}02^2)}$$

$$= 0{,}101 \text{ cm} \leq \frac{L}{360} = \frac{4.100}{360} = 1{,}11 \text{ cm (Ok!)}$$

a = jumlah sagrod dalam satu bentang = 1 buah

Jadi digunakan jenis profil Light Lip Channels 125x50x20x3,2

4.1.2 Perencanaan Sagrod dan Tierod



Gambar 4.6 Rencana Pembebanan Kuda – Kuda

1. Sagrod

   Beban Sagrod dan Tierod :

- Berat penutup atap x sisi miring = 11 x 5,85 = 63,4 kg/m'
- Beban Hidup x sisi miring = 20 x 5,85 = 117 kg/m'
- Beban gording = berat gording x jlm gording = 7,43 x 6 = 44,6 kg/m'

225 kg/m'

Ss = L/2 = 4/2 = 2 m

$P_{//}$ = P.sinα.Ss = 225 x sin 17 x 2 = 131,567 kg/m'

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

$$D = \sqrt{\frac{P_{//}\,4}{0,33.Fu.\pi}} = \sqrt{\frac{131,567.4}{0,33.4077.3,14}} = 0,35 \text{ cm} = 3,5 \text{ mm}$$

Dipakai sagrod = D_{sagrod} + 3 mm = 3,5 + 3 = 10 mm

2. Tierod

Beban Tierod = T = $P_{//}$ .cosα = 131,567 x cos 17 = 125,818 kg

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

$$D = \sqrt{\frac{T.4}{0,33.Fu.\pi}} = \sqrt{\frac{125,818.4}{0,33.4077.3,14}} = 0,35 \text{ cm} = 3,5 \text{ mm}$$

Dipakai tierod = D_{tierod} + 3 mm = 3,5 + 3 = 8 mm

Sagrod dan Tierod dipakai diameter = 8 mm

4.1.3 Perencanaan Kuda – Kuda (KK1)

4.1.3.1 Pembebanan Kuda – Kuda

Beban Tetap :

- Berat Gording (Light Lip Channel) = 7,43 kg/m'
- Berat eternity (Tabel 2.1 PPIUG '83) = 11 kg/m^2

- Penggantung langit – langit dari kayu (Tabel 2.1 PPIUG '83)

  = 7 kg/m^2

- Berat penutup atap dari asbes (Tabel 2.1 PPIUG '83)

  = 11 kg/m^2

- Beban hidup (pasal 3.2.2.b. PPIUG'83) = 20 kg/m^2

- Berat Kuda – Kuda Taksiran :

  Tabel baja siku – siku sama kaki

  Ditaksir mernggunakan profil 2L 50x50x5

  W = 2 x 3,77 = 7,54 kg/m'

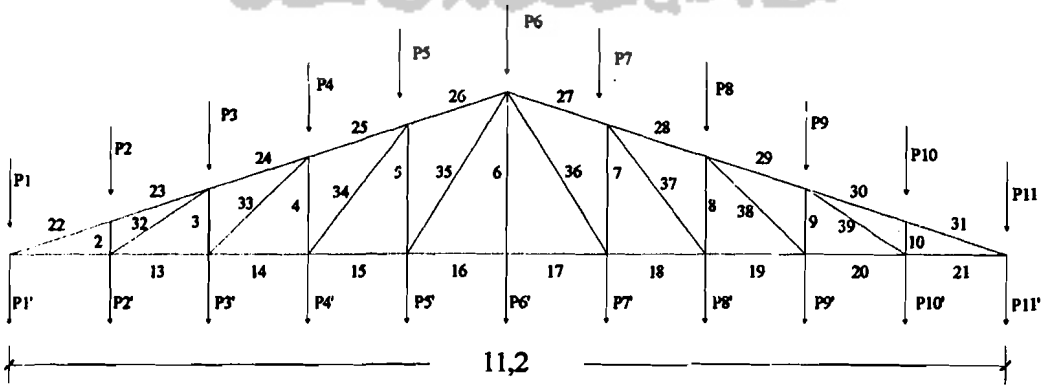
  Berat kuda – kuda = $\frac{W.L_{total}\,kuda-kuda}{Lkuda-kuda} = \frac{7{,}54x44{,}65}{11{,}20} = 30{,}06$ kg/m'

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

  = 0,2 x 30,06 = 6,012 kg/m'

  Beban kuda – kuda = 30,06 + 6,012 = 36,072 kg/m'

  Beban – beban pada joint :



Gambar 4.7 Gaya Reaksi Beban Tetap

a. $P_1 = P_{11}$

Beban Gording	= 7,43 x 4	=	29,72 kg
Berat Penutup atap	= 11 x 4 x ½ (1,12/cos17)	=	25,74 kg
	qD	=	55,46 kg
Beban hidup (qL)	= 20 x 4 x ½ (1.12/cos17)	=	46,84 kg

b. $P_2 = P_3 = P_4 = P_5 = P_6 = P_7 = P_8 = P_9 = P_{10}$

Beban Gording	= 7,43 x 4	=	29,72 kg
Berat Penutup atap	= 11 x 4 x (1,12/cos17)	=	51,48 kg
	qD	=	81,20 kg
Beban hidup (qL)	= 20 x 4 x (1,12/cos17)	=	93,60 kg

c. $P_{1'} = P_{11'}$

Beban Eternit	= 11 x 4 x ½ .1,12	=	24,64 kg
Berat Kuda - Kuda	= 36,072 x ½ . 1,12	=	20,20 kg
	qD	=	44,84 kg

d. $P_{2'} = P_{3'} = P_{4'} = P_{5'} = P_{6'} = P_{7'} = P_{8'} = P_{9'} = P_{10'}$

Beban Eternit	= 11 x 4 x 1,12	=	49,28 kg
Berat Kuda-kuda	= 36,072 x 1,12	=	40,40 kg
	qD	=	89,68 kg

Beban Angin

Wa = 25 Kg/m^2 (pasal 4.2.1 PPIUG'83)

a. Angin Kiri

- Angin Tekan (Wt)

  Wt = 0 (karena sudut yang terlalu landai $\alpha = 17°$)

- Angin Hisap (Wh)

  Wt = 0 (karena sudut yang terlalu landai $\alpha = 17°$)

  $W_{\perp} = 0$ (karena sudut yang terlalu landai $\alpha = 17°$)

  W// = 0 (karena beban angin bekerja di atap PPIUG'83)

b. Angin Kanan

  Besar angin kanan sama dengan angin kiri.

4.1.3.2 Perhitungan Rangka

Analisa rangka menggunakan SAP 2000 versi 9.03 dapat dilihat dalam lampiran dan beban rencana kuda – kuda KK dapat dilihat pada tabel

1. Data profil baja yang digunakan

   Modulus of Elasticity (Es) = $2{,}1 \times 10^6$ Kg/cm^2

   Fy = 2531 Kg/cm^2

   Asumsi profil 2L 50x50x5 dengan :

   b = 50 mm tf = 5 mm tw = 5 mm

2. Data – data pembebanan yang dimasukkan pada SAP 2000 versi 9.03

Untuk pembebanan P_1' s/d P_9' pada perhitungan SAP 2000, berat kuda – kuda sudah termasuk berat sendiri maka tidak dimasukkan dalam perhitungan.

Tabel 4.1 Gaya P_1 sampai dengan P_9

Nama Gaya	Beban Mati (qD) kg	Beban Hidup (qL) kg
$P_1 = P_{11}$	- 55,46	- 46,84
$P_2 = P_3 = P_4 = P_5 = P_6 = P_7 = P_8 = P_9 = P_{10}$	- 81,20	- 93,60

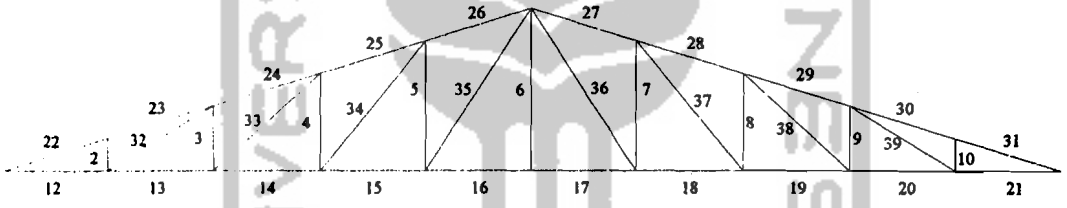
Tabel 4.2 Gaya P_1' s/d P_9'

Nama Gaya	Beban Mati (qD) kg
$P_1' = P_{11}'$	- 44,84
$P_2' = P_3' = P_4' = P_5' = P_6' = P_7' = P_8' = P_9' = P_{10}'$	- 89,68

b. Akibat beban angin

Tidak diperhitungkan karena sudut yang terlalu landai $\alpha = 17°$

4.1.3.3 Perencanaan Profil Kuda-Kuda



Gambar 4.8 Penomoran Batang

1. Batang Tekan (btg 22 dan btg 31)

   Gaya batang (tekan) maksimum = 4862.35 kg

   Panjang 1,17 m = 117 cm

   r = 1,51 cm

   k = 1 (Salmon Johnson hal 278 gambar 691 (c))

   Dari Rumusan Salmon Johnson hal 274

   Untuk angka kelangsingan $\frac{k.L}{r} \leq Cc = \frac{6440}{\sqrt{fy}}$

Ambil $\frac{k.L}{r} = \frac{1.117}{1,51} = 77,48$

$Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}} = 128,009 > \frac{k.L}{r} = 77,48$, maka :

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{k.L/r}{Cc} - \frac{1(k.L/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{77,48}{128,009} - \frac{1(77,48)^3}{8.128,009^3} = 1,866$$

$$Fa_{perlu} = \frac{fy}{Fs}\left(1 - 0,5\left(\frac{k.L/r}{Cc}\right)^2\right) = \frac{2531}{1,866}\left(1 - 0,5\left(\frac{77,48}{128,009}\right)^2\right)$$

$= 1107,922 \text{ kg/cm}^2$

Profil 2L 50x50x5 (profil yang biasa digunakan dilapangan), dengan

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

Kontrol Local Buckling :

$\frac{bf}{tf} \leq \frac{76}{\sqrt{fy}}$ (fy dalam Ksi)

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

$10 \leq 12,667$ (ok)

Kontrol Beban :

$\frac{k.L}{r} = \frac{1.117}{1,51} \leq Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}}$

$77,48 \leq 128,01$ (terjadi tekuk elastis)

$A_{perlu} = Fa_{perlu}.A_{ada} \geq P_{tjd}$

$= 1107,922 \,.\, 9,4$

$= 10414,47 \text{ kg} \geq 4862.35 \text{ kg} \rightarrow$ (okey...)

2. Batang Tarik (btg 12 – btg 21)

Gaya batang (tarik) maksimum = 4576.95 kg

Panjang 1,12 m = 112 cm

$$R_{min} = \frac{L}{240} = \frac{112}{240} = 0{,}47 \text{ cm}$$

$$A_{eff} = \frac{P}{0{,}5.Fu} = \frac{4576{,}95}{0{,}5.4077} = 2{,}24 \text{ cm}^2$$

μ = Faktor reduksi luas netto = 0,85

$$A_{netto} = \frac{A_{eff}}{\mu} = \frac{2{,}24}{0{,}85} = 2{,}64 \text{ cm}^2$$

Dicoba profil 2L 50x50x5

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

r = 1,51 cm

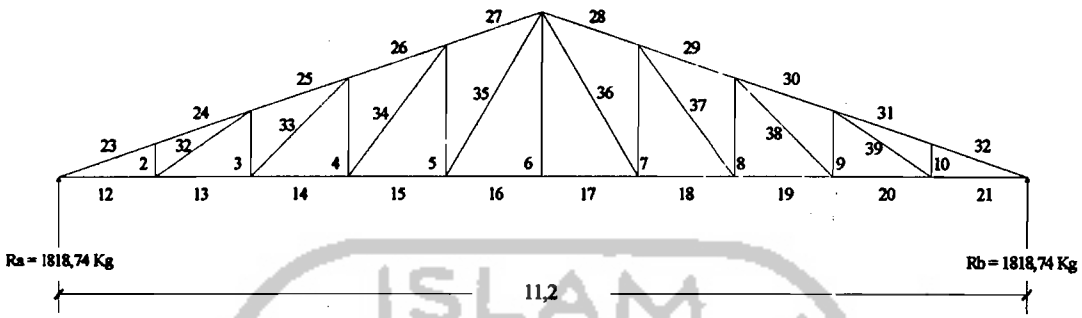
Kontrol kelangsingan (λ):

$$\lambda_{ada} = \frac{k.L}{r_{ada}} = \frac{1.112}{1{,}51}$$

= 74,172 ≤ 240 (Untuk elemen atau batang utama) → **ok**

Jadi profil kuda – kuda 2L 50x50x5 dapat digunakan.

4.1.3.4 Perencanaan Pelat Kuda-Kuda



Gambar 4.9 Reaksi Pada Dukungan

Beban P diambil dari reaksi dukungan dari perhitungan SAP 2000 v 9.03:

$R_a = R_b = P_{maks} = 1791{,}98$ kg f'c = 25 Mpa = 250 kg/cm^2

$$A_{perlu} = \frac{P}{0{,}33.f'c} = \frac{1791{,}98}{0{,}33.250} = 21{,}72 \text{ cm}^2$$

Diambil ukuran pelat : 15 x 20 = 300 cm^2 > A_{perlu}

$$q = \frac{P}{BxL} = \frac{1791{,}98}{15.20} = 5{,}97 \text{ kg/cm}^2$$

$$x = \frac{20-(5+1+5)}{2} = 4.5 \text{ cm}$$

M = ½ q. x^2 = ½ x 5,97 x 4,5^2 = 60,44 kgcm

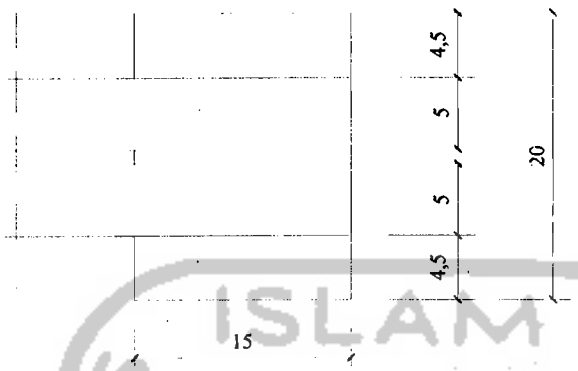
Syarat :

$$0{,}66 \text{ fy} = \frac{M}{1/6.1.tp^2} \rightarrow tp = \sqrt{\frac{10 \times M}{fy}}$$

$$tp = \sqrt{\frac{10 \times 60{,}44}{2400}} = 0{,}51 \text{ cm} \approx 1 \text{ cm}$$

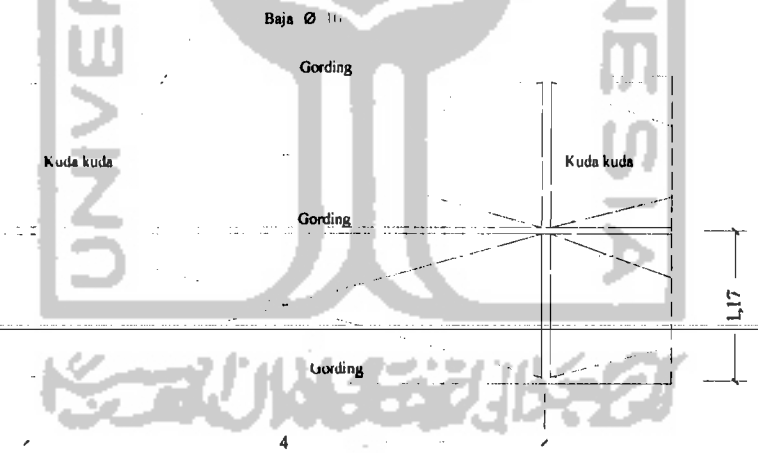
Sehingga dipakai pelat dengan tebal 1 cm

Pelat kuda-kuda berukuran : 20 x 15 x 1



Gambar 4.10 Sambungan Pelat Dukungan

4.1.3.5 Perencanaan Dukungan Arah lateral



Gambar 4.11 Dukungan Arah Lateral

Diketahui :

Lb = jarak antar gording = 1,17 m

Lc = jarak antar kuda-kuda = 4 m

$L = \sqrt{Lb^2 + Lc^2} = \sqrt{1{,}17^2 + 4^2} = 4{,}1676$ m = 416,76cm

Syarat L/r ≤ 300, sehingga :

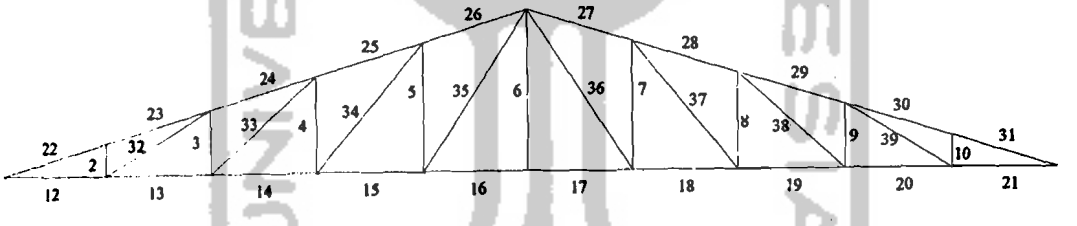
$$r \text{ min} \geq \frac{L}{300} = \frac{416{,}76}{300} = 1{,}39 \text{ cm}$$

Keterangan :

- L ≤ 3 m → dipakai baja tulangan ∅ 12 mm
- L ≥ 3 m → dipakai baja tulangan ∅ 19 mm
- 3 m < L ≤ 5 m → dipakai baja tulangan ∅ 16 mm

⇒ Sehingga dipakai baja tulangan ∅ 16 mm > r min = 1,39 cm

4.1.3.6 Perencanaan Sambungan



Gambar 4.12 Nomor Batang

- Tebal Pelat sambung (t) = 1 cm, dbaut = ½" = 1,27 cm
- Mutu Baja Profil : **2L 50x50x5**

  Tegangan Leleh (fy) = 36 Ksi = 2531 kg/cm^2

  Kuat Tarik (fu) = 58 Ksi = 4077 kg/cm^2

- Mutu baut A325X (Non Full Draat) :

  Tegangan leleh (fy) = 44 Ksi = 3093 kg/cm^2

  Kuat tarik (fu) = 30 Ksi = 2109 kg/cm^2

Tinjauan tegangan geser 1 baut :

P_{geser} $= ¼ . \pi . D^2_{baut} . fu .$ jumlah bidang geser (n)

$= ¼ . \pi . 1{,}27^2 . 2109 . 2$

$= 5340{,}52$ kg

Tinjauan tegangan tumpu 1 baut :

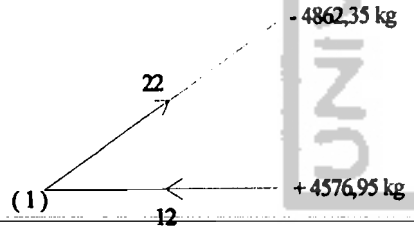
P_{geser} $= 1{,}2 . t . D^2_{baut} . fu .$ jumlah bidang geser (n)

$= 1{,}2 . 1 . 1{,}27 . 4077 . 1$

$= 6213{,}35$ kg

Maka P_{baut} diambil $\rightarrow$ $P_{geser} = 5340{,}52$ kg

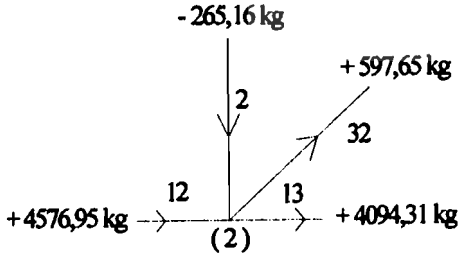
Jumlah baut (N) $= \frac{P_{terjadi}}{P_{baut}}$



Joint 1

$$N\,22 = \frac{4862{,}35}{5340{,}42} = 0{,}91 = 2 \text{ bh}$$

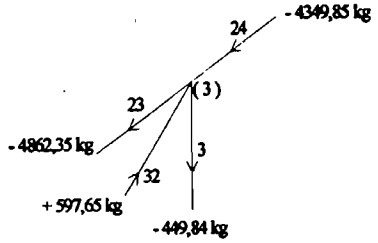
$$N\,12 = \frac{4576{,}95}{5340{,}42} = 0{,}86 = 2 \text{ bh}$$



Joint 2

$$N\,2 = \frac{265{,}16}{5340{,}42} = 0{,}05 = 2 \text{ bh}$$

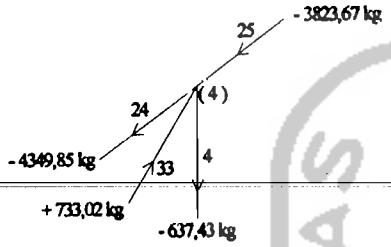
Joint 3



$$N\,32 = \frac{597,66}{5340,42} = 0,11 = 2\ bh$$

$$N\,3 = \frac{449,84}{5340,42} = 0,08 = 2\ bh$$

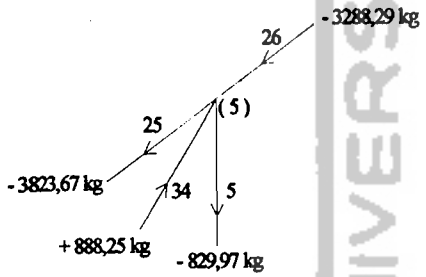
Joint 4



$$N\,33 = \frac{733,02}{5340,42} = 0,14 = 2\ bh$$

$$N\,4 = \frac{637,43}{5340,42} = 0,12 = 2\ bh$$

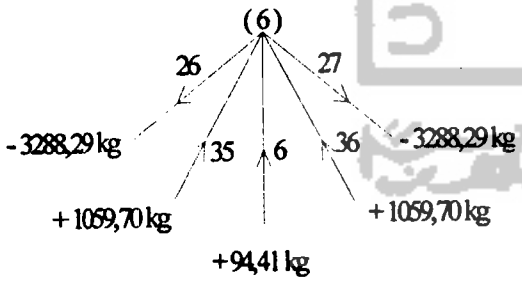
Joint 5



$$N\,34 = \frac{888,25}{5340,42} = 0,17 = 2\ bh$$

$$N\,5 = \frac{829,97}{5340,42} = 0,16 = 2\ bh$$

Joint 6

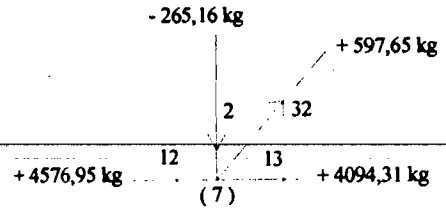


$$N\,26 = N\,27 = \frac{3288,29}{5340,42} = 0,28 = 2\ bh$$

$$N\,36 = N\,35 = \frac{1059,70}{5340,42} = 0,33 = 2\ bh$$

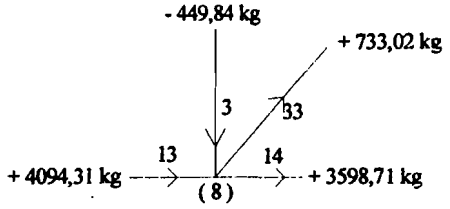
$$N\,6 = \frac{94,41}{5340,42} = 0,07 = 2\ bh$$

Joint 7



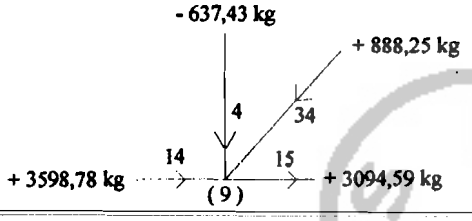
$$N\,13 = \frac{4094,31}{5340,42} = 1,33 = 2\ bh$$

Joint 8

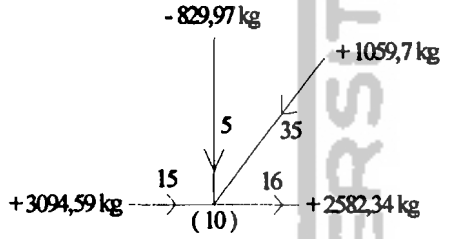


$$N\ 14 = \frac{3598,71}{5340,42} = 1,16 = 2\ bh$$

Joint 9



$$N\ 15 = \frac{3094,59}{5340,42} = 1,00 = 2\ bh$$



Joint 10

$$N\ 16 = \frac{2582,34}{5340,42} = 0,84 = 2\ bh$$

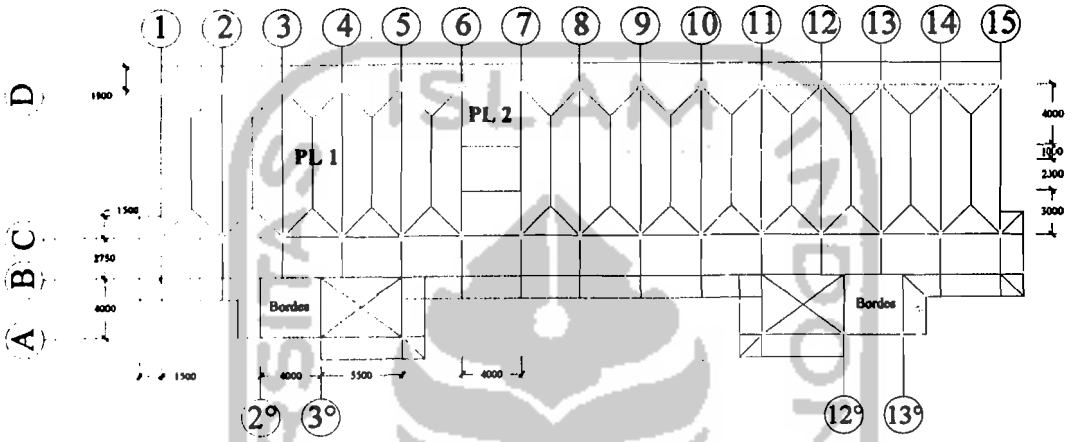
Tabel 4.3 Jumlah baut pada kuda kuda (KK 1)

Joint	Batang	Jumlah baut
1	12 ; 22 ; 21 ; 31	2
2	22 ; 2 ; 33 = 30 ; 10 ; 31	2
3	23 ; 24 ; 32 ; 3 = 29 ; 30 ; 39 ; 9	2
4	24 ; 25 ; 33 ; 4 ; 28 ; 29 ; 38 ; 8	2
5	25 ; 26 ; 34 ; 5 = 27 ; 28 ; 37 ; 7	2
6	25 ; 27 ; 35 ; 36 ; 6	2
7	12 ; 13 ; 32 ; 2 = 20 ; 21 ; 39 ; 10	2
8	13 ; 14 ; 33 ; 3 = 20 ; 19 ; 38 ; 9	2
9	14 ; 15 ; 34 ; 4 = 19 ; 18 ; 37 ; 8	2
10	15 ; 16 ; 35 ; 5 = 18 ; 17 ; 36 ; 7	2
11	16 ; 17 ; 6	2

4.2 Perencanaan Pelat Lantai

4.2.1 Pelat Satu Arah

4.2.1.1 Pembebanan Pelat Lantai



Gambar 4.13 Pembebanan Pelat Lantai 1

Perhitungan beban pelat lantai sesuai ketentuan PPPURDG 1987, sebagai berikut :

- Beban Mati Pelat Lantai :

1. Berat sendiri pelat lantai : 0,12 x 1,0 x 1,0 x 24 = 2,88 kN/m^2
2. Pasir (tebal 5 cm) : 0,05 x 1,0 x 1,0 x 16 = 0,80 kN/m^2
3. Spesi (tebal 3 cm) : 0,03 x 1,0 x 1,0 x 21 = 0,63 kN/m^2
4. Keramik : 0,02 x 1,0 x 1,0 x 20 = 0,20 kN/m^2 +

Beban Mati Total (qD) = 4,51 kN/m^2

- Beban Hidup Pelat Lantai :

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

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

  qU = 1,2. qD + 1,6. qL = 1,2. 4,51 + 1,6. 2,5 = 9,412 kN/m^2

  Koefisien momen (distribusi momen) (x) = ly/lx

4.2.1.2 Penulangan Pelat Lantai

Beton : f'c = 22,5 Mpa ; ⟶ $\beta_1 = 0,85$

Baja : fy = 300 Mpa. ⟶ $\varepsilon y = \frac{fy}{Es} = \frac{300}{200000} = 0,0015$

$$\rho b = \frac{0,85.f'c.}{fy}.\beta_1.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{300}.0,85.\left(\frac{600}{600+300}\right) = 0,0361$$

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

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{300} = 0,00467$$

$$m = \frac{fy}{0,85.f'c} = \frac{300}{0,85.22,5} = 15,686$$

4.2.1.3 Perhitungan Pelat Type PL-1 (4000 x 10000)

Panjang Bentang Pendek Lx = 4000 = 4,0 m

Panjang Bentang Panjang Ly = 10000 = 10,0 m

Ly/Lx = 2,5 m ; Ly/Lx ≥ 2 dianggap⟶ "pelat satu arah".

Lebar balok (asumsi) = 450 mm= 0,45 m

Menentukan Tebal Pelat, dengan f'c = 22,5 Mpa ; fy = 300 MPa :

- *Pelat dengan satu tepi menerus*

$$h_{min} = \frac{lx}{24}\left(0{,}35 + \frac{Fy}{700}\right)$$

$$h_{min} = \frac{4.10^3}{24}\left(0{,}35 + \frac{300}{700}\right) = 129{,}76\ mm$$

- *Pelat dengan dua tepi menerus*

$$h_{min} = \frac{lx}{28}\left(0{,}35 + \frac{Fy}{700}\right)$$

$$h_{min} = \frac{4.10^3}{28}\left(0{,}35 + \frac{300}{700}\right) = 111{,}22\ mm$$

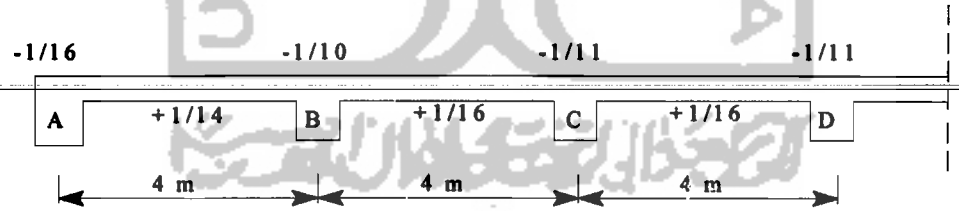
→ Digunakan h = 120 mm

Ln (bentang bersih) = 4000 - 450 = 3550 mm = 3,55 m

Untuk Lebar 1 meter, maka qU = 9,412 kN/m[1]

Menghitung Momen dengan Metode Pendekatan Momen.

Skema koefisien momen dikalikan dengan qU.Ln^2 :



Gambar 4.14 Koefisien Momen

Mu = koef. qU. Ln^2

Momen Tumpuan Tepi (A):

Mu$^-$ (1/16) = 1/16 . 9,412 . 3,55^2 = 7,41 kN.m

Momen Lapangan Tepi (A-B):

$Mu^{+}(1/14) = 1/14 . 9,412 . 3,55^2 = 8,47$ kN.m

Momen Tumpuan Tengah Tepi (B):

$Mu^{-}(1/10) = 1/10 . 9,412 . 3,55^2 = 11,86$ kN.m

Momen Lapangan Tengah (B-C),(C-D) dst :

$Mu^{+}(1/16) = 1/16 . 9,412 . 3,55^2 = 7,41$ kN.m

Momen Tumpuan Tengah (C) :

$Mu^{-}(1/11) = 1/11 . 9,412 . 3,55^2 = 10,78$ kN.m

- Vu $= 1,15 . 1/2 . qU . Ln = 1,15 . 1/2 . 9,412 . 3,55 = 19,21$ kN
- Vu $= 1/2 . qU . Ln = 1/2 . 9,412 . 3,55 = 16,71$ kN

Diperkirakan tulangan yang dipakai P10, penutup beton 20 mm, maka :

- $d = 120 - 20 - \frac{10}{2} = 95$ mm

Periksa Kuat Geser (tanpa tulangan geser) :

- $\Phi Vn = \Phi . \left(\frac{1}{6} . \sqrt{f'c} . b_w . d \right) = 0,6 . \left(\frac{1}{6} . \sqrt{22,5} . 1000 . 95 \right) = 45062,46$ N

  $= 45,062$ kN $> 16,71$ kN $\longrightarrow$ OK "*Aman*"

⇒ **Perhitungan Tulangan Lentur:**

a. Diambil $Mu^{-}(1/16) = 7,41$ kN.m

$$Mn = \frac{Mu^{-}(1/16)}{\phi} = \frac{7,41}{0,8} = 9,26 \text{ kN.m}$$

$$Rn = \frac{Mn}{b.d^2} = \frac{9,26.10^6}{1000.95^2} = 1,026 MPa$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{300}{0{,}85.22{,}5} = 15{,}686$$

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

$$= \frac{1}{15{,}686}\left[1 - \sqrt{1 - \frac{2.15{,}686.1{,}026}{300}}\right]$$

$$= 0{,}0035$$

Ternyata $\rho = 0{,}0035 < \rho_{min} = 0{,}00467$

dan $1{,}33.\ \rho = 1{,}33\ .0{,}0035 = 0{,}00465 < \rho_{min} = 0{,}00467$

↳ Maka dipakai $\rho = \rho_{min} = 0{,}00467$

Luas Tulangan Pokok (As) :

$As = \rho\ .b.d = 0{,}00467.1000.95 = 443{,}65\ mm^2$

Luas 1 batang tulangan $P10_{mm}$; $A_{1\phi} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.10^2 = 78{,}54\ mm^2$

$S_{pokok} = (A_{1\phi}.b)/As$

$= (78{,}54.1000)/\ 443{,}65 = 177{,}03\ mm$

↳ Dipakai **P10 - 170 mm**

Jarak tulangan maksimum : $3.h = 3.\ 120 = 360\ mm < 500\ mm$

Jarak tulangan dipakai 170 mm < 360 mm ⟶ OK !!!

Luas tulangan dipakai; $As_{pakai} = \frac{(A_{1\phi}.1000)}{S}$

$$= \frac{(78{,}54.1000)}{170} = 462\ mm^2 > 443{,}65\ mm^2$$

↳ OK !!! (aman)

Luas Tulangan Susut (As_{susut}) :

$A_{sst} = 0{,}0018.\ b.\ h = 0{,}0018.\ 1000.\ 120 = 216\ mm^2$

Terlihat bahwa $As_{pok} = 443{,}65\ mm^2 > A_{sst} = 216\ mm^2 \longrightarrow$ OK !!!

Dicoba tulangan pokok P6 mm $\rightarrow$ $A_{1\phi} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.8^2 = 50{,}24\ mm^2$

$$S_{tul.susut} \leq \frac{A_{1\phi}.b}{As_{sst}} = \frac{50{,}24.1000}{216} = 232{,}59\ mm$$

$\rightarrow$ Dipakai Tul. Susut : **P8 - 200 mm**

Jarak tulangan maksimum : 5.h = 5. 120 = 600 mm > 500 mm

Jarak tulangan dipakai 200 mm < 500 mm $\longrightarrow$ OK !!!

b. **Diambil Mu^+ ($1/14$) = 8,47 kN.m**

$$Mn = \frac{Mu^+(1/14)}{\phi} = \frac{8{,}47}{0{,}8} = 10{,}59\ kN.m$$

$$Rn = \frac{Mn}{b.d^2} = \frac{10{,}59.10^6}{1000.95^2} = 1{,}17\ MPa$$

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

$$= \frac{1}{15{,}686}.\left[1 - \sqrt{1 - \frac{2.15{,}686.1{,}17}{300}}\right]$$

$= 0{,}004$

Ternyata $\rho = 0{,}004 < \rho_{min} = 0{,}00467$

dan $1{,}33.\ \rho = 1{,}33\ .0{,}004 = 0{,}00532 > \rho_{min} = 0{,}00467$

$\rightarrow$ Maka dipakai $\rho = \rho_{min} = 0{,}00467$

Luas Tulangan Pokok (As) :

As = ρ .b.d = 0,00467.1000.95 = 443,65 mm^2

Luas 1 batang tulangan P10 mm ; $A_{1\phi} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.10^2 = 78,54\ mm^2$

S_{pokok} = $(A_{1\phi}.b)/As$

= (78,54.1000)/ 443,65 = 177,03 mm

↳ Dipakai **P10 - 170 mm**

Jarak tulangan maksimum : 3.h = 3. 120 = 360 mm < 500 mm

Jarak tulangan dipakai 170 mm < 360 mm ⟶ OK !!!

Luas tulangan dipakai; $As_{pakai} = \frac{(A_{1\phi}.1000)}{S}$

$= \frac{(78,54.1000)}{170} = 462\ mm^2 > 443,65\ mm^2$

↳ OK !!! (aman)

Luas Tulangan Susut ($As_{.susut}$) :

A_{sst} = 0,0018. b. h = 0,0018. 1000. 120 = 216 mm^2

Terlihat bahwa As_{pok} = 443,65 mm^2 > A_{sst} = 216 mm^2 ⟶ OK !!!

Dicoba tulangan pokok P6 mm → $A_{1\phi} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.8^2 = 50,24\ mm^2$

$S_{tul.susut} \leq \frac{A_{1\phi}.b}{As_{sst}} = \frac{50,24.1000}{216} = 232,59$ mm

↳ Dipakai Tul. Susut : **P8 - 200 mm**

Jarak tulangan maksimum : 5.h = 5. 120 = 600 mm > 500 mm

Jarak tulangan dipakai 200 mm < 500 mm ⟶ OK !!!

c. **Diambil Mu$^-$ ($\frac{1}{10}$) = 11,86 kN.m**

$$Mn = \frac{Mu^-(\frac{1}{10})}{\phi} = \frac{11,86}{0,8} = 14,83 \text{ kN.m}$$

$$Rn = \frac{Mn}{b.d^2} = \frac{14,83.10^6}{1000.95^2} = 1,64\, MPa$$

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

$$= \frac{1}{15,686}\left[1 - \sqrt{1 - \frac{2.15,686.1,64}{300}}\right]$$

$$= 0,0057$$

Ternyata $\rho = 0,0057 < \rho_{min} = 0,00467$

dan $1,33.\ \rho = 1,33\ .0,0057 = 0,00761 > \rho_{min} = 0,00467$

↳ Maka dipakai $\rho = \rho_{min} = 0,00467$

Luas Tulangan Pokok (As) :

$As = \rho\ .b.d = 0,00467.1000.95 = 443,65\ mm^2$

Luas 1 batang tulangan P10 mm ; $A_{1\emptyset} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.10^2 = 78,54\ mm^2$

$S_{pokok} = (A_{1\emptyset}.b)/As$

$= (78,54.1000)/\ 443,65 = 177,03$ mm

↳ Dipakai **P10 - 170 mm**

Jarak tulangan maksimum : 3.h = 3. 120 = 360 mm < 500 mm

Jarak tulangan dipakai 170 mm < 360 mm ⟶ OK !!!

Luas tulangan dipakai; $As_{pakai} = \frac{(A_{1\phi}.1000)}{S}$

$= \frac{(78,54.1000)}{170} = 462\ mm^2 > 443,65\ mm^2$

↳ OK !!! (aman)

Luas Tulangan Susut ($As_{.susut}$) :

$A_{sst} = 0,0018.\ b.\ h = 0,0018.\ 1000.\ 120 = 216\ mm^2$

Terlihat bahwa $As_{pok} = 443,65\ mm^2 > A_{sst} = 216\ mm^2$ ⟶ OK !!!

Dicoba tulangan pokok P6 mm → $A_{1\phi} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.8^2 = 50,24\ mm^2$

$S_{tul.susut} \leq \frac{A_{1\phi}.b}{As_{sst}} = \frac{50,24.1000}{216} = 232,59\ mm$

↳ Dipakai Tul. Susut : **P8 - 200 mm**

Jarak tulangan maksimum : 5.h = 5. 120 = 600 mm > 500 mm

Jarak tulangan dipakai 200 mm < 500 mm ⟶ OK !!!

d. Diambil Mu$^-$ ($\frac{1}{11}$) = 10,78 kN.m

$Mn = \frac{Mu^-(\frac{1}{11})}{\phi} = \frac{10,78}{0,8} = 13,48\ kN.m$

$Rn = \frac{Mn}{b.d^2} = \frac{13,48.10^6}{1000.95^2} = 1,49\ MPa$

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

$= \frac{1}{15,686}\left[1 - \sqrt{1 - \frac{2.15,686.1,49}{300}}\right]$

$= 0,00518$

Ternyata $\rho = 0,00518 < \rho_{min} = 0,00467$

dan 1,33. ρ = 1,33 .0,00518 = 0,00688 < ρ_{min} = 0,00467

↳ Maka dipakai $\rho = \rho_{min}$ = 0,00467

Luas Tulangan Pokok (As) :

As = ρ .b.d = 0,00467.1000.95 = 443,65 mm^2

Luas 1 batang tulangan P10 mm ; $A_{1\varnothing} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.10^2$ = 78,54 mm^2

S_{pokok} = $(A_{1\varnothing}.b)/As$

= (78,54.1000)/ 443,65 = 177,03 mm

↳ Dipakai **P10 - 170 mm**

Jarak tulangan maksimum : 3.h = 3. 120 = 360 mm < 500 mm

Jarak tulangan dipakai 170 mm < 360 mm ⟶ OK !!!

Luas tulangan dipakai; $As_{pakai} = \frac{(A_{1\phi}.1000)}{S}$

$= \frac{(78,54.1000)}{170}$ = 462 mm^2 > 443,65 mm^2

↳ OK !!! (aman)

Luas Tulangan Susut ($As_{.susut}$) :

A_{sst} = 0,0018. b. h = 0,0018. 1000. 120 = 216 mm^2

Terlihat bahwa As_{pok} = 443,65 mm^2 > A_{sst} = 216 mm^2 ⟶ OK !!!

Dicoba tulangan pokok P6 mm → $A_{1\varnothing} = \frac{1}{4}.\pi.\phi^2 = \frac{1}{4}.\pi.8^2$ = 50,24 mm^2

$S_{tul.susut} \leq \frac{A_{1\phi}.b}{As_{sst}} = \frac{50,24.1000}{216}$ = 232,59 mm

↳ Dipakai Tul. Susut : **P8 - 200 mm**

Jarak tulangan maksimum : 5.h = 5. 120 = 600 mm > 500 mm

Jarak tulangan dipakai 200 mm < 500 mm ⟶ OK !!!

⟶ *Untuk kebutuhan tulangan pelat lantai dan atap satu arah secara keseluruhan dapat dilihat pada tabel 4.1 (terlampir)*

4.2.2 Perencanaan Pelat Lantai Dua Arah

4.2.2.1 Pembebanan Pelat Lantai

Perhitungan beban pelat lantai sesuai ketentuan PPPURDG 1987, sebagai berikut :

- Beban Mati Pelat Lantai :

1. Berat sendiri pelat lantai : 0,12 x 1,0 x 1,0 x 24 = 2,88 kN/m^2
2. Pasir (tebal 5 cm) : 0,05 x 1,0 x 1,0 x 16 = 0,80 kN/m^2
3. Spesi (tebal 3 cm) : 0,03 x 1,0 x 1,0 x 21 = 0,63 kN/m^2
4. Keramik : 0,01 x 1,0 x 1,0 x 20 = 0,20 kN/m^2 +

Beban Mati Total (qD) = 4,51 kN/m^2

- Beban Hidup Pelat Lantai :

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

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

qU = 1,2. qD + 1,6. qL = 1,2. 4,51 + 1,6. 2,5 = 9,412 kN/m^2

Koefisien momen (distribusi momen) (x) = ly/lx

4.2.2.2 Penulangan Pelat Lantai

Beton : f'c = 22,5 Mpa ; ⟶ $\beta_1 = 0,85$

Jika : f'c ≤ 30 Mpa maka $\beta_1 = 0,85$

f'c > 30 Mpa maka $\beta_1 = 0,85 - 0,008.(f'c - 30)$

Baja Polos : fy = 300 Mpa. ⟶ $\varepsilon y = \frac{fy}{Es} = \frac{300}{200000} = 0,0015$

$$\rho b = \frac{0,85.f'c.}{fy}.\beta_1.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{300}.0,85.\left(\frac{600}{600+300}\right) = 0,0361$$

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

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{300} = 0,00467$$

$$m = \frac{fy}{0,85.f'c} = \frac{300}{0,85.22,5} = 15,686$$

4.2.2.3 Perhitungan Pelat Type PL-2 (4000 x 4000)

Panjang Bentang Pendek Lx = 4000 = 4,0 m

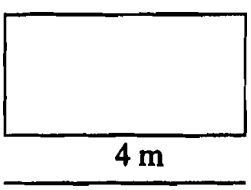
Panjang Bentang Panjang Ly = 4000 = 4,0 m

Ly/Lx = 1,0 m ; Ly/Lx ≤ 2,0 dianggap ⟶ "pelat dua arah".

Lebar balok (asumsi) = 450 mm= 0,45 m

Menentukan Tebal Pelat, dengan f'c = 22,5 Mpa ; fy = 300 MPa :

- Menghitung distribusi momen



ly = 4 m

lx = 4 m

ly/lx = 1,0

Dari tabel 13.3.2.PBI 1971 (tumpuan tepi dianggap menerus atau jepit elastis)

Didapat : clx = 36 ctx = 36

cly = 36 cty = 36

Mulx = 0,001.qu.lx^2.clx = 0,001 . 9,412 . 4^2 . 36 = 5,43 KNm

Mutx = - 0,001.qu.lx^2.ctx = - 0,001 . 9,412 . 4^2 . 36 = - 5,43 KNm

Muly = 0,001.qu.lx^2.cly = 0,001 . 9,412 . 4^2 . 36 = 5,43 KNm

Muty = - 0,001.qu.lx^2.cty = - 0,001 . 9,412 . 4^2 . 36 = - 5,43 KNm

4.2.2.4 Perhitungan Tulangan Pelat Lantai

- Perencanaan tulangan lx = tx

h = 120 mm

d = h – pb – ½ Øtul tx = 120 – 20 – ½.8 = 94 mm

Mu = 5,43 KNm

$Mu/\varnothing = \frac{5,43}{0,8} = 6,79$ KNm

$Rn = \frac{Mu./\phi}{b.d^2} = \frac{6,79 .10^6}{1000.94^2} = 0,77$ Mpa

$m = \frac{fy}{0,85.f'c} = \frac{300}{0,85.22,5} = 15,69$

$\rho_{perlu} = \frac{1}{m}.\left(1-\sqrt{1-\frac{2.m.Rn}{fy}}\right) = \frac{1}{15,69}.\left(1-\sqrt{1-\frac{2 .15,69. 0,77}{300}}\right)$

$= 0,0026 < \rho_{min} = \frac{1,4}{fy} = \frac{1,4}{300} = 0,0047$

$\rho b = \frac{0,85.f'c.}{fy}.\beta_1.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{300}.0,85.\left(\frac{600}{600+300}\right) = 0,0361$

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

1,33 ρ perlu = 1,33 . 0,0026 = 0,00349

Syarat :

1) Bila $\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{perlu}$

2) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

   Bila $1,33.\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

3) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1,33 . \rho_{min}$

   Bila $1,33.\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1,33 . \rho_{min}$

Menurut Syarat (2) maka $\rho_{pakai} = \rho_{min} = 0,00349$

As perlu = ρ terpakai.b.d = 0,00349. 1000 . 94 $\geq$ 0,002 . 1000 . 120

$= 328,06 \text{ mm}^2$

As perlu pakai = 328,06 mm^2

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

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = \frac{50.1000}{328,06}$

$= 152,4\text{mm} \leq 2.h = 240 \text{ mm}$

$\leq 250 \text{ mm}$

Dipakai jarak (s) = 150 mm

$$\text{As ada} = \frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{50.1000}{150} = 333,33 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0,85 \cdot f'c \cdot b} = \frac{333,33 \,.\, 300}{0,85 \,.\, 22,5 \,.1000} = 5,22 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq Mu/\phi$$

$= 333{,}33 \;.\; 300 \cdot \left(94 - {}^{5{,}22}/_{2}\right) \geq 6{,}79.1{,}33 = 9{,}03 \;\text{KNm}$

$= 9{,}14 \;\text{KNm} \geq 9{,}03 \;\text{KNm}$

$\Rightarrow$ Dipakai tulangan P8 – 150 mm

- Perencanaan tulangan ly

h = 120 mm

d = h – pb – Øtul lx – ½ Øtul ly = 120 – 20 – 8 – 8/2 = 88 mm

Mu = 5,43 KNm

$Mu/\varnothing = \frac{5{,}43}{0{,}8} = 6{,}79 \;\text{KNm}$

$Rn = \frac{Mu./\phi}{b.d^2} = \frac{6{,}79 \;.10^6}{1000.94^2} = 0{,}77 \;\text{Mpa}$

$m = \frac{fy}{0{,}85.f'c} = \frac{300}{0{,}85 \;.\; 22{,}5} = 15{,}69$

$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot Rn}{fy}}\right) = 0{,}00114 \;<\; \rho_{min} = \frac{1{,}4}{300} = 0{,}0047$

$\rho b = \frac{0{,}85.f'c.}{fy} . \beta_1 . \left(\frac{600}{600 + fy}\right) = \frac{0{,}85.22{,}5}{300} . 0{,}85 . \left(\frac{600}{600 + 300}\right) = 0{,}0361$

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

1,33 ρ perlu = 1,33 . 0,0026 = 0,00349

Syarat :

1) Bila $\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{perlu}$

2) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

   Bila $1{,}33.\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

3) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1{,}33 \;.\; \rho_{min}$

Bila $1{,}33.\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1{,}33 \cdot \rho_{min}$

Menurut Syarat (3) maka $\rho_{pakai} = 1{,}33.\rho_{perlu} = 0{,}00349$

As perlu = ρ terpakai.b.d = 0,00349 . 1000 .85 ≥ 0,002 . 1000.120

$= 296{,}65 \text{ mm}^2 \geq 240 \text{ mm}^2$

As perlu pakai = $296{,}65 \text{ mm}^2$

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

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = \frac{50.1000}{296{,}65} = 168{,}55$ mm

≥ 2.h = 240 mm

≥ 250 mm

Dipakai jarak (s) = 150 mm

As ada = $\frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{50.1000}{150} = 333{,}33 \text{ mm}^2$

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0{,}85 \cdot f'c \cdot b} = \frac{333{,}33 \,.\, 300}{0{,}85 \,.\, 22{,}5 \,.1000} = 5{,}22 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq Mu/\phi \,.1{,}33$$

$$= 333{,}33 \,.\, 300 \cdot \left(88 - 4{,}9/2\right) \geq 6{,}79.1{,}33 = 9{,}03 \text{ KNm}$$

$= 9{,}15 \text{ KNm} \geq 9{,}03 \text{ KNm}$

⇒ Dipakai tulangan P10 – 150 mm

▲ Perencanaan tulangan ty

h = 120 mm

d = h – pb – ½ Øtul ty = 120 – 20 – ½ 8 = 96 mm

Mu = 5,43 KNm

$$Mu/\varnothing = \frac{5,43}{0,8} = 6,79 \text{ KNm}$$

$$Rn = \frac{Mu./\phi}{b.d^2} = \frac{6,79 .10^6}{1000.94^2} = 0,77 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{300}{0,85.22,5} = 15,69$$

$$\rho_{perlu} = \frac{1}{m}.\left(1-\sqrt{1-\frac{2\cdot m\cdot Rn}{fy}}\right) = \frac{1}{15,69}.\left(1-\sqrt{1-\frac{2\ .15,69.\ 0,77}{300}}\right)$$

$$= 0,0026 < \rho_{min} = \frac{1,4}{fy} = \frac{1,4}{300} = 0,0047$$

$$\rho b = \frac{0,85.f'c.}{fy}.\beta_1.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{300}.0,85.\left(\frac{600}{600+300}\right) = 0,0361$$

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

1,33 ρ perlu = 1,33 . 0,0026 = 0,00349

Syarat :

1) Bila $\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{perlu}$

2) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

   Bila $1,33.\rho_{perlu} > \rho_{min}$ maka $\rho_{pakai} = \rho_{min}$

3) Bila $\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1,33\ .\ \rho_{min}$

   Bila $1,33.\rho_{perlu} < \rho_{min}$ maka $\rho_{pakai} = 1,33\ .\ \rho_{min}$

Menurut Syarat (2) maka $\rho_{pakai} = \rho_{min} = 0{,}00349$

As perlu = ρ terpakai.b.d = 0,00349. 1000 . 94 ≥ 0,002 . 1000 . 120

$= 328{,}06 \text{ mm}^2$

As perlu pakai = $328{,}06 \text{ mm}^2$

Dipakai tulangan pokok Ø8 mm dengan A1Ø = 50 mm^2

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = \frac{50.1000}{328{,}06}$

$= 152{,}4\text{mm} \leq 2.h = 240 \text{ mm}$

$\leq 250 \text{ mm}$

Dipakai jarak (s) = 150 mm

As ada = $\frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{50.1000}{150} = 333{,}33 \text{ mm}^2$

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0{,}85 \cdot f'c \cdot b} = \frac{333{,}33 \,.\, 300}{0{,}85 \,.\, 22{,}5 \,.1000} = 5{,}22 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq Mu/\phi \,.1{,}33$$

$$= 333{,}33 \,.\, 300 \cdot \left(94 - 5{,}22/2\right) \geq 6{,}79.1{,}33 = 9{,}03 \text{ KNm}$$

$= 9{,}14 \text{ KNm} \geq 9{,}03 \text{ KNm}$

⇒ Dipakai tulangan P8 – 150 mm


ISLAM
UNIVERSITAS
INDONESIA

4.3 Perencanaan Struktur Portal Dengan Daktilitas Penuh

Pada perencanaan ulang gedung kampus Jurusan Akuntansi Fakultas Ekonomi UPN "Veteran" Yogyakarta ini, perencanaan portal dianalisis dengan SAP2000 dengan analisis struktur tiga (3) Dimensi. Dan beban yang bekerja pada struktur adalah sebagai berikut :

a. Beban Mati

- Pembebanan Pelat Lantai :

  Beban Pelat Lantai (qD) = 4,51 kN/m^2 (terlampir)

- Pembebanan Pelat Atap :

  Beban Pelat Atap (qD) = 3,06 kN/m^2 (terlampir)

- Beban Dinding ½ bata = (tinggi lantai x bj x tebal dinding)

  = (4,2 .18 .0,15) = 11,34 kN/m'

- Beban pegangan selasar ½ bata = (tgg pegangan x bj x tebal dinding)

  = (1 .18 .0,10) = 2,7 kN/m'

- Beban Listplank beton = (pjg listplank x tebal listplank) . bj

  = (2,1 .0,12) .24 = 6,9 kN/m'

- Beban balok Listplank beton = (pjg x tebal x tinggi) . bj

  = {(2,1 .0,2.0,3) .24}= 3,024 kN

- Beban Konsol Atap Genteng = {(pjg x tebal x tinggi) . bj}

  = {(3,3 .0,2.0,3) .24}= 4,752 kN

- Beban Plafond = 0,18 kN/m^2

• Beban Atap Genteng pada Konsol = { lebar . bj}

= 3,3 . 0,5 kN/m^2 = 1,65 kN/m'

(PPIUG'1983 tabel 3.1, halaman 17)

• Beban Kuda-Kuda Baja (kk1 dan kk2)

$R_a = R_b = P_{maks}$ = 1819 kg = 18,19 kN

P1 dan P2 = 18,19 kN

• Beban Kuda-Kuda Baja (kk3, kk4 dan kk5)

Diperkirakan $\frac{1}{3}$ berat kk1 dan jumlahnya ada 3 titik

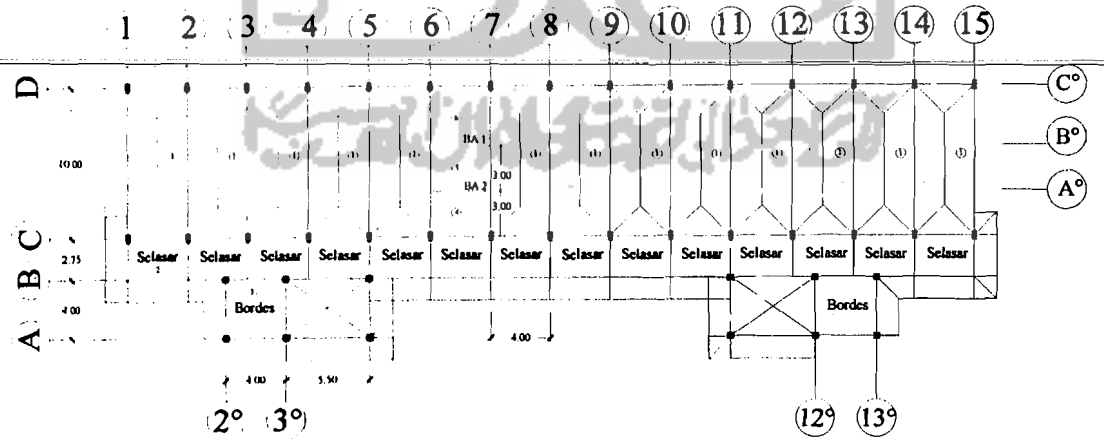
P3, P4 dan P5 = beban Kuda-kuda = ($\frac{1}{3}$ .18,19) = 6,0625,125 kN

b. Beban Hidup

• Beban Hidup (qL) Pelat Lantai untuk Ruang Kuliah = 2,5 kN/m^2

(PPIUG'1983 tabel 3.1, halaman 17)

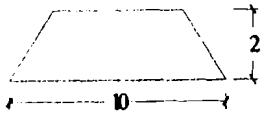
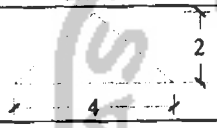
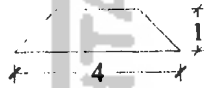
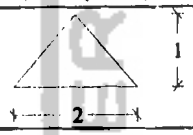
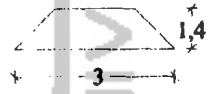
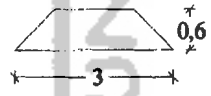
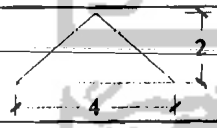
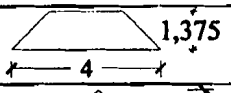
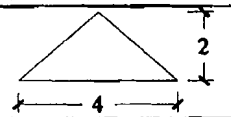
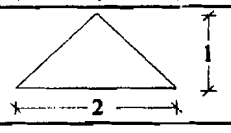
• Beban Hidup (qL) Pelat Atap = 1.0 kN/m^2

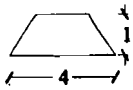
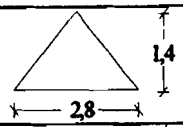
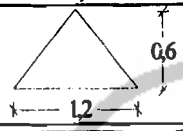
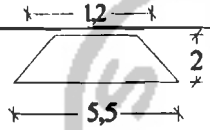


Gambar 4.15 Pembebanan Pelat Lantai 1 - 3

Pemodelan jenis beban pada SAP 2000 sebagai berikut :

Tabel 4.4 Type Pembebanan

Tinjauan Arah Melintang (sb Y)					
Nama Type	**Gambar Type**	**Panjang (arah y)**	**Lebar (arah x)**	**Tinggi (z)**	**As:Bntang**
Type - 1		10	4	2	1 : C- D
Type - 2		2,75	4	1,375	1 : B-C
Type - 3		4	4	2	2^0 : A-B
Type - 4		4	2	1	7 : C^0-D^0
Type - 5		4	1	0,5	7 : B^0-C^0
Type - 6		2	4	1	7 : A^0-B^0
Type - 7		3	2,8	1,4	7 : C-A^0
Type - 8		3	1,2	0,6	7 : C-A^0
Type - 9		4	5,5	2	3^0 : A-B
Tinjauan Arah Memanjang (sb X)					
Type – 1		10	4	2	1 : C- D
Type – 2		2,75	4	1,375	1 : B-C
Type – 3		4	4	2	2^0 : A-B
Type – 4		4	2	1	7 : C^0-D^0

Type – 5		4	1	0,5	7 : B^0-C^0
Type – 6		2	4	1	7 : A^0-B^0
Type – 7		3	2,8	1,4	7 : C-A^0
Type – 8		3	1,2	0,6	7 : C-A^0
Type - 9		4	5,5	2	3^0 : A-B

4.3.1 Perhitungan Gaya Geser Dasar Horisontal Total Akibat Gempa

Tabel 4.5 Beban Yang Terjadi

Lantai	Elemen	b (m) (a)	h (m) (b)	l (m) (c)	Faktor BJ (KN) (d)	Faktor reduksi (e)	Total beban (KN) (a).(b).(c).(d).(e)
L. Dasar							
	B. Mati						
	Sloof	0,3	0,5	249	24	1	747
		0,3	0,5	63,5	24	1	57,15
	Kolom	0,6	0,6	116	24	1	445,44
		0,5	0,5	50,4	24	1	314,50
	Dinding	0.15	4,2	250	18	1	2835
Total beban (Wt1)							4399,09
Lt 1							
	B. Mati						
	pelat	12,75	0,12	56	24	1	2056,32
	balok	0,45	0,8	191,25	24	1	1101,60
		0,2	0,4	8	24	1	15.36
	kolom	0,6	0,6	168	24	1	1048,32
	Dinding selasar	0,15	1	45	18	1	121,50
	dinding	0,15	4,2	152	18	1	1723,68
	keramik	12,75	0,01	56	24	1	171,36

	spesi	12,75	0,03	56	21	1	449,82
	~~konsol beton~~	~~0,2~~	~~0,3~~	~~71,4~~	~~24~~	~~1~~	~~102,82~~
	pelat konsol	0,12	2,1	35,5	24	1	214,70
	ringbalk konsol	0,15	0,3	144	24	1	155,52
	pasir	12,75	0,05	56	18	1	642,60
	penggantung	12,75	0,05	56	0,18	1	6,43
	B. Hidup	12,75	1	56	2,5	0,5	892,50
Total beban (Wt2)							**8702,53**
Lt 2							
	B. Mati						
	pelat	12,75	0,12	56	24	1	2056,32
	balok	0,45	0,8	191,25	24	1	1101,60
		0,2	0,4	8	24	1	15,36
	kolom	0,4	0,65	168	24	1	1048,32
	dinding	0,15	1	152	18	1	1723,68
	Dinding selasar	0,15	1	45	18	1	121,50
	keramik	12,75	0,01	56	24	1	171,36
	spesi	12,75	0,03	56	21	1	449,82
	pasir	12,75	0,05	56	18	1	642,60
	konsol beton	0,2	0,3	71,4	24	1	102,82
	pelat konsol	0,12	2,1	35,5	24	1	214,70
	ringbalk konsol	0,15	0,3	144	24	1	155,52
	penggantung	15,75	0,05	56	0,18	1	7,94
	B. Hidup	12.75	1	56	2.5	0,5	1440
Total beban (Wt3)							**9251,54**
Lt 3							
	B. Mati						
	Dinding selasar	0,15	1	45	18	1	121,50
	pelat	12,75	0,12	56	24	1	2056,32
	balok	0,45	0,8	191,25	24	1	1101,60
		0,2	0,4	8	24	1	19,20
	kolom	0,6	0,6	168	24	1	1048,32
	keramik	12,75	0,01	56	24	1	171,36
	spesi	12,75	0,03	56	21	1	449,82
	pasir	12,75	0,05	56	18	1	642,60
	konsol beton	0,2	0,3	71,4	24	1	102,82
	pelat konsol	0,12	2,1	35,5	24	1	214,70

	ringbalk konsol	0,15	0,3	144	24	1	155,52
	penggantung	15,75	0,05	56	0,18	1	7,94
	B. Hidup	12,75	1	56	2,5	0,5	892,50
Total beban (Wt4)							**6984,20**
Atap							
	B. Mati						
	Genteng pd konsol	3,3	149,5		0,5	1	246,675
	konsol beton	0,2	0,3	112,2	24	1	161,57
	ringbalk konsol	0,15	0,3	144	24	1	155,52
	ringbalk	0,45	0,8	132	24	1	253,44
	penggantung	15	1	36	0,18	1	97,2
	kuda kuda 1 = (kk 1 + atap + gording) x 11					1	400,1228
	kuda kuda 2 = kk 2 + atap + gording) x 2					1	36,3748
	kuda kuda 3 = (1/3.(kk1+ atap + gording)) x 6					1	72,7498
	kuda kuda 4 = (1/3.(kk1+ atap + gording)) x 2					1	24,2499
	kuda kuda 5 = (1/3.(kk1+ atap + gording)) x 6					1	72,7498
Total beban (Wt5)							**1447,90**
			W. TOTAL				**30785,25**

Gaya geser dasar horisontal akibat gempa dipengaruhi oleh berat total dari keseluruhan struktur yang direncanakan ditambah dengan beban hidup yang bekerja. Sesuai fungsi penggunaan gedung, yaitu sebagai gedung kuliah, maka menurut Peraturan Pembebanan Indonesia 1983 (tabel 3.3) untuk perencanaan beban gempa, beban hidup direduksi sebesar 0,5.

A. Waktu Getar Bangunan (T)

Waktu getar struktur untuk struktur portal terbuka beton bertulang dapat dihitung dengan :

$T = 0{,}06 \cdot H^{3/4} \cdot = 0{,}06 \cdot 16{,}8^{3/4} = 0{,}98$ detik

B. Koefisien Gempa Dasar (C)

Pada redesain ini, bangunan berada pada wilayah gempa tiga (3), pada kondisi tanah lunak. Waktu getar struktur (T) = 0,98 dtk, maka berdasarkan respon spektrum wilayah 3 didapatkan koefisien gempa dasar C = 0,76 (Gambar 2 SNI-1726-2002)

C. Faktor Keutamaan (I) dan Faktor Jenis Struktur (K)

Berdasarkan fungsi bangunan, maka faktor keutamaan bangunan (T) diambil = 1,0 (tabel 1 SNI-1726-2002 (page 7)).

Sedangkan untuk faktor faktor reduksi gempa (R) diambil = 8,5 (tabel 2 SNI-1726-2002 (page 10))yaitu untuk daktail penuh.

D. Gaya Geser Horisontal Akibat Gempa (V)

Gaya geser horisontal akibat gempa yang bekerja dapat dihitung dengan :

$$V = \frac{C_1.I}{R} W_T = \frac{0,76.1}{8,5}.30785,25 = 2771,34 \text{ KN}$$

E. Distribusi Gaya Horisontal Akibat Gempa Kesepanjang Tinggi Gedung

1) Arah X

$$\frac{H}{B} = \frac{16,8}{56} = 0,3 < 3, \text{ maka :}$$

$$\text{Fix} = \frac{Wi.hi}{\sum Wi.hi}.V$$

2) Arah Y

$$\frac{H}{B} = \frac{16,8}{12,75} = 1,32 < 3, \text{ maka :}$$

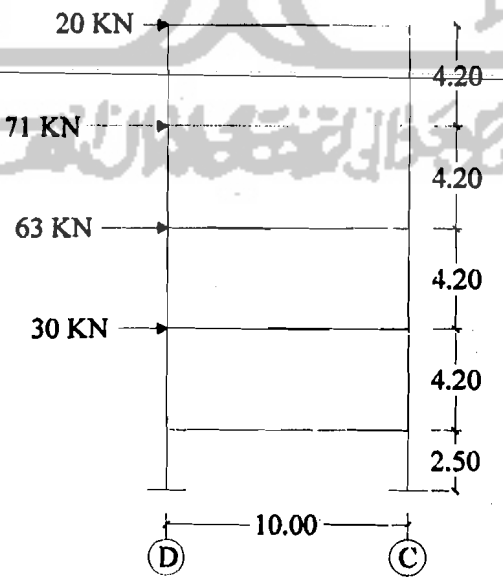
$$\text{Fiy} = \frac{Wi.hi}{\sum Wi.hi}.V$$

Tabel 4.6 Distribusi Gaya Geser Horisontal Total Akibat Gempa Arah X dan Y

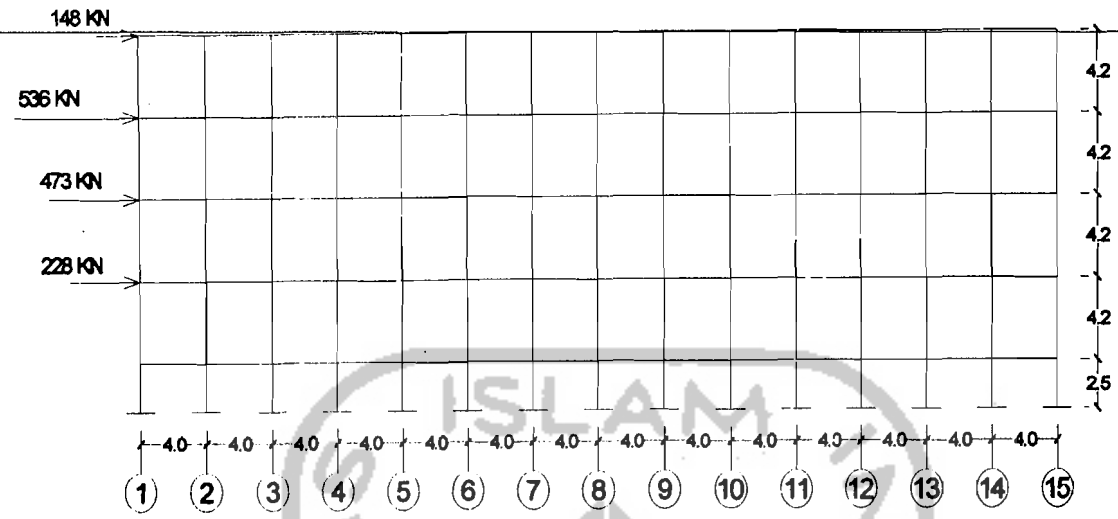
Tingkat	hi (m)	Wi (KN)	V (KN)	Wi.hi (KNm)	$F_i = \frac{W_i.H_i}{\sum W_i.H_i} \cdot V$
Atap	16.8	1447,90	2771,34	24324,72	296,35
Lt 3	12.6	6984,20	2771,34	88000,92	1072,14
Lt 2	8.4	9251,54	2771,34	77712,936	946,767
Lt 1	4.2	8912,53	2771,34	37432,626	456,05
Lt dasar	0	2835	2771,34	0	0
		Σ		227471,202	2771,307

Tabel 4.7 distribusi Gaya Geser Horisontal untuk Tiap Portal ArahX dan ArahY

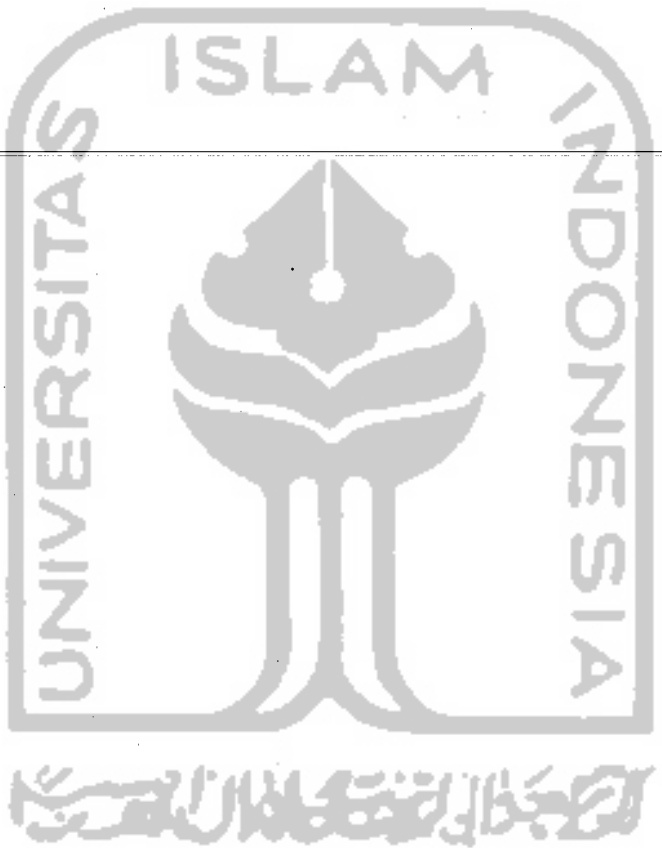
Tingkat	Arah X	Arah Y
	1/15 .Fi, x	(1/2.Fi,y)
Atap	20	148
Lt 3	71	536
Lt 2	63	473
Lt 1	30	228
Lt dasar	0	0



Gambar 4.16 Distribusi Gaya Geser Horisontal Arah X



Gambar 4.17 Distribusi Gaya Geser Horisontal Arah Y


ISLAM
UNIVERSITAS
INDONESIA

4.4 Desain Balok Induk

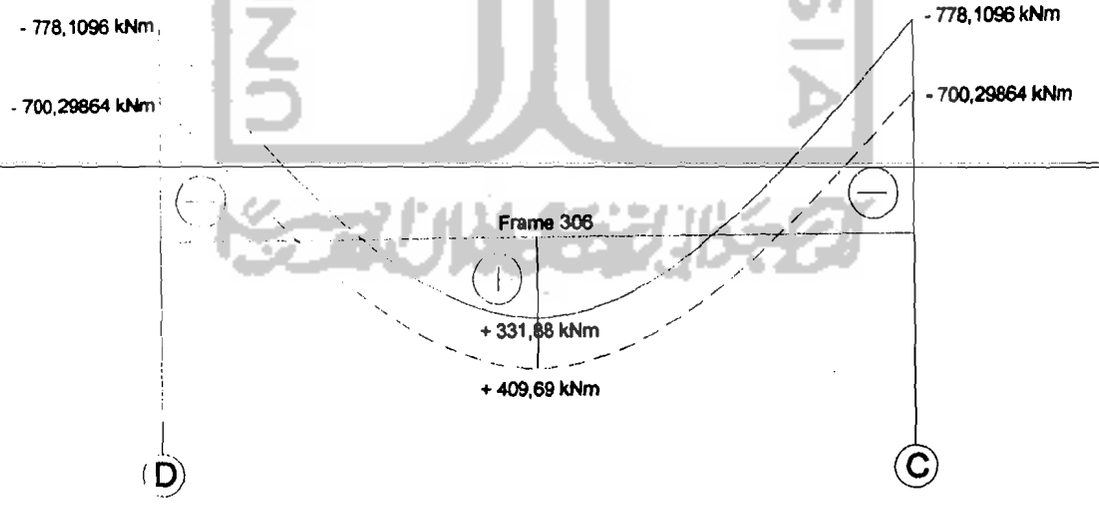
4.4.1 Portal Arah – Y (Melintang)

A. Momen Rencana Balok

Momen rencana balok diambil yang terbesar dari hasil kombinasi beban sebagai berikut :

- $1{,}2\ M_D + 1{,}6\ M_L$
- $0{,}9\ M_D \pm M_E$
- $1{,}05\ (M_D + 0{,}65.M_L \pm M_E)$

Berikut ini contoh perhitungan balok Balok Induk portal–Y melintang, As 7 (bentang D-C) lantai 1.



Gambar 4.18 Distrubusi Momen Pada Portal Y-Melintang As-7 Bentang D-C

B. Tulangan Tumpuan

Momen tumpuan diambil yang paling besar dari semua kombinasi, dan momen yang digunakan adalah yang terbesar dari kedua tumpuan (kiri & kanan).

Dimensi rencana balok Induk $^{450}/_{800}$, maka :

fc' = 22,5 Mpa fy = 350 Mpa

- Tulangan tumpuan momen negatif

  Mu_{awal} = 778,1096 kNm (-)

  Dalam perencanaan kapasitas balok portal, momen negatif akibat kombinasi beban gravitasi dan beban gempa balok boleh diredistribusikan dengan menambah atau mengurangi dengan prosentase yang tidak melebihi:

$$q = 30\left(1 - \frac{4}{3}\frac{\rho - \rho'}{\rho b}\right)\%$$

dengan syarat apabila tulangan lentur balok portal telah direncanakan sehingga (ρ-ρ ') tidak melebihi 0,5.ρb. (SKSNI T-15-1991-03 Pasal 3.1.4)

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{350}.0,85\left(\frac{600}{600+350}\right) = 0,029$$

$$= 30\left(1 - \frac{4}{3}\frac{0,5.0,029'}{0,029}\right)\%$$

$$= 10\,\%$$

Momen tumpuan didistribusikan ke momen lapangan sebesar = 10 %

$$\frac{10}{100}x778,1096 = 77,81096\ kNm$$

Mu akibat distribusi momen = 778,1096 - 77,81096 = 700,29864 kNm

$$\frac{Mu}{\phi} = \frac{700,29864}{0,8} = 875,52 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{350}.0,85\left(\frac{600}{600+350}\right) = 0,0293$$

$\rho_{maks} = 0,75 \cdot \rho b = 0,75 \cdot 0,029 = 0,022$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{350} = 0,004$$

$$\rho_{pakai} = 0,5 \cdot \rho_{maks} = 0,5 \cdot 0,022 = 0,011 \rightarrow (\rho - \rho')$$

$$m = \frac{fy}{0,85.f'c} = \frac{350}{0,85.22,5} = 18,30$$

$$Rn = \rho\, fy\,(1 - \tfrac{1}{2}\rho\, m) = 0,011.350\,(1 - \tfrac{1}{2} \cdot 0,011.18,30) = 3,462 \text{ Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{875,52 \cdot 10^6}{3,462 \cdot 450}} = 749,59 \text{ mm}$$

$d_{ada} = h - d' = 800 - 100 = 700 \text{ mm} < d_{perlu} = 749,59 \text{ mm}$

(d' = 100 mm, diasumsikan menggunakan tul 2 lapis)

maka dipakai **Tulangan Rangkap**

$$\rho_1 = (\rho - \rho') \rightarrow 0,011$$

Luas Tulangan Tarik

$As_1 = \rho_1 \cdot b \cdot d_{ada}$

$As_1 = 0,011 .450 .700 = 3465,15 \text{ mm}^2$

Asumsi Baja Tulangan Tarik Telah Leleh

$$a = \frac{As_1.fy}{0,85.fc'.b} = \frac{3465,15.350}{0,85.22,5.450} = 140,921 \text{ mm}$$

$$Mn_1 = As_1 \cdot fy \cdot \left(d_{ada} - a/2\right) < Mu/\phi$$

$$= 3465{,}15 \cdot 350 \cdot \left(700 - 140{,}921/2\right) \cdot 10^{-6}$$

$$= 763{,}507 \text{ KNm} < Mu/\phi = 875{,}52 \text{ KNm}$$

Kelebihan Mn ditahan oleh tambahan tulangan tarik dan tulangan tekan:

$$Mn_2 = Mu/\phi - Mn1$$

$$Mn_2 = 875{,}52 - 763{,}507 = 112{,}013 \text{ KNm}$$

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

$$fs' = 600 \cdot \left\{1 - \frac{0{,}85 \cdot 22{,}5 \cdot 0{,}85}{0{,}011 \cdot 350} \cdot \frac{100}{700}\right\} = 238{,}095 \text{ Mpa} \leq 350 \text{ Mpa}$$

Syarat:

jika $fs' \leq fy$ maka $fs' = fs'$

jika $fs' \geq fy$ maka $fs' = fy$

Luas Tulangan Tekan

$$As' = As_2 = \frac{Mn_2}{fs' \cdot \left(d_{ada} - d'\right)}$$

$$As' = As_2 = \frac{112{,}013 \cdot 10^6}{238{,}095 \cdot (700 - 100)} = 784{,}094 \text{ mm}^2$$

Dipakai tulangan Ø22 dengan $A_{1Ø} = 380{,}1336 \text{ mm}^2$

$$\text{jumlah tulangan (n')} = \frac{As'}{A_{1\phi}} = \frac{784{,}094}{380{,}1336} = 2{,}063 \text{ batang}$$

dipakai **3D22**, maka $As_{ada} = 3 \cdot 380{,}1336 = 1140{,}398 \text{ mm}^2 > As_2$

Luas Tulangan Tarik

$As = As_1 + As_2 = 3465,15 + 784,094 = 4249,242\ mm^2$

Dipakai tulangan Ø22 dengan $A_{1\emptyset} = 380,1336\ mm^2$

jumlah tulangan (n') = $\frac{As}{A_{1\phi}} = \frac{4294,242}{380,1336} = 11,178$ batang

dipakai **12D22**, maka $As_{ada} = 12 \,.\, 380,1336 = 4561,593\ mm^2 > As$

Kontrol kapasitas lentur:

$$\rho = \frac{As_{ada}}{b \cdot d_{diketahui}}$$

$$\rho = \frac{4561,593}{450 \cdot 700} = 0,014$$

$$\rho' = \frac{As'}{b \cdot d_{diketahui}}$$

$$\rho' = \frac{784,094}{450.700} = 0,004$$

$$fs' = 600 \cdot \left\{1 - \frac{0,85 \cdot 22,5 \cdot 0,85}{(0,014 - 0,004) \cdot 350} \cdot \frac{100}{700}\right\} = 233,446\ Mpa \leq 350\ Mpa$$

$$a = \frac{As \cdot fy - As' \cdot fs'}{0,85 \cdot f'c \cdot b} = \frac{4561,593.350 - 784,094.233,446}{0,85.22,5.450} = 154,578\ mm$$

$$Mn = (As \cdot fy - As' \cdot fs') \cdot \left(d_{diketahui} - a/2\right) + (As' \cdot fs') \cdot (d_{diketahui} - d')$$

$$= (((4561,593.350) - (784,094.233,446)).(700 - (154,578/2)))$$

$$+ ((784,094.233,446).(700 - 100))$$

$$= 988,148\ KNm > Mu/\phi = 875,52\ KNm \text{ -OK-}$$

Jarak bersih antar tulangan $= \dfrac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul.}{(n-1)}$

$$= \frac{450 - 2.40 - 2.10 - 12.22}{(12-1)}$$

$$= 7{,}81 \text{ mm} < 25 \text{ mm}$$

maka dipakai tulangan 2 lapis

- Tulangan tumpuan momen positif

  Untuk menghitung tulangan yang digunakan pada bangunan tahan gempa SNI mengisyaratkan untuk mengambil nilai terbesar dari keterangan berikut :

  1. Jumlah tulangan momen positif diambil ≥ 50 % jumlah tulangan momen negative pada tumpuan.
  2. Jumlah tulangan momen positif diambil ≥ 30 % jumlah tulangan momen positif pada lapangan.

  Dikarenakan besarnya momen negative tumpuan dengan momen positif lapangan selisihnya tidak terlalu besar maka peraturan no 2 tidak perlu digunakan. Maka jumlah tulangan tumpuan momen positif

  = 50% . jumlah tulangan tumpuan

  = 50% . 12 tulangan D22

  = 6 tulangan D22

$$\text{Jarak bersih antar tulangan} = \frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul.}{(n-1)}$$

$$= \frac{450 - 2.40 - 2.10 - 12.22}{(12-1)}$$

$$= 7{,}81 \text{ mm} < 25 \text{ mm}$$

maka dipakai tulangan 2 lapis

- Tulangan tumpuan momen positif

  Untuk menghitung tulangan yang digunakan pada bangunan tahan gempa SNI mengisyaratkan untuk mengambil nilai terbesar dari keterangan berikut :

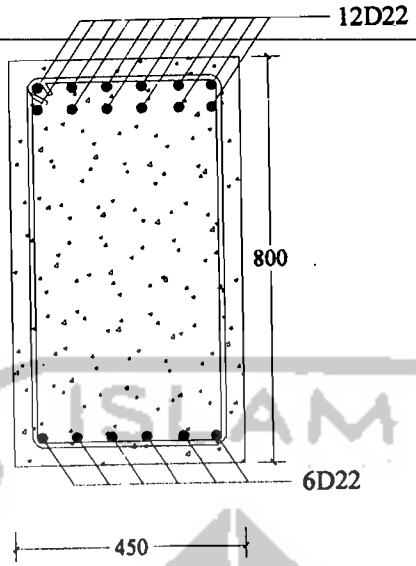
  1. Jumlah tulangan momen positif diambil ≥ 50 % jumlah tulangan momen negative pada tumpuan.
  2. Jumlah tulangan momen positif diambil ≥ 30 % jumlah tulangan momen positif pada lapangan.

  Dikarenakan besarnya momen negative tumpuan dengan momen positif lapangan selisihnya tidak terlalu besar maka peraturan no 2 tidak perlu digunakan. Maka jumlah tulangan tumpuan momen positif

  = 50% . jumlah tulangan tumpuan

  = 50% . 12 tulangan D22

  = 6 tulangan D22



Gambar 4.19 Tulangan Tumpuan Balok Induk

C. Tulangan Lapangan

Mu_{awal} = 331,88 KNm (+) : Mu dist. momen = 331,88 + 77,81 = 409,69 kNm

$$\frac{Mu}{\phi} = \frac{409,69}{0,8} = 509,1 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{350}.0,85\left(\frac{600}{600+350}\right) = 0,029$$

ρ_{maks} = 0,75 . ρb = 0,75 . 0,029 = 0,022

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{350} = 0,004$$

ρ_{pakai} = 0,5 . ρ_{maks} = 0,5 . 0,022 = 0,011

$$m = \frac{fy}{0,85.f'c} = \frac{350}{0,85.22,5} = 18,3$$

Rn = ρ fy (1 – ½ ρ m) = 0,011 . 350 (1 - ½ .0,011 . 18,3) = 3,46 Mpa

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{509{,}1 \,.\, 10^6}{3{,}46 \,.\, 450}} = 571{,}62 \text{ mm} < d_{ada} = 800 - 100 = 700 \text{ mm}$$

maka dipakai **tulangan sebelah**.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{509{,}1.10^6}{450.700^2} = 2{,}31 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho_{pakai} = \frac{2{,}31}{3{,}46}.0{,}011 = 0{,}0073 > \rho_{min} = 0{,}004$$

$$< \rho_{maks} = 0{,}022$$

$$\rho_{pakai} = \rho_{ada} = 0{,}0076$$

$As_{perlu} = \rho_{ada}.b.d = 0{,}0073.450.700 = 2311 \text{ mm}^2$

Dipakai tulangan Ø22 dengan $A_1Ø = 380{,}1336 \text{ mm}^2$

jumlah tulangan (n) $= \frac{As_{perlu}}{A_1\phi} = \frac{2311}{380{,}1336} = 6{,}1 \approx 7$ batang

dipakai **7D22**, maka $As_{ada} = 7 \,.\, 380{,}1336 = 2661 \text{ mm}^2 > As_{perlu} = 2350 \text{ mm}^2$

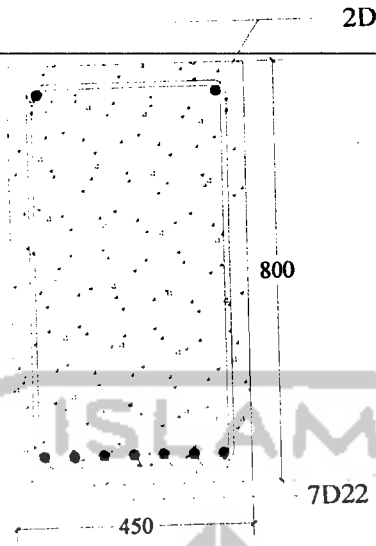
Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{2661 \,.350}{0{,}85.22{,}5.450} = 115{,}97 \text{ mm}$$

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

$= 2661 \,.\, 350 \,.\, (\, 700 - {}^{108{,}2}/_{2}\,) .10^{-6}$

$= 602 \text{ KNm} > {}^{Mu}/_{\phi} = 509{,}1 \text{ KNm} \rightarrow$ OK!



Gambar 4.20 Tulangan Lapangan Balok Induk

D. Momen Nominal Aktual Balok (As 7, bentang D-C)

1) Momen Aktual Balok Negatif Tumpuan kiri = kanan

tulangan atas = 12D22 dengan $As_{ada} = 4561{,}6\ mm^2$

tulangan bawah = 3D22 dengan $As_{ada} = 1140{,}4\ mm^2$

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{4561{,}6}{450.700} = 0{,}0145$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{1140{,}4}{450.700} = 0{,}0036$$

$\rho_1 = \rho - \rho' = 0{,}0145 - 0{,}0036 = 0{,}0109$

$$fs' = 600\left\{1 - \frac{0{,}85.f'c.\beta_1}{(\rho - \rho').fy}.\frac{d'}{d}\right\} = 600\left\{1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}0054.350}.\frac{100}{700}\right\}$$

$= 233{,}45$ Mpa

$fs' < fy$ dipakai $fs' = 233{,}45$ Mpa

$$a = \frac{(As_{ada}.fy)-(As'_{ada}.fs')}{0,85.fc'.b} = \frac{(4561,6.350)-(1140,4.233,45)}{0,85.22,5.450}$$

$= 154,58 \text{ mm}^2$

$Mn_1 = (As_{ada}.fy - As'_{ada}.fs').(d - a/2)$

$= (4561,6 .350 - 1140,04.233,45).(700 - 154,58/2) .10^{-6}$

$= 828,42$ KNm

$Mn_2 = (As'_{ada}.fs'). (d - d') = (1140,04 .154,58).(700-100) .10^{-6}$

$= 159,73$ KNm

Mnak$^-$ $= Mn_1 + Mn_2 = 828,42 + 159,73 = \mathbf{988,15}$ KNm

2) Momen Aktual Balok Positif

$$\rho_{aktual} = \frac{As_{ada}}{b.d_{pakai}}\ \text{lapangan} = \frac{2660,94}{450.700} = 0,0084$$

$$m = \frac{fy}{0,85.f'c} = \frac{350}{0,85.22,5} = 18,3$$

$Rn = \rho.fy(1 - 1/2.\rho..m) = 0,0084.350.(1 - 1/2 . 0,0084. 18,3) = 2,73$ Mpa

Mnak$^+$ $= Rn.b.d^2 = 2,73 . 450 . 700^2 . 10^{-6} = \mathbf{601,54}$ KNm

E. Perencanaan Tulangan Geser Balok Induk, As 7 (Bentang D -C), Lantai 1

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

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

dimana :

Vu,b = gaya geser rencana balok

Øo = 1,25 untuk fy < 400 Mpa (SKSNI 3.14.4-1.2)

$M_{nak,b}$ = momen nominal/leleh balok pada ujung komponen ($Mnak^-$)

$M_{nak,b'}$ = momen nominal/leleh balok pada bidang muka disebelahnya. ($Mnak^+$)

Ln = panjang bentang bersih balok

Vg = geser akibat beban gravitasi. (VD+VL)

K = tingkat daktalitas struktur, untuk daktalitas penuh dipakai = 1,0 (Gedeon dan Takim, beton seri 3)

V_D = 220,98 KN ; V_L = 42,76 KN ; V_E = 54,93 KN

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

$$Vu,b = 0,7.1,25\left[\frac{988,15 + 601,54}{9,4}\right] + 1,05(220,98 + 42,76) = 424,35 \text{ KN}$$

Dengan syarat tidak lebih besar dari :

$$Vu,b = 1,05\,(V_{D,b} + V_{L,b} + 4/K\,.V_{E,b})$$

$$Vu,b = 1,05\,(220,98 + 42,76 + (4/1)\,.\,54,93) = 505,521 \text{ KN} \ldots\ldots \text{OK !!!}$$

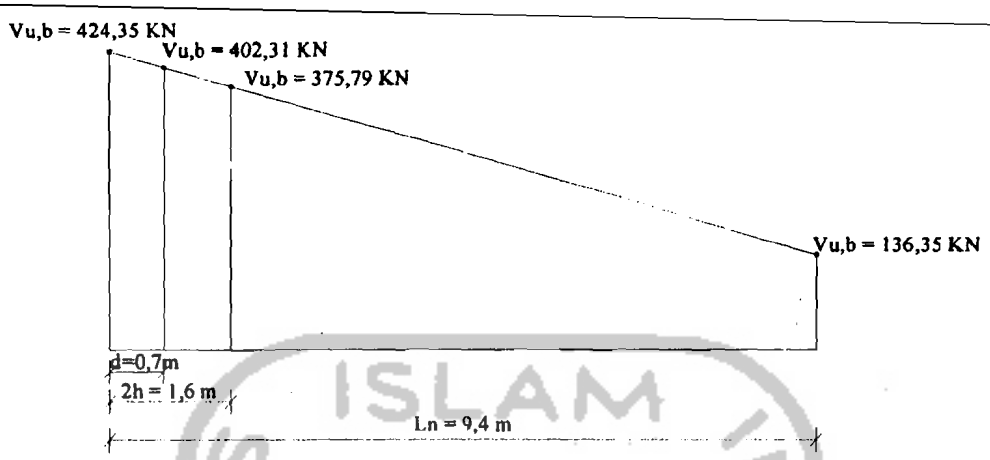
$$Vu,b_{pakai} = \left[1,05Vg - 0,7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] +$$

$$\frac{Ln - d}{Ln}\left[Vu,b - \left[1,05Vg - 0,7\phi_0\left[\frac{Mnak,b + Mnak,b'}{Ln}\right]\right]\right]$$

$$= \left[(1,05.263,213) - 0,7.1,25\left(\frac{988,15 + 601,54}{9,4}\right)\right] +$$

$$\frac{9,4 - 0,7}{9,4}\left[424,35 - \left[(1,05.263,213) - 0,7.1,25\left[\frac{988,15 + 601,54}{9,4}\right]\right]\right]$$

$$= 402,31 \text{ KN}$$



Gambar 4.21 Diagram Tegangan Geser Balok

1) Dalam daerah sendi plastis

Vu,b untuk perencanaan di dalam daerah sendi plastis diambil sejauh **d** dari tumpuan, yaitu :

Vu,b = 402,31 KN

Vc = 0

$$\frac{Vu,b}{\phi} = \frac{402,31}{0,6} = 671,96 \text{ KN}$$

Digunakan sengkang Ø P10 mm, maka : $Av = 2 \cdot \frac{1}{4} \cdot \pi \cdot 10^2 = 157,08 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vu,b/\phi - Vc} = \frac{157,08.300.700}{671,96 - 0} \cdot 10^{-3} = 49,09 \text{ mm}$$

$$\leq d/4 = 700/4 = 175 \text{ mm}$$

Jadi dipakai tulangan geser **P10 – 40 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2 . 800 = 1600 mm dengan Vu,b = 375,79 kN

$Vc = 1/6.\sqrt{fc'}b.d = 1/6.\sqrt{22,5}.450.700.10^{-3} = 249,03$ kN

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{375,79}{0,6} - 249,03 = 375,71 \text{ kN}$$

Digunakan sengkang P10 mm, maka : $Av = 2 . \frac{1}{4} . \pi . 10^2 = 157,08 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vs} = \frac{157,08.300.700}{375,71} . 10^{-3} = 87,80 \text{ mm}$$

$$\leq d/2 = 700/2 = 350 \text{ mm}$$

≤ 600 mm

Jadi dipakai tulangan geser **P10 – 80 mm**

F. Perencanaan Tulangan Torsi

Tu = 33,5127 KNm → Portal arah Y, As 7 (Bentang D - C), Lantai 1.

$\sum x^2.y = 450^2 . 800 = 162.10^6 \text{ mm}^3$

Pada redesign ini komponen struktur portal merupakan komponen statis tak tentu. Untuk komponen statis tak tentu setelah terjadi retak akibat torsi, dalam rangka untuk mencapai keseimbangan terjadi redistribusi tegangan torsional yang mempengaruhi komponen lain yang bertemu pada satu titik buhul. Maka untuk menganalisis torsi dipakai torsi keserasian.

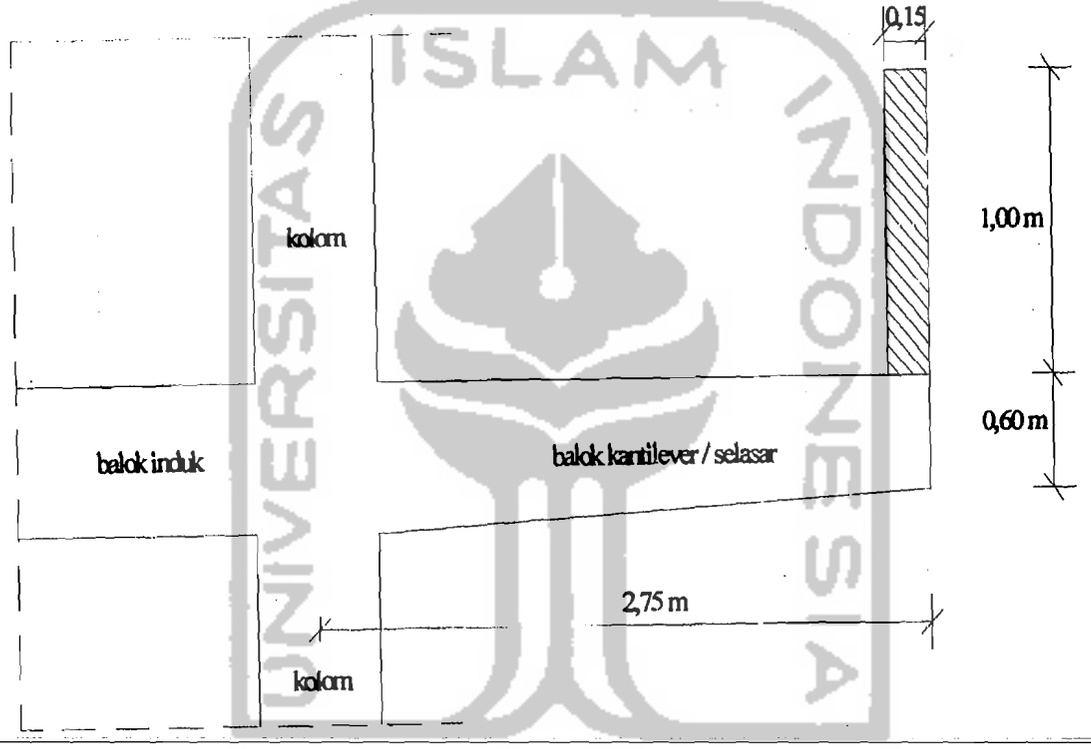
Kemampuan penampang beton menahan torsi untuk torsi keserasian :

$$Tu,b = \phi\left(\frac{1}{9}\sqrt{f'c}.\sum x^2.y\right) = 0,6.\left(\frac{1}{9}\sqrt{22,5}.162\right)10^6$$

= 51,23 kNm > Tu = 33,4315 KNm , tulangan torsi diabaikan.

4.4.2 Perencanaan Balok Kantilever Portal Arah Y, as 7, Bentang B - C

Pada Proyek Pembangunan Kampus UPN "Veteran" Jurusan Akuntansi, menggunakan balok kantilever yang dapat dilihat seperti pada gambar :

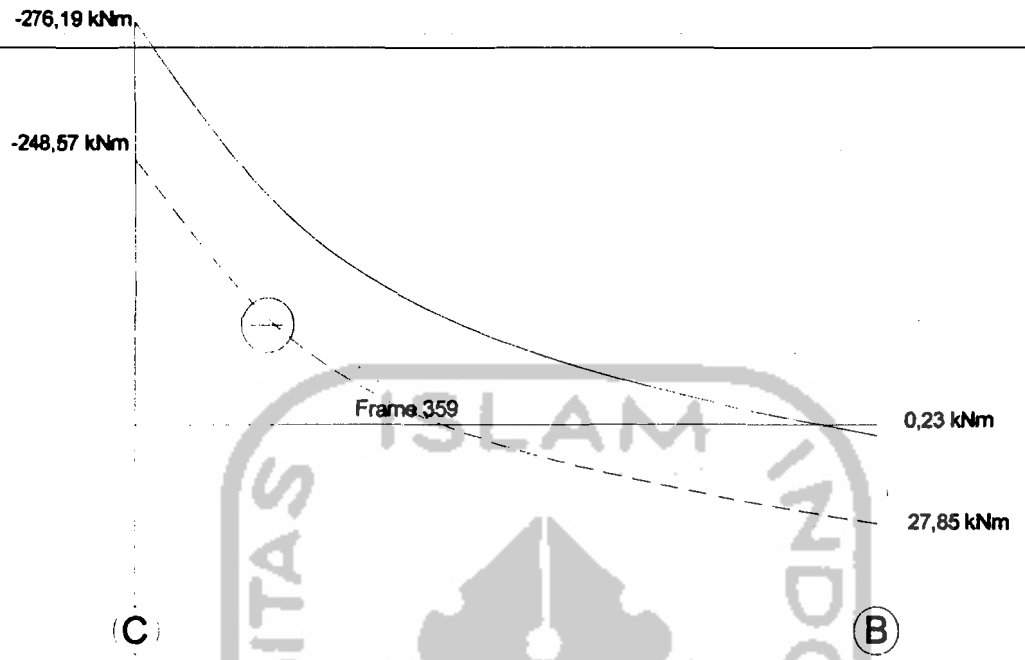


Gambar 4.22 Balok Selasar / Kantilever

A. Momen Rencana Balok Kantilever

Berikut ini contoh perhitungan balok Kantilever portal–Y,

As 7 (bentang B – C) lantai 1.



Gambar 4.23 Distrubusi Momen Pada Portal Y-Melintang As-7 Bentang C-B

B. Tulangan Tumpuan

Momen tumpuan diambil yang paling besar dari semua kombinasi, dan momen yang digunakan adalah yang terbesar dari kedua tumpuan(kiri & kanan).

Dimensi rencana balok anak $^{450}/_{800}$, maka :

fc' = 22,5 Mpa fy = 350 Mpa

- Tulangan tumpuan momen negatif

  Mu_{awal} = 276,19 kNm (-)

  Dalam perencanaan kapsitas balok portal, momen negatif akibat kombinasi beban gravitasi dan beban gempa balok boleh diredistribusikan dengan menambah atau mengurangi dengan prosentase yang tidak melebihi:

$$q = 30\left(1 - \frac{4}{3}\frac{\rho - \rho'}{\rho b}\right)\%$$

dengan syarat apabila tulangan lentur balok portal telah direncanakan sehingga (ρ-ρ') tidak melebihi 0,5.ρb. (Gideon Kusuma dan Takim Andriono)

Momen tumpuan didistribusikan ke momen lapangan sebesar = 10 %

$$\frac{10}{100} x 276{,}19 = 27{,}619\ kNm$$

Mu akibat distribusi momen = 276,19 – 27,619 = 248,57 kNm

$$\frac{Mu}{\phi} = \frac{248{,}57}{0{,}8} = 310{,}65\ \text{KNm}$$

$$\rho b = \frac{0{,}85.f'c}{fy}\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5}{350}.0{,}85\left(\frac{600}{600+350}\right) = 0{,}029$$

ρ_{maks} = 0,75 . ρb = 0,75 . 0,029 = 0,022

rasio tulangan rencana = 0,5 . ρ_{maks} = 0,5 . 0,022 = 0,011

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{350} = 0{,}004$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{350}{0{,}85.22{,}5} = 18.30$$

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

$$= 0{,}011.\ 350\ (1 - \tfrac{1}{2} .\ 0{,}011.\ 18{,}30) = 3{,}462\ \text{Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{310{,}65\ .\ 10^6}{3{,}462\ .\ 450}} = 446{,}50\ \text{mm}$$

d_{ada} = h - d' = 800 -100 = 700 mm > d_{perlu} = 446,50 mm

(d' = 100 mm, diasumsikan menggunakan tul 2 lapis)

maka dipakai **tulangan sebelah**

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{310,65.10^6}{450.700^2} = 1,41 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1,41}{3,462}.0,011 = 0,0045 > \rho_{min} = 0,004$$

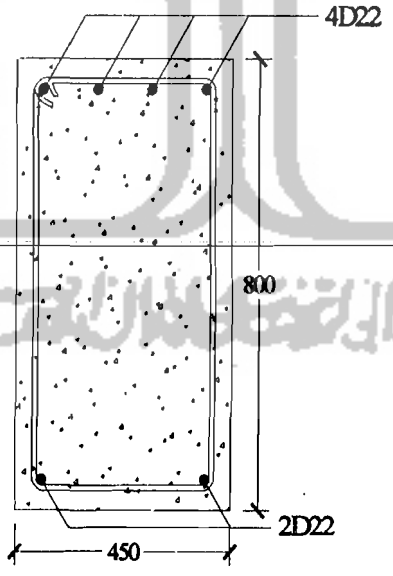
$$< \rho_{maks} = 0,022 \rightarrow \rho_{pakai} = 0,0045$$

$As_{perlu} = \rho_{pakai} . b . d = 0,0045 . 450 . 700 = 1410 \text{ mm}^2$

Dipakai tulangan Ø22 dengan A1Ø = 380,1336 mm^2

jumlah tulangan (n) = $\frac{As_{perlu}}{A_{1\phi}} = \frac{1410}{380,1336} = 3,7 \approx$ **4 batang**

dipakai **4D22,** maka $As_{ada} = 4 .380,1336 = 1520,53 \text{ mm}^2 > As_{perlu} =$ 1260 mm^2



Gambar 4.24 Tulangan Tumpuan dan Lapangan Balok Kantilever

Kontrol kapasitas momen nominal :

$$a = \frac{As._{ada}.fy}{0{,}85.fc'.b} = \frac{1520{,}53.350}{0{,}85.22{,}5.450} = 61{,}84\ mm$$

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

$= 1520{,}53 \,.\, 350 \,.\, (700 - {}^{61{,}84}/_{2})$

$= 356080000$ Nmm $= 356{,}08$ kNm $> {}^{Mu}/_{\phi} = 310{,}65\ kNm$ .... OK

C. Tulangan Lapangan

$Mu_{awal} = 0{,}23$ KNm (+)

Mu dist. momen = 27,619 + (0,23) = 27,85 kNm

$$\frac{Mu}{\phi} = \frac{27{,}85}{0{,}8} = 34{,}84 \text{ KNm}$$

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5}{350}.0{,}85\left(\frac{600}{600+350}\right) = 0{,}029$$

$\rho_{maks} = 0{,}75 \,.\, \rho b = 0{,}75 \,.\, 0{,}029 = 0{,}022$

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{350} = 0{,}004$$

$\rho_{pakai} = 0{,}5 \,.\, \rho_{maks} = 0{,}5 \,.\, 0{,}022 = 0{,}011$

$$m = \frac{fy}{0{,}85.f'c} = \frac{350}{0{,}85.22{,}5} = 18{,}3$$

$Rn = \rho\, fy\, (1 - \frac{1}{2}\rho\, m) = 0{,}011 \,.\, 350\, (1 - \frac{1}{2}\,.0{,}011 \,.\, 18{,}3) = 3{,}46$ Mpa

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{34{,}84 \,.\, 10^6}{3{,}46 \,.\, 450}} = 149{,}53 \text{ mm} < d_{ada} = 800 - 100 = 700 \text{ mm}$$

maka dipakai **tulangan sebelah.**

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{34,84.10^6}{450.700^2} = 0,16 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho_{pakai} = \frac{0,11}{3,46}.0,016 = 0,0004 < \rho_{min} = 0,004$$

$$\rho_{pakai} = \rho_{min} = 0,004$$

$As_{perlu} = \rho_{pakai}.b.d = 0,004.450.700 = 1260 \text{ mm}^2$

Dipakai tulangan Ø22 dengan $A_1Ø = 380,1336 \text{ mm}^2$

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{1260}{380,1336} = 3,3 \approx 4 \text{ batang}$$

dipakai **4D22**,maka $As_{ada} = 4 . 380,1336 = 1520,53 \text{ mm}^2 > As_{perlu} = 1260 \text{ mm}^2$

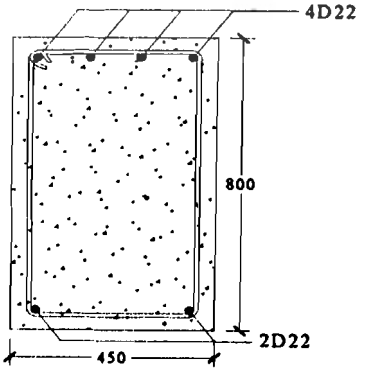
Kontrol kapasitas momen nominal :

$$a = \frac{As._{ada}.fy}{0,85.fc'.b} = \frac{1520,53.350}{0,85.22,5.450} = 61,84\ mm$$

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

$= 1520,53 . 350 . (700 - 61,84/2)$

$= 356080000 \text{ Nmm} = 356,08 \text{ kNm} > Mu/\phi = 34,84\ kNm$ ⟶ OK



Gambar 4.25 Tulangan Lapangan Balok Kantilever

D. Momen Nominal Aktual Balok Kantilever (As 7, bentang C-B)

1) Momen Aktual Balok Negatif Tumpuan

tulangan atas = 4D22 dengan $As_{ada} = 1520{,}53\ mm^2$

tulangan bawah = 2D22 dengan $As_{ada} = 760{,}27\ mm^2$

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{1520{,}53}{450.700} = 0{,}0048$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{760{,}27}{450.700} = 0{,}0024$$

$\rho_1 = \rho - \rho' = 0{,}0048 - 0{,}0024 = 0{,}0024$

$$fs' = -600\left\{1 - \frac{0{,}85.f'c.\beta_1}{(\rho - \rho').fy}.\frac{d'}{d}\right\} = -600\left\{1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}0018.350}.\frac{100}{700}\right\}$$

$= -1049{,}49$ Mpa

fs'> fy dipakai fy= 350 Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0{,}85.fc'.b} = \frac{(1520{,}53.350) - (760{,}27.350)}{0{,}85.22{,}5.450}$$

$= 30{,}92\ mm^2$

$$Mn_1 = (As_{ada}.fy - As'_{ada}.fs').(d - {}^{a}/_{2})$$

$$= (1520{,}53 .350 - 760{,}27 .350).(700 - {}^{30{,}92}/_{2}) .10^{-6} = 182{,}15\ KNm$$

$Mn_2 = (As'_{ada} .fs').(d - d') = (760{,}27 .350).(700–100).10^{-6} = 159{,}66$ KNm

$Mnak^{-} = Mn1 + Mn_2 = 182{,}15 + 159{,}66 = 341{,}81$ KNm

2) Momen Aktual Balok lapangan

$$\rho_{aktual} = \frac{As_{ada}}{b.d_{pakai}}\ lapangan = \frac{1520,53}{450.700} = 0,0048$$

$$Rn = \rho.fy(1 - \tfrac{1}{2}.\rho.m) = 0,0048\ .350.(1-\tfrac{1}{2}\ .\ 0,0048.\ 18,3) = 1,61\ \text{Mpa}$$

$$Mnak^{+} = Rn.b.d^2 = 1,61\ .450.700^2.10^{-6} = 356,08\ \text{kNm}$$

E. Perencanaan Tulangan Geser Balok Kantilever

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

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

dimana : Vu,b = gaya geser rencana balok

$Øo$ = 1,25 untuk fy < 400 Mpa (SKSNI 3.14.4-1.2)

$M_{nak,b}$ = momen nominal/leleh balok pada ujung komponen

$M_{nak,b'}$ = momen nominal/leleh balok pada bidang muka disebelahnya.

Ln = panjang bentang bersih balok

Vg = geser akibat beban gravitasi. (VD+VL)

$V_D = 107$ KN ; $V_L = 18,8$ KN ; $V_E = 0,018$ KN

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

$$Vu,b = 0,7.1,25\left[\frac{341,81 + 356,08}{2,45}\right] + 1,05.(107 + 18,8) = 380,91\ \text{KN}$$

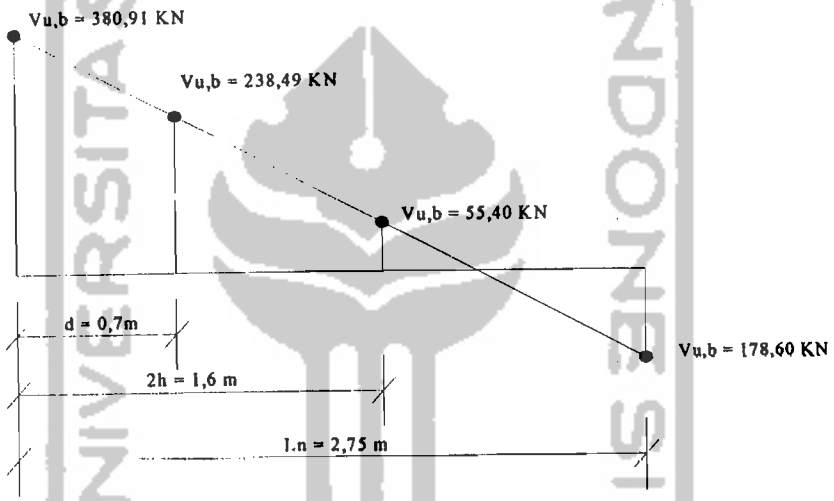
$$Vu,b_{\ pakai} = \left[1,05Vg - 0,7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] +$$

$$\frac{Ln-d}{Ln}\left[Vu,b-\left[1{,}05Vg-0{,}7\phi_0\left[\frac{Mnak,b+Mnak,b'}{Ln}\right]\right]\right]$$

$$=\left[(1{,}05.125{,}4)-0{,}7.1{,}25.\left(\frac{341{,}81+356{,}08}{2{,}45}\right)\right]+$$

$$\frac{2{,}45-0{,}7}{2{,}45}\left[380{,}91-\left[(1{,}05.125{,}4)-0{,}7.1{,}25\left[\frac{341{,}81+356{,}08}{2{,}45}\right]\right]\right]$$

$$=238{,}49\ KN$$



Gambar 4.26 Diagram Tegangan Geser Balok Kantilever

1) Dalam dacrah sendi plastis

Vu,b untuk perencanaan di dalam daerah sendi plastis diambil sejauh **d** dari tumpuan, yaitu :

Vu,b = 238,49 KN

Vc = 0

$$\frac{Vu,b}{\phi}=\frac{238{,}49}{0{,}6}=397{,}48\ KN$$

Digunakan sengkang Ø P10 mm, maka : Av = 2 . ¼ . π . 10^2 = 157,08 mm^2

Jarak sengkang :

$$s \geq \frac{Av.fy.d}{Vu,b/\phi - Vc} = \frac{157,08.350.700}{346,85-0} . 10^{-3} = 82,99 \text{ mm}$$

$$> d/4 = 700/4 = 175 \text{ mm}$$

Jadi dipakai tulangan geser **P10 – 80 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2 . 800 = 1600 mm dengan Vu,b = 249,9 kN

$$Vu,b = \frac{\left[\left(Vu,b.\frac{d}{1000}\right)-\left((Vu,b-Vu,b_{pakai}).2.\frac{h}{1000}\right)\right]}{\frac{d}{1000}}$$

$$= \frac{\left[\left(380,91.\frac{700}{1000}\right)-\left(142.2.\frac{800}{1000}\right)\right]}{\frac{700}{1000}}$$

$$= 55,37 \text{ kN}$$

$$Vc = 1/6.\sqrt{fc'}b.d = 1/6.\sqrt{22,5}.450.700.10^{-3} = 249,03 \text{ kN}$$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{55,37}{0,6} - 249,03 = -156,75 \text{ kN}$$

Digunakan sengkang P10 mm, maka : Av = 2 . ¼ . π . 10^2 = 157,08 mm^2

Jarak sengkang :

$$s \geq \frac{Av.fy.d}{Vs} = \frac{157,08.450.700}{-156,75} . 10^{-3} = -210,45 \text{ mm}$$

$$\geq d/2 = 700/2 = 350 \text{ mm}$$

≥ 600 mm

Jadi dipakai tulangan geser **P10 – 200 mm**

F. Perencanaan Tulangan Torsi

Tu = 3,7325 KNm (Portal Y melintang, Frame 359)

$\sum x^2.y = 450^2 . 800 = 288.10^6 \text{ mm}^3$

Pada redesign ini komponen struktur portal merupakan komponen statis tak tentu. Untuk komponen statis tak tentu setelah terjadi retak akibat torsi, dalam rangka untuk mencapai keseimbangan terjadi redistribusi tegangan torsional yang mempengaruhi komponen lain yang bertemu pada satu titik buhul. Maka untuk menganalisis torsi dipakai torsi keserasian.

Kemampuan penampang beton menahan torsi untuk torsi keserasian :

$$Tu,b = \phi\left(\frac{1}{9}\sqrt{f'c}.\sum x^2.y\right) = 0{,}6.\left(\frac{1}{9}\sqrt{22{,}5}.288.\right)10^6$$

= 91,07 kNm > Tu = 3,7325 KNm , tulangan torsi diabaikan.


ISLAM
UNIVERSITAS
INDONESIA

4.5 Perencanaan Kolom

4.5.1 Perhitungan Momen dan Gaya Aksial Rencana

Contoh perhitungan pada kolom 941 As – 7 portal C lantai 1 **(V3 dan M2)**

1. Momen untuk portal arah X

   $M_{Dy\ atas}$ = 92,84 KNm

   $M_{Dy\ bawah}$ = -106,80 KNm

   $M_{Ly\ atas}$ = 21,17 KNm

   $M_{Ly\ bawah}$ = -28,41 KNm

   $M_{Ey\ atas}$ = 132,81 KNm

   $M_{Ey\ bawah}$ = -100,48 KNm

   Kombinasi pembebanan :

   Atas

   $1{,}2\ M_{Dy} + 1{,}6\ M_{Ly}$ = 145,28 KNm

   $1{,}05\ (M_{Dy} + 0{,}65.M_{Ly} + M_{Ey})$ = 251,38 KNm

   Bawah

   $1{,}2\ M_{Dy} + 1{,}6\ M_{Ly}$ = 173,62 KNm

   $1{,}05\ (M_{Dy} + 0{,}65.\ M_{Ly} + M_{Ey})$ = 235,54 KNm

2. Momen untuk portal arah Y

   Data Momen

   $M_{Dx\ atas}$ = 92,84 KNm

   $M_{Dx\ bawah}$ = -106,8 KNm

   $M_{Lx\ atas}$ = 21,17 KNm

   $M_{Lx\ bawah}$ = -28,41 KNm

$M_{Ex\ atas}$ = 4,92 KNm

$M_{Ex\ bawah}$ = -4,48 KNm

Kombinasi pembebanan :

Atas

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 145,28 KNm

$1,05\ (M_{Dx} + 0,65.M_{Lx} + M_{Ex})$ = 115,99 KNm

Bawah

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 173,62 KNm

$1,05\ (M_{Dx} + 0,65.M_{Lx} + M_{Ex})$ = 134,742 KNm

3. Gaya Aksial

Data Gaya Aksial

$P_{D\ atas}$ = - 942 KNm

$P_{D\ bawah}$ = - 977 KNm

$P_{L\ atas}$ = -162 KNm

$P_{L\ bawah}$ = -162 KNm

$P_{Ex\ atas}$ = - 2,3 KNm

$P_{Ex\ bawah}$ = - 2,3 KNm

$P_{Ey\ atas}$ = 58 KNm

$P_{Ey\ bawah}$ = 58 KNm

Kombinasi pembebanan :

Atas

$1,2\ P_D + 1,6\ P_L$ = 1390 KNm

$1,05\ (P_D + 0,65.P_L + P_{Ex})$ = 1102 KNm

$1{,}05\ (P_D + 0{,}65.P_L + P_{Ey})$ = 1161 KNm

Bawah

$1{,}2\ P_D + 1{,}6\ P_L$ = 1432 KNm

$1{,}05\ (P_D + 0{,}65.P_L + P_{Ex})$ = 1198 KNm

$1{,}05\ (P_D + 0{,}65.P_L + P_{Ey})$ = 1236 KNm

4.5.2 Kriteria Kolom dan Pembesaran Kolom

Menghitung Kekakuan Kolom

1. Arah X

Ec = Eg $= 4700\sqrt{f'c}$

$= 4700\ \sqrt{22{,}5}$

= 22294 Mpa

rencana dimensi kolom = 600x600 mm

Ic (Inersia kolom) $= \frac{1}{12}.600^3.600 = 10{,}8.10^9\ mm^4$

$$\beta d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.106{,}8}{1{,}2.106.8 + 1{,}6.28{,}41} = 0{,}75$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta.d)} = \frac{22294.10{,}8.10^9}{2{,}5(1+0{,}75)} = 5{,}5.\ 10^{13}\ Nmm^2$$

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

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

$$I_{cr} = \frac{1}{2}\left(\frac{1}{12}.b.h^3\right) = \frac{1}{2}\left(\frac{1}{12}.450.800^3\right) = 9{,}6.10^9 \text{ mm}^4$$

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

Lc (panjang bersih kolom) = 3,2 m

Lg (panjang bersih balok) = 9,4 m

Lg (panjang bersih balok) = 2,45 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{Ec.Icr}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{5{,}5.10^{13}}{3200} + \frac{5{,}5.10^{13}}{3200}\right)}{\left(\frac{22294.10{,}8.10^9}{9400} + \frac{22294.10{,}8.10^9}{2450}\right)} = 0{,}28$$

ψ_{bawah} = 0 (ujung jepit)

Dari Nomogram portal tanpa pengaku, didapat k = 1,08 (Desain Beton Bertulang; Charles G. Salmon, Chu-Kia Wang, hal 69 gambar 15.8.1)

$$\frac{k.lu}{r} = \frac{1{,}08.3200}{0{,}3.600} = 19{,}2 < 22 \text{ (termasuk kolom pendek)}$$

karena dari perhitungan diketahui merupakan kolom pendek maka analisis orde dua diabaikan.

2. Arah Y

Ec = Eg $= 4700\sqrt{f'c}$

$= 4700\ \sqrt{22,5}$

$= 22294$ Mpa

rencana dimensi kolom = 600x600 mm

Ic (Inersia kolom) $= \frac{1}{12}.600^3.600 = 10,8.10^9\ mm^4$

$$\beta d = \frac{1,2M_D}{1,2M_D + 1,6M_L} = \frac{1,2.106,8}{1,2.106.8 + 1,6.28,41} = 0,75$$

$$EI = \frac{E_c.I_c}{2,5(1+\beta.d)} = \frac{22294.10,8.10^9}{2,5(1+0,75)} = 5,5.\ 10^{13}\ Nmm^2$$

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

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

$$I_{cr} = \frac{1}{2}.\left(\frac{1}{12}.b.h^3\right) = \frac{1}{2}.\left(\frac{1}{12}.450.800^3\right) = 9,6.10^9\ mm^4$$

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

Lc (panjang bersih kolom) = 3,2 m

Lg (panjang bersih balok) = 3,4 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{Ec.Icr}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\frac{5,5.10^{13}}{3200} + \frac{5,5.10^{13}}{3200}\right)}{\left(\frac{22294.10,8.10^9}{3400} + \frac{22294.10,8.10^9}{3400}\right)} = 0,24$$

ψ_{bawah} = 0 (ujung jepit)

Dari Nomogram portal tanpa pengaku, didapat k = 1,07 (Desain Beton Bertulang; Charles G. Salmon, Chu-Kia Wang, hal 69 gambar 15.8.1)

$$\frac{k.lu}{r} = \frac{1,07.3200}{0,3.600} = 19,02 < 22 \text{ (termasuk kolom pendek)}$$

karena dari perhitungan diketahui merupakan kolom pendek maka analisis orde dua diabaikan.

4.5.3 Analisis Gaya Aksial dan Momen akibat balok

Perhitungan kolom (941) As – 7 portal C lantai 1 Akibat Gempa arah Y

h = 4,2 m

hn = Tinggi kolom - (2 . ½ .h_{balok}) = (4,2 - 0,8) = 3,4 m

Rv = faktor reduksi *(Gideon Kusuma)*

(jumlah lantai 1< n < 4) ; Rv = 1

(jumlah lantai 4< n < 20) ; Rv = 1,1 - 0,025.n

(jumlah lantai >20) ; Rv = 0,6

ωd = (factor pembesar dinamis) = 1,3; kecuali untuk kolom lantai 1 dan lantai paling atas yang kemungkinan terjadi sendi plastis pada kolom, ωd = 1

k = 1

Ng = (P_D + P_L)

l_{ki} = bentang balok $_{kiri}$

l_{ka} = bentang balok $_{kanan}$

l'_{ki} = bentang bersih balok $_{kiri}$

l'_{ka} = bentang bersih balok $_{kanan}$

a. Perhitungan Arah X

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25. 601,54 = 751,92 KNm

$Mkap_{(kanan)}$ = 1,25. Mnak = 1,25 . 356,08 = 445,10 KNm

menghitung gaya aksial rencana :

$$Pu,k_y = 0,7.\ Rv.\left(\frac{Mkap_{kanan}}{l}+\frac{Mkap_{kiri}}{l}\right)+1,05.\ Ng$$

$$= 0,7.\ 1.\left[\frac{445,1}{2,75}+\frac{751,92}{10}\right]+1,05\ .(977+162)$$

$$= 1362\ KN$$

Tidak perlu melebihi :

$$Pu,k_y = 1,05\ (N_D+N_L+\frac{4}{k}\ .\ N_E)$$

$$= 1,05\ (977+162+\frac{4}{1}\ .58\ )$$

$$= 1440\ KN$$

menghitung α :

$M_{Ey\ atas}$ = 132,81 KNm

$M_{Ey\ bawah}$ = -100,48 KNm

$$\alpha\ ka = \frac{M_{E,k(lti+latas)}}{M_{E,k(lti+latas)}+M_{E,k(ltibawah)}} = \frac{132,81}{132,81+100,48} = 0,57$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+latas)}+M_{E,k(ltibawah)}} = \frac{100,48}{132,81+100,48} = 0,43$$

menghitung momen rancang kolom :

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

$$= \frac{3,6}{4,2}.1,3\ .\ 0,57\ .\ 0,7.\left(\frac{2,75}{2,45}.445,1 + \frac{10}{9,4}.751,92\right)$$

$$= 545\ \text{KNm}$$

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

$$= \frac{3,6}{4,2}.1,3\ .\ 0,57\ .\ 0,7.\left(\frac{2,75}{2,45}.445,1 + \frac{10}{9,4}.751,92\right)$$

$$= 412\ \text{KNm}$$

tidak perlu melebihi :

$$Mu,k = 1,05(M_{Dy} + M_{Ly} + \frac{4}{k}\ M_{Ey})$$

$$= 1,05\ (106,80 + 28,41\ + \frac{4}{1}\ .\ 132,81)$$

$$= 710,27\ \text{KNm}$$

b. Perhitungan Arah Y

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25 . 333,02 = 416 KNm

$Mkap_{(kanan)}$ = 1,25. Mnak = 1,25 . 333,02 = 416 KNm

menghitung gaya aksial rencana :

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

$$= \{0,7.\ 1.(\ \frac{416}{4} + \frac{416}{4}\ )\} + \{1,05\ .(977 + 162)\}$$

$$= 1341,55\ \text{KN}$$

tidak perlu melebihi :

$Pu,k_x = 1,05\ (N_D + N_L + 4/K\ .N_E)$

$= 1,05\ (977 + 162 + \frac{4}{1}\ .2,3\ )$

$= 1205,96$ KN

menghitung α :

$M_{Ex\ atas} = 4,92$ KNm

$M_{Ex\ bawah} = -4,48$ KNm

$$\alpha \text{ ka} = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{4,92}{4,92+4,48} = 0,53$$

$$\alpha \text{ kb} = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{4,48}{4,92+4,48} = 0,47$$

menghitung momen rancang kolom :

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

$$= \frac{3,4}{4,2}.1,3\ .0,53\ .\ 0,7.\left(\frac{4}{3,4}.416 + \frac{4}{3,4}.416\right)$$

$= 401$ KNm

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

$$= \frac{3,4}{4,2}.1,3.0,47\ .\ 0,7.\left(\frac{4}{3,4}.416 + \frac{4}{3,4}.416\right)$$

$= 356$ KNm

tidak perlu melebihi :

$$Mu,k_x = 1{,}05(M_{Dx} + M_{Lx} + \frac{4}{k} M_{Ex})$$

$$= 1{,}05\ (106{,}8 + 28{,}41\ + \frac{4}{1} 4{,}92)$$

$$= 160{,}78 \text{ KNm}$$

4.5.4 Perencanaan Tulangan Lentur Kolom

Untuk perencanaan penulangan kolom dipakai nilai terbesar dari hasil analisis SAP 2000 dan momen akibat momen kapasitas balok, maka :

$Pu,k_x = 1205{,}96$ KN

$Pu,k_y = 1362$ KN

$Mu,k_x = 160{,}78$ KNm

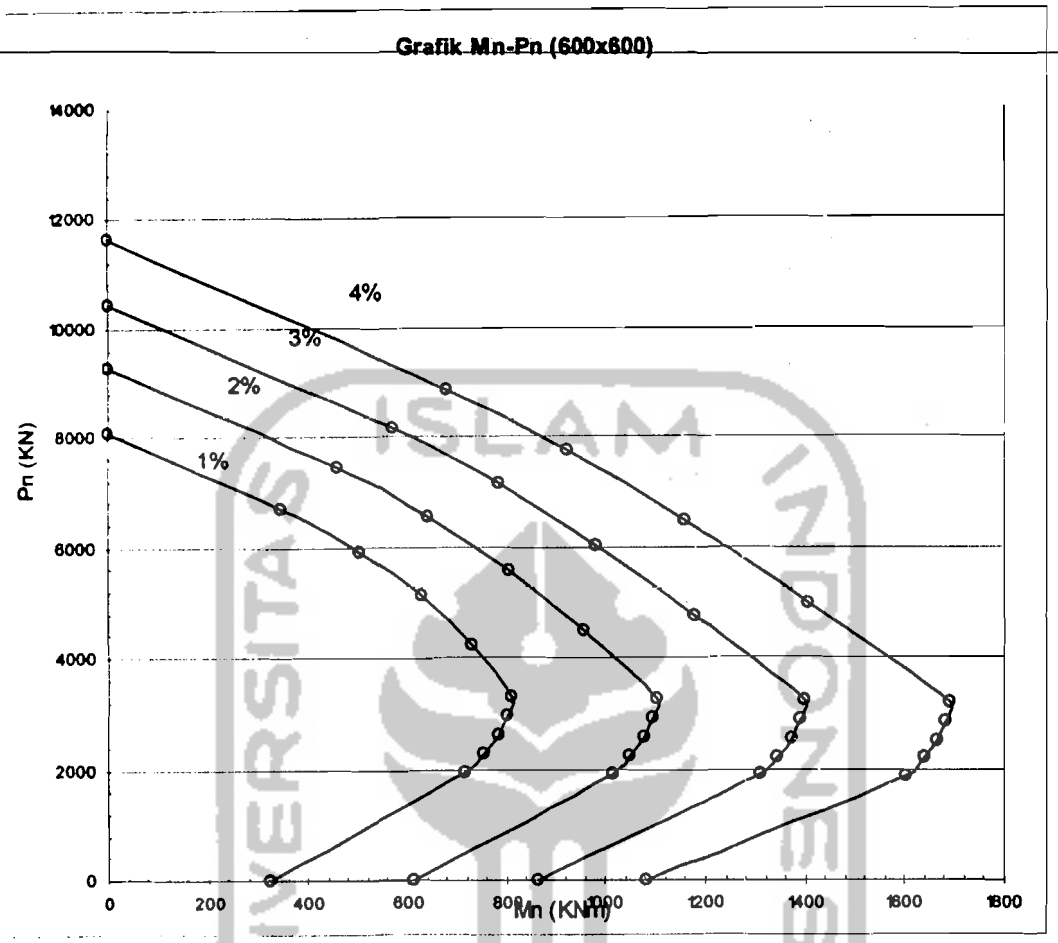
$Mu,k_y = 545$ KNm

$$\frac{Pu,k_x}{\phi} = \frac{1205{,}96}{0{,}65} = 1855{,}32 \text{ KN}$$

$$\frac{Pu,k_y}{\phi} = \frac{1362}{0{,}65} = 2095{,}86 \text{ KN}$$

$$\frac{Mu,k_x}{\phi} = \frac{160{,}78}{0{,}65} = 247{,}35 \text{ KNm}$$

$$\frac{Mu,k_y}{\phi} = \frac{545}{0{,}65} = 838{,}46 \text{ KNm}$$



Gambar 4.27 Grafik Mn - Pn Kolom

a. Arah x

$$\frac{Pu,k_y}{\phi} = \frac{1362}{0,65} = 2095,86 \text{ KN}$$

$$\frac{Mu,k_y}{\phi} = \frac{545}{0,65} = 838,46 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 1,4$ %

Ast = 0,014.600.600 = 5040 mm^2

As = As' = 0,5.Ast = 2520 mm^2

dipakai **7D22** dengan $As_{ada} = As'_{ada} = 2660\ mm^2$

Cek eksentrisitas balance (e_b)

$$yb = \frac{600.d}{600 + fy} = \frac{600.540}{600 + 350} = 341\ mm$$

$$ab = \beta_1 . yb = 0{,}85 . 341 = 290\ mm$$

$$fs' = 600\frac{(yb - d')}{yb} = 600\frac{(341 - 60)}{341} = 495\ MPa > fy = 350\ MPa$$

Dengan demikian digunakan fs' = fy = 350 MPa (Keadaan Simbang)

$$Ccb = 0{,}85.f'c.b.ab = 0{,}85.22{,}5.600.290 = 3327750\ N$$

$$Csb = As'(fs' - 0{,}85.f'c) = 2660.(350 - 0{,}85.22{,}5) = 880128\ N$$

$$Tsb = As.fy = 2660 .350 = 931000\ N$$

$$Pnb = Ccb + Csb - Tsb = 3327750 + 880128 - 931000$$

$$= 3276878\ N = 3277\ KN$$

$$Mnb = Ccb\left[\frac{h}{2} - \frac{ab}{2}\right] + Csb\left(\frac{h}{2} - d'\right) + Tsb\left(d - \frac{h}{2}\right)$$

$$= 3327750.\left[\frac{600}{2} - \frac{290}{2}\right] + 880128.\left(\frac{600}{2} - 60\right) + 931000.\left(540 - \frac{600}{2}\right)$$

$$= 950471970\ Nmm = 951\ KNm$$

$$eb = \frac{Mnb}{Pnb} = \frac{951}{3277} = 0{,}29\ m$$

$$e = \frac{Mu_k, y/\phi}{Pu_k/\phi} = \frac{838{,}46}{2095{,}86} = 0{,}4\ m$$

karena $e > e_b$ ⟶ kolom mengalami patah tarik

Kontrol tegangan pada daerah tarik :

$$Pn = \frac{As'.fy}{\frac{e}{(d-d')} + 0{,}5} + \frac{b.h.fc'}{\frac{3.h.e}{d^2} + 1{,}18}$$

$$= \frac{2660.350}{\frac{400}{(540-60)} + 0{,}5} + \frac{600.600.22{,}5}{\frac{3.600.400}{540^2} + 1{,}18}$$

$$= 2917970 \text{ N} = 2917{,}97 \text{ KN}$$

$$Pn = 2917{,}97 \text{ KN} > \frac{Pu_k}{\phi} = 2095{,}86 \text{ KN} \ldots\ldots\ldots\ldots \textbf{Ok !}$$

$$Mn = Pn \,.\, eb$$

$$= 2917{,}97 \,.\, 0{,}29$$

$$= 848{,}81 \text{ KNm} > \frac{Mu, k_y}{\phi} = 838{,}46 \text{ KNm}$$

b. Arah y

$$\frac{Pu, k_x}{\phi} = \frac{1205{,}96}{0{,}65} = 1855{,}32 \text{ KN}$$

$$\frac{Mu, k_x}{\phi} = \frac{160{,}78}{0{,}65} = 247{,}35 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 1\ \%$

$Ast = 1{,}00.600.600 = 3600 \text{ mm}^2$

$As = As' = 0{,}5.Ast = 1800 \text{ mm}^2$

dipakai **4D22** dengan $As_{ada} = As'_{ada} = 1966 \text{ mm}^2$

Cek eksentrisitas balance (e_b)

$$xb = \frac{600.d}{600 + fy} = \frac{600.540}{600 + 350} = 341 \text{ mm}$$

$ab = \beta_1 \,.\, xb = 0{,}85 \,.\, 341 = 290 \text{ mm}$

$$fs' = 600\frac{(xb - d')}{xb} = 600\frac{(341 - 60)}{341} = 495\ MPa > fy = 350\ MPa$$

digunakan fs' = fy = 350 MPa

Ccb = 0,85.f'c.b.ab = 0,85.22,5.600. 290 = 3327750 N

Csb = As'(fs'- 0,85.f'c) = 1520.(350 – 0,85.22,5) = 502930 N

Tsb = As.fy = 1520.350 = 532000 N

Pnb = Ccb + Csb – Tsb = 3327750 + 502930 – 532000

= 3298680 N = 3299 KN

$$Mnb = Ccb\left[\frac{h}{2} - \frac{ab}{2}\right] + Csb\left(\frac{h}{2} - d'\right) + Tsb\left(d - \frac{h}{2}\right)$$

$$= 3327750.\left[\frac{600}{2} - \frac{290}{2}\right] + 502930.\left(\frac{600}{2} - 60\right) + 532000\left(540 - \frac{600}{2}\right)$$

= 734234700 Nmm = 734 KNm

$$eb = \frac{Mnb}{Pnb} = \frac{734}{3299} = 0,25$$

$$e = \frac{Mu_{,kx}/\phi}{Pu_{,k}/\phi} = \frac{247,35}{1855,32} = 0,13$$

karena $e < e_b$ ⟶ kolom mengalami patah desak

Kontrol tegangan pada daerah desak :

$$Pn = \frac{As'.fy}{\frac{e}{(d - d')} + 0,5} + \frac{b.h.fc'}{\frac{3.h.e}{d^2} + 1,18}$$

$$= \frac{1520.350}{\frac{330}{(540 - 60)} + 0,5} + \frac{600.600.22,5}{\frac{3.600.330}{540^2} + 1,18}$$

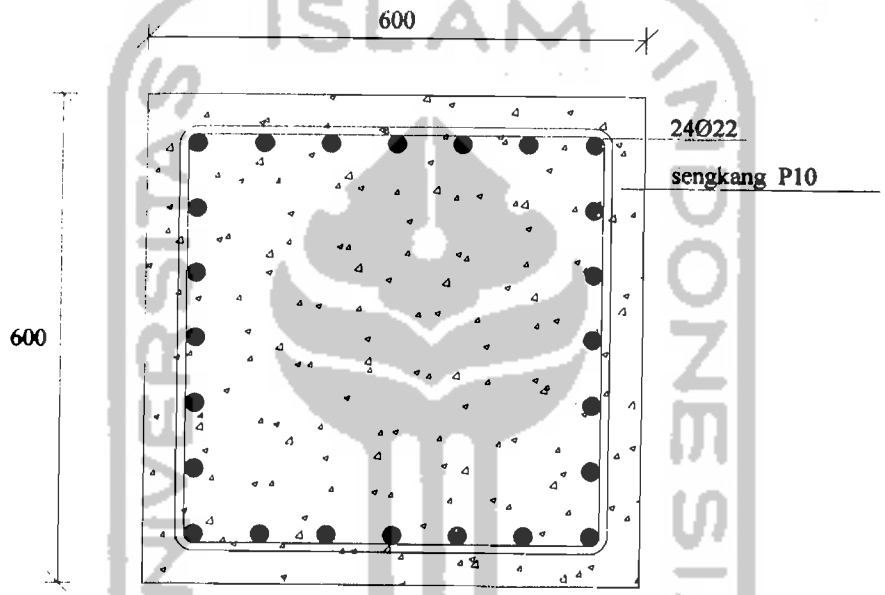
= 4899240 N = 4899,24 KN

Pn = 4899,24 KN > $\frac{Pu_k}{\phi}$ = 1319 KN ..... **Ok !**

Mn = Pn . eb

= 1520 . 0,25

= 1230,51 KNm > $\frac{Mu,k_x}{\phi}$ = 247,35 KNm



Gambar 4.28 Gambar Penulangan Kolom

4.5.5 Perencanaan Tulangan Geser Kolom (941)

$Mu_{,k}$ atas = 545 KNm

$Mu_{,k}$ bwh = 412 KNm

$V_{D,}k$ = 47,54 KN

$V_{L,}k$ = 11,8 KN

$V_{Ex,}k$ = - 22,4 KN

$V_{Ey,}k$ = 55,55 KN

h_n = 3,40 m

$$V_{u,k} = \frac{Mu,k_{atas} + Mu,k_{bawah}}{h_n} = \frac{545 + 412}{3,40} = 282 \text{ KN}$$

tetapi tidak perlu lebih besar dari :

$$V_{u,k} = 1,05\ (V_{D,k} + V_{L,k} + \frac{4}{k}(V_{E,k})$$

$$= 1,05\ (47,54 + 11,8 + (\frac{4}{1}.55,55))$$

$$= 296 \text{ KN}$$

$$\frac{V_U,k}{\phi} = \frac{282}{0,6} = 470 \text{ KN}$$

di daerah sejauh *lo*

Kekuatan beton dalam menahan gaya geser dianggap 0 (Vc)

$$Vs = \frac{V_U,k}{\phi} = 470 \text{ KN}$$

Dipakai tulangan geser rangkap P10 mm, maka :

$Av = 2\ .\ ¼\ .\pi\ .\ 10^2 = 157\ mm^2$

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157.300.540}{470.10^3} = 54 \text{ mm}$$

$< 16.D = 160$ mm

Digunakan sengkang **2P10 - 50 mm**

di luar daerah *lo*

$$Vc = \left(1 + \frac{Puk_y}{14.Ag}\right).\frac{1}{6}\sqrt{f'c}.b.d$$

$$= \left(1 + \frac{1362.10^3}{14.600.600}\right).\frac{1}{6}\sqrt{22,5}.600.540 = 325380\ N$$

$$= 325\ KN < \frac{V_U,k}{\phi} = 470\ KN,\ \text{maka perlu tulangan geser}$$

$$Vs = \frac{V_U,k}{\phi} - Vc = 470 - 325 = 145\ N$$

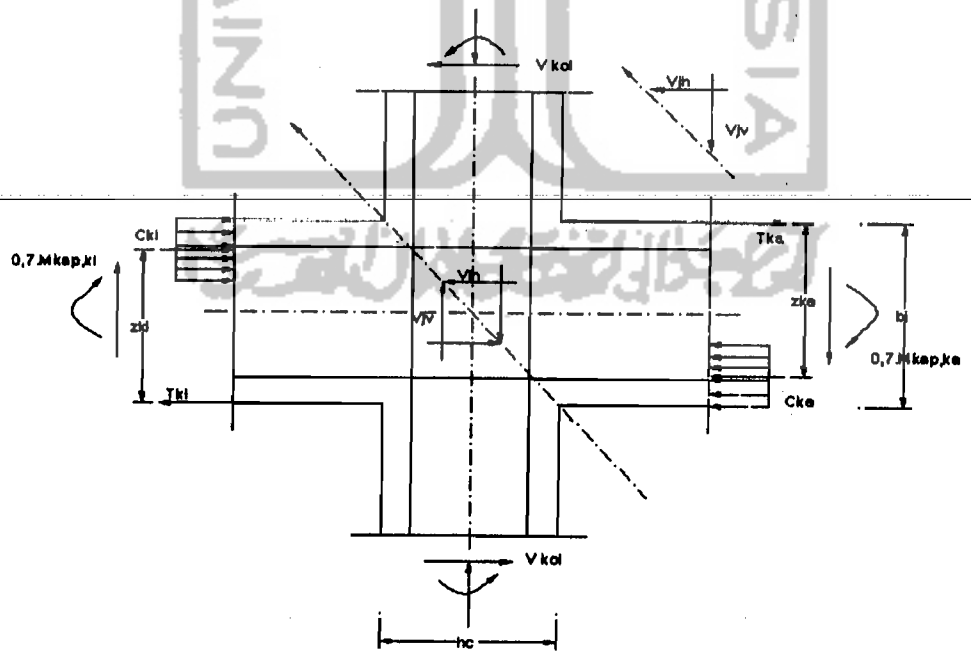
Dipakai tulangan geser rangkap 2P10 mm, maka :

$$Av = 2\ .\ ¼\ .\pi\ .\ 10^2 = 157\ mm^2$$

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157.300.540}{145.10^3} = 175\ mm$$

Dipakai sengkang **2P10 – 160 mm**, maka :

4.5.6 Pertemuan Balok Kolom



Gambar 4.29 Joint Balok Kolom Dalam

a. Perhitungan gaya-gaya dalam

1) Sumbu Y

b_j = bc = 800 mm

= bb + 0,5.hc = 450 + 0,5.600 = 750 mm

b_j pakai = 750 mm

h_c = 600 mm

$$V_{kol,y} = \frac{0,7 \cdot \left(\frac{lki}{lki'} \cdot M_{kap,ki} + \frac{lka}{lka'} M_{kap,ka}\right)}{\frac{1}{2} \cdot (h_{k,a} + h_{k,b})}$$

$$V_{kol,y} = \frac{0,7 \cdot \left(\frac{10}{9,4} \cdot 751,92 + \frac{2,75}{2,45} \cdot 445,10\right)}{\frac{1}{2} \cdot (4,2 + 4,2)} = 221,86 \text{ KN}$$

$z_{ki,y}$ = 700 = 0,70 m

$z_{ka,y}$ = 700 = 0,70 m

$C_{ki,y} = T_{ki,y} = 0,7 \cdot (M_{kap,by\text{-}ki})/z_{ki,y}$

= 0,7.(751,92/0,70) = 751,92 KN

$C_{ka,y} = T_{ka,y} = 0,7 \cdot (M_{kap,by\text{-}ka})/z_{ka,y}$

= 0,7. (445,1/0,70) = 445,1 KN

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y} = 751,92 + 445,1 - 221,86$

= 975,16 KN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j \cdot h_c} \leq 1,5\sqrt{f'c}$$

$$v_{jh,y} = \frac{975,16}{0,75.0,60} = 2167,02 \text{ KN/m}^2$$

$= 2{,}167\ N/mm^2 < 1{,}5.\sqrt{22{,}5} = 7{,}115\ N/mm^2$ .....**Ok !**

$$V_{ch,y} = 2/3\ .\sqrt{\left\{\left(Puk_y/Ag\right) - 0{,}1.f'c\right\}}.b_j.h_c$$

$$V_{ch,y} = 2/3.\sqrt{\left\{\left(1362.10^3/600.600\right) - 0{,}1.22{,}5\right\}}.750.600$$

$= 371484\ N = 371\ KN$

$V_{sh,y} = V_{jh,y} - V_{ch,y}$

$= 975{,}16 - 371 = 604{,}16\ KN$

2) Sumbu X

$b_j = bc = 800\ mm$

$= bb + 0{,}5.hc = 450 + 0{,}5.600 = 750\ mm$

b_j pakai $= 750\ mm$

$h_c = 600\ mm$

$$V_{kol,x} = \frac{0{,}7\ .\left(\frac{lki}{lki'}.M_{kap,ki} + \frac{lka}{lka'}.M_{kap,ka}\right)}{1/2\ .\left(h_{k,a} + h_{k,b}\right)}$$

$$V_{kol,x} = \frac{0{,}7\ .\left(\frac{4}{3{,}2}.333{,}02 + \frac{4}{3{,}2}.333{,}02\right)}{1/2\ .\left(4{,}2 + 4{,}2\right)} = 171{,}4\ KN$$

$z_{ki,x} = 700 = 0{,}7\ m$

$z_{ka,x} = 700 = 0{,}7\ m$

$C_{ki,x} = T_{ki,x} = 0{,}7.\ (M_{kap,bx\text{-}ki})/z_{ki,x}$

$= 0{,}7.(333{,}02)/0{,}7 = 333{,}02\ KN$

$C_{ka,x} = T_{ka,x} = 0{,}7.\ (M_{kap,bx\text{-}ka})/z_{ka,x}$

$= 0{,}7.(333{,}02)/0{,}7 = 333{,}02\ KN$

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x}$

$= 333,02 + 333,02 - 171,4 = 495$ KN

Kontrol tegangan geser horizontal :

$$v_{jh.x} = \frac{V_{jh,x}}{b_j.h_c} \leq 1,5\sqrt{f'c}$$

$$v_{jh.x} = \frac{495}{0,75.0,6} = 1100 \text{ KN/m}^2$$

$= 1,1 \text{ N/mm}^2 < 1,5.\sqrt{22,5} = 7,115 \text{ N/mm}^2$ .....**Ok !**

$$V_{ch.x} = 2/3 \cdot \sqrt{\left\{\left(Pu, k_x / Ag\right) - 0,1.f'c\right\}}.b_j.h_c$$

$$V_{ch.x} = 2/3. \sqrt{\left\{\left(1205,96.10^3 / 600.600\right) - 0,1.22,5\right\}}.750.600$$

$= 314628$ N $= 315$ KN

$V_{sh,x} = V_{jh,x} - V_{ch,x}$

$= 495 - 315 = 180$ KN

b. Penulangan Geser Horisontal

$V_{sh,x\ mak} < V_{sh,y\ max}$ maka dipakai $V_{sh,y\ max} = 604,16$ KN

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{604,16.10^3}{300} = 1726 \text{ mm}^2$$

Digunakan sengkang rangkap 4 diameter tul. P10 dengan Av = 314 mm^2

Jumlah lapis sengkang = $\frac{1646}{314} = 5,2$ lapis

digunakan sengkang rangkap **6P10**

c. Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}} V_{jh,mak} \cdot \left(0{,}6 + {}^{Pu,k_y}\!/_{Ag.f'c}\right)$$

$$V_{cv} = 1 \,.\, 777 \,.10^3 \cdot \left(0{,}6 + {}^{972.10^3}\!/_{600.600.22{,}5}\right)$$

$= 559440$ N $= 560$ KN

$V_{jv} = bj/h_c . V_{jh,mak}$

$= (0{,}75/0{,}6).\ 777 = 971$ KN

$V_{sv} = V_{jv} - V_{cv} = 971 - 560 = 411$ KN

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{411.10^3}{300} = 1174 \text{ mm}^2$$

Digunakan sengkang rangkap 4 diameter tul. P10 dengan Av = 314 mm^2

Jumlah lapis sengkang $= \frac{1174}{314} = 3{,}74$ lapis

digunakan sengkang rangkap **4P10**


UNIVERSITAS ISLAM INDONESIA

4.6 Perencanaan Tangga

4.6.1 Spesifikasi Struktur

1. Tinggi lantai (h) = 4,2 m = 420 cm
2. Lebar Bordes (LB) = 1,75 m = 175 cm
3. Tinggi optrade rencana diambil 16 cm

   Jumlah Optrade = 420/17 = 24,7 dipakai 25 Buah

   Tinggi Optrade pakai = 420/26 = 16,8 cm

   Jumlah Antrade = 25 – 2 = 23

   Diambil Panjang Antrade = 30 cm
4. Sudut kemiringan Tangga = 16,8/30 = arc tg α $\rightarrow$ α = 29,54°
5. Dimensi Tangga

   Panjang Tangga (Lo)

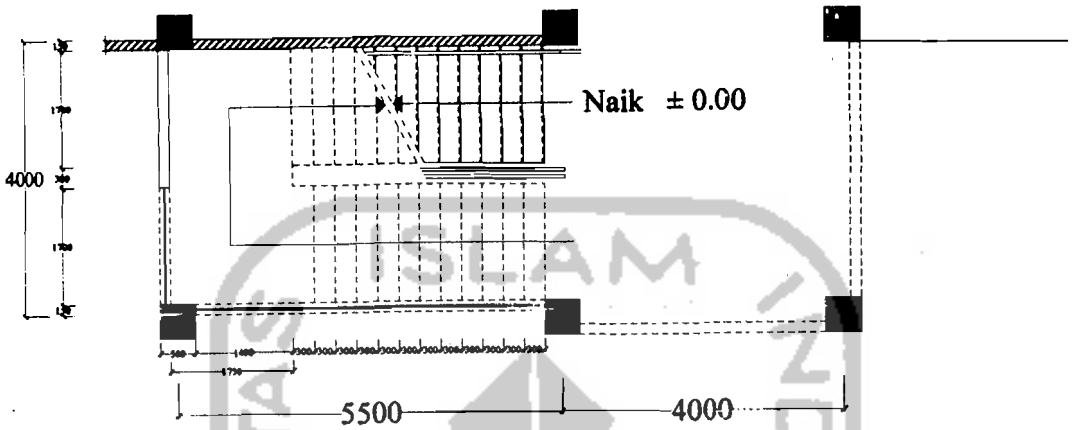
   Lo = (panjang antrade x jumlah antrade/2) + LB

   = (30 x 23/2) + 175 = 346,75 cm

   Lebar bersih tangga = 170 cm
6. Tebal Pelat bordes Diambil 13 cm

   Tebal pelat sisi tegak (h') = $\dfrac{13}{Sin.(90-29{,}54)}$ = 15 cm

4.6.2 Pembebanan



Gambar 4.30 Denah Tangga

1. Pembebanan Bordes Untuk SAP

Beban mati:

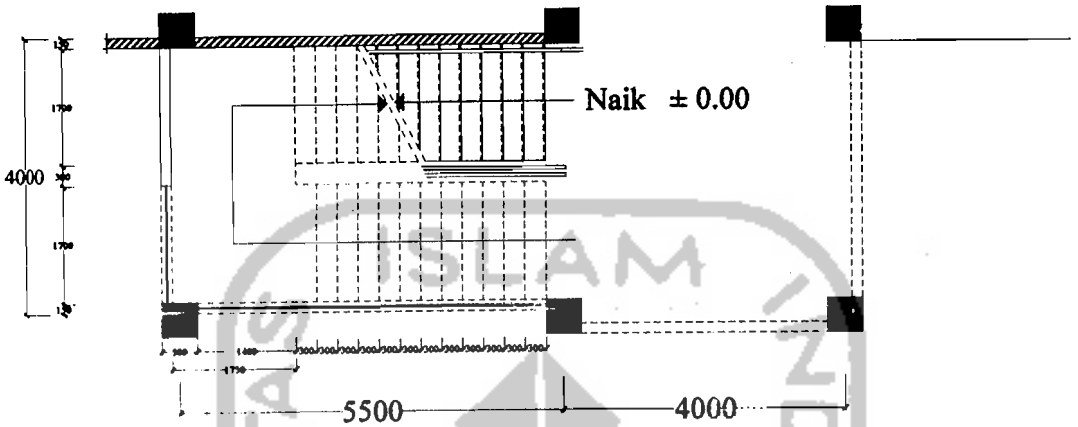
- Berat sendiri pelat = 0,13 . (1,70+30+1,70) . 24 = 11,23 KN/m
- Berat spesi = 0,03 . (1,70+30+1,70) . 24 = 1,22 KN/m
- Berat keramik = 0,01 . (1,70+30+1,70) . 24 = 0,41 KN/m

qD = 12,86 KN/m

Beban hidup:

QL = 300 kg/m = 3 KN/m^2 . 4 = 12 KN/m^2

4.6.2 Pembebanan



Gambar 4.30 Denah Tangga

1. Pembebanan Bordes Untuk SAP

Beban mati:

- Berat sendiri pelat = 0,13 . (1,70+30+1,70) . 24 = 11,23 KN/m
- Berat spesi = 0,03 . (1,70+30+1,70) . 24 = 1,22 KN/m
- Berat keramik = 0,01 . (1,70+30+1,70) . 24 = 0,41 KN/m

qD = 12,86 KN/m

Beban hidup:

QL = 300 kg/m = 3 KN/m^2 . 4 = 12 KN/m^2

4.6.4 Perencanaan Balok Bordes

Dimensi rencana balok :

Tinggi (h) = 400 mm

Lebar (b) = 200 mm

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

d = 400 – 30 – 10 – ½ 19 = 350,5 mm

Pembebanan :

1. Pembebanan Bordes

Beban mati

- Berat sendiri pelat = 0,13 .24 = 3,12 KN/m^2
- Berat spesi = 0,03 .24 = 0,72 KN/m^2
- Berat keramik = 0,01 .24 = 0,24 KN/m^2

qD = 4,08 KN/m^2

Beban hidup

QL = 300 kg/m = 3 KN/m^2

Qu = 1,2 qD + 1,6 qL = 1,2 . 4,08 + 1,6 . 3 = 9,7 KN/m^2

Untuk lebar = 4 m → qu = 9,7 . 4 = 38,80 KN/m

- berat pelat bordes = 9,7 . 4 = 38,80 KN/m
- berat sendiri = 1,2.0,20.0,40.24 = 2,30 KN/m +

qu = 41,10 KN/m

4.6.4 Perencanaan Balok Bordes

Dimensi rencana balok :

Tinggi (h) = 400 mm

Lebar (b) = 200 mm

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

d = 400 – 30 – 10 – ½ 19 = 350,5 mm

Pembebanan :

1. Pembebanan Bordes

Beban mati

- Berat sendiri pelat = 0,13 .24 = 3,12 KN/m^2
- Berat spesi = 0,03 .24 = 0,72 KN/m^2
- Berat keramik = 0,01 .24 = 0,24 KN/m^2

qD = 4,08 KN/m^2

Beban hidup

QL = 300 kg/m = 3 KN/m^2

Qu = 1,2 qD + 1,6 qL = 1,2 . 4,08 + 1,6 . 3 = 9,7 KN/m^2

Untuk lebar = 4 m → qu = 9,7 . 4 = 38,80 KN/m

- berat pelat bordes = 9,7 . 4 = 38,80 KN/m
- berat sendiri = 1,2.0,20.0,40.24 = 2,30 KN/m +

qu = 41,10 KN/m

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{68,5.10^6}{3,46.200}} = 314,62 \text{ mm} < d = 400 \text{ mm},$$

maka dipakai tulangan sebelah.

$h = d + z$ (z = 60 mm , karena dianggap tul. satu lapis)

$h = 228,75 + 60 = 288,75 \text{ mm} \approx 400 \text{ mm}$

$d_{ada} = h - z = 400 - 60 = 340 \text{ mm}$

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{68,5.10^6}{200.340^2} = 2,96 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{2,96}{3,46}0,011 = 0,0094 > \rho_{min} = 0,004$$

$\rho_{pakai} = 0.0094$

$As = \rho_{ada}.b.d_{ada} = 0,0094 \cdot 200 \cdot 340 = 639,2 \text{ mm}^2$

Dipakai diameter tulangan D19, maka : $A_1Ø = 283,39 \text{ mm}^2$

$$n = \frac{As}{A_1\phi} = \frac{639,2}{283,39} = 2,26 \text{ batang}$$

Dipakai tulangan memanjang **3D19 mm**, maka :

$As_{ada} = 3 \cdot 283,39 = 850,17 \text{ mm}^2 > As = 639,2 \text{ mm}^2$

Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0,85.f'c\, b} = \frac{850,17.350}{0,85.22,5.200} = 77,79 \text{ mm}$$

$$Mn = As_{ada}\, fy\,(d - \frac{a}{2}) \geq Mu/\phi$$

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{68,5.10^6}{3,46.200}} = 314,62 \text{ mm} < d = 400 \text{ mm},$$

maka dipakai tulangan sebelah.

$h = d + z$ (z = 60 mm , karena dianggap tul. satu lapis)

$h = 228,75 + 60 = 288,75 \text{ mm} \approx 400 \text{ mm}$

$d_{ada} = h - z = 400 - 60 = 340 \text{ mm}$

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{68,5.10^6}{200.340^2} = 2,96 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{2,96}{3,46}0,011 = 0,0094 > \rho_{min} = 0,004$$

$\rho_{pakai} = 0.0094$

$As = \rho_{ada}.b.d_{ada} = 0,0094 . 200 . 340 = 639,2 \text{ mm}^2$

Dipakai diameter tulangan D19, maka : $A_1Ø = 283,39 \text{ mm}^2$

$$n = \frac{As}{A_1\phi} = \frac{639,2}{283,39} = 2,26 \text{ batang}$$

Dipakai tulangan memanjang **3D19 mm**, maka :

$As_{ada} = 3 . 283,39 = 850,17 \text{ mm}^2 > As = 639,2 \text{ mm}^2$

Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada} fy}{0,85.f'c\ b} = \frac{850,17.350}{0,85.22,5.200} = 77,79 \text{ mm}$$

$$Mn = As_{ada}\ fy\ (d - \frac{a}{2}) \geq Mu/\phi$$

maka dipakai tulangan sebelah.

$h = d + z$ (z = 60 mm , karena dianggap tul. satu lapis)

$h = 222,47 + 60 = 282,47 \text{ mm} \approx 400 \text{ mm}$

$d_{ada} = h - z = 400 - 60 = 340 \text{ mm}$

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{34,25.10^6}{200.340^2} = 1,48 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{1,48}{3,46}0,011 = 0,00471 > \rho_{min} = 0,004$$

$\rho_{pakai} = 0,00471$

$$As = \rho_{pakai}.b.d_{ada} = 0,00471 \text{ . } 200 \text{ . } 340 = 320,28 \text{ mm}^2$$

Dipakai diameter tulangan D19, maka : $A_1Ø = 283,39 \text{ mm}^2$

$$n = \frac{As}{A_1\phi} = \frac{320,28}{283,39} = 1,1 \text{ batang}$$

Dipakai tulangan memanjang **2D19 mm**, maka :

$As_{ada} = 2 \text{ . } 283,39 = 566,78 \text{ mm}^2 > As = 345,6 \text{ mm}^2$

Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada}\, fy}{0,85.f'c\, b} = \frac{566,78.350}{0,85.22,5.200} = 51,86 \text{ mm}$$

$$Mn = As_{ada}\, fy\,(d - \frac{a}{2}) \geq Mu/\phi$$

$$= 566,78 \text{ . } 350\,(340 - \frac{51,86}{2}).\, 10^{-6} \geq Mu/\phi$$

$$= 62,30 \text{ KNm} > Mu/\phi = 34,25 \text{ KNm} \quad \text{(Ok!)}$$

b. Perencanaan tulangan geser balok bordes

a) Gaya Geser Dukungan

Vu dukungan = $\frac{1}{2}.q_U.L$

$= \frac{1}{2} . 41{,}1 . 4{,}0 = 82{,}2$ KN

maka $Vu/\phi = \frac{82{,}2}{0{,}6} = 137$ KN

$Vc = \left(\frac{1}{6}\sqrt{fc'}\right).b.d = \left(\frac{1}{6}\sqrt{22{,}5}\right).200.340 = 53758{,}72$ N = 53,76 kN

$Vu/\phi = 137 > Vc = 53{,}76$, maka perlu tulangan geser.

$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3} . 200 . 340 . 10^{-3} = 22{,}7$ kN

b) Gaya Geser Tengah Bentang

$Vu_{\text{tengah bentang}} = \frac{1}{8}.qU.L = \frac{1}{8}.41{,}1.4 = 20{,}55$ kN

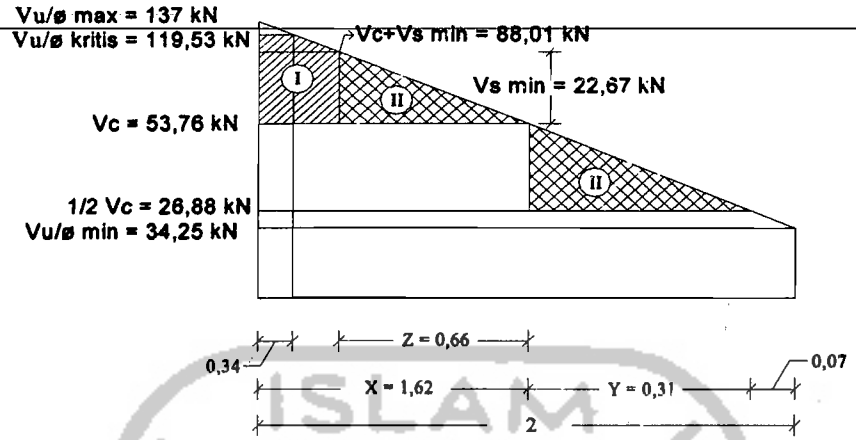
maka $Vu/\phi = \frac{20{,}55}{0{,}6} = 34{,}25$ kN

Tegangan Geser Beton (Vc) :

$Vc = \left(\frac{1}{6}\sqrt{fc'}\right).b.d = \left(\frac{1}{6}\sqrt{22{,}5}\right).200.340 = 53{,}76$ N = 53,76 kN

$Vu/\phi = 34{,}25 < Vc = 53{,}76$, maka tidak perlu tulangan geser.

$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3} . 200 . 340 . 10^{-3} = 22{,}67$ kN



Gambar 4.32 Diagram Tegangan Geser Balok Anak

$$Vu/\phi\ kritis = \frac{\left((½.L - d_{ada})(Vu/\phi_{max} - Vu/\phi_{min})\right)}{½.L} + Vu/\phi_{min}$$

$$= \frac{\left((2,0 - 0,34)(137 - 34,25)\right)}{2,0} + 34,25 = 119,53 \text{ kN}$$

$$x = \frac{(Vu/\phi_{max} - Vc).½L}{Vu/\phi_{max} - Vu/\phi_{min}} = \frac{(137 - 53,76).½.4}{137 - 34,25} = 1,62 \text{ m}$$

$$y = \frac{(Vc - ½Vc).½L}{Vu/\phi_{max} - Vu/\phi_{min}} = \frac{(53,76 - ½.53,76).½.4}{137 - 34,25} = 0,31 \text{ m}$$

$$z = \frac{Vu/\phi_{min} \cdot x}{Vu/\phi_{max} - Vc} = \frac{34,25 \,.1,62}{137 - 53,76} = 0,66 \text{ m}$$

Daerah I :

$(Vc + Vs_{min}) = 88,01 < Vu/\phi_{kritis} = 119,53$

Digunakan sengkang Ø P10 mm

$Av = 2 \,.\, ¼ \,.\, \pi \,.\, 10^2 = 157 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vu/\phi - Vc} = \frac{157.300.340}{137 - 53{,}76} . 10^{-3} = 192{,}38 \text{ mm}$$

$$< d/2 = 340/2 = 170 \text{ mm}$$

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

Jadi dipakai tulangan sengkang **P10 - 190 mm**

Daerah II :

$Vu/\phi = Vc + Vs_{min} = 88{,}01$ kN

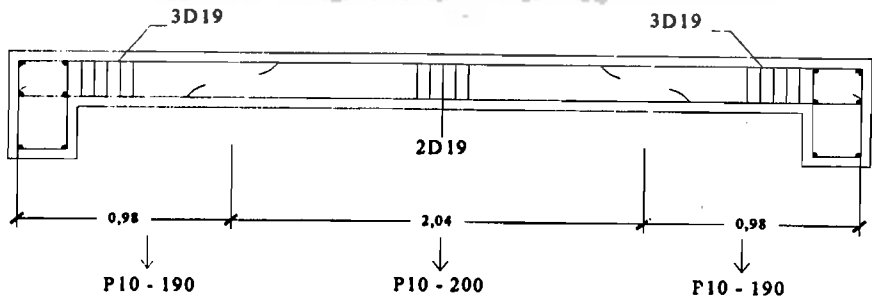
Digunakan sengkang Ø P10 mm, maka : $Av = 2 . ¼ . \pi . 10^2 = 157$ mm^2

$$s \leq \frac{Av.fy.d}{Vs_{min}} = \frac{157.300.340}{22{,}67} . 10^{-3} = 706 \text{ mm}$$

$$> d/2 = 340/2 = 175 \text{ mm}$$

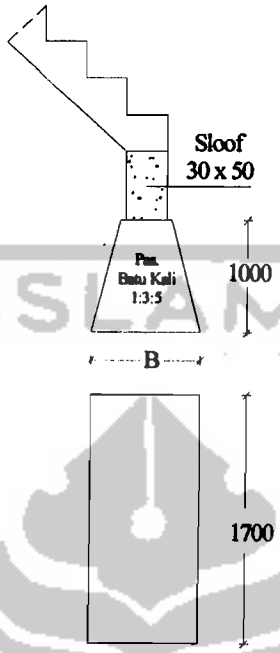
$$\geq 600 \text{ mm}$$

Jadi dipakai tulangan sengkang **P10 - 200 mm**



Gambar 4.33 Tulangan Geser Balok Anak

4.6.5 Perencanaan Pondasi Tangga



Gambar 4.34 Pondasi tangga

σ_{tanah} = 275 KN/m^2 ; γbtanah = 15 KN/m^3

f'c = 22,5 KN/m^2 ; γbeton = 24 KN/m^3

Pembebanan :

- Akibat beban tangga = 164,75 KN
- Berat balok diatas pondasi = 1,2x0,30x0,5x1,7x24 = 7,34 KN

Pu = 172,18 KN

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

$= 275 - (0,6.24) - (1,5.15)$

$= 238$ kN/m^2

$\sigma_{ijin} = \frac{Pu}{A}$

$A = \frac{Pu}{\sigma_{ijin}} = \frac{172,18}{238} = 0,72\ m^2$

A = B x L ; dimana L = 1,7 m, maka:

$B = \frac{0,72}{1,7} = 0,42$, dipakai B = 0,6 m, maka:

$A = 0,6 \times 1,7 = 1,02\ m^2$

Kontrol tegangan yang terjadi:

$\sigma_{terjadi} = \frac{Pu}{A} \geq \sigma_{ijin}$

$= \frac{172,18}{1,05} \leq \sigma_{ijin} = 172,18\ KN$

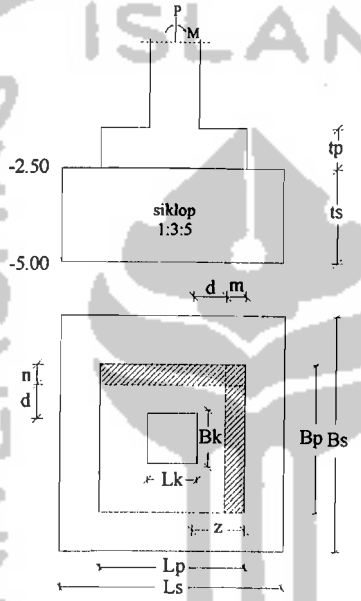
$= 163,98\ KN/m^2 \leq \sigma_{ijin} = 172,18\ KN$


ISLAM
UNIVERSITAS
INDONESIA

4.7 Perencanaan Pondasi

4.7.1 Perencanaan Dimensi Pondasi

4.7.1.1 Perencanaan Pondasi Telapak Setempat Pada Portal Arah X As 7 C Untuk Gempa Y Arah Kiri



Gambar 4.35 Pondasi Pelat Kaki dengan Siklop

fy = 350 Mpa

fc' = 22,5 Mpa

γbtanah = 15 KN/m^3 (dari data lab)

γbeton = 24 KN/m^3

lk kolom = 0,6 m

bk kolom = 0,6 m

Pd kolom dasar = 1603 KN

Pl kolom dasar = 243 KN

Pey kolom dasar = 127 KN

Pu kolom dasar = 1982 KN

Md kolom dasar = 44,43 KNm

Ml kolom dasar = 6,55 KNm

Mey kolom dasar = 239,44 KNm

σ tanah ijin(q) = 275 KN/m^3

Asumsi tebal plat (tp) = 0,60 m

a) Perhitungan luasan siklop (1:3:5) dan telapak Pondasi

Py = (Pd + Pl + Pey) = (1603 + 243 + 127) = 1972 KN

My = (Md + Ml + Mey) = (22 + 0,37 + 282) = 290 KNm

$$e = \frac{My}{Py} = \frac{290}{1972} = 0{,}15\,m$$

i) Dimensi Siklop (Ls)

Tinggi Cyclop (ts) = 2,5m = 2500 mm

$\sigma_{\text{tanah netto}} = \sigma \text{ tanah} - [(Df - tp - ts).\gamma\text{tanah}] - (tp.24) - (ts.\ \gamma_{cyclop})$

= 275 – [(5 – 2,5 – 0,60).15] – (0,60.24) – (2,5 . 22)

= 178 KN/m^3

Asumsi Lebar siklop Ls = 3,7 m, 1/6.Ls ≥ 0,16, maka eksentrisitas kecil

Untuk eksentrisitas kecil maka Bs dapat dicari dengan rumus:

$$Bs \geq \left(\frac{(Ls.Py) + (6.Py.e)}{\sigma_{tnh.netto}.Ls^2} \right)$$

$$Bs \geq \left(\frac{(3{,}7.1972) + (6.1972.0{,}15)}{(178.3{,}7^2)} \right)$$

Dicoba Bs = 3,7 m

$$3{,}7\ \text{m} \geq \left(\frac{(3{,}7.1972)+(6.1972.0{,}15)}{(178.3{,}7^2)} \right)$$

$$3{,}7 \geq 3{,}7\ \text{m}$$

Sehingga dari perhitungan trial dan error dipakai :

Panjang siklop (Ls) = 3,7 m

Lebar siklop (Bs) = 3,7 m

Syarat :

$$\text{Teg. Maks} = \frac{Py}{Bs.Ls} + \frac{My}{1/6.Bs.Ls^2} \leq \text{Teg. Netto tanah} = 178\ \text{KN/m}^3$$

$$= \frac{1972}{3{,}7.3{,}7} + \frac{1972.0{,}16}{1/6.3{,}7.3{,}7^2} \leq 178\ \text{KN/m}^3$$

$$= 178\ \text{KN/m}^3 \leq 178\ \text{KN/m}^3 \qquad (\textbf{Aman})$$

ii) Dimensi footplat (L_P)

$q_{\text{siklop ijin}}$ (1:3:5) = 1000 KN/m^2 ~ 1500 KN/m^2

maka dipakai 1000 KN/m^2

$$\sigma_{\text{ijin siklop netto}} = \sigma\ \text{siklop} - [(Df - tp).\gamma\text{tanah}] - (tp.24)$$

$$= 1000 - [(2{,}5 - 0{,}60).15] - (0{,}60.24)$$

$$= 957\ \text{KN/m}^3$$

Asumsi Lebar siklop Lp = 1,8 m, 1/6.Ls ≥ 0,16, maka eksentrisitas kecil

Untuk eksentrisitas kecil maka Bp dapat dicari dengan rumus:

$$Bp \geq \left(\frac{(Lp.Py)+(6.Py.e)}{\sigma_{siklop.netto}.Lp^2} \right)$$

$$Bp \geq \left(\frac{(1{,}8.1972)+(6.1972.0{,}16)}{(957.1{,}8^2)} \right)$$

Dicoba Bp = 1,8 m

$$1{,}8 \text{ m} \geq \left(\frac{(1{,}8.1972)+(6.1972.0{,}16)}{(957.1{,}8^2)} \right)$$

$$1{,}8 \geq 1{,}71 \text{ m}$$

Sehingga dari perhitungan trial dan error dipakai :

Panjang pondasi (Lp) = 1,8 m

Lebar pondasi (Bp) = 1,8 m

Syarat :

$$\text{Teg. Maks} = \frac{Py}{Bp.Lp} + \frac{My}{1/6.Bp.Lp^2} \leq \text{Teg. Netto tanah} = 957 \text{ KN/m}^3$$

$$= \frac{1972}{1{,}8.1{,}8} + \frac{1972.0{,}16}{1/6.1{,}8.1{,}8^2} \leq 957 \text{ KN/m}^3$$

$$= 908 \text{ KN/m}^3 \leq 957 \text{ KN/m}^3 \qquad (\textbf{Aman})$$

b) Kontrol Terhadap Geser

tebal pondasi (tp) diambil = 600 mm; d` (didalam tanah) = 100 mm

d = tp - d` = 600-100 = 500 mm

$$m = \frac{Lp - Lk - 2d}{2} = \frac{1{,}8 - 0{,}6 - 2.0{,}50}{2} = 0{,}1 \text{ m}$$

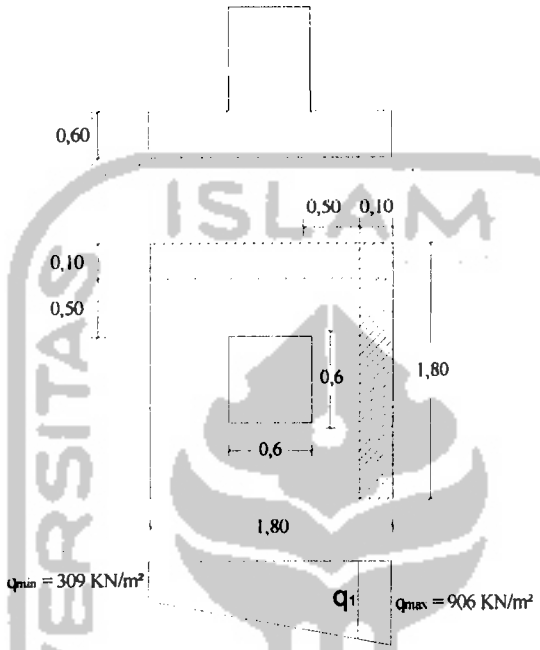
$$n = \frac{Bp - bk - 2d}{2} = \frac{1{,}8 - 0{,}6 - 2.0{,}50}{2} = 0{,}1 \text{ m}$$

Pu terhadap kombinasi gempa y kiri = 1,05.(PD + PLr + PEy) = 1982 KN

$$q \text{ max} = \frac{Pu}{Lp^2}\left(1 + \frac{6.e}{Lp}\right) = \frac{1982}{1{,}8^2}\left(1 + \frac{6.0{,}15}{1{,}8}\right) = 906\, KN/m^2$$

$$q\ min = \frac{Pu}{Lp^2}\left(1-\frac{6.e}{Lp}\right)=\frac{1982}{1{,}8^2}\left(1-\frac{6.0{,}15}{1{,}8}\right)=309\,KN/m^2$$

❖ Kontrol Geser Satu Arah



Gambar 4.36 Pondasi dengan geser satu arah

$$q_1 = q\min+\frac{Lp-d}{Lp}(q\max-q\min)$$

$$= 309+\frac{1{,}8-0{,}5}{1{,}8}(906-306)=740\ KN/m^2$$

$$Vu_1 = \frac{q\max+q1}{2}.Lp.m.Bp.$$

$$= \frac{906+740}{2}.1{,}8.0{,}1.1{,}8$$

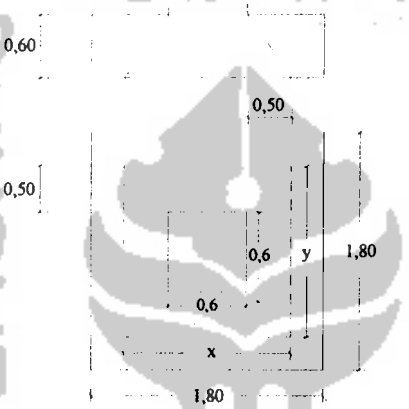
$$= 267\ KN$$

$$Vu_1/\phi = 267/0{,}6 = 445\ KN$$

$$Vc = \frac{1}{6}\sqrt{f`c}.Lp.d = \frac{1}{6}.\sqrt{22,5}.1800.500 = 712\,KN$$

$$Vc = 712\ KN \geq {}^{Vu_1}\!/_{\phi} = 445\ KN\ (\textbf{Aman})$$

❖ **Kontrol Geser Dua Arah**



Gambar 4.37 Pondasi dengan geser dua arah

Teg. Ultimit rata-rata (σ_u) $= \dfrac{Pu}{Bp.Lp} = \dfrac{1982}{1,8.1,8} = 608\ KN/m^2$

x = Lk + d = 0,6 + 0,50 = 1,10 m = 1100 mm

y = Bk + d = 0,6 + 0,50 = 1,10 m = 1100 mm

bo = (x + y).2 = (1,10+1,10).2 = 4,4 m

Vu = σ_u . {(Bp.Lp)-(x.y)}

= 608 . {(1,8.1,8)-(1,10.1,10)}

= 1234 KN

${}^{Vu}\!/_{\phi} = {}^{1223}\!/_{0,6} = 2056$ KN

$$Bc = \frac{Bp}{Lp} = \frac{1}{1} \leq 2$$

$$Vc = 4.\sqrt{f`c}.bo.d = 4.\sqrt{22,5}.4400.500 = 41742\ KN$$

$$Vc = 41742\ KN \geq \frac{Vu_1}{\phi} = 2056\ KN \quad \textbf{(Aman)}$$

c) Perhitungan Tulangan Lentur Pondasi

• Kuat tumpuan Pondasi :

$$\phi.Pn = \phi.(0,85.f`c.A_1.\sqrt{\frac{A_2}{A_1}})$$

Luas penampang kolom (A_1) = bk . hk = 0,60. 0,60 = 0,36 m^2

Luas pelat pondasi (A_2) = Bx . By = 1,8 . 1,8 = 3,24 m^2

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{3,24}{0,36}} = 3 > 2$$ (jika lebih besar dari 2, dipakai nilai 2)

$$\phi.Pn = \phi.(0,85.f`c.A_1.2)$$

$$= 0,7.(0,85 . 22,5 . 0,36 . 2).10^3 = 9639\ KN$$

• Kuat tumpuan kolom :

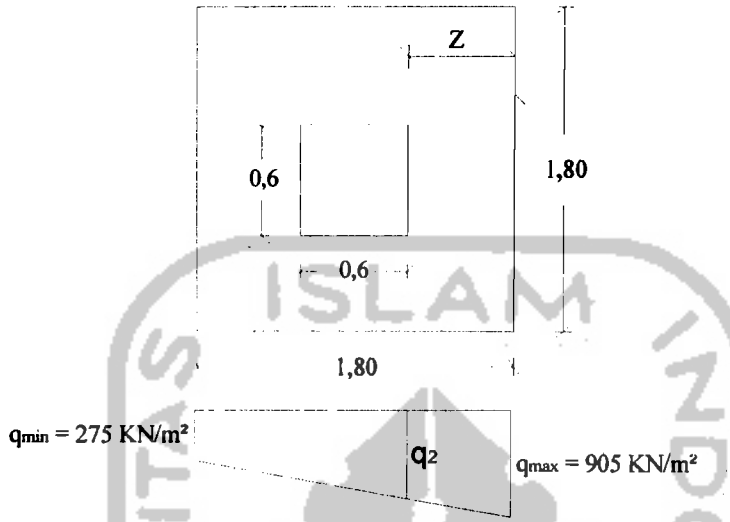
$$\phi.Pn = \phi.(0,85.f`c.A_1)$$

$$= 0,7.(0,85.22,5.0,36).10^3 = 4820\ KN$$

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 9639\ KN > \phi.Pn_{kolom} = 4820\ KN$ **.....Ok !**

d) Perhitungan Tulangan Lentur Pondasi



Gambar 4.38 Tulangan Lentur Pondasi

$$Z = \frac{Lp - Lk}{2} = \frac{1{,}8 - 0{,}6}{2} = 0{,}6 \text{ m}$$

$$q_2 = q\min + \frac{Lp - Z}{Lp}(q\max - q\min)$$

$$= 306 + \frac{1{,}8 - 0{,}6}{1{,}8}(906 - 306) = 707\ KN/m^2$$

$$Mu_{pakai} = \frac{q_2 . Z}{2} \frac{1}{3}.Z + \frac{qu_{max} . Z}{2} \frac{2}{3}.Z$$

$$= \left(\frac{707.0{,}6}{2} \frac{1}{3}.0{,}6 \right) + \left(\frac{906.0{,}6}{2} \frac{2}{3}.0{,}6 \right)$$

$$= 151 \text{ KNm}$$

$$\frac{Mu_{pakai}}{\phi} = \frac{151}{0{,}8} = 189 \text{ KNm}$$

- Digunakan tulangan pokok $\varnothing_{16}$ mm, maka : $A_{1\varnothing} = 201$ mm^2

- Tebal pelat pondasi : tp = 600 mm, selimut beton (Pb) = 70 mm

$$d = tp - Pb - 0{,}5.\varnothing_{tul.\ pokok} = 600 - 70 - 0{,}5.\ 16 = 522 \text{ mm}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{350}{0{,}85.22{,}5} = 18{,}3$$

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

$$Rn = \frac{Mu_{pakai}/\phi}{b.d^2} = \frac{189.10^6}{1000.522^2} = 0{,}7 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{350} = 0{,}004$$

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5.0{,}85}{350}.\left(\frac{600}{600+350}\right) = 0{,}029$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}029 = 0{,}022$$

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

$$= \frac{1}{18{,}3}.\left(1 - \sqrt{1 - \frac{2.18{,}3.0{,}7}{350}}\right) = 0{,}002 \quad < \rho_{mak} = 0{,}0211$$

$$< \rho_{min} = 0{,}004$$

$1{,}33\ .\ 0{,}002 = 0{,}0027 < \rho_{min} = 0{,}004$

$\rho_{pakai} = 0{,}0027$

$As_{perlu} = \rho_{pakai}\ .\ b\ .\ d = 0{,}0027\ .\ 1000\ .\ 522 = 1401 \text{ mm}^2$

$As_{susut} = 0{,}002.b.h = 0{,}002.1000.600 = 1200 \text{ mm}^2 < As_{perlu}$,

maka, $As_{perlu} = 1401 \text{ mm}^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{201.1000}{1401} = 143 \text{ mm}$$

$$s \leq 2.h = 2.600 = 1200 \text{ mm}$$

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

→ Dipakai Tulangan Pokok : D_{16} – 140 mm

$$As_{ada} = \frac{A_{16}.1000}{s} = \frac{201.1000}{140} = 1436 \text{ mm}^2$$

Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1436.350}{0{,}85.22{,}5.1000} = 26 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - \frac{a}{2}\right)$$

$$= 1436 . 350 \left(522 - \frac{26}{2}\right) . 10^{-6}$$

$$= 256 \text{ KNm} \geq \frac{Mu_{pakai}}{\phi} .1{,}33 = 252 \text{ KNm} \ldots\ldots\ldots \textbf{Ok !}$$

<u>Perencanaan Tulangan Susut Pondasi</u>

Syatat: Jika tp > 200 mm, maka perlu tulangan susut

Karena tp = 600 mm > 200 mm, maka perlu tulangan susut

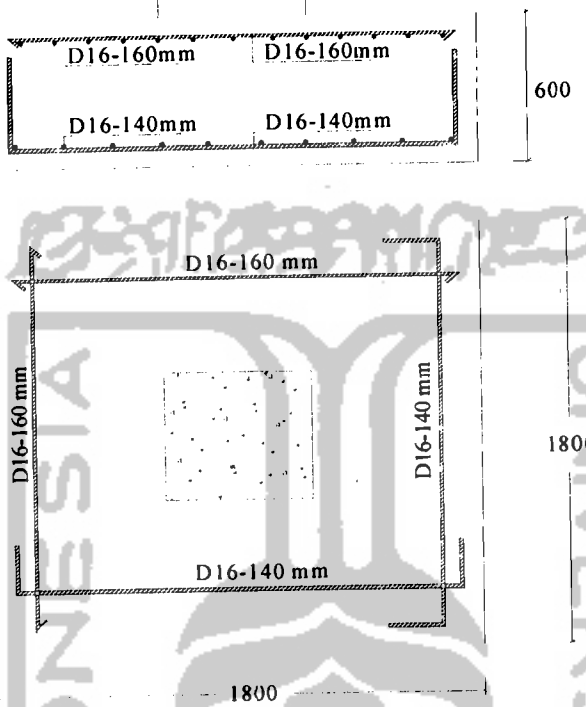
$As_{bagi} = 0{,}002. b. tp = 0{,}002. 1000. 600 = 1200 \text{ mm}^2$

Digunakan tulangan bagi ∅16 mm, maka: $A_{1\varnothing} = 201 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{201.1000}{1200} = 167 \text{ mm}$$

→ Dipakai Tulangan Susut : P_{16} – 160 mm



Gambar 4.39 Tulangan Pondasi Pelat Kaki